CHAPTER 3
SOLUTIONS

3-1 Lake Pleasant elevation drop

Given: Inflow = 0.0; Outflow = 0.0; Evaporation = 6.8 mm/d; Seepage = 0.01 mm/d

Solution:

a. The mass balance for the lake is

$$\text{Storage} = P + Q_{in} + I_{in} - Q_{out} - R - E - T$$

$$\text{Storage} = 0.0 + 0.0 - (0.01\text{mm/d})(31\text{d}) - 0.0 - 0.0 - (6.8\text{mm/d})(31\text{d}) - 0.0$$

$$\text{Storage} = -0.31 \text{ mm} - 210.8 \text{ mm} = -211.11 \text{ mm or } -210 \text{ mm}$$

b. With a vertical lake shore the elevation drop is equal to the change in storage.

Elevation drop = 210 mm or 21 cm

c. With a slope of 5°

$$r = y^*\csc\Theta$$

$$r = (211.11 \text{ mm})^*\csc(5^\circ)$$

$$r = (211.11 \text{ mm})(11.47) = 2421.4 \text{ mm or } 242 \text{ cm}$$

3-2 Mass balance on storage reservoir

Given: Dimensions of Lake Kickapoo = 12 km x 2.5 km; Inflow = 3.26 m^3/s; outflow = 2.93 m^3/s; precipitation = 15.2 cm; evaporation = 10.2 cm; seepage = 2.5 cm

Solution:

a. The mass balance diagram is shown below.

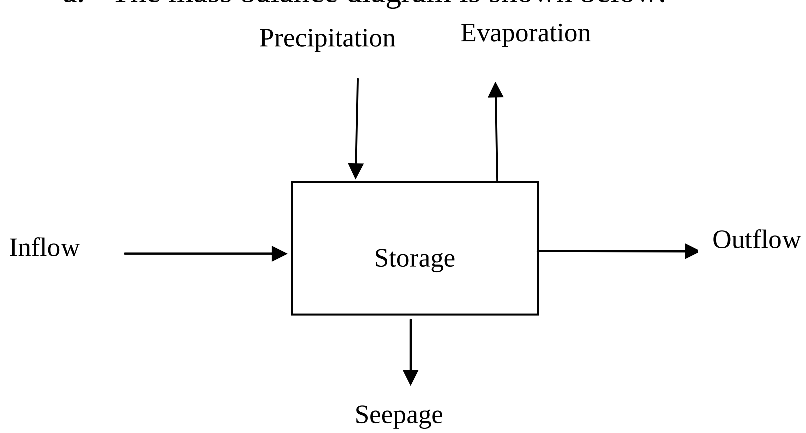


Figure S-3-2 Mass Balance Diagram

b. The mass balance equation is:

$$\Delta\text{Storage} = \text{Precipitation} + \text{Inflow} - \text{Evapotranspiration} - \text{Outflow} - \text{Seepage}$$

c. Convert all units to volumes

Area of Lake Kickapoo = (12 km)(2.5 km) $(1 \times 10^6\ m^2/km^2) = 3.0 \times 10^7\ m^2$

Precip. = (15.2 cm)$(3.0 \times 10^7\ m^2)(10^{-2}\ m/cm) = 4.56 \times 10^6\ m^3$

Inflow = $(3.26\ m^3/s)$(86,400 s/d)(31 d/mo of MAR) = $8.73 \times 10^6\ m^3$

Evap. = (10.2 cm)$(3.0 \times 10^7\ m^2)(10^{-2}\ m/cm) = 3.06 \times 10^6\ m^3$

Outflow = $(2.93\ m^3/s)$(86,400 s/d)(31 d/mo of MAR) = $7.85 \times 10^6\ m^3$

Seepage = (2.5 cm)$(3.0 \times 10^7\ m^2)(10^{-2}\ m/cm) = 7.5 \times 10^5\ m^3$

d. Compute change in storage

$$\Delta\text{Storage} = 4.56 \times 10^6\ m^3 + 8.73 \times 10^6\ m^3 - 3.06 \times 10^6\ m^3 - 7.85 \times 10^6\ m^3 - 7.5 \times 10^5\ m^3$$

$$\Delta\text{Storage} = 1.63 \times 10^6\ m^3$$

3-3 Mass balance on storage reservoir and runoff coefficient

Given: watershed area = 4,000 km^2; precipitation = 102 cm/y; flow of river = 34.2 m^3/s; infiltration = 5.5×10^{-7} cm/s; evapotranspiration = 40 cm/y

Solution:

a. The mass balance diagram is shown below.

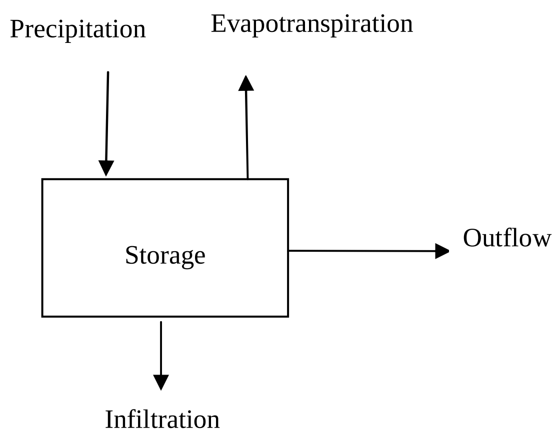


Figure S-3-3 Mass Balance Diagram

b. The mass balance equation is:

ΔStorage = Precipitation - Outflow - Evapotranspiration - Infiltration.

c. It is convenient to solve the mass balance equation in units of cm/y, so converting flow and infiltration:

$$\text{Flow} = \frac{(34.2\,\text{m}^3/\text{s})(86400\,\text{s/d})(365\,\text{d/y})(100\,\text{cm/m})}{(4000\text{km}^2)(1\times10^6\ \text{m}^2/\text{km}^2)} = 26.96\,\text{cm/y}$$

Infiltration = (5.5 x 10^{-7} cm/s)(86,400 s/d)(365 d/y) = 17.34 cm/y

d. Compute the change in storage.

ΔStorage = 102 cm/y - 26.96 cm/y - 40 cm/y - 17.34 cm/y = 17.70 cm/y

The volume for the 4,000 km^2 area,

Volume = (17.70 cm/y)(10^{-2} m/cm)(4,000 km^2)(1 x 10^6 m^2/km^2)

Volume = 7.08 x 10^8 m^3 or 7 x 10^8 m^3

e. The runoff coefficient is

$$C = \frac{\text{runoff}}{\text{precipitation}} = \frac{26.96\text{cm}}{102\text{cm}} = 0.26$$

3-4 Infiltration rates and total volume

Given: Values for Horton constants for Fuquay pebbly lam sand

Solution:

a. For 12 minutes (.020 h)

$f = 61 + (159 – 61) \exp[(-4.7)(0.20 \text{ h})] = 99.28$ or 99 mm/h

b. For 30 minutes (0.50 h)

$f = 61 + (159 – 61) \exp[(-4.7)(0.50 \text{ h})] = 70.35$ or 70 mm/h

c. For 60 minutes (1.0 h)

$f = 61 + (159 – 61) \exp[(-4.7)(1.0 \text{ h})] = 61.89$ or 62 mm/h

d. For 120 minutes (2.0 h)

$f = 61 + (159 – 61) \exp[(-4.7)(2.0 \text{ h})] = 61.00$ or 61 mm/h

e. Volume over 120 minutes (2.0 h)

$$V = (61)(2) + \frac{159-61}{4.7}\{1-\exp[(4.7)(2.0)]\} = 122 + 20.85 = 142.85\text{mm}$$

3-5 Total volume of infiltration

Given: Values for Horton constants: f_o = 4.70 cm/h or 47.0 mm/h; f_c = 0.70 cm/h or 7.0 mm/h; k = 0.1085 h^{-1} and three sequential storms of 30 minute duration with precipitation rates of 30 mm/h, 53 mm/h, and 23 mm/h.

Solution:

a. First 30 minutes

V_{storm} = (30 mm/h)(0.5 h) = 15 mm

$$V_{horton} = (7.0)(0.5) + \frac{47.0-7.0}{0.1085}\{1-\exp[(-0.1085)(0.5)]\} = 22.97\,mm$$

Since the volume of precipitation is less than the infiltration, the volume of infiltration is 15 mm

b. Second 30 minutes

V_{storm} = (53 mm/h)(0.5 h) = 26.5 mm

$$\forall_{horton} = (7.0)(0.5) + \frac{47.0 - 7.0}{0.1085}\{1 - \exp[(-0.1085)(1.0)]\} = 41.4\text{mm}$$

Since the volume of precipitation is 15 + 26.5 = 41.5 mm, the volume of infiltration is 41.5 mm

c. Third 30 minutes

$\forall_{storm}$ = (23 mm/h)(0.5 h) = 11.5 mm

$$\forall_{horton} = (7.0)(0.5) + \frac{47.0 - 7.0}{0.1085}\{1 - \exp[(-0.1085)(1.5)]\} = 58.87\text{mm}$$

Since the volume of precipitation is 15 + 26.5 + 11.5 = 53.0 mm, the volume of infiltration is 53 mm

3-6 Estimated evaporation

Given: Lake Hefner equations; air temperature= 30 °C; water temperature = 15 °C; wind speed 9 m/s; and RH = 30 %.

Solution:

a. From Table 3-1, at 15 °C the saturation vapor pressure is estimated to be 1.704 kPa

b. Using a vapor pressure of 4.243 at 30 °C and 30 % RH, the vapor pressure in the overlying air is estimated to be:

e_a = (4.243 kPa)(0.30) = 1.2729 kPa

c. Estimated evaporation

E = 1.22(1.704 - 1.2729)(9) = 4.73 or 4.7 mm

3-7 Estimated evaporation – hot and dry

Given: Lake Hefner equations; air temperature = 40˚C; water temperature = 25˚C; wind speed is 2.0 m/s; and relative humidity is 5 %

Solution:

a. From Table 3-1 with a water temperature of 25˚C, the saturation vapor pressure is estimated to be 3.167 kPa

b. Using a vapor pressure of 7.378 at 40˚C and 5% RH, the vapor pressure of the overlying air is estimated to be

$e_a = (7.378)(0.05) = 0.3689$ kPa

c. Estimated evaporation

$E = 1.22(3.167 - 0.3689)(2.0) = 6.83$ or 6.8 mm/d

3-8 Estimated humidity to reduce evaporation to nil

Given: Water temperature = 10 °C; air temperature = 25 °C

Solution:

a. From Table 3-1, at 10 °C the saturation vapor pressure is 1.227 kPa

b. For E to = 0 regardless of wind speed, the values of e_a and e_s must be equal. At 25 °C the value of e_a must be 1.227, so

$e_a = 1.227 = (3.167)(RH)$

Solving for RH

$$RH = \frac{1.227}{3.167} = 0.387 \text{ or } 39\ \%$$

3-9 IDF curve for 2 y storm

Given: T = 2 y; n = 45 y; Table 3-1

Solution:

$$m = \frac{n+1}{T} = \frac{46}{2} = 23$$

Starting with the 5-minute duration, note that the 23rd ranked storm lies between the 49th and 16th ranked storms, that is:

Intensity, mm/h

	120		140
Rank	49		16
		< 23 >	

By interpolating find the intensity is 135.8 mm/h. Using m = 23 interpolate to find intensities for selected durations.

Duration (min)	Intensity (mm/h)
5	135.8
10	116.7
15	98.7
20	70.0
30	37.8
40	33.5
50	23.9
60	--

3-10 IDF curve for 10 y storm

Given: T = 2 y ; n = 45; Table 3-1

Solution:

$$m = \frac{46}{10} = 4.60$$

By interpolation find intensities for selected durations:

Duration (min)	Intensity (mm/h)
5	172.0
10	156.0
15	129.3
20	98.0
30	72.6
40	49.7
50	38.2
60	26.3

3-11 IDF curve for 5 y storm

Given: T = 5 y; n = 10; Annual max data

Solution:

$$m = \frac{11}{5} = 2.20$$

Under each duration find intensity of 2.20 ranked storm by interpolation:

Duration (min)	Intensity (mm/h)
30	118.8
60	96.8
90	77.8
120	52.2

3-12 IDF curve for 2 y storm

Given: T = 2 y; n = 10; Annual max. data from 2-3

Solution:

$$m = \frac{11}{2} = 5.50$$

Under each duration find intensity of 5.50 ranked storm by interpolation:

Duration (min)	Intensity (mm/h)
30	82.0
60	61.2
90	41.1
120	16.2

3-13 Parking lot configuration

Given: Vertical and horizontal configurations
For "a" D = 830 m, S = 6.00 %
For "b" D = 600 m, S = 6.00 %

Solution:

a. From Table 3-3 under pavement select C = 0.95 for "asphaltic"

b. For configuration "a"

$$t_c = \frac{1.8(1.1-0.95)[(3.28)(830)]^{1/2}}{6.00^{1/3}} = \frac{14.08}{1.817} = 7.75\,\text{min or } 7.8\text{ min}$$

c. For configuration "b"

$$t_c = \frac{1.8(1.1-0.95)[(3.28)(600.00)]^{1/2}}{6.00^{1/3}} = \frac{11.977}{1.817} = 6.59\,\text{min or } 6.6\text{ min}$$

3-14 Mechanicsville runoff by rational method

Given: Figure P-3-14, 2 y storm and building types with areas shown in table below.

Solution:

Area Type	Area (m^2)	% of Total	C*
Slate roofs	15831	21.39	0.95
Asphalt streets	18886	25.52	0.95
Flat (2%) sandy soil	39293	53.09	0.10
SUM	74010	100.00	

* From Table 3-3. The most conservative estimates of C are those that yield the greatest runoff and, hence, result in the largest (most conservative) storm sewer.

Composite value for C

C = .2139(.95) + .2552(.95) + .5309(.10)
C = 0.2032 + 0.2424 + 0.0531
C = 0.4987

Calculate t_c using "flat" slope of 2.0% from Eqn. 3-16

$$t_c = \frac{1.8(1.1-0.4987)[(3.28)(272)]^{1/2}}{2.0^{1/3}} = \frac{32.3266}{1.2599} = 25.65\,\text{min}$$

From IDF Curve (Figure P-3-14) find i at Duration = 25.65 min
i = 59 mm/h

Compute peak discharge from Eqn. 3-15

Q = 0.0028(0.4987)(59)(74,010)(1 x 104 m^2/ha)

Q = 0.6097 m^3/s or 0.61 m^3/s

3-15 Mechanicsville runoff in Miami, FL

Given: Same as 3.14

Solution:

a. Calculate composite C and t_c as in Problem 3.14

b. From IDF curve for Miami, FL find C at Duration = 25.65 min (0.42 h)
From Figure 3-10c read i $\cong$ 110 mm/h

c. Peak discharge
Q = 0.0028(0.4987)(110)(74010)(1 x 10^{-4})
Q = 1.136 or 1.14 m^3/s

3-16 Little League/pasture runoff by rational method

Given:
A = 9.94 ha
D = 450 m
S = 2.00 %
C = 0.20
IDF curves from Boston, MA (Figure 3-10a)
5 year return period

Solution:

a. Compute t_c

$$t_c = \frac{1.8(1.1 - 0.20)[(3.28)(450.0)]^{1/2}}{2.00^{1/3}} = \frac{62.24}{1.2599} = 49.398 \text{ min}$$

b. For 5 y storm in Boston

$$\frac{49.398 \text{ min}}{60 \text{ min}/h} = 0.82h$$

From Figure 3-10a read i = 38 mm/h

c. Peak discharge
Q = 0.0028(0.20)(38)(9.94)
Q = 0.21 m^3/s

3-17 Little League/parking lot runoff

Given:

A = 2.64 ha
D = 200.0 m
S = 1.80 %
C = 0.70
IDF curves for Boston, MA (Figure 3-10a)
5 y return period

Solution:

a. Compute t_c

$$t_c = \frac{1.8(1.1 - 0.70)[(3.28)(200)]^{1/2}}{1.80^{1/3}} = \frac{18.44}{1.216} = 15.15\,\text{min}$$

b. From IDF curve for Boston

$$\frac{15.15\,\text{min}}{60\,\text{min}/h} = 0.25h$$

From Figure 3-10a read i = 76 mm/h

c. Peak discharge

Q = 0.0028(0.70)(76)(2.64)
Q = 0.39 m^3/s

d. Culvert does NOT have enough capacity

0.39 m^3/s > 0.21 m^3/s

3-18 Peak discharge at Holland, MI

Given:

A = 4.8 ha
D = 219.0 m
C = 0.85
S = 1.00 %
IDF curve equation for Holland, MI

Solution:

a. Calculate t_c

$$t_c = \frac{1.8(1.1 - 0.85)[(3.28)(219.0)]^{1/2}}{1.00^{1/3}} = \frac{12.06}{1.0} = 12.06\,\text{min}$$

b. Calculate i

$$i = \frac{1193.80}{12.06^{0.8} + 7} = \frac{1193.80}{7.33 + 7} = 83.3\,\text{mm/h}$$

c. Peak discharge

Q = 0.0028(0.85)(83.31)(4.8)
Q = 0.95 m^3/s

3-19 Shopping mall runoff by rational method

Given: Sketch shown in Figure P-3-19 and IDF curve from Figure P-3-14

Solution:

PART I: Frequency of flooding with existing culvert

a. First calculate t_c for pasture alone (Eqn. 3-16)

$$t_c = \frac{1.8(1.1 - 0.20)[(3.28)(1000)]^{1/2}}{2.0^{1/3}} = 74\,\text{min}$$

b. From Fig. P-3-14 at duration = 74 min. find

i = 33 mm/h

c. Now determine design flow (maximum Q) for existing culvert from Eqn. 3-15

Q = 0.0028(0.20)(33)(40.0)

Q = 0.7392 or 0.74 m^3/s

d. Calculate t_c for parking lot alone

$$t_c = \frac{1.8(1.1 - 0.70)[(3.28)(447.21)]^{1/2}}{2.0^{1/3}} = 22\,\text{min}$$

e. The intensity of rainfall that will cause flooding.

Since the t_c from the parking lot is substantially less than that for the pasture, the peak flows will not coincide and the controlling discharge will be for the shorter duration from the parking lot. Thus, ignoring the pasture, the intensity on the parking lot that will yield the peak discharge may be found by solving Eqn 3-15 for the intensity (i):

$$i = \frac{0.7392\,m^3/s}{(0.0028)(0.70)(10.0\,ha)} = 37.71\,mm/h$$

f. Using the t_c for the parking lot and the intensity calculated in "e" and plotting the intersection of these two lines on Figure P-3-14, find, by interpolation, that the frequency of flooding is approximately 4 times per year.

PART II

Peak discharge for 10 y storm (again ignoring pasture because t_c is so much greater):

a. From P-3-14 using t_c = 22 min and freq. = 10 y

i = 100 mm/h

b. Then peak discharge (and design flow) for 10 y storm is

Q = 0.0028(0.70)(100 mm/h)(10.0 ha)

Q = 1.96 or 2.0 m^3/s

3-20 Clinic runoff by rational method

Given: Sketch shown in Figure P-3-20 and IDF curve in P-3-14.

Solution:

Part I: Frequency of flooding with existing culvert

a. Calculate the time of concentration (t_c) for the pasture alone using Eqn 3-16:

$$t_c = \frac{1.8(1.1 - 0.16)[(3.28)(350)]^{1/2}}{4.40^{1/3}} = 35.0\,min$$

b. From Figure P-3-14 at a duration = 35 min and a 5 y storm find

i = 63 mm/h

c. Now calculate the design flow (maximum Q) for the existing culvert using Eqn 3-15:

Q = (0.0028)(0.16)(63 mm/h)(12.65 ha) = 0.36 m^3/s

d. Now calculate t_c for the parking lot alone:

$$t_c = \frac{1.8(1.1 - 0.70)[(3.28)(117.83)]^{1/2}}{1.70^{1/3}} = 14.6 \text{ min}$$

e. The intensity of rainfall that will cause flooding.

Since the t_c from the parking lot is substantially less than that for the pasture, the peak flows will not coincide and the controlling discharge will be for the shorter duration from the parking lot. Thus, ignoring the pasture, the intensity on the parking lot that will yield the peak discharge may be found by solving Eqn 3-15 for the intensity (i):

$$i = \frac{0.36\, m^3/s}{(0.0028)(0.70)(3.16\, ha)} = 58.1\, mm/h$$

f. Using the t_c for the parking lot and the intensity calculated in "e" and plotting the intersection of these two lines on Figure P-3-14, find, by interpolation, that the frequency of flooding is approximately once every 3/4 year.

Part II

The design flow for a culvert that can handle a 10 year storm runoff from the parking lot (again ignoring the pasture because the t_c for the parking lot is so much smaller than that for the pasture) may be found as follows:

a. From Figure P-3-14 with a t_c = 14.6 min and a 10 year recurrence interval, the intensity is found to be 127 mm/h.

b. The peak flow and, hence, the design flow is

$$Q = (0.0028)(0.70)(127 \text{ mm/h})(3.16 \text{ ha}) = 0.79 \text{ m}^3/\text{s}$$

3-21 Peak discharge from two adjacent parcels

Given: Figure labeled with variables

	Upstream Parcel	Downstream Parcel
A	3.0 ha	4.0 ha
C	0.35	0.30
D	193.5 m	100.0 m
S	1.50 %	4.40 %

Drainage ditch flows at 0.60 m/s
Drainage ditch is 200.0 m long
Seattle, WA IDF curves in Figure 3-10d

Solution:

a. Calculate runoff t_c for the west parcel

$$t_c = \frac{1.8(1.1-0.35)[(3.28)(193.5)]^{1/2}}{1.50^{1/3}} = 29.71 \text{ min}$$

b. Calculate runoff t_c for the east parcel

$$t_c = \frac{1.8(1.1-0.30)[(3.28)(100.0)]^{1/2}}{4.40^{1/3}} = 15.92 \text{ min}$$

c. Total t_c for each parcel

$$t_{c(west)} = t_c + \text{traveltime} = 29.71 + \left(\frac{200.0\text{m}}{0.60\,\text{m/s}}\right)\left(\frac{1}{60\,\text{s/min}}\right) = 29.71\text{min} + 5.55\text{min} = 35.26\text{min}$$

$t_{c(east)} = 15.92$ min

d. Therefore use 29.71 min as the maximum time of concentration and, hence, the duration of the rainfall. From the Seattle, WA IDF curve at 29.71 min (0.5 h) find i = 17.5 mm/h

e. Sum the CA values

$$\sum CA = (0.35)(3.0) + (0.30)(4.0) = 2.25$$

f. Calculate peak discharge

Q = 0.0028(17.5)(2.25)

Q = 0.11 m^3/s

3-22 Peak discharge from three adjacent parking lots

Given: Figure labeled with variables

	West	Center	East
A	8.0 ha	12.0 ha	6.0 ha
C	0.90	0.90	0.90
D	282.8 m	244.9 m	273.8 m
S	0.90 %	1.20 %	1.80 %

Storm sewer flows at 0.90 m/s. The sewer lengths are 250.0 m and 400.0 m. The 5 year storm at Miami, FL (Figure 3-10c) is to be used.

Solution:

a. Calculate the runoff t_c for the west lot

$$t_c = \frac{1.8(1.1-0.90)[(3.28)(282.8)]^{1/2}}{0.90^{1/3}} = \frac{10.96}{0.965} = 11.36\,\text{min}$$

b. Calculate the total t_c for the west lot

$$t_{c(\text{west})} = t_c + \text{traveltime} = 11.36\,\text{min} + 0.0\,\text{min} = 11.36\,\text{min}$$

c. Calculate the runoff t_c for the center lot

$$t_c = \frac{1.8(1.1-0.90)[(3.28)(244.9)]^{1/2}}{1.20^{1/3}} = \frac{10.203}{1.063} = 9.60\,\text{min}$$

d. Calculate the total t_c for the center lot

$$t_{c(\text{center})} = t_c + \text{traveltime} = 9.60\,\text{min} + \left(\frac{400\text{m}}{0.90\,\text{m/s}}\right)\left(\frac{1}{60\,\text{s/min}}\right) = 17.9\,\text{min}$$

e. Calculate the runoff t_c for the east lot

$$t_c = \frac{1.8(1.1-0.90)[(3.28)(273.8)]^{1/2}}{1.80^{1/3}} = \frac{10.788}{1.216} = 8.87\,\text{min}$$

f. Calculate the total t_c for the east lot

$$t_{c(\text{east})} = t_c + \text{traveltime} = 8.87\,\text{min} + \left(\frac{250.0\text{m}}{0.90\,\text{m/s}}\right)\left(\frac{1}{60\,\text{s/min}}\right) + 7.41\text{min} = 20.91\text{min}$$

Note that the 7.41 min is the travel time calculated in "d" above for the 400.0 m from M.H. in the center lot to the last M.H.

g. The largest total t_c is 20.91 min. This is the maximum time of concentration and, thus, is the duration of the rainfall. From the Miami, FL IDF curve find i = 120 mm/h for the five year storm at 20.91 min (0.35 h).

h. Sum the CA values

$$\sum CA = (0.90)(8.0) + (0.90)(12.0) + (0.90)(6.0) = 23.40$$

i. Calculate peak discharge

Q = 0.0028(23.40)(120)

Q = 7.9 m^3/s

3-23 Unit hydrograph for Isoceles River

Given: Basin area = 14.40 km^2; stream discharge graph

Solution:

The direct runoff ordinates at 1500, 1600 and 1700 hours are shown in Figure S-3-23. The volume may be computed by the method shown in the book or from the observation that the area under the curve is equal to the volume and, hence, is equal to 1/2 (base)(height) of the triangle.

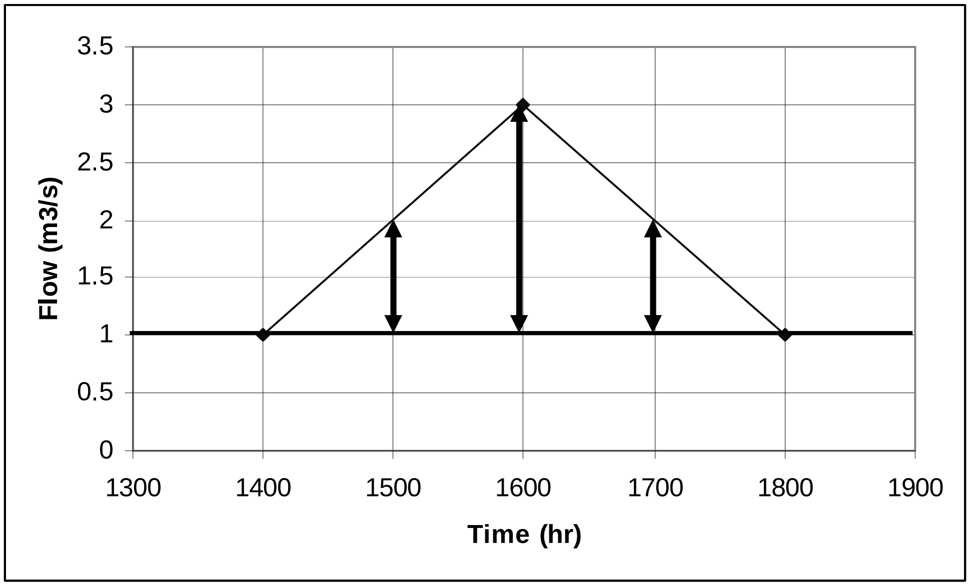


Figure S-3-23

Using the book method:

Time Interval (h)	Total Ord.	Base Ord.	DRH Ord.	Volume Increment (m^3)
1430 – 1530	2.0	1.0	1.0	3600
1530 – 1630	3.0	1.0	2.0	7200
1630 – 1730	2.0	1.0	1.0	3600
				SUM = 14400

Volumes were computed as in 3-25.

The unit depth as in 3-25 is

$$\frac{14400\text{m}^3}{(14.40\text{km}^2)(1\times10^6\ \text{m}^2/\text{km}^2)}\times100\,\text{cm/m} = 0.10\text{cm}$$

Compute U.H. Ord. as in 3-25

Plotting Time (h)	U.H. Ord. (m^3/s-cm)
1.0	10.0
2.0	20.0
3.0	10.0

3-24 Unit hydrograph for Convex River

Given: Area of watershed = 100.0 ha; Total stream flow ordinates

Solution:

a. The volume may be computed by the method shown in the book or from the observation that the area under the curve is equal to the volume and, hence, is equal to $(1/2)(\pi)(D^2/4)$ of the circle.

b. Using the book method

Time Interval (h)	Total Ord. (m^3/s)	Base Ord. (m^3/s)	DRH Ord. (m^3/s)	Volume Increment (m^3)
2030 – 2130	3.0	1.5	1.5	5400
2130 – 2230	3.8	1.5	2.3	8280
2230 – 2330	4.0	1.5	2.5	9000
2330 – 0030	3.8	1.5	2.3	8280
0030 – 0130	3.0	1.5	1.5	5400
				SUM = 36360

Total ordinates were provided in the problem statement. Baseline ordinates are read from the extrapolated baseline (a horizontal line).

The volume increment is calculated as:

$$\not{V} = (\text{DRH})(\text{time interval})(3600 \text{ s/h})$$

For example:

$$\not{V} = (1.50 \text{ m}^3/\text{s})(1\text{h})(3600 \text{ s/h}) = 5400 \text{ m}^3$$

c. Determine the unit depth

$$\frac{36360\text{m}^3}{(100\text{ha})(1\times10^4 \text{ m}^2/\text{ha})}\times 100 \text{ cm/m} = 3.64\text{cm}$$

d. Compute the U.H. ordinates

1 h $\quad \dfrac{1.5\,m^3/s}{3.64cm} = 0.41\,m \ /s - cm$

2h $\quad \dfrac{2.3\,m^3/s}{3.64cm} = 0.63\,m^3/s - cm$

3h $\quad \dfrac{2.5\,m^3/s}{3.64cm} = 0.69\,m^3/s - cm$

4h $\quad \equiv$ 2h = 0.63 m^3/s-cm

5h $\quad \equiv$ 1h = 0.41 m^3/s-cm

3-25 Unit hydrograph for Verde River

Given: Basin area = 64.0 km^2; Stream flow data for 5 h storm

Solution: Begin by plotting stream flow data as in Figure S-3-25 on following page.

a. Plot base flow as straight line extrapolation from A to B.

b. Beginning of U.H. is at beginning of DH. Arbitrarily select time intervals as shown and find ordinates from Figure S-3-25.

Time Interval (h)	Total Ord. (m^3/s)	Base Ord. (m^3/s)	DRH Ord. (m^3/s)	Volume Increment (m^3)
10 to 15	1	0.4	0.6	10,800
15 to 20	3.4	0.4	3	54,000
20 to 30	6.24	0.4	5.84	210,240
30 to 40	5.77	0.4	5.37	193,320
40 to 50	4.29	0.4	3.89	140,040
50 to 60	2.72	0.38	2.34	84,240
60 to 70	1.64	0.38	1.26	45,360
70 to 80	0.79	0.3	0.49	17,640
80 to 90	0.25	0.25	0	0
		Total Volume =		755,640

Total ordinate and base ordinate are read at midpoint of time interval. DRH = (Total Ord.) - (Base Ord.). Volume increment is calculated as

~~V~~ = (DRH)(time interval)(3600 s/h)

For example 10-15 h

~~V~~ = (0.60 m^3/s)(5 h)(3600 s/h) = 10800 m^3

Determine the unit depth

$$\frac{755640}{\left(64\text{km}^2\right)\left(1\times10^6\ \text{m}^2/\text{km}^2\right)}\times100\,\text{cm/m}=1.18\text{cm}$$

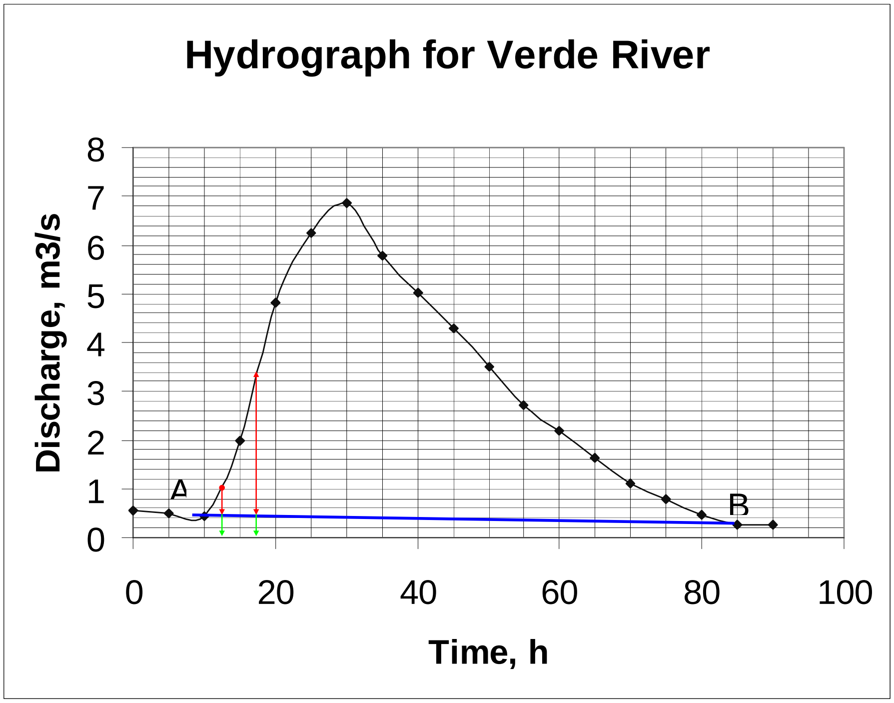


Figure S-3-25 Hydrograph for Verde River

Compute U.H. ordinates

$$\text{U.H.Ord}=\frac{\text{DRH.Ord}}{\text{UnitDepth}}=\frac{0.6}{1.18}=0.51$$

Time Interval (h)	Plotting Time (h)	U.H. Ordinate (m^3/s-cm)
0 to 5	2.5	0.51
5 to 10	7.5	2.54
10 to 20	15	4.95
20 to 30	25	4.55
30 to 40	35	3.29
40 to 50	45	1.98
50 to 60	55	1.07
60 to 70	65	0.42

Plotting time is time from beginning of precipitation excess. In essence new time zero is established at point A in Figure S-3-25.

3-26 Unit hydrograph for Crimson River

Given: Basin area = 626 km^2; Stream flow data for 5 h storm

Solution: As in Problem 3-25 begin with plot. See Figure S-3-26.

Construct base flow line as in Problem 3-25. Determine direct runoff volume as in 3-25:

Time Interval (h)	Total Ord. (m^3/s)	Base Ord. (m^3/s)	DRH Ord. (m^3/s)	Volume Increment (m^3)
15 to 20	3.5	1.4	2.1	37,800
20 to 30	15.14	1.3	13.84	498,240
30 to 40	21.55	1.2	20.35	732,600
40 to 50	15.77	1.1	14.67	528,120
50 to 60	11.03	1	10.03	361,080
60 to 70	6.88	1	5.88	211,680
70 to 80	3.46	0.9	2.56	92,160
80 to 90	1.48	0.8	0.68	24,480
90 to 100	0.77	0.77	0	0
		Total Volume =		2,486,160

Total ordinate and base ordinate are read at midpoint of time interval. DRH = (Total Ord.) - (Base Ord.). Volume increment is calculated as

$$\forall = (\text{DRH})(\text{time interval})(3600 \text{ s/h})$$

For example 15-20 h

$$\forall = (2.1 \text{ m}^3/\text{s})(5 \text{ h})(3600 \text{ s/h}) = 37800 \text{ m}^3$$

Determine unit depth as in 3-25

$$\frac{2486160}{(626\text{km}^2)(1\times10^6 \text{ m}^2/\text{km}^2)} \times 100 \text{ cm/m} = 0.40\text{cm}$$

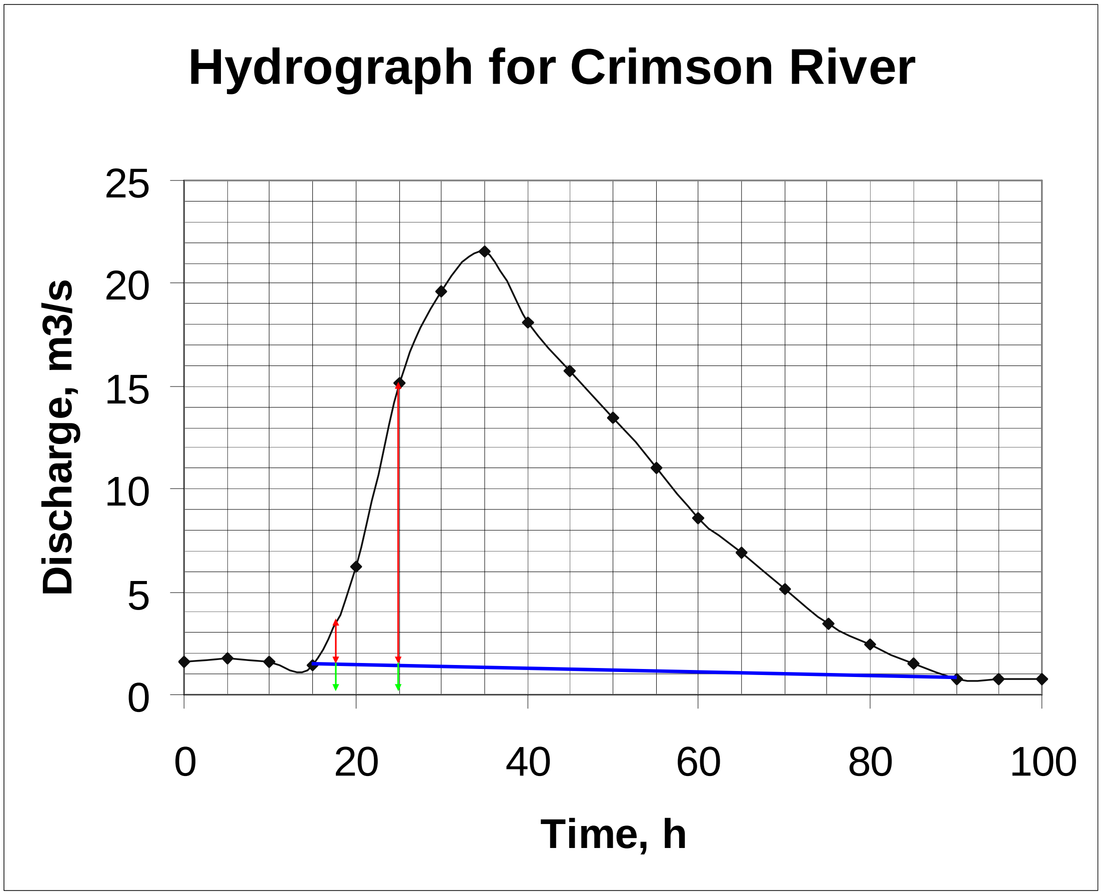


Figure S-3-26 Hydrograph for Crimson River

Compute U.H. Ord.

Time Interval (h)	Plotting Time (h)	U.H. Ordinate (m^3/s-cm)
0 to 5	2.5	5.29
5 to 15	10	34.85
15 to 25	20	51.24
25 to 35	30	36.94
35 to 45	40	25.25
45 to 55	50	14.81
55 to 65	60	6.45
65 to 75	70	1.71
75 to 85	80	0.00
85 to 90	90	0.00

3-27 Applying U.H. Ordinates to a storm sequence

Given: U.H. Ordinates and storm sequence

Solution:

Day	Rainfall Excess (cm)	DRH Ord. (m^3/s)				Compound Runoff (m^3/s)
		1	2	3	4	
1	0.30	0.04	N/A-1	N/A-2	N/A-2	0.04
2	0.20	0.23	0.02	N/A-2	N/A-2	0.25
3	0.0	0.04	0.15	N/A-2	N/A-2	0.19
4	0.0	0.0	0.03	N/A-2	N/A-2	0.03

Rainfall Excess = Precipitation - Abstractions

For day 1, R.E. = 0.50 – 0.20 = 0.30

N/A-1: Rain that falls in day 2 cannot appear in the stream the day before it rains.

N/A-2: No rain falls in days 3 and 4 so there cannot be any runoff.

3-28 Compound runoff hydrograph for Isoceles River

Given: Rainfall excess for 1st hour = 0.1 cm; for 2nd hour R.E. = 0.20 cm; for 3rd hour R.E. = 0.05 cm; U.H. ordinates from Prob. 3-23.

Solution:

Time	R.E. (cm)	DRH Ordinates			Compound Runoff (m^3/s)
		1	2	3	
1	0.10	1.0	N/A	N/A	1.0
2	0.20	2.0	2.0	N/A	4.0
3	0.05	1.0	4.0	0.5	5.5
4	0.0	0.0	2.0	1.0	3.0
5	0.0	0.0	0.0	0.5	0.5

3-29 Compound runoff hydrograph for Verde River (Problem 3-25)

Given: Rainfall excess of 15 mm/h for 5 h from 1500 to 2000 and 10 mm/h for 5 h from 0500 to 1000 and unit hydrograph ordinates from problem 3-25

Solution: R.E. = (15 mm/h)(5 h)/(10 mm/cm) = 7.50 cm.

Note: 1500 h = 0 h for computations.

Time Interval (h)	Plotting Time (h)	UH Ord. (m^3/s-cm)	Rainfall Excess (cm)	DRH Ordinates		Compound Runoff (m^3/s)	
				1	2		
0 to 5	2.5	0.51	7.5	3.825	n/a	3.825	
5 to 10	7.5	2.54		19.05	n/a	19.05	
10 to 20	15	4.95		37.125	n/a	37.125	
20 to 30	25	4.55	5	34.125	2.55	36.675	See Note
30 to 40	35	3.29		24.675	12.7	37.375	See Note
40 to 50	45	1.98		14.85	24.75	39.6	
50 to 60	55	1.07		8.025	22.75	30.775	
60 to 70	65	0.42		3.15	16.45	19.6	
70 to 80	75				9.9	9.9	
80 to 90	85				5.35	5.35	
90 to 100	95				2.1	2.1	

NOTE: The plotting time for these two points must be the same time after the start of precipitation excess as the U.H. times, I.e. 2.5 and 7.5 h. Hence, the compound runoff will not be the sum as shown.

3-30 Compound runoff hydrograph for Crimson River

Given: Rainfall excess = 25 mm/h for 5 h and 15 mm/h for 5 h

Solution:

R.E. = (25 mm/h)(5 h)/(10 mm/cm) = 12.50 cm
R.E. = (15 mm/h)(5 h)/(10 mm/cm) = 7.50 cm

Note: 1200 h = 0 h for computations and plot

Time Interval (h)	Plotting Time (h)	UH Ord. (m^3/s-cm)		Rainfall Excess (cm)	DRH Ordinates 1	DRH Ordinates 2	Interpolation*	Compound Runoff (m^3/s)
0 to 5	2.5	5.29		12.5	66.13	n/a		66.13
	7.5		5.29	7.5		39.68		
5 to 15	10	34.85			435.63		150.53	586.15
	15		34.85			261.38		
15 to 25	20	51.24			640.50		322.84	963.34
	25		51.24			384.30		
25 to 35	30	36.94			461.75		330.68	792.43
	35		36.94			277.05		
35 to 45	40	25.25			315.63		233.21	548.84
	45		25.25			189.38		
45 to 55	50	14.81			185.13		150.23	335.35
	55		14.81			111.08		
55 to 65	60	6.45			80.63		79.73	160.35
	65		6.45			48.38		
65 to 75	70	1.71			21.38		30.60	51.98
	75		1.71			12.83		
75 to 85	80						6.41	6.41

*Because of the odd offset (2.5 h) the DRH ordinates for the first and second storms do not plot at the same time. I have linearly interpolated values from the second strom to determine runoff at the same plotting time as the first storm. The value is used to compute compound runoff.

3-31 Reservoir volume for droughts

Given: Design volume = 7.00×10^6
Inflow = 3.2 m^3/s
Outflow = 2.0 m^3/s

Solution:

a. Write the mass balance equation in terms of volumes

$$\not{V} = (Q_{in})(t) - (Q_{out})(t)$$

b. Solve for t

$$t = \frac{\not{V}}{Q_{in} - Q_{out}} = \left(\frac{7.00 \times 10^6}{3.2\,m^3/s - 2.0\,m^3/s}\right)\left(\frac{1}{86400\,s/d}\right) = 67.5d$$

3-32 Woebegone water tower

Given: Estimeated demand cylce
Pump capacity = 36 L/s

Solution:

a.This is a mass balance problem. Set up tabular form as shown below.

Time	Q_{in} (L/s)	$\not{V}_{in}$ (L)	Q_{out} (L/s)	$\not{V}_{out}$ (L)	ΔS	ΣΔs
12am – 6am	36	777600*	0.0	0.0	777600	777600
6am – 12noon	36	777600	54.0**	1166400	-388800	388800
12noon – 6pm	36	777600	54.0	1166400	-388800	0.0
6pm – 12midnight	36	777600	36.0	777600	0.0	0.0

*(36 L/s)(6 h)(3600 s/h) = 777600
**(54 L/s)(6 h)(3600 s/h) = 1166400

b. Volume of water tower required is 777600 L

3-33 Water supply from Clear Fork Trinity River

Given: Table of mean monthly discharge; Demand = 0.35 m^3/s

Solution:

a. Mass balance by spreadsheet

Year	Month	Q_{in} (m^3/s)	Q_{in}*Δt (m^3)	Q_{out} (m^3/s)	Q_{out}*Δt (m^3)	ΔS (m^3)	ΣΔS (m^3)	
1951	Jul	0.98	2,624,832	0.35	937,440	1,687,392	0	
	Aug	0	0	0.35	937,440	-937,440	-937,440	
	Sep	0	0	0.35	907,200	-907,200	-1,844,640	
	Oct	0	0	0.35	937,440	-937,440	-2,782,080	
	Nov	0.006	15,552	0.35	907,200	-891,648	-3,673,728	
	Dec	0.09	241,056	0.35	937,440	-696,384	-4,370,112	
1952	Jan	0.175	468,720	0.35	937,440	-468,720	-4,838,832	<=
	Feb	0.413	999,130	0.35	846,720	152,410	-4,686,422	
	Mar	0.297	795,485	0.35	937,440	-141,955	-4,828,378	
	Apr	1.93	5,002,560	0.35	907,200	4,095,360	-733,018	
	May	3.65	9,776,160	0.35	937,440	8,838,720	8,105,702	<= Reservoir is full and overflows excess

b. Volume required is 4838832 m^3

c. Reservoir is full and overflows excess at end of May.

Year	Month	Q_{in} (m^3/s)	Q_{in}*Δt (m^3)	Q_{out} (m^3/s)	Q_{out}*Δt (m^3)	ΔS (m^3)	ΣΔS (m^3)
1964	Jan	3.77	10,097,568	1.76	4,713,984	5,383,584	0
	Feb	2.57	6,217,344	1.76	4,257,792	1,959,552	0
	Mar	7.33	19,632,672	1.76	4,713,984	14,918,688	0
	Apr	6.57	17,029,440	1.76	4,561,920	12,467,520	0
	May	1.85	4,955,040	1.76	4,713,984	241,056	0
	Jun	0.59	1,529,280	1.76	4,713,984	-3,184,704	-3,184,704
	Jul	0.38	1,017,792	1.76	4,713,984	-3,696,192	-6,880,896
	Aug	0.25	669,600	1.76	4,713,984	-4,044,384	-10,925,280
	Sep	0.21	544,320	1.76	4,561,920	-4,017,600	-14,942,880
	Oct	0.27	723,168	1.76	4,713,984	-3,990,816	-18,933,696
	Nov	0.36	933,120	1.76	4,561,920	-3,628,800	-22,562,496
	Dec	0.79	2,115,936	1.76	4,713,984	-2,598,048	-25,160,544
1965	Jan	0.65	1,740,960	1.76	4,713,984	-2,973,024	-28,133,568
	Feb	1.33	3,217,536	1.76	4,257,792	-1,040,256	-29,173,824
	Mar	2.38	6,374,592	1.76	4,713,984	1,660,608	-27,513,216
	Apr	3.79	9,823,680	1.76	4,561,920	5,261,760	-22,251,456
	May	1.47	3,937,248	1.76	4,713,984	-776,736	-23,028,192
	Jun	0.59	1,529,280	1.76	4,713,984	-3,184,704	-26,212,896
	Jul	0.23	616,032	1.76	4,713,984	-4,097,952	-30,310,848
	Aug	0.2	535,680	1.76	4,713,984	-4,178,304	-34,489,152
	Sep	0.19	492,480	1.76	4,561,920	-4,069,440	-38,558,592
	Oct	0.27	723,168	1.76	4,713,984	-3,990,816	-42,549,408
	Nov	0.45	1,166,400	1.76	4,561,920	-3,395,520	-45,944,928
	Dec	0.64	1,714,176	1.76	4,713,984	-2,999,808	-48,944,736
1966	Jan	0.61	1,633,824	1.76	4,713,984	-3,080,160	-52,024,896
	Feb	1.96	4,741,632	1.76	4,257,792	483,840	-51,541,056
	Mar	5.55	14,865,120	1.76	4,713,984	10,151,136	-41,389,920
	Apr	2.92	7,568,640	1.76	4,561,920	3,006,720	-38,383,200
	May	2.46	6,588,864	1.76	4,713,984	1,874,880	-36,508,320
	Jun	0.8	2,073,600	1.76	4,713,984	-2,640,384	-39,148,704
	Jul	0.26	696,384	1.76	4,713,984	-4,017,600	-43,166,304
	Aug	0.18	482,112	1.76	4,713,984	-4,231,872	-47,398,176
	Sep	0.27	699,840	1.76	4,561,920	-3,862,080	-51,260,256
	Oct	0.52	1,392,768	1.76	4,713,984	-3,321,216	-54,581,472
	Nov	1.75	4,536,000	1.76	4,561,920	-25,920	-54,607,392
	Dec	1.35	3,615,840	1.76	4,713,984	-1,098,144	-55,705,536

3-34 Water supply for Squannacook River

Given: Table of mean monthly discharge; Demand = 0.60 m^3/s

Solution:

a. Mass balance by spreadsheet

1967	Jan	1.68	4,499,712	1.76	4,713,984	-214,272	-55,919,808	
	Feb	1.53	3,701,376	1.76	4,257,792	-556,416	-56,476,224	<=
	Mar	2.64	7,070,976	1.76	4,713,984	2,356,992	-54,119,232	
	Apr	10.62	27,527,040	1.76	4,561,920	22,965,120	-31,154,112	
	May	6.29	16,847,136	1.76	4,713,984	12,133,152	-19,020,960	
	Jun	3.17	8,216,640	1.76	4,713,984	3,502,656	-15,518,304	
	Jul	2.22	5,946,048	1.76	4,713,984	1,232,064	-14,286,240	
	Aug	0.72	1,928,448	1.76	4,713,984	-2,785,536	-17,071,776	
	Sep	0.47	1,218,240	1.76	4,561,920	-3,343,680	-20,415,456	
	Oct	0.6	1,607,040	1.76	4,713,984	-3,106,944	-23,522,400	
	Nov	1.07	2,773,440	1.76	4,561,920	-1,788,480	-25,310,880	
	Dec	3.03	8,115,552	1.76	4,713,984	3,401,568	-21,909,312	<= Reservoir is not full yet

b. Volume required is 56476224 m^3

c. The reservoir is not full at the end of December 1967.

3-35 Water supply from the Hoko River

Given: Table of mean monthly discharge; Demand = 0.325 m^3/s

Solution:

a. Mass balance by spreadsheet

Note: Flow restriction implies

$$\frac{0.325\,\mathrm{m}^3/\mathrm{s}}{Q_{in}} = 0.10$$

$$Q_{in} = \frac{0.325\,\mathrm{m}^3/\mathrm{s}}{0.10} = 3.25\,\mathrm{m}^3/\mathrm{s}$$

Year	Month	Q_{in} (m^3/s)	$Q_{in}*\Delta t$ (m^3)	Q_{out} (m^3/s)	$Q_{out}*\Delta t$ (m^3)	ΔS (m^3)	$\Sigma\Delta S$ (m^3)	
1972	May	3	8,035,200	3.25	8,704,800	-669,600	-669,600	
	Jun	1	2,592,000	3.25	8,704,800	-6,112,800	-6,782,400	
	Jul	5.32	14,249,088	3.25	8,704,800	5,544,288	-1,238,112	
	Aug	0.841	2,252,534	3.25	8,704,800	-6,452,266	-7,690,378	
	Sep	2	5,184,000	3.25	8,424,000	-3,240,000	-10,930,378	
	Oct	1.14	3,053,376	3.25	8,704,800	-5,651,424	-16,581,802	<=
	Nov	11.8	30,585,600	3.25	8,424,000	22,161,600	5,579,798	<= reservoir is full

b. Volume or reservoir is 16,581,802 m^3

3-36 Bar Nunn retention pond

Given: Tabulation of inflow and outflow; Each interval is 1 h

Solution:

a. Using mass balance technique complete the table.

Interval	Q_{in} (L/s)	V_{in} (m^3)	Q_{out} (L/s)	V_{out} (m^3)	ΔV (m^3)	$\Sigma\Delta V$ (m3)
1	10.0	36.0*	10.0	36.0	0	0
2	20.0	72.0	10.0	36.0	36.0	36.0
3	30.0	108.0	10.0	36.0	72.0	108.0
4	20.0	72.0	10.0	36.0	36.0	144.0
5	15.0	54.0	10.0	36.0	18.0	162.0
6	5.0	18.0	10.0	36.0	Outflow exceeds inflow	

*Example calculation
$(10.0 \text{ L/s})(1 \text{ h})(3600 \text{ s/h})(1 \times 10^{-3} \text{ m}^3/\text{L}) = 36.0 \text{ m}^3$

b. The maximum V is 162.0 m^3. Therefore, the volume of the retention basin should be 162.0 m^3.

3-37 Menominee River flood storage

Given: Table of mean monthly discharges January 1, 1959 – December 31, 1960.
Flood stage is at 100 m^3/s
Downstream discharge is 100 m^3/s

Solution:
a. Mass balance by spreadsheet (following page)

Year	Month	Q_{in} (m^3/s)	Q_{in}*Δt (m^3)	Q_{out} (m^3/s)	Q_{out}*Δt (m^3)	ΔS (m^3)	ΣΔS (m^3)	
1959	Jan	46.7	125,081,280	100	267,840,000	-142,758,720	0	
	Feb	43.1	104,267,520	100	241,920,000	-137,652,480	0	
	Mar	55	147,312,000	100	267,840,000	-120,528,000	0	
	Apr	110	285,120,000	100	259,200,000	25,920,000	25,920,000	
	May	105	281,232,000	100	267,840,000	13,392,000	39,312,000	
	Jun	56.7	146,966,400	100	259,200,000	-112,233,600	0	<= Reservoir completely empty at end of JUN
	Jul	48.3	129,366,720	100	267,840,000	-138,473,280	0	
	Aug	78	208,915,200	100	267,840,000	-58,924,800	0	
	Sep	142	368,064,000	100	259,200,000	108,864,000	108,864,000	
	Oct	155	415,152,000	100	267,840,000	147,312,000	256,176,000	
	Nov	122	316,224,000	100	259,200,000	57,024,000	313,200,000	
	Dec	78.2	209,450,880	100	267,840,000	-58,389,120	254,810,880	See note
1960	Jan	82.3	220,432,320	100	267,840,000	-47,407,680	207,403,200	See note
	Feb	71	171,763,200	100	241,920,000	-70,156,800	137,246,400	See note
	Mar	62.4	167,132,160	100	267,840,000	-100,707,840	36,538,560	See note
	Apr	242	627,264,000	100	259,200,000	368,064,000	404,602,560	
	May	373	999,043,200	100	267,840,000	731,203,200	1,135,805,760	
	Jun	135	349,920,000	100	259,200,000	90,720,000	1,226,525,760	<= max. vol
	Jul	83.4	223,378,560	100	267,840,000	-44,461,440	1,182,064,320	
	Aug	72.1	193,112,640	100	267,840,000	-74,727,360	1,107,336,960	
	Sep	80.8	209,433,600	100	259,200,000	-49,766,400	1,057,570,560	
	Oct	60.5	162,043,200	100	267,840,000	-105,796,800	951,773,760	
	Nov	102	264,384,000	100	259,200,000	5,184,000	956,957,760	
	Dec	68.2	182,666,880	100	267,840,000	-85,173,120	871,784,640	<= Reservoir not empty at end of DEC

NOTE: Although Q_{in} is less than 100 m^3, the reservoir is not empty at the end of DEC (or JAN, FEB, MAR).

b. Volume required is 1,226,525,760 m^3

3-38 Spokane River flood storage

Given: Table of mean monthly discharges for March 1957 through October 1958
Flood stage is ≥ 250 m^3/s
Discharge is 250 m^3/s for each flood

Solution:

a. Mass balance by spreadsheet

Year	Month	Q_{in} (m^3/s)	Q_{in}*Δt (m^3)	Q_{out} (m^3/s)	Q_{out}*Δt (m^3)	ΔS (m^3)	ΣΔS (m^3)	
1957	Jan	99	265,161,600	250	669,600,000	-404,438,400	0	
	Feb	61	147,571,200	250	604,800,000	-457,228,800	0	
	Mar	278	744,595,200	250	669,600,000	74,995,200	74,995,200	
	Apr	461	1,194,912,000	250	648,000,000	546,912,000	621,907,200	
	May	792	2,121,292,800	250	669,600,000	1,451,692,800	2,073,600,000	
	Jun	329	852,768,000	250	648,000,000	204,768,000	2,278,368,000	<= max. vol
	Jul	33.6	89,994,240	250	669,600,000	-579,605,760	1,698,762,240	
	Aug	12.5	33,480,000	250	669,600,000	-636,120,000	1,062,642,240	
	Sep	15.7	40,694,400	250	648,000,000	-607,305,600	455,336,640	
	Oct	55.5	148,651,200	250	669,600,000	-520,948,800	0	<= Reservoir completely empty by end of OCT
	Nov	66.9	173,404,800	250	648,000,000	-474,595,200	0	
	Dec	73	195,523,200	250	669,600,000	-474,076,800	0	
1958	Jan	80	214,272,000	250	669,600,000	-455,328,000	0	
	Feb	245	592,704,000	250	604,800,000	-12,096,000	0	
	Mar	234	626,745,600	250	669,600,000	-42,854,400	0	
	Apr	408	1,057,536,000	250	648,000,000	409,536,000	409,536,000	
	May	548	1,467,763,200	250	669,600,000	798,163,200	1,207,699,200	
	Jun	152	393,984,000	250	648,000,000	-254,016,000	953,683,200	
	Jul	29.5	79,012,800	250	669,600,000	-590,587,200	363,096,000	
	Aug	4.5	12,052,800	250	669,600,000	-657,547,200	0	<= Reservoir not empty at end of AUG
	Sep	24.4	63,244,800	250	648,000,000	-584,755,200	0	
	Oct	36.8	98,565,120	250	669,600,000	-571,034,880	0	
	Nov	153	396,576,000	250	648,000,000	-251,424,000	0	
	Dec	240	642,816,000	250	669,600,000	-26,784,000	0	

b. Storage required = 2.28×10^9 m^3

c. The reservoir is not empty.

3-39 Rappahannuck River flood storage

Given: Table of mean monthly discharge
Flood stage is 5.8 m^3/s
Downstream discharge is constant at average flow for the period

Solution:

a. Mass balance by spreadsheet

Year	Month	Q_{in} (m^3/s)	Q_{in}*Δt (m^3)	Q_{out} (m^3/s)	Q_{out}*Δt (m^3)	ΔS (m^3)	ΣΔS (m^3)	
1960	Jan	4.11	11,008,224	5.8	15,534,720	-4,526,496	0	
	Feb	9.71	23,490,432	5.8	14,031,360	9,459,072	9,459,072	
	Mar	7.7	20,623,680	5.8	15,534,720	5,088,960	14,548,032	
	Apr	13.3	34,473,600	5.8	15,033,600	19,440,000	33,988,032	
	May	11.3	30,265,920	5.8	15,534,720	14,731,200	48,719,232	
	Jun	9.97	25,842,240	5.8	15,033,600	10,808,640	59,527,872	
	Jul	2.97	7,954,848	5.8	15,534,720	-7,579,872	51,948,000	
	Aug	1.85	4,955,040	5.8	15,534,720	-10,579,680	41,368,320	
	Sep	2.77	7,179,840	5.8	15,033,600	-7,853,760	33,514,560	
	Oct	1.1	2,946,240	5.8	15,534,720	-12,588,480	20,926,080	
	Nov	1.23	3,188,160	5.8	15,033,600	-11,845,440	9,080,640	
	Dec	1.31	3,508,704	5.8	15,534,720	-12,026,016	0	<= Reservoir completely empty by end of DEC
1961	Jan	3.31	8,865,504	5.8	15,534,720	-6,669,216	0	
	Feb	15.4	37,255,680	5.8	14,031,360	23,224,320	23,224,320	
	Mar	9.85	26,382,240	5.8	15,534,720	10,847,520	34,071,840	
	Apr	15.5	40,176,000	5.8	15,033,600	25,142,400	59,214,240	
	May	11.1	29,730,240	5.8	15,534,720	14,195,520	73,409,760	
	Jun	6.82	17,677,440	5.8	15,033,600	2,643,840	76,053,600	
	Jul	3.23	8,651,232	5.8	15,534,720	-6,883,488	69,170,112	<= max. vol
	Aug	2.24	5,999,616	5.8	15,534,720	-9,535,104	59,635,008	
	Sep	1.7	4,406,400	5.8	15,033,600	-10,627,200	49,007,808	
	Oct	1.16	3,106,944	5.8	15,534,720	-12,427,776	36,580,032	
	Nov	1.77	4,587,840	5.8	15,033,600	-10,445,760	26,134,272	
	Dec	4.25	11,383,200	5.8	15,534,720	-4,151,520	21,982,752	
1962	Jan	5.44	14,570,496	5.8	15,534,720	-964,224	21,018,528	
	Feb	5.61	13,571,712	5.8	14,031,360	-459,648	20,558,880	
	Mar	16.8	44,997,120	5.8	15,534,720	29,462,400	50,021,280	
	Apr	10.7	27,734,400	5.8	15,033,600	12,700,800	62,722,080	
	May	5.27	14,115,168	5.8	15,534,720	-1,419,552	61,302,528	
	Jun	6.88	17,832,960	5.8	15,033,600	2,799,360	64,101,888	
	Jul	3.57	9,561,888	5.8	15,534,720	-5,972,832	58,129,056	
	Aug	1.51	4,044,384	5.8	15,534,720	-11,490,336	46,638,720	
	Sep	0.855	2,216,160	5.8	15,033,600	-12,817,440	33,821,280	
	Oct	0.932	2,496,269	5.8	15,534,720	-13,038,451	20,782,829	
	Nov	4.73	12,260,160	5.8	15,033,600	-2,773,440	18,009,389	
	Dec	3.6	9,642,240	5.8	15,534,720	-5,892,480	12,116,909	

b. Storage required = 6.92×10^7 m^3

3-40 Hydraulic gradient size and direction

Given: A 100 ha square; total piezometric head at each corner.

Solution:

a. Determine the dimensions of the square by converting to m^2 and taking square root.

Area = (100 ha)(10^4 m^2/ha) = 1.0 x 10^6 m^2

Distance between wells = length of side = (1.0 x 10^6 m^2)$^{0.5}$ = 1,000 m

b. The hydraulic gradient is exactly from west to east because both western total heads and the eastern total heads are the same. There is no need to plot the points. Either the NE/NW line or the SE/ SW line can be used to calculate the magnitude of the hydraulic gradient:

$$\text{HydraulicGradient} = \frac{30.6 - 30}{1000} = 6.0 \times 10^{-4}$$

3-41 Hydraulic gradient size and direction

Given: Same wells as in Problem 3-40; water levels measured from ground surface.

Solution:

a. Distances are the same as Problem 3-40.

b. Note that water elevations are measured from the surface, so the gradient is from the east to the west. The NW corner is 0.4 below the NE and SE corners and the SW corner is 0.6 m below the NE and SE corners.

c. The flow pattern is a little more complex. Since the total head at the NE corner is the same as that of the SE corner, a point midway between the two corners is assumed to have the same total head. Begin the construction with the point midway between the NE and SE corners and the other two corners as shown below.

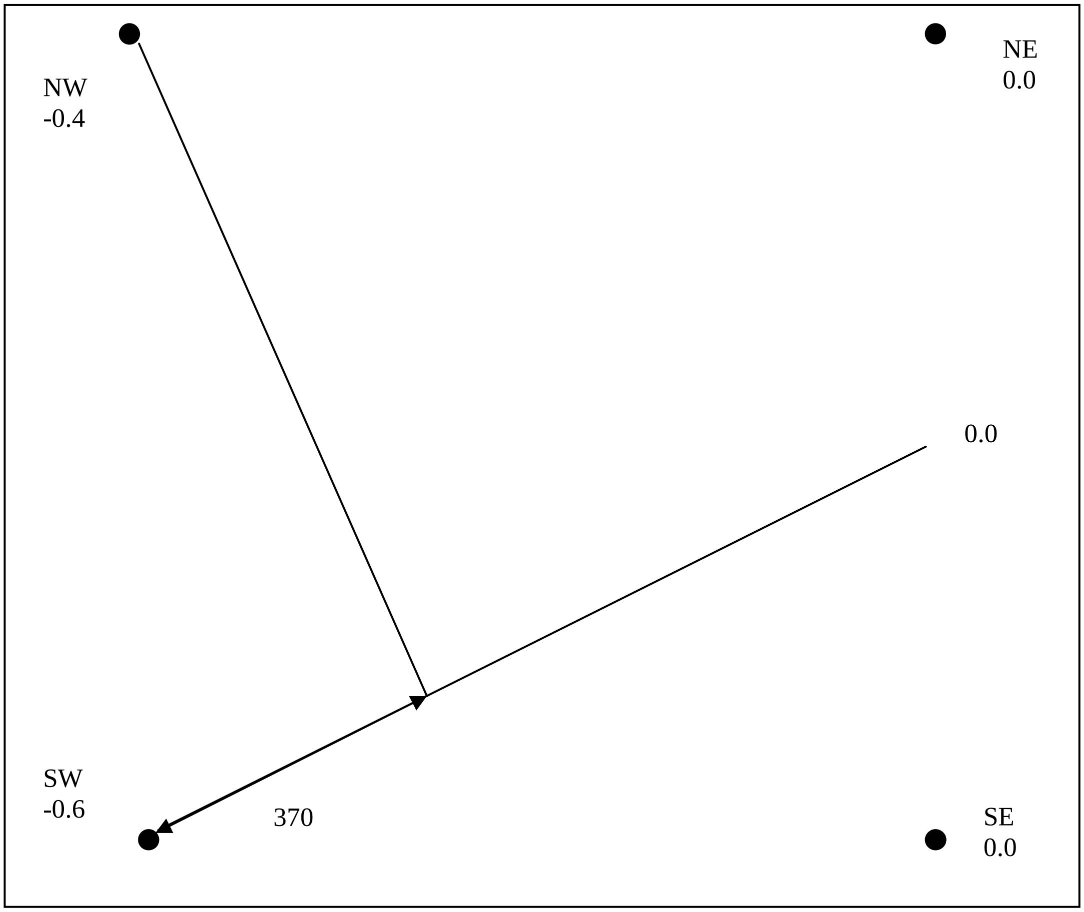


Figure S-3-41

d. Graphically determine the distance from A to B to be 370 m and calculate the hydraulic gradient as:

$$\text{HydraulicGradient} = \frac{0.6 - 0.4}{370} = 5.4 \times 10^{-4}$$

3-42 Hydraulic gradient for modeling study

Given: Well grid; ground surface elevation; and depth to ground water table in each well.

Solution:

a. Note that water elevations are measured from the surface

b. Plot the wells and elevations as shown below

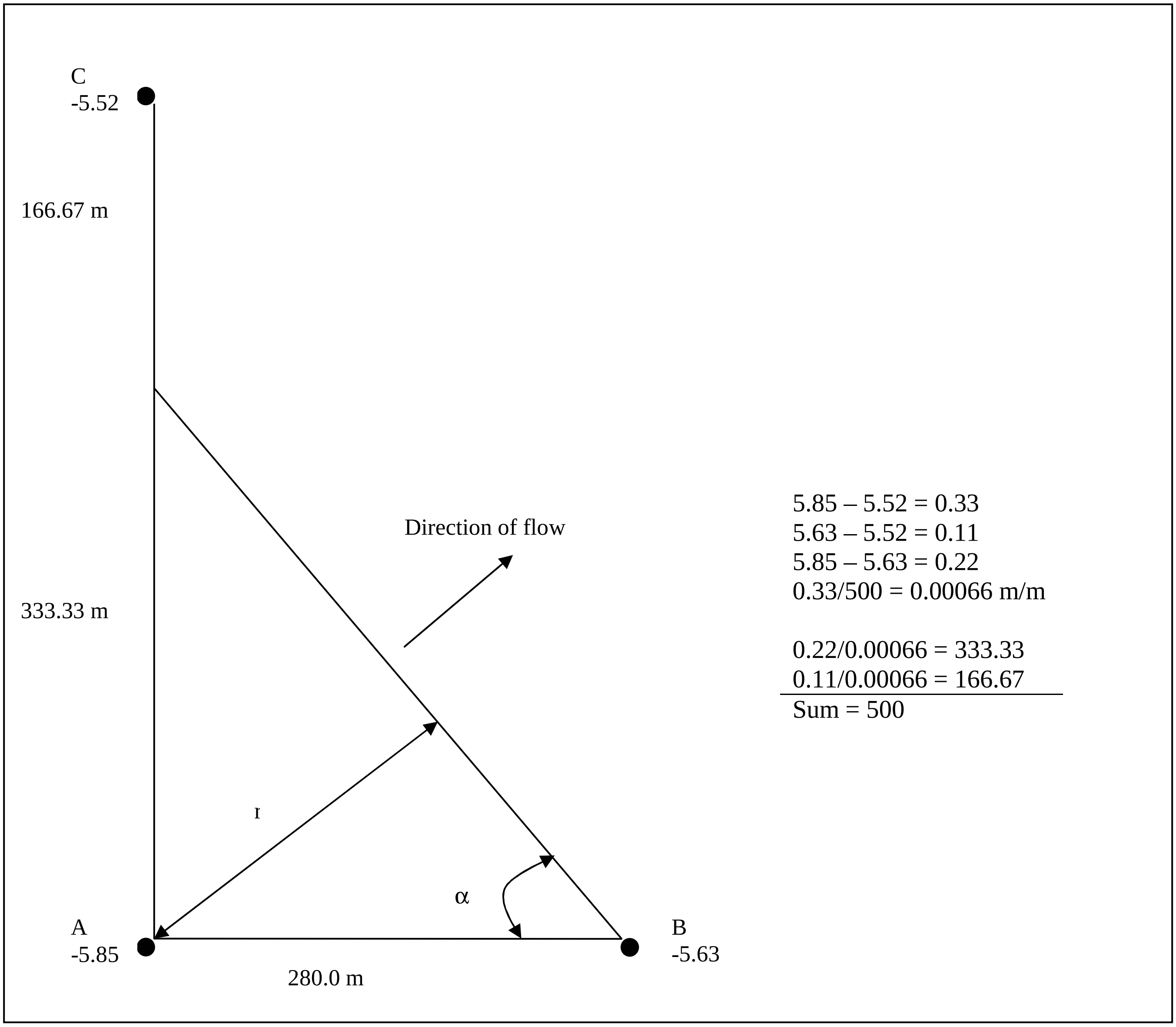


Figure S-3-42

c. Follow directions given under "Definition of Terms" in Section 3-5. The graphical construction is shown on the next page.

The calculation of the pont on $\overline{AC}$ with head equal to B is as follows:

$$\frac{5.85-5.52}{500\text{m}}=\frac{0.33}{500}=0.00066$$

$$\frac{5.63-5.52}{0.00066}=\frac{0.11}{0.00066}=166.67\text{m}$$

Measure 166.67 m from point C to point of equal head.
500 – 166.67 = 333.33

d. Calculate the distance

$$\tan^{-1}(\alpha)=\frac{333.33}{280.0}$$
$\alpha = 49.97^\circ$

The distance is then r = 280.0*sin(α) = 280.0*sin(49.97) = 214.396 m

$$\text{HydraulicGradient} = \frac{5.85-5.63}{214.396} = 0.00103$$

3-43 Darcy velocity

Given: hydraulic conductivity = 6.9 x 10^{-4}m/s; hydraulic gradient = 0.00141; porosity = 0.20

Solution:

a. The Darcy velocity is given by Equation 3-21

$v = (6.9 \times 10^{-4}\ \text{m/s})(0.00141) = 9.73 \times 10^{-7}\ \text{m/s}$

b. The average linear velocity is given by Equation 3-25

$$v' = \frac{9.73\times10^{-7}\ \text{m/s}}{0.20} = 4.86\times10^{-6}\ \text{m/s}$$

3-44 Darcy velocity for fine sand

Given: hydraulic conductivity = 3.5 x 10^{-5} m/s; hydraulic gradient = 0.00141; porosity = 0.45

Solution:

a. Darcy velocity is given by Equation 3-21
$v = (3.5 \times 10^{-5}\ \text{m/s})(0.00141) = 4.94 \times 10^{-8}\ \text{m/s}$

b. Average linear velocity is given by Equation 3-25

3-45 Darcy velocity and hydraulic gradient

Given: Average linear velocity = 0.60 m/d; porosity = 0.30; hydraulic conductivity = 4.75 x 10^{-4} m/s

Solution:

a. Solve Equation 3-25 for the Darcy velocity
v = (v')(porosity)
v = (0.60 m/d)(0.30) = 0.18 m/d
In m/s
v = (0.18 m/d)/(86400 s/d) = 2.08 x 10^{-6} m/s

b. Solve Equation 3-21 for the hydraulic gradient

$$\frac{dh}{dr} = \frac{v}{k} = \frac{2.08\times10^{-6}\ \text{m/s}}{4.75\times10^{-4}\ \text{m/s}} = 0.00439$$

3-46 Travel time in an aquifer

Given: two piezometers 280 m apart; difference in levels = 1.4 m; hydraulic conductivity = 50 m/d; porosity = 0.20

Solution:

a. First compute the Darcy velocity

$$v = 50\,\text{m/d}\left(\frac{1.4\text{m}}{280\text{m}}\right) = 0.25\,\text{m/d}$$

b. Compute the average linear velocity as

$$v' = \frac{0.25\,\text{m/d}}{0.20} = 1.25\,\text{m/d}$$

3-47 Steady-state drawdown for artesian aquifer

Given: Artesian aquifer 28.0 m thick; piezometric surface 94.05 m above the confining layer; pumping rate of 0.00380 m^3/s; drawdown of 64.05 m at observation well 48.00 m away; sandstone aquifer.

Find: Drawdown at observation well 68.00 m away.

Solution:

A sketch of the problem is shown below

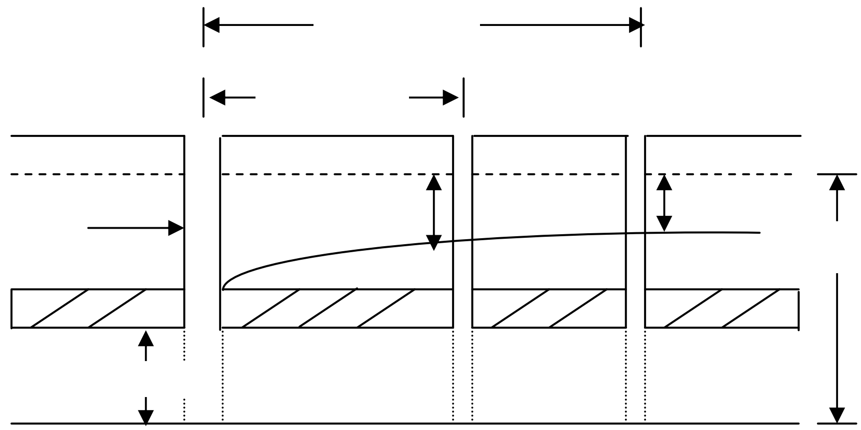


Figure S-3-47

a. From the aquifer material (sandstone) and Table 3-5, find that the hydraulic conductivity (K) is 5.8×10^{-7}.

b. Calculate h_1

$$h_1 = 94.05 \text{ m} - 64.05 \text{ m} = 30.00 \text{ m}$$

c. Using Eqn 3-28 and $T = KD = (5.8 \times 10^{-7})(28.0 \text{ m}) = 1.624 \times 10^{-5} \text{ m}^2/\text{s}$

$$0.00380 = \frac{2\pi(1.624\times10^{-5}\ \text{m}^2/\text{s})(h_2 - 30.00\text{m})}{\ln\left(\frac{68.00}{48.00}\right)}$$

d. Solve for h_2

$$h_2 = \frac{(Q)\left(\ln\left(\frac{r_2}{r_1}\right)\right)}{2\times\pi\times T} + 30.00 = \frac{(0.00380)\left(\ln\left(\frac{68.00}{48.00}\right)\right)}{2\pi(1.624\times10^{-5})} + 30.00 = 12.97 + 30.00 = 42.97\text{m}$$

e. Compute the drawdown

$$s_2 = 94.05 - 42.97 = 51.08 \text{ m}$$

3-48 Steady-state drawdown in artesian aquifer

Given: Artesian aquifer 99.99 m thick; piezometric surface 170.89 m above bottom confining layer; pumping rate of 0.0020 m^3/s; drawdown of 12.73 m at well 280.0 m away; sandstone aquifer.

Solution:

Sketch is similar to 3-47 with furthest well 1492.0 m away.

a. From the aquifer material (sandstone) in Table 3-5 find a hydraulic conductivity of 5.8 x 10^{-7} m/s

b. Calculate h_1

$h_1 = 170.89 - 12.73 = 158.16$ m

c. Using Equation 3-28
and T = KD = (5.8 x 10^{-7} m/s)(99.99 m) = 5.80 x 10^{-5} m^2/s
Solve Equation 3-28 for h_2

$$h_2 = \frac{(Q)\ln\left(\frac{r_2}{r_1}\right)}{2\pi T} + 158.16 = \frac{(0.0020\,m^3/s)\ln\left(\frac{1492.0m}{280.0m}\right)}{2\pi(5.80\times10^{-5})} + 158.16 = 9.19 + 158.16 = 167.35m$$

d. The drawdown is
$s_2 = 170.89 - 167.35 = 3.54$ m

3-49 Steady-state artesian drawdown

Given: Artesian aquifer 42.43 m thick; piezometric surface 70.89 m above bottom confining layer; pumping rate of 0.0255 m^3/s; drawdown of 5.04 m in well 272.70 m from the pumping well; fractured rock aquifer.

Solution:

Sketch similar to that for Problem 3-47 but the unknown drawdown is between the pumping well and the observation well given at $r_1 = 64.28$ m

a. From the aquifer material (fractured rock), in Table 3-5 find a hydraulic conductivity of 5.8 x 10^{-5} m/s

b. Calculate h_2

$h_2 = 70.89 – 5.04 = 65.85$

c. Using Equation 3-28

and $T = KD = (5.8 \times 10^{-5}\ m/s)(42.43\ m) = 2.46 \times 10^{-3}\ m^2/s$

Solve Equation 3-28 for h_1

$$h_1 = h_2 - \frac{(Q)\ln\left(\frac{r_2}{r_1}\right)}{2\pi T} = 65.85 - \frac{(0.0255)\ln\left(\frac{272.70}{64.28}\right)}{2\pi\left(2.46\times10^{-3}\right)} = 65.85 - 2.38 = 63.47$$

d. The drawdown is

$s = 70.89 – 63.47 = 7.42\ m$

3-50 Hydraulic conductivity of a confined aquifer

Given: Confined aquifer 82.0 m thick; drawdown of 3.55 m at 41.0 m away; drawdown of 1.35 m at 63.5 m away; pumping rate of 0.0280 m^3/s; non-pumping piezometric surface = 109.5 m

Solution:

a. Calculate h_1 and h_2

$h_1 = 109.5 – 3.55 = 105.95\ m$

$h_2 = 109.5 – 1.35 = 108.15\ m$

b. Solve Equation 3-28 for T

$$T = \frac{(Q)\ln\left(\frac{r_2}{r_1}\right)}{2\pi(h_2 - h_1)} = \frac{(0.280)\ln\left(\frac{63.5}{41.0}\right)}{2\pi(108.15 - 105.95)} = \frac{0.0122}{13.82} = 8.86\times10^{-4}\ m^2/s$$

c. Solve T = KD for K

$$K = \frac{T}{D} = \frac{8.86\times10^{-4}\ m^2/s}{82.0m} = 1.08\times10^{-5}\ m/s$$

3-51 Steady-state drawdown to aquiclude (Q_{max} ?)

Given: Example Prob. 3-10; drawdown at (h_1) observation well 2.0 m from pumping well is lowered to bottom of aquiclude

Solution:

A sketch of the problem is shown below

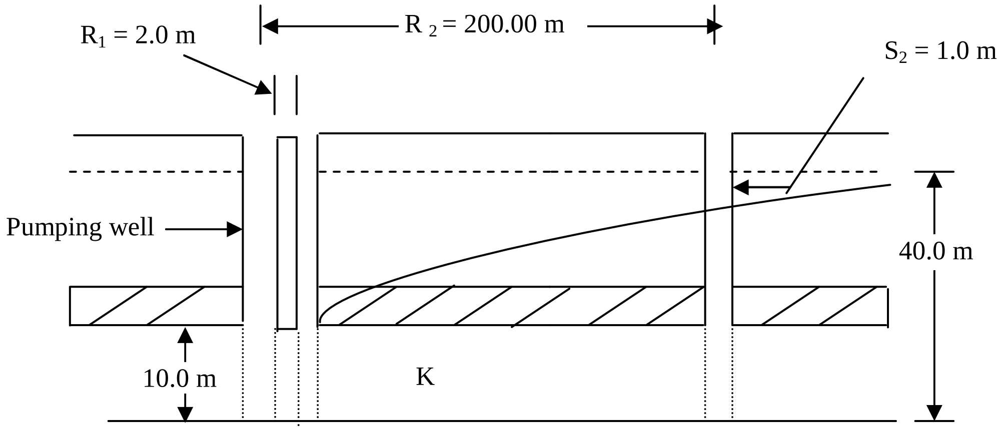


Figure S-3-51

a. From Example Problem 3-10: K = 1.5 x 10^{-4} m/s

b. As in example problem

h_1 = 40.0 - 10.0 = 30.0 m

h_2 = 40.0 - 1.0 = 39.0 m

$$Q = \frac{2\pi(1.5\times10^{-4}\ \mathrm{m/s})(10.0\mathrm{m})(39.0\mathrm{m}-30.0\mathrm{m})}{\ln\left(\frac{200.0\mathrm{m}}{2.0\mathrm{m}}\right)} = \frac{8.48\times10^{-2}}{4.61} = 0.0184\,\mathrm{m^3/s}$$

3-52 Radius of steady-state drawdown of 2.0 m

Given: Artesian aquifer 5 m thick; aquifer material is mixture of sand and gravel; piezometric surface 65 m above confining layer; drawdown of 7 m at observation well 10 m from pumping well; pumping rate is 0.020 m^3/s

Solution:

A sketch of the problem is shown below.

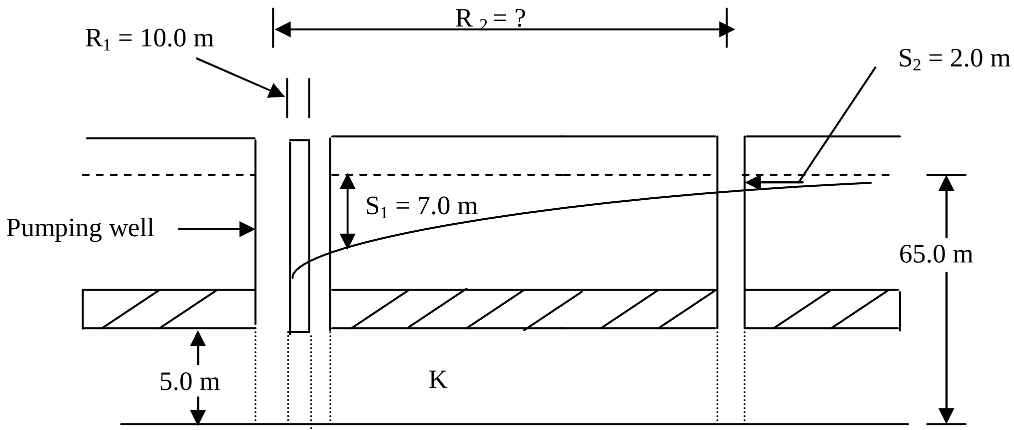


Figure S-3-52

a. From Table 3-5 with "sand and gravel” aquifer material find: $K = 6.1 \times 10^{-4}$ m/s

b. The transmissibility is:

$$T = KD = (6.1 \times 10^{-4}\ \text{m/s})(5\ \text{m}) = 0.00305\ \text{m}^2/\text{s}$$

c. Calculate heights above confining layer

$$h_1 = 65.0 - 2.0 = 63.0\ \text{m}$$

$$h_2 = 65.0 - 7.0 = 58.0\ \text{m}$$

d. Setup Eqn. 3-29

$$0.020\,\text{m}^3/\text{s} = \frac{2\pi\left(3.05\times10^{-3}\ \text{m}^2/\text{s}\right)\left(63.0\text{m} - 58.0\text{m}\right)}{\ln\left(\dfrac{r_2}{10.0}\right)}$$

e. Solve for r_2

$$\ln\left(\frac{r_2}{10.0}\right) = \frac{2\pi\left(3.05\times10^{-5}\ \text{m}^2/\text{s}\right)\left(63.0\text{m} - 58.0\text{m}\right)}{0.020\,\text{m}^3/\text{s}}$$

Taking exponential of both sides of eqn.

$$\left(\frac{r_2}{10.0}\right) = \exp\left\{\frac{2\pi\left(3.05\times10^{-3}\ \text{m}^2/\text{s}\right)\left(63.0\text{m} - 58.0\text{m}\right)}{0.020\,\text{m}^3/\text{s}}\right\} = \exp(4.79) = 120.41$$

$$r_2 = (10.0)(120.41) = 1204.1 \text{ or } 1200\ \text{m}$$

3-53 Steady-state unconfined aquifer hydraulic conductivity

Given: Depth of well = 18.3 m; static water level = 4.57 m below grade; test well pumped at rate of 0.0347 m3/s; drawdown at observation wells: 2.78 m at distance of 20 m; 0.73 m at distance of 110.0 m

Solution:

A similar sketch of the problem can be found in Problem 3-54

a. Calculate depth of aquifer
D = 18.3 – 4.57 = 13.73 m

b. Calculate h_1 and h_2
h_1 = 13.73 – 2.78 = 10.95 m
h_2 = 13.73 – 0.73 = 13.0 m

c. Solve Equation 3-29 for K

$$K = \frac{(Q)\ln\left(\frac{r_2}{r_1}\right)}{\pi\left(h_2^{\;2} - h_1^{\;2}\right)} = \frac{\left(0.0347\, m^3/s\right)\ln\left(\frac{110.0}{20.0}\right)}{\pi\left[(13.0m)^2 - (10.95m)^2\right]} = \frac{5.92\times10^{-2}}{1.54\times10^{2}} = 3.84\times10^{-4}\ m/s$$

3-54 Steady-state drawdown to bottom of aquifer

Given: Example Prob. 3-12; s_1 = 30.0 m

Solution:

A sketch of the problem is shown below.

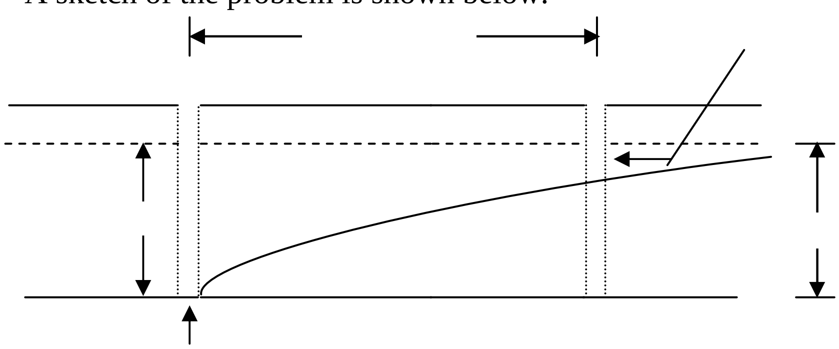


Figure S-3-54

a. From the example problem: K = 6.4 x 10^{-3}

b. Calculate heights above confining layer

h_1 = 30.0 - 30.0 = 0.0 m
h_2 = 30.0 - 9.90 = 20.10 m

c. Calculate the maximum pumping rate

$$Q = \frac{\pi\left(6.4\times10^{-3}\ \text{m/s}\right)\left[(20.10\text{m})^2 - (0.0\text{m})^2\right]}{\ln\left(\frac{100.0\text{m}}{0.25\text{m}}\right)} = \frac{8.1231}{5.9915} = 1.36\,\text{m}^3/\text{s}$$

3-55 Contractor dewatering unconfined aquifer

Given: Q = 0.0280 m^3/s; aquifer = medium sand; dimensions shown on drawing below

Solution:

A sketch of the problem is shown below. From number of days of pumping (1066) assume that this is steady-state.

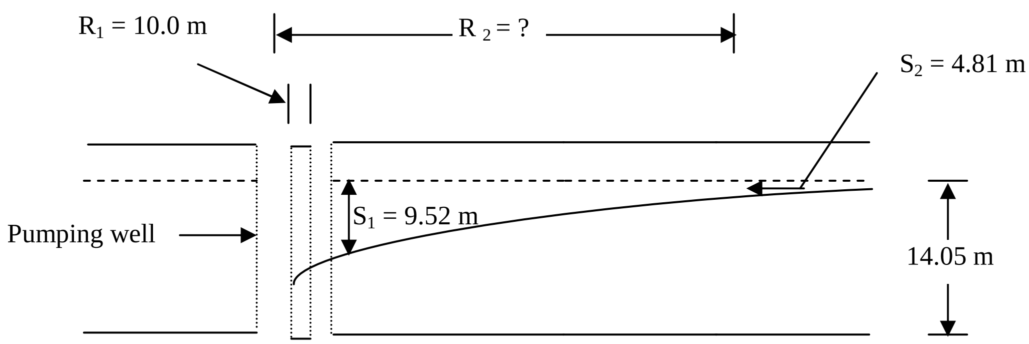


Figure S-3-55

a. Using Table 3-5 and "medium sand" find: K = 1.5 x 10^{-4} m/s

b. Calculate heights above confining layer

h_1 = 14.05 - 9.52 = 4.53 m $(h_1)^2$ = 20.52

$h_2 = 14.05 - 4.81 = 9.24$ m $(h_2)^2 = 85.38$

c. Setup Eqn. 3-30

$$0.0280\,\text{m}^3/\text{s} = \frac{\pi\left(1.5\times10^{-4}\ \text{m}^2/\text{s}\right)\left(85.38-20.52\right)}{\ln\left(\dfrac{r_2}{10.0}\right)}$$

e. Solve for r_2

$$\ln\left(\frac{r_2}{10.0}\right) = \frac{\pi\left(1.5\times10^{-4}\ \text{m}^2/\text{s}\right)\left(85.38-20.52\right)}{0.0280\,\text{m}^3/\text{s}}$$

Taking exponential of both sides of eqn.

$$\frac{r_2}{10.0} = \exp\left\{\frac{\pi\left(1.5\times10^{-4}\ \text{m}^2/\text{s}\right)\left(85.38-20.52\right)}{0.0280\,\text{m}^3/\text{s}}\right\} = \exp\{1.0916\} = 2.9790$$

$$r_2 = (10.0)(2.9790) = 29.79\text{m}$$

3-56 Dewatering aquifer

Given: Unconfined, steady-state, lower piezometric surface 5.25 m below static at 45.45 m from well and 2.50 m at 53.56 m; aquifer material is loam.

Solution:

a. Calculate heights above confining layer
$h_1 = 30.0 – 5.25 = 24.75$ m
$h_2 = 30.0 – 2.50 = 27.50$ m

b. From Table 3-5 find $K = 6.4 \times 10^{-6}$ m/s for loam soil

c. Set up Eqn. 3-30 and solve for Q

$$Q = \frac{\pi\left(6.4\times10^{-6}\right)\left[(27.5)^2-(24.75)^2\right]}{\ln\left(\dfrac{53.56}{45.45}\right)} = \frac{2.89\times10^{-3}}{\ln(1.18)} = \frac{2.89\times10^{-3}}{0.164} = 0.0176\,\text{m}^3/\text{s}$$

3-57 Height of piezometric surface

Given: Unconfined aquifer 20 m thick; K = 1.5 x 10^{-4} ; pumping rate is 0.015 m^3/s; drawdown at 0.25 m diameter well is 8.0 m.

Find: Height of piezometric surface 80.0 m away.

Solution:

A sketch of the problem is shown below.

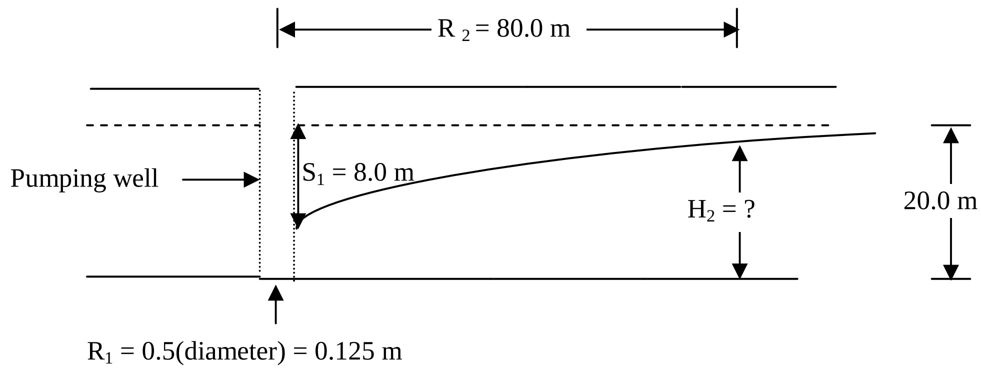


Figure S-3-57

a. Calculate height above confining layer

$h_1 = 20.0 - 8.0 = 12.0$ m $(h_1)^2 = 144.0$

b. Setup Eqn. 3-30 (Note that r_1 = diameter/2)

$$0.0150\,m^3/s = \frac{\pi(1.5\times10^{-4}\ m^2/s)((h_2^{\ 2}) - 144.0)}{\ln\left(\frac{80.0}{0.125}\right)}$$

c. Solve for h_2

$$h_2^{\ 2} = \frac{(0.0150\,m^3/s)\ln\left(\frac{80.0}{0.125}\right)}{\pi(1.5\times10^{-4}\ m^2/s)} + 144.0 = \frac{9.69\times10^{-2}}{4.71\times10^{-4}} + 144.0 = 349.73$$

$$h_2 = \sqrt{349.73} = 18.70\text{m}$$

3-58 Height of piezometric surface (0.5 m diameter well)

Given: See Problem 3-57; new well diameter

Solution:

a. Sketch is same as Problem 3-57

b. Set up Eqn. 3-30 (Note that r_1 = diameter/2)

$$0.0150\,\text{m}^3/\text{s} = \frac{\pi\left(1.5\times10^{-4}\ \text{m/s}\right)\left[(h_2)^2 - 144.0\right]}{\ln\left(\dfrac{80.0}{0.250}\right)}$$

c. Solve for h_2

$$h_2{}^2 = \frac{(0.0150)\ln\left(\dfrac{80.0}{0.250}\right)}{\pi\left(1.5\times10^{-4}\right)} + 144.0 = \frac{8.65\times10^{-2}}{4.71\times10^{-4}} + 144.0 = 183.6 + 144.0 = 327.61$$

$$h_2 = \sqrt{327.61} = 18.10\text{m}$$

3-59 Unsteady flow in a confined aquifer

Given: Aquifer thickness = 28.00 m; aquifer material is fractured rock; drawdown in pumping well is 6.21 m after 48 hours of pumping; pumping rate is 0.0075 m^3/s

Solution:

a. From Table 3-5 find K = 5.8 x 10^{-5} m/s

b. Note from Eqn. 3-30 that T = KD and calculate transmissivity

T = KD = (5.8 x 10^{-5} m/s)(28.00 m) = 1.62 x 10^{-3} m^2/s

c. Convert 48 hour pumping time to days so units are consistent

(48 h)/(24 h/d) = 2 d

d. Solve Eqn. 3-35 for s_2

$$s_2 = \frac{0.0075\,\text{m}^3/\text{s}}{4\pi\left(1.62\times10^{-3}\ \text{m}^2/\text{s}\right)}\ln\left(\frac{48}{2}\right) + 6.21 = \left(3.68\times10^{-1}\right)(3.18) + 6.21 = 7.38\text{m}$$

Note: Well diameter is not relevant for solution.

3-60 Transmissibility from pumping data (unsteady flow)

Given: Well dia. = 0.61 m; Q = 0.0303 m /s; s = 0.98 m at 8 min; s = 3.87 m at 24 h

Solution: (following example 3-10)

$$T = \frac{0.0303\,\text{m}^3/\text{s}}{4\pi(3.87\text{m} - 0.98\text{m})}\ln\left(\frac{24\text{h}(60\,\text{min/h})}{8\,\text{min}}\right) = \frac{0.0303\,\text{m}^3/\text{s}}{36.3168}\ln(180.0)$$

$$T = 0.000834(5.1929) = 4.33\times10^{-3}\ \text{m}^2/\text{s}$$

3-61 Transmissivity of confined aquifer

Given: Pumping rate = 0.0076 m^3/s; drawdown at 0.10 min = 3.00m and at 1.00 min = 34.0 m

Solution:

a. Use Eqn 3-36

$$T = \frac{0.0076\,\text{m}^3/\text{s}}{4\pi(34.0\text{m} - 3.0\text{m})}\ln\left(\frac{1.00}{0.10}\right) = (1.95\times10^{-5})(2.30) = 4.49\times10^{-5}\ \text{m}^2/\text{s}$$

Note: Diameter of well is not relevant for solution.

3-62 Storage coefficient from pumping data

Given: At 96.93 m drawdown = 1.04 m after 80 min of pumping at 0.0170 m /s; t_0 = 0.6 min; T = 5.39 x 10^{-3} m^2/s

Solution: (following example 3-13)

$$S = \frac{(2.25)(5.39\times10^{-3}\ \text{m}^2/\text{s})(0.6\,\text{min})(60\,\text{s/min})}{(96.93)^2}$$

$$S = \frac{0.4366}{9.395\times10^3} = 4.647\times10^{-5}\,(\text{NoUnits!})$$

Check u assumption

$$u = \frac{(96.93)^2(4.647\times10^{-5})}{4(5.39\times10^{-3})(80\,\text{min})(60\,\text{s/min})} = \frac{4.366\times10^{-1}}{1.305\times10^2} = 4.2\times10^{-3}$$

u = 4.2 x 10^{-3} < 0.01 , therefore okay.

3-63 Drawdown at observation well

Given: Data in Problem 3-62; 80 days of pumping

Solution:

a. Using Equation 3-35

$$s_2 - 1.04\text{m} = \frac{0.0170\,\text{m}^3/\text{s}}{4\pi\left(5.39\times10^{-3}\right)}\ln\left(\frac{80\text{d}(1440\,\text{min/d})}{80\,\text{min}}\right)$$

$$s_2 = \frac{0.0170}{0.0677}\ln(1440) + 1.04 = (0.2510)(7.27) + 1.04 = 2.87\text{m}$$

3-64 Drawdown at pumping well

Given: T = 2.51 x 10^{-3} m^2/s; S = 2.86 x 10^{-4} ; well diameter is 0.5 m; pumping rate is 0.0194 m^3/s; two days of pumping.

Solution:

a. Calculate u (see Eqn. 3-31) using r = well diameter/2

$$u = \frac{(0.25\text{m})^2\left(2.86\times10^{-4}\right)}{4\left(2.51\times10^{-3}\right)(2\text{d})(86400\,\text{s/d})} = \frac{1.79\times10^{-5}}{1.73\times10^{3}} = 1.03\times10^{-8}$$

b. At this point two solution techniques are possible. One is to work the problem using the W(u) method of Eqn. 3-31. The other is to recognize that u is << 0.01 and use Eqn. 3-34. Although not required in the problem statement, both methods are employed here.

c. By the exact W(u) method.

Using u as calculated above and Table 3-6 find

W(u) = 17.8435

Compute drawdown using Eqn. 3-31

$$s = \frac{0.0194\,\text{m}^3/\text{s}}{4\pi\left(2.51\times10^{-3}\,\text{m}^2/\text{s}\right)}(17.8435)$$

$$s = (0.615)(17.8435) = 10.97\text{m}$$

d. By the approximation of Eqn. 3-34

$$s = \frac{0.0194\,m^3/s}{4\pi(2.51\times10^{-3}\,m^2/s)}\ln\left(\frac{2.25(2.51\times10^{-3}\,m^2/s)(2d)(86400\,s/d)}{(0.25)^2(2.86\times10^{-4})}\right)$$

$$s = \frac{0.0194}{3.15\times10^{-2}}\ln\left(\frac{9.76\times10^{2}}{1.79\times10^{-5}}\right)$$

$$s = (0.615)\ln(5.46\times10^{7}) = (0.615)(17.8) = 10.96m$$

3-65 Storage coefficient from pumping test data

Given: Q = 0.0350 m^3/s; r = 300.0 m; drawdown values at three time intervals.

Solution:

Using the first and last times and following Example 3-13

a. Calculate transmissibility

$$T = \frac{0.0350\,m^3/s}{4\pi(5.90m - 3.10m)}\ln\left(\frac{1700\,min}{100\,min}\right) = \frac{0.0350}{35.1858}\ln(17.0)$$

$$T = 0.00099(2.833) = 2.818\times10^{-3}\,m^2/s$$

b. From plot below determine t_0 = 4.4 min

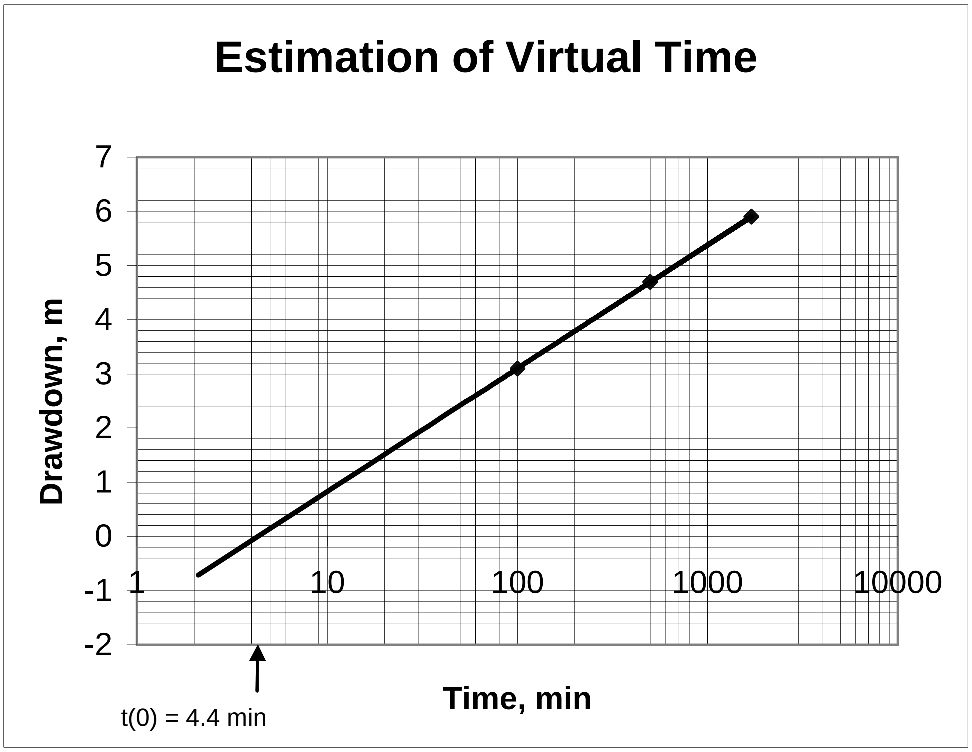


Figure S-3-65

c. Calculate storage coefficient

$$S = \frac{2.25\left(2.818\times10^{-3}\right)\left(4.4\,\text{min}\right)\left(60\,\text{s/min}\right)}{300^2} = \frac{1.674}{9.0\times10^4} = 1.9\times10^{-5}\,(\text{NoUnits!})$$

3-66 Storage coefficient from pumping test data

Given: Data in Problem 3-65; observation well 100.0 m form pumping well

Solution:

a. Calculate T as in Prob. 3-65 solution.

$T = 2.818 \times 10^{-3}\ m^2/s.$

b. Use t_0 from Prob. 3-65 solution.

c. Calculate storage coefficient

$$S = \frac{2.25\left(2.818\times10^{-3}\right)\left(4.4\,\text{min}\right)\left(60\,\text{s/min}\right)}{100^2} = \frac{1.674}{1.0\times10^4} = 1.67\times10^{-5}\left(\text{NoUnits!})\right)$$

3-67 Storage coefficient from pumping test data

Given: Q = 0.0221 m^3/s; r = 100.0 m; drawdown values at three time intervals.

Solution: Using the first and last times and following Example 3-13

a. Calculate transmissibility

$$T = \frac{0.0221\,m^3/s}{4\pi(6.30m - 1.35m)} \ln\left(\frac{1440\,min}{10\,min}\right) = \frac{0.0221}{62.2035} \ln(144.0) = 1.766 \times 10^{-3}\ m^2/s$$

b. From plot below determine t_0 = 2.5 min

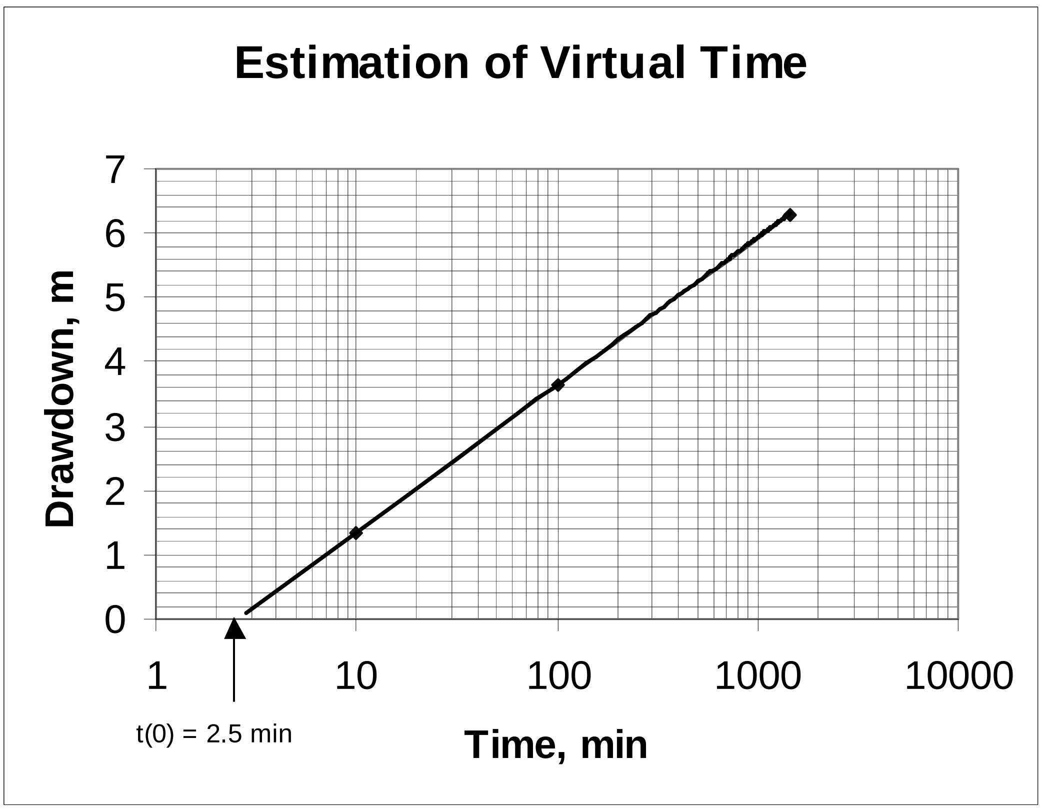


Figure S-3-67

c. Calculate storage coefficient

$$S = \frac{2.25(1.766 \times 10^{-3}\ m^2/s)(2.5\,min)(60\,s/min)}{(100m)^2} = \frac{0.5960}{1 \times 10^4} = 5.96 \times 10^{-5}\ (\text{NoUnits!})$$

3-68 Storage coefficient from pumping test data

Given: Data in Problem 3-67; observation well 60.0 m form pumping well

Solution:

a. Calculate T as in Prob. 3-67 solution.

$T = 1.766 \times 10^{-3}\ m^2/s.$

b. Use t_0 from Prob. 3-67 solution.

c. Calculate storage coefficient

$$S = \frac{2.25(1.766\times10^{-3}\ m^2/s)(2.5\,min)(60\,s/min)}{(60.0m)^2} = \frac{0.5960}{3.6\times10^3} = 1.656\times10^{-4}\,(\text{NoUnits!})$$

3-69 Storage coefficient from pumping data

Given: 0.76 m diameter well pumping at 0.0035 m^3/s; drawdown at three time intervals

Solution:

a. Using the first and last time and following Example 3-14, calculate the tranmissivity

$$T = \frac{0.0035\,m^3/s}{4\pi(5.00m - 2.00m)}\ln\left(\frac{10.0}{0.20}\right) = (9.28\times10^{-5})(3.91) = 3.63\times10^{-4}\ m^2/s$$

b. From plot of drawdown versus time below find t_0 = 0.015 min

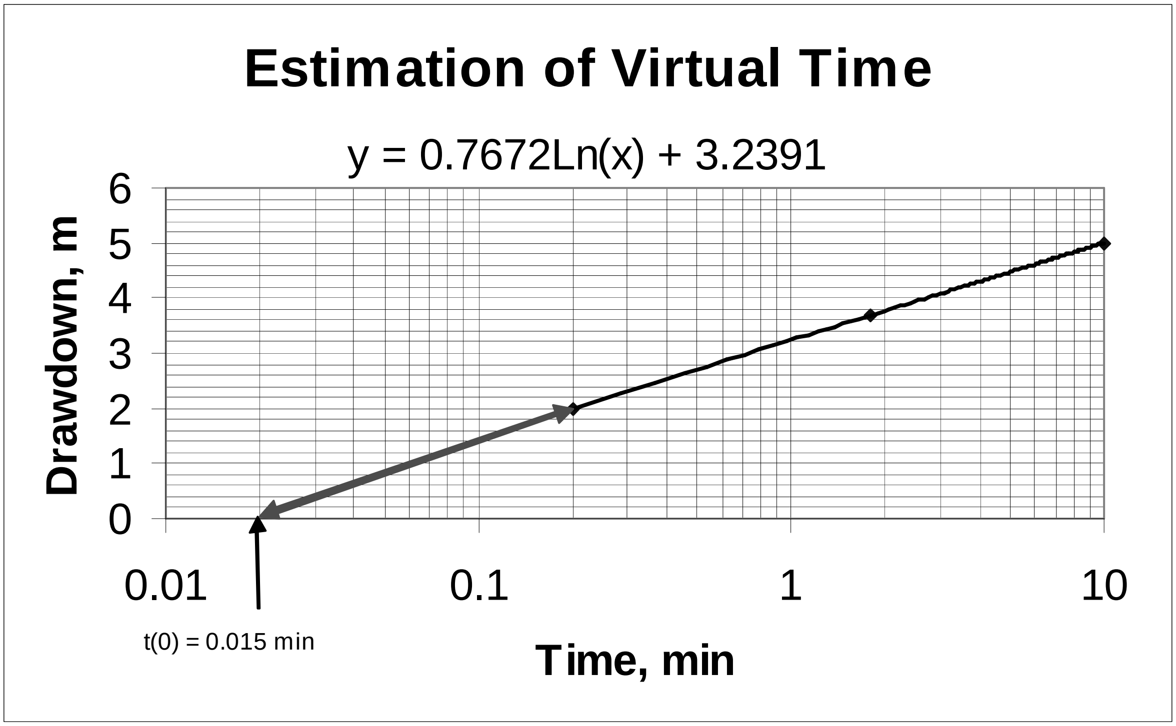


Figure S-3-69

c. Calculate storage coefficient using Eqn. 3-37
Note that r = (0.76 m)/2 = 0.38 m

$$S = \frac{2.25\left(3.63\times10^{-4}\ \mathrm{m^2/s}\right)\left(0.015\,\mathrm{min}\right)\left(60\,\mathrm{s/min}\right)}{\left(0.38\mathrm{m}\right)^2} = \frac{7.35\times10^{-4}}{0.144} = 5.09\times10^{-3}\left(\text{NoUnits!}\right)$$

3-70 Interference of well A on B

Given: Wells 106.68 m apart; $Q_A = 0.0379\ m^3/s$; $T = 4.35 \times 10^{-3}\ m^2/s$; $S = 4.1 \times 10^{-5}$

Solution:

a. Calculate u

$$u = \frac{\left(106.68\mathrm{m}\right)^2\left(4.1\times10^{-5}\right)}{4\left(4.35\times10^{-3}\ \mathrm{m^2/s}\right)\left(365\mathrm{d}\right)\left(86400\,\mathrm{s/d}\right)} = \frac{4.67\times10^{-1}}{5.49\times10^{5}} = 8.50\times10^{-7}$$

b. From Table 3-6 find W(u) = 13.4008

c. Calculate interference

$$s_{\mathrm{AonB}} = \frac{0.0379\,\mathrm{m^3/s}}{4\pi\left(4.35\times10^{-3}\ \mathrm{m^2/s}\right)}\times13.4008 = 9.29\mathrm{m}$$

3-71 Total drawdown in well B

Given: Data in Prob. 3-70

Solution:

a. Calculate u for well B (r = 0.460/2 = 0.230)

$$u = \frac{(0.230)^2(4.1\times10^{-5})}{4(4.35\times10^{-3})(365\text{d})(86400\,\text{s/d})} = \frac{2.1689\times10^{-6}}{5.49\times10^{5}}$$

$u = 3.95 \times 10^{-12}$

b. From Table 3-6 find W(u) = 25.6675

c. Calculate drawdown in well B pumping alone

$$s_B = \frac{0.0252}{4\pi(4.35\times10^{-3})}(25.6675) = 11.83\text{m}$$

d. Total drawdown in well B

$s_{\text{Total in B}} = s_B + s_{\text{A on B}}$

where $s_{\text{A on B}}$ was determined in Prob. 3-70

$s_{\text{Total in B}} = 11.83 + 9.29 = 21.12$ m

3-72 Interference of well 12 on 13

Given: Wells 100.0 m apart; $Q_{12} = 0.0250\ m^3/s$; $T = 1.766 \times 10^{-3}\ m^2/s$; $S = 6.675 \times 10^{-5}$

Solution:

a. Calculate u

$$u = \frac{(100)^2(6.675\times10^{-5})}{4(1.766\times10^{-3})(280\text{d})(86400\,\text{s/d})} = \frac{6.675\times10^{-1}}{1.709\times10^{5}}$$

$u = 3.906 \times 10^{-6}$

b. From Table 3-6 find W(u) = 11.8773

c. Calculate interference

$$s_{12on13} = \frac{0.0250}{4\pi\left(1.766\times10^{-3}\right)}(11.8773) = 13.38\,m$$

3-73 Total drawdown in well 13

Given: Data in Prob. 3-72

Solution:

a. Calculate u for well 13 (r = 0.500/2 = 0.250)

$$u = \frac{(0.250)^2\left(6.675\times10^{-5}\right)}{4\left(1.766\times10^{-3}\right)(280\,d)(86400\,s/d)} = \frac{4.17\times10^{-6}}{1.709\times10^{5}}$$

$u = 2.44 \times 10^{-11}$

b. From Table 3-6 find W(u) = 23.8594

c. Calculate drawdown in well 13 pumping alone

$$s_{13} = \frac{0.0300}{4\pi\left(1.766\times10^{-3}\right)}(23.8594) = 32.25\,m$$

d. Total drawdown in well 13

$$s_{Total\ in\ 13} = s_{13} + s_{12\ on\ 13}$$

Where $s_{12\ on\ 13}$ was determined in Prob. 2-72.

$$s_{Total\ in\ 13} = 32.25 + 13.38 = 45.63\ m$$

3-74 Interference of well X on wells Y and Z

Given: Wells located at 100 m intervals; Q_X = 0.0315 m^3/s; Q_Y = 0.0177 m^3/s; Q_Z = 0.0252 m^3/s; all well diameters = 0.3 m; T = 1.77 x 10^{-3} m^2/s; S = 6.436 x 10^{-5}

Solution:

a. Calculate u

$$u_{XY} = \frac{(100)^2 (6.436 \times 10^{-5})}{4(1.77 \times 10^{-3})(100 d)(86400 s/d)} = 1.05 \times 10^{-5}$$

$$u_{XZ} = \frac{(200)^2 (6.436 \times 10^{-5})}{4(1.77 \times 10^{-3})(100 d)(86400 s/d)} = 4.21 \times 10^{-5}$$

b. From Table 3-6 find $W(u)_{XY}$ = 10.8849 and $W(u)_{XZ}$ = 9.4986

c. Calculate drawdown in Y from X

$$s_{XY} = \frac{0.0315}{4\pi(1.77 \times 10^{-3})}(10.8849) = 15.42 m$$

d. Calculate drawdown in Z from X

$$s_{XZ} = \frac{0.0315}{4\pi(1.77 \times 10^{-3})}(9.4986) = 13.45 m$$

3-75 Total drawdown in well X

Given: Data in Prob. 3-74

Solution

a. Calculate u

For well X (r = 0.3/2 = 0.150 m)

$$u_{X} = \frac{(0.150)^2 (6.436 \times 10^{-5})}{4(1.77 \times 10^{-3})(100 d)(86400 s/d)} = 2.37 \times 10^{-11}$$

$$u_{YX} = \frac{(100)^2 (6.436 \times 10^{-5})}{4(1.77 \times 10^{-3})(100 d)(86400 s/d)} = 1.05 \times 10^{-5}$$

$$u_{ZX} = \frac{(200)^2 (6.436 \times 10^{-5})}{4(1.77 \times 10^{-3})(100 d)(86400 s/d)} = 4.21 \times 10^{-5}$$

b. From Table 3-6 find $W(u)_{X}$ = 23.8895, $W(u)_{YX}$ = 10.8849, $W(u)_{ZX}$ = 9.4986

c. Calculate drawdown in well X pumping alone

$$s_X = \frac{0.0315}{4\pi(1.77\times10^{-3})}(23.8895) = 33.83\text{m}$$

d. Calculate drawdown in well X from wells Y and Z

$$s_{YX} = \frac{0.0177}{4\pi(1.77\times10^{-3})}(10.8895) = 8.66\text{m}$$

$$s_{YX} = \frac{0.0177}{4\pi(1.77\times10^{-3})}(9.4986) = 10.76\text{m}$$

e. Calculate total drawdown in well X

$$s_{X=\text{Total}} = s_X + s_{YX} + s_{ZX} = 33.83 + 8.66 + 10.76 = 53.26\text{m}$$

3-76 Effect of adding 6th well

Given: S = 6.418 x 10^{-5} ; T = 1.761 x 10^{-3} m^2/s; static pumping level = 6.90 m below ground level; depth from ground surface to top of artesian = 87.0 m; depth of wells and pumping rates; t = 100 days.

Solution:

a. A spreadsheet can be used to perform the calculations.

b. Set up grid coordinates on well field. The following were used to solve this problem (differences in scaling will affect the results):

Well	x-coord. (m)	y-coord. (m)
1	0	0
2	100	0
3	0	100
4	100	100
5	0	200
6	100	200

c. Calculate the distance between wells

Distance btw wells	#1	#2	#3	#4	#5	#6
to #1	0	100	100	141.4214	200	223.6068
to #2	100	0	141.4214	100	223.6068	200
to #3	100	141.4213562	0	100	100	141.4214
to #4	141.4213562	100	100	0	141.4214	100
to #5	200	223.6067977	100	141.4214	0	100
to #6	223.6067977	200	141.4214	100	100	0

d. Calculate u

u for well	#1	#2	#3	#4	#5	#6
to #1	2.37273E-11	1.05455E-05	1.05E-05	2.11E-05	4.22E-05	5.27E-05
to #2	1.05455E-05	2.37273E-11	2.11E-05	1.05E-05	5.27E-05	4.22E-05
to #3	1.05455E-05	2.1091E-05	2.37E-11	1.05E-05	1.05E-05	2.11E-05
to #4	2.1091E-05	1.05455E-05	1.05E-05	2.37E-11	2.11E-05	1.05E-05
to #5	4.21819E-05	5.27274E-05	1.05E-05	2.11E-05	2.37E-11	1.05E-05
to #6	5.27274E-05	4.21819E-05	2.11E-05	1.05E-05	1.05E-05	2.37E-11

e. Calculate W(u)

W(u) for well	#1	#2	#3	#4	#5	#6
to #1	23.88717694	10.88260715	10.88261	10.18947	9.496344	9.273211
to #2	10.88260715	23.88717694	10.18947	10.88261	9.273211	9.496344
to #3	10.88260715	10.18947051	23.88718	10.88261	10.88261	10.18947
to #4	10.18947051	10.88260715	10.88261	23.88718	10.18947	10.88261
to #5	9.496344423	9.273211417	10.88261	10.18947	23.88718	10.88261
to #6	9.273211417	9.496344423	10.18947	10.88261	10.88261	23.88718

f. Calculate drawdown in each well

Drawdown for well	#1	#2	#3	#4	#5	#6	Well #	Total Drdwn (m)
on well #1	23.86	15.49	9.29	8.15	12.19	10.56	1	79.54
on well #2	10.87	34.00	8.70	8.70	11.90	10.81	2	84.99
on well #3	10.87	14.50	20.40	8.70	13.97	11.60	3	80.05
on well #4	10.18	15.49	9.29	19.11	13.08	12.39	4	79.54
on well #5	9.48	13.20	9.29	8.15	30.66	12.39	5	83.18
on well #6	9.26	13.52	8.70	8.70	13.97	27.20	6	81.35

g. There is potential for adverse effects if the piezometric surface is drawn below the bottom of the aquiclude (top of aquifer). To see if this has happened, 6.90 m must be added to the total drawdown for each well because drawdown is calculated from undisturbed piezometric surface (non-pumping water level) and not the ground surface. This total must then be compared with the distance to the top of the artesian aquifer as measured from the ground surface.

Well #	Depth to piezometric surface* (m)	Depth to top of aquifer* (m)	Okay?
1	86.44	87	Yes
2	91.89	87	No
3	86.95	87	Yes
4	86.44	87	Yes
5	90.08	87	No
6	88.25	87	No

* From ground surface

3-77 Effect of adding 6th well

Given: S = 2.11 x 10^{-6} ; T = 4.02 x 10^{-3} m^2/s; static pumping level = 9.50 m below ground level; depth from ground surface to top of artesian = 50.1 m; depth of wells and pumping rates, t = 100 days.

Solution:

a. A spreadsheet can be used to perform the calculations.

b. Set up grid coordinates on well field. The following were used to solve this problem (differences in scaling will affect the results):

Well	x-coord. (m)	y-coord. (m)
1	0	0
2	600	150
3	0	300
4	600	450
5	0	600
6	600	750

c. The effect of adding the sixth well is the increase in drawdown caused by the sixth well on the total drawdown of each individual well:

Well No.	Drawdown w/ 6th Well (m)	Drawdown w/o 6th Well (m)	Increase in Drawdown (m)
1	39.6	34.35	5.25
2	44.71	39	5.71
3	40.78	35.29	5.49
4	39.46	33.06	6.4
5	43.16	37.48	5.68
6	41.16	N/A	N/A

d. There is potential for adverse effects if the piezometric surface is drawn below the bottom of the aquiclude (top of aquifer). To see if this has happened, 9.50 m must be added to the total drawdown for each well because drawdown is calculated from undisturbed piezometric surface (non-pumping water level) and not the ground surface. This total must then be compared with the distance to the top of the artesian aquifer as measured from the ground surface.

Well No.	Depth to Piez. Surface* (m)	Depth to Top of Aquif.* (m)	Okay?
1	49.09	50.1	Yes
2	54.21	50.1	No
3	50.28	50.1	No
4	48.96	50.1	Yes
5	52.66	50.1	No
6	50.65	50.1	No

* From ground surface.

3-78 Acceptable pumping rate/time

Given: Problem 3-77; Drawdown in wells 2-5 too deep; new well diameter = 1.50 m

Solution:

The new, larger well diameter mitigates the problem of excessive drawdown. A wide variety of combinations of pumping rate or time adjustments may be used to achieve an acceptable drawdown. Another solution is to lower the pumping rate of well 6 to 0.0170 m^3/s.

3-79 Effect of adding 6th well

Given: $S = 2.8 \times 10^{-5}$; $T = 1.79 \times 10^{-3}$ m^2/s; static pumping level = 7.60 m below ground level; depth from ground surface to top of artesian = 156.50 m; depth of wells and pumping rates, t = 180 days.

Solution:

A spreadsheet can be used to perform the calculations.

a. Set up grid coordinates on well field. The following were used to solve this problem (differences in scaling will affect the results):

Well	x-coord. (m)	y-coord. (m)
1	125	0
2	0	240
3	190	125
4	65	360
5	245	240
6	125	475

b. The effect of adding the sixth well is the increase in drawdown caused by the sixth well on the total drawdown of each individual well:

Well No.	Drawdown w/ 6th Well (m)	Drawdown w/o 6th Well (m)	Increase in Drawdown (m)
1	145.3	125.96	19.34
2	152.58	130.8	21.78
3	151.5	130.94	20.56
4	150.8	126	24.8
5	152.21	130.39	21.82
6	148.55	N/A	N/A

c. There is potential for adverse effects if the piezometric surface is drawn below the bottom of the aquiclude (top of aquifer). To see if this has happened, 7.60 m must be added to the total drawdown for each well because drawdown is calculated from undisturbed piezometric surface (non-pumping water level) and not the ground surface. This total must then be compared with the distance to the top of the artesian aquifer as measured from the ground surface.

Well No.	Depth to Piez. Surface* (m)	Depth to Top of Aquif.* (m)	Okay?
1	152.9	156.5	Yes
2	160.18	156.5	No
3	159.1	156.5	No
4	158.4	156.5	No
5	159.81	156.5	No
6	156.15	156.5	Yes but close

* From ground surface.

3-80 Acceptable pumping rate/time

Given: Problem 3-79; Drawdown in wells 2-5 too deep; new well diameter = 1.80 m

Solution:

The new, larger well diameter mitigates the problem of excessive drawdown. A wide variety of combinations of pumping rate or time adjustments may be used to achieve an acceptable drawdown. Another solution is to lower the pumping rate of well 6 to 0.0454 m^3/s.

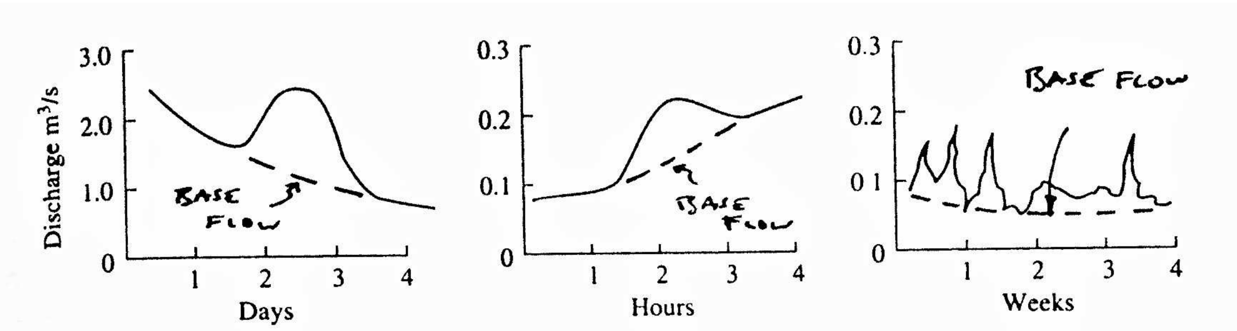
DISCUSSION QUESTIONS

3-1 The answer is False. Rewrite the statement as:

An artesian aquifer is under pressure because of the weight of the overlying water.

The height of the water to the piezometric surface provides the pressure.

3-2 The base flow may be determined by sketching the extrapolation of discharge curve as shown below.



(a) (b) (c)

3-3 The discharge would have to be measured until the time of concentration was achieved. The data to be gathered to determine the time of concentration would include: topographic data to determine the overland flow distance (D) and the slope (S) and a description of the surface characteristics to determine the runoff coefficient (C).

3-4 The variables in the rational formula include the runoff coefficient (C), the rainfall intensity (i) and the area (A). The rational formula assumes that steady state conditions have been achieved. This occurs when the precipitation has fallen long enough to achieve the time of concentration. For a constant rainfall intensity, the discharge does not increase for durations of precipitation greater then the time of concentration. Thus, the rainfall intensity selected for use in the rational formula is the intensity that corresponds to the time of concentration. An IDF curve is entered with the time of concentration and an intensity reading taken at the desired return period.

3-5 The answer is False. Rewrite the statement as:

When a flood has a recurrence interval (return period) of 5 years, it means that a chance of another flood of the same or greater intensity occurring next year is 20 percent.

3-6 The answer is False. Rewrite the statement as:

The hydrologic year used for data presentation of rainfall or runoff events is from October 1 to September 30.

3-7 Because water is encountered in the drilling at 1.8 m and the well is terminated with a screen at 6.0 -8.0 m, we can hypothesize that this is an unconfined aquifer with the ground water table at 1.8 m below the ground surface.

3-8 The notes indicate the casing is sealed to the clay at 13.7-17.5 m and that the static water level is 10.2 m below the ground surface. We hypothesize that the fine sand is a confined aquifer between the clay and the bedrock. The clay serves as an aquiclude, the bedrock as a confining layer. The piezometric surface for the confined aquifer is 10.2 m below grade.

3-9 The sketch showing well interference is shown below. It is not symmetrical because the two wells are pumping at different rates. Figure 3-26 shows the symmetric case.