Instructor's Manual
to accompany

BASIC ENVIRONMENTAL TECHNOLOGY

Water Supply, Waste Management, and Pollution Control

Fifth Edition

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This manual provides instructors with (a) text page references and Internet URLs where answers to the end-of-chapter *Review Questions* can be found, (b) worked out solutions to each of the *Practice Problems,* and (c) supplemental problems and 100 multiple choice questions (and answers) that can be incorporated in tests or a final examination.

Generally, answers to end-of-chapter Practice Problems are rounded-off to reflect the precision of the data and/or the accuracy of the assumed factors in the problems. These answers are also listed in Appendix F of the text for students to use in checking their work. (The author has made every attempt to keep errors to a minimum. He can be notified of any mistakes that may be found in the text or in this manual at: nathanson1@comcast.net).

CHAPTER 1 BASIC CONCEPTS

<u>Review Question Page References</u>

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(31) http://www.epa.gov/epahome/laws.htm
(32) http://www.usgs.gov/nawqa/
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(There are no Practice Problems for Chapter 1)

CHAPTER 2 HYDRAULICS

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(21) 51 (22) www.iihs.uiowa.edu/projects/index.html
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Solutions to Practice Problems

1. $P = 0.43 \times h$ (Equation 2-2b)
   $P = 0.43 \times 50$ ft $= 22$ psi at the bottom of the reservoir
   $P = 0.43 \times (50 - 30) = 0.43 \times 20$ ft $= 8.6$ psi above the bottom

2. $h = 0.1 \times P = 0.1 \times 50 = 5$ m (Equation 2-3a)

3. Depth of water above the valve: $h = (78\text{ m} - 50\text{ m}) + 2\text{ m} = 30$ m
   $P = 9.8 \times h = 9.8 \times 30 = 294$ kPa ≈ 290 kPa (Equation 2-2a)

4. $h = 2.3 \times P = 2.3 \times 50 = 115$ ft, in the water main
   $h = 115 - 40 = 75$ ft
   $P = 0.43 \times 75 = 32$ psi, 40 ft above the main (Equation 2-2b)

5. Gage pressure $P = 30 + 9.8 \times 1 = 39.8$ kPa ≈ 40 kPa
   Pressure head (in tube) $= 0.1 \times 40$ kPa $= 4$ m

6. $Q = A \times V$ (Eq. 2-4), therefore $V = Q/A$
   $A = \pi D^2/4 = \pi (0.3)^2/4 = 0.0707\text{ m}^2$
   $100\text{ L/s} \times 1\text{ m}^3/1000\text{L} = 0.1\text{ m}^3/\text{s}$
   $V = 0.1\text{ m}^3/\text{s} / 0.707\text{m}^2 = 1.4\text{ m/s}$

7. $Q = (500\text{ gal/min}) \times (1\text{ min}/60\text{ sec}) \times (1\text{ ft}^3/7.5\text{ gal}) = 1.11$ cfs
   $A = Q/V$ (from Eq. 2-4) .
   $A = 1.11\text{ ft}^3/\text{sec} / 1.4\text{ ft/sec} = 0.794\text{ ft}^2$
   $A = \pi D^2/4$, therefore $D = \sqrt{4A/\pi} = \sqrt{(4)(0.794)/\pi} = 1$ ft $= 12$ in.

8. $Q = A_1 \times V_1 = A_2 \times V_2$ (Eq. 2-5)

Since $A = \pi D^2/4$, we can write

$D_1^2 \times V_1 = D_2^2 \times V_2$ and $V_2 = V_1 \times (D_1^2/D_2^2)$

In the constriction, $V_2 = (2 \text{ m/s}) \times (4) = 8$ m/s

9. Area of pipe A = $\pi(0.3)^2/4 = 0.0707$ m^2

Area of pipe B = $\pi(O.1)^2/4 = 0.00785$ m^2

Area of pipe C= $\pi(0.2)^2/4 = 0.03142$ m^2

$Q_A = Q_B + Q_C = 0.00785 \text{ m}^2 \times 2 \text{ m/s} + 0.03142 \text{ m}^2 \times 1 \text{ m/s}$

$= 0.04712$ m^3/s (from continuity of flow: $Q_{IN} = Q_{OUT}$)

$V_A = Q_A/A_A = 0.4712/0.0707 \approx 0.67$ m/s (from Eq. 2-4)

10. $p_1/w + V_1^2/2g = p_2/W + V_2^2/2g$ (Eq. 2-8)

$A_1 = \pi(1.33)^2/4 = 1.4$ ft^2 $\qquad$ $A_2 = \pi(0.67)^2/4 = 0.349$ ft^2

$V_1 = 6/1.4 = 4.29$ ft/sec $\qquad$ $V_2 = 6/0.349 = 17.2$ ft/sec

$w = 62.4$ Ib/ft^3 and $g = 32.2$ ft/sec^2

From Eq. 2-8, and multiplying *psi* x 144 in^2/ft^2 to get Ib/ft^2

$50(144)/62.4 + 4.29^2/2(32.2) = p_2(144)/62.4 + 17.2^2/2(32.2)$

$115.38 + 0.28578 = 2.3076p_2 + 4.5937$

$p_2 = 111.07/2.307 \approx 48$ psi

11. $p_1/w + v_1^2/2g = p_2/w + v_2^2/2g$ (Eq. 2-8)

$A_1 = \pi(0.300)^2/4 = 0.0707$ m^2 $\qquad$ $A_2 = \pi(0.100)^2/4 = 0.00785$ m^2

$Q = 50 \text{ L/s} \times 1 \text{ m}^3/1000 \text{ L} = 0.05$ m^3/s

$V_1 = 0.05/0.0707 = 0.70721$ m/sec $\qquad$ $V_2 = 0.05/0.00785 = 6.369$ m/sec

$w = 9.81$ kN/m^3 and $g = 9.81$ m/s^2; From Eq. 2-8,

$700/2(9.81) + 0.70721^2/2(9.81) = p_2/2(9.81) + 6.369^2/2(9.81)$

$35.67789 + 0.02549 = 0.05097p_2 + 2.06775$ and $p_2 = 660$ kPa

12. From Figure 2.15, with Q = 200 L/s and D = 600 mm, read S = 0.0013.
Therefore h_L = S x L = 0.0013 x 1000 m = 1.3 m

Pressure drop p = 9.8 x 1.3 = 12.7 ≈ 13 kPa per km

13. h_L = 2.3 x 20 = 46 ft and S = 46/5280 = 0.0087 (where 1 mi = 5280 ft)

From Figure 2.15, with Q = 1000 gpm and S = 0.0087, read D = 10.3 in.
Use a 12 in. standard diameter pipe

14. S = 10/1000 = 0.01

From the nomograph (Figure 2.15) read Q ≈ 100 L/s = 0.1 m^3/s

Check with Eq. 2-9: $Q = 0.28 \times 100 \times 0.3^{2.63} \times 0.01^{0.54} \approx 0.1\ m^3/s$ OK

15. Use (Eq. 2-10): $Q = C \times A_2 \times \{(2g(p_1 - p_2)/w)/(1 - (A_2/A_1)^2\}^{1/2}$

where $A_1 = \pi(6)^2/4 = 28.27\ in^2$ and $A_2 = \pi(3)^2/4 = 7.07\ in^2$

$g = 32.2\ ft/s^2 = 386.4\ in/s^2$

$w = 62.4\ lb/ft^3 \times 1\ ft^3/12^3\ in^3 = 0.0361\ lb/in^3$

$Q = 0.98 \times 7.07 \times \{(2(386.4)(10)/0.0361)/(1 - (7.07/28.27)^2)\}^{1/2}$

$Q = 0.98 \times 7.07 \times \sqrt{228{,}354} = 3311\ in^3/s = 1.9$ cfs ≈ 2 cfs

16. Use (Eq. 2-10): $Q = C \times A_2 \times \{(2g(p_1 - p_2)/w)/(1 - (A_2/ A_1)^2)\}^{1/2}$

$A_1 = \pi(0.15)^2/4 = 0.01767\ m^2$ and $A^2 = \pi(0.075)^2/4 = 0.00442\ m^2$

$g = 9.81\ m/s^2$ $w = 9.81\ kN/m^3$

$1 - (A_2/A_1)^2 = 1 - (0.00442/0.01767)^2 = 0.93743$

$Q = 0.98 \times 0.00442 \times \{(2(9.81)(100)/9.81)/0.93743)\}^{1/2} = 0.063\ m^3/s$

(or, Q = 0.063 m/s x 1000 Lm^3 = 63 L/s)

17. Use Manning's nomograph (Figure 2.21): With D = 800 mm = 80 cm, and
S = 0.2% = 0.002, read Q= 0:56 m^3/s = 560 L/s and V = 1.17 m/s

18. S = 1.5/1000 = 0.015; from Fig. 2.21, Q ≈ 1800 gpm and V ≈ 2.3 ft/s

19. Q = 200 L/s = 0.2 m^3/s; from Fig. 2.21, D ≈ 42 cm; use 450 mm pipe

20. Q = 7 mgd = 7,000,000 gal/day x 1 day/1440 min ≈ 4900 gpm
From Fig. 2.21, with 36 in and 4900 gpm: S = 0.00027, V = 1.54 ft/s
Since 1.54 ft/s is less than the minimum self-cleansing velocity of 2 ft/s, it is necessary to increase the slope of the 36 in pipe.
From Fig. 2.21, with 36 in and 2 ft/s: S = 0.00047 = 0.047% ≈ 0.05%

21. For full-flow conditions, with D = 300 mm and S = 0.02, read from

Fig. 2.21: Q = 0.135 m^3/s = 135 L/s and V = 2 m/s "
q/Q = 50/135 = 0.37 From Fig. 2.22, d/D = 0.42 and v/V = 0.92
Depth at partial flow d = 0.42 x 300 = 126 mm ≈ 130 mm
Velocity at partial flow v = 0.92 x 2 ≈ 1.8 m/s

22. For full-flow conditions, from Fig. 2.21 read Q = 1800 gpm. From Fig. 2.22, the maximum value of q/Q = 1.08 when d/D = 0.93. Therefore, the highest discharge capacity for the 18 in pipe, q_{max} = 1800 x 1.08 ≈ 1900 gpm, would occur at a depth of d = 18 x 0.93 ≈ 17 in.

23. For full-flow conditions, from Fig. 2.21 read Q = 0.55 m^3/s = 550 L/s. From Fig. 2.22, the maximum value of v/V = 1.15 when d/D = 0.82. Therefore, the highest flow velocity for the 900 mm pipe, v_{max} =

0.9 x 1.15 ≈ 1 m/s, would occur at a depth of d = 900 x 0.82 ≈ 740 mm

When the flow occurs at that depth, q/Q = 1.05 and the discharge q = 580 L/s

24. S = 0.5/100 = 0.005
For full-flow conditions, Q = 0.44 m^3/s = 440 L/s and V = 1.6 m/s
Since d/D = 200/600 = 0.33, from Fig. 2.22 q/Q = 0.23 and v/V = 0.8
Therefore, q = 440 x 0.23 ≈ 100 L/s and v = 1.6 x 0.8 ≈ 1.3 m/s

25. Q = A x V = 2 x 0.75 x 25/75 = 0.5 m^3/s = 500 L/s

26. From Eq. 2-12, Q = 2.5 x $(4/12)^{2.5}$ = 0.16 cfs

27. 150 mm x 1 in/25.4 mm x 1 ft/12 in = 0.492 ft
From Eq. 2-12, Q = 2.5 x $(0.492)^{2.5}$ = 0.425 cfs x 28.32 L/ft^3 ≈ 12 L/s

28. From Eq. 2-13, Q = 3.4 x (20/12) x $(10/12)^{1.5}$ = 4.3 cfs ≈ 120 L/s

CHAPTER 3 HYDROLOGY

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(31) http://hydrolab.arsL/sda.gov (32) www.epa.gov/surf

(33) www.envirosources.com

Solutions to Practice Problems

1. Intensity = 500 mm/ 10 h = 50 mm/h
   Volume = depth x area = 0.5 m x 750 000 m^3 = 375 000 m^3 = 375 ML

2. Intensity = 1 in./0.5 h = 2 in./h
   Volume = depth x area = 1 in. x 1 ft/12 in. x 96 ac = 8 ac-ft
   Volume = 8 ac-ft x 43,560 ft^2/ac ≈ 350,000 ft^3

3. (a) 100 mm/h (4 in./h); (b) 45 mm/h (1.7 in./h); (c) 50 mm/h (2 in./h)

4. 75 mm/0.5 h = 150 mm/h; line up 30 min and 150 mm/h in Fig. 3.5. The intersection falls on the 100-yr storm curve. The probability of a greater storm occurring within the next year is P = 1/100 = 0.01 = 1%

5. From Eq. 3-3, i = 3000/(90 + 20) = 27 mm/h

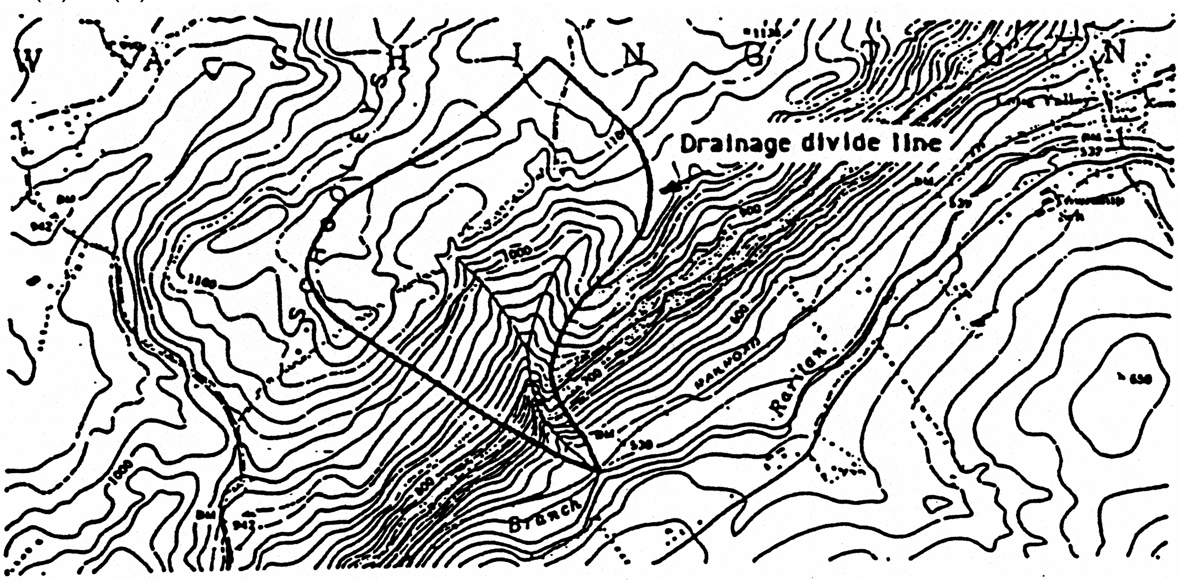
6. P = 1/N = 1/20 = 0.05 = 5%

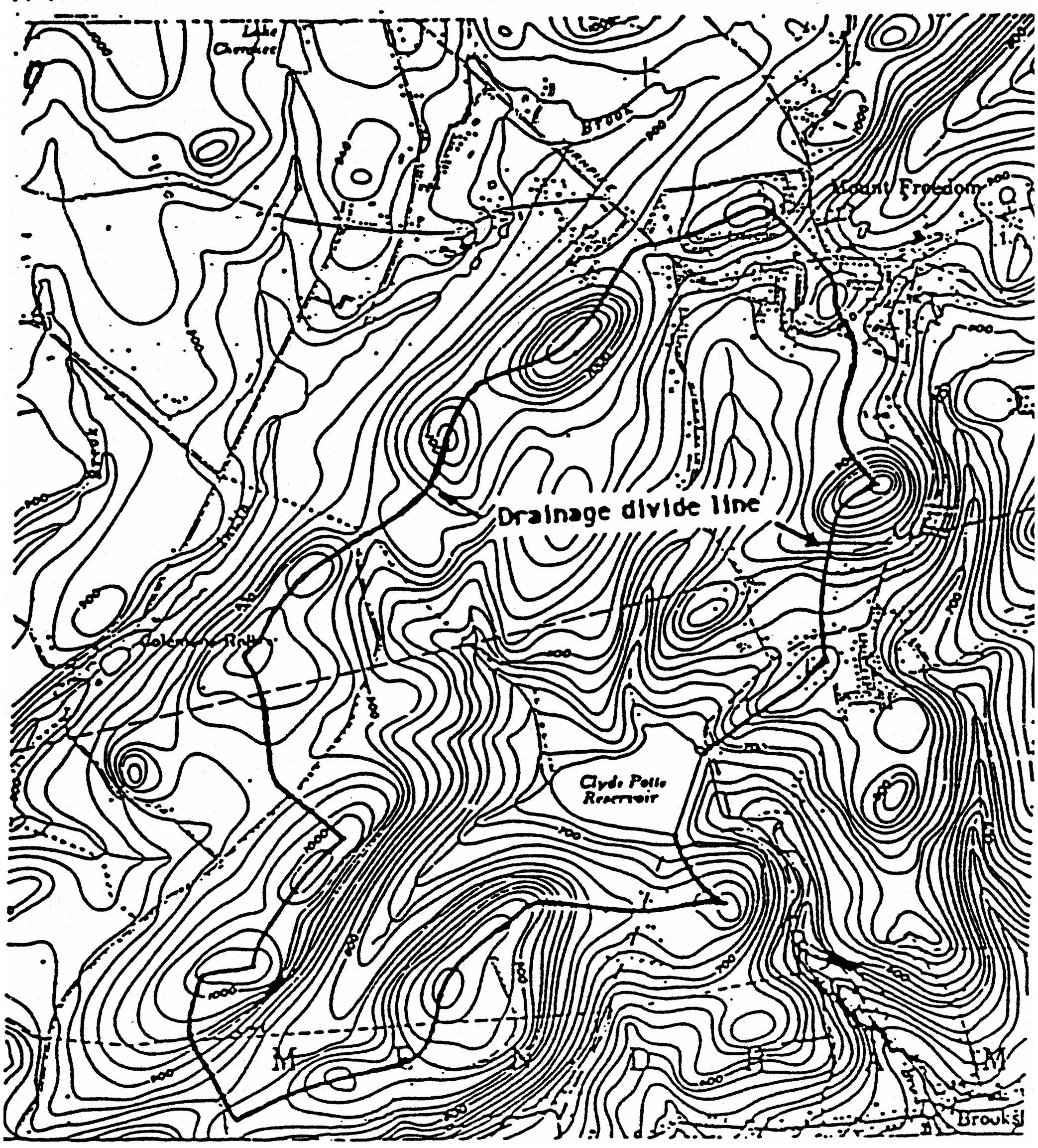
7.

Low Flow	Rank	Probability	Low Flow	Rank	Probability
57	1	0.059	45	9	0.529
53	2	0.117	44	10	0.588
50	3	0.176	42	11	0.647
50	4	0.235	41	12	0.706
50	5	0.294	40	13	0.765
48	6	0.353	39	14	0.824
47	7	0.412	36	15	0.882
45	8	0.471	33	16	0.941

Multiply vertical axis values on Figure 3.16 by 10, and plot Low Flow versus Probability. Read MA7CD10 flow to be approximately 35m^3/s (where the recurrence value = 10 yrs.)

8. (a) & (b)


Drainage divide line
Raritan
Branch

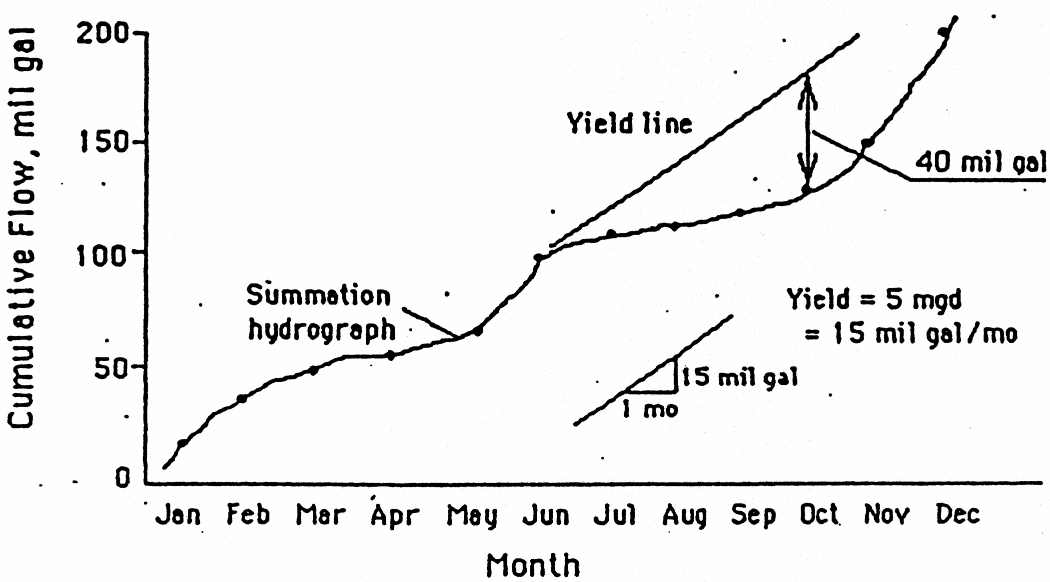

Mount Freedom
Drainage divide line
Clyde Potts Reservoir

9.

Cumulative Flows, mil gal

Month	Flow	Month	Flow	Month	Flow	Month	Flow
Jan	20	Apr	52	Jul	108	Oct	119
Feb	40	May	62	Aug	111	Nov	149
Mar	50	Jun	100	Sep	113	Dec	199

The required reservoir volume is approximately 40 million gallons.



10. $V = K \times S$ (Darcy's Law, Equation 3-4)
    $V = 0.05$ mm/s x 0.5/100 = 0.05 x 0.005 = 0.00025 mm/s
    $V = 0.00025$ mm/s x 3600 s/h x 24 h/d = 0.9 mm/h ≈ 22 mm/d

11. K = VIS (From Eq. 3-4)
    V = 0.05 m/h x 1000 mm/m x 1h/3600 s = 0.0139 mm/s
    K = 0.0139/0.035 ≈ 0.4 mm/s (For sand, K = 0.01 to 10 mm/s)

12. Yield = 2 m^3/h/m x 15 m = 30 m^3/h
    10% of 30 = 3; new yield ≈ 33 m^3/h

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(41) www.wqa.org	(42) www.epa.gov/nerlcwww	(43) www.envirosources.com	

Solutions to Practice Problems

1. 275 ppm = 275 mg/L (since 1 ppm = 1 mg/L)
   275 mg/L x 1 gpg/17.1 mg/L = 16.1 gpg

2. 4 x 17.1 = 68 mg/L of hardness as $CaCO_3$
   This level is not objectionable; it is considered to be soft water and not cause scaling problems or interfere noticeably with lathering.

3. Since 1 mg/L = 8.34 lb/mil gal, we can write
   50 mg/L x (8.34 lb/mil gal)/(1 mg/L) x 3 mil gal/day = 1250 lb/d

4. 30 ac-ft x 43,560 ft^3/ac-ft x 7.48 gal/ft^3 = 9,775,000 gal
   50 lb/9.775 mil gal x (1 mg/L)/(8.34 lb/mil gal) = 0.6 mg/L

5. 1 L of water has a mass of 1 kg
   2 ppm = 2 mg/L = 2 kg/ML (multiply mg/L by $10^6/10^6$)
   2 kg/ML x 5 ML = 10 kg of chlorine

6. lb/d = concentration in ppm x flow rate in mgd x 8.34
   lb/d = 0.5 ppm x 25 mil gal/d x (8.34 lb/mil gal)/(1 ppm) ~ 100 lb/d

7. 0.005 mg/0.200 L = 0.025 mg/L = 25 μg/L = 25 ppb

8. BOD_5 = 14 - 6 = 8 mg/L; there apparently is some organic material in the stream, but it is not possible to determine if the organics are from decaying leaves and animal wastes or from sewage, from this one test alone.

9. We could say that the BOD_5 of the stream is at least 14 mg/L, but it could also be higher than that. Since this exceeds 10 mg/L, it is likely that the stream is polluted with sewage.

10. $BOD_t = BOD_L \times (1 - 10^{-kt}\}$ (Equation 4-2)

Since t = 5 d and k = 0.15/d, k x t = 0.75

$BOD_L = 270/(1 - 10^{-0.75}) = 304 \approx 300$ mg/L

11. $BOD_5 = (DO_0 - DO_5) \times 300/V$ (Equation 4-3)

$BOD_5 = (9.2 - 4.7) \times 300/5 = 270$ mg/L

$BOD_L = 270/(1 - 10^{-0.14 \text{ X } 5}) = 337 \approx 340$ mg/L

12; $BOD_5 = 280 \times (1 - 10^{-0.1 \text{ X } 5}) = 190$ mg/L (Eq. 4-2)

$190 = (9.0 - DO_5) \times 300/5$ (Eq. 4-3)

$DO_5 = 9.0 - (190/60) = 9.9 - 3.2 = 5.8$ mg/L

13. TDS = (A - B) x 1000/C (Equation 4-4)

TDS = (38 845 mg - 38 820 mg) x 1000/50 ml = 500 mg/L

14. TSS = (A - B) x 1000/C (Equation 4-4)

TSS = (580 - 545) x 1000/100 = 350 mg/L

(560 - 545)/(580 - 545) x 100 = 43% volatile solids

15. coliforms/100 mL = 22 x 100/10 = 220 per 100 mL

16. From Table 4.4, the MPN of this sample = 120 per 100 ml

CHAPTER 5 WATER POLLUTION

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(1) 123	(8) 128	(14) 133	(21) 138	(28) www.epa.gov/owow/iakes/lakes.htm/
(2) 124	(9) 129	(15) 134	(22) 140	(29) www.epa.gov/owow/estuaries/about1.html
(3) 124	(10) 130	(16) 135	(23) 140	(30) www.epa.gov/owowwtr1
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Solutions to Practice Problems

1. $c_d = (c_sQ_s + c_WQ_W)/(Q_s + Q_W)$ (Equation 5-1)

TDS = (100 x 6 + 500 x 1.5)/(6 + 1.5) = 177 ≈ 180 mg/L

2. $c_d = (c_sQ_s + c_WQ_W)/(Q_s + Q_W)$ (Equation 5-1)

$10 = (3 \times Q_s + 100 \times 1.5)/(Q_s + 1.5)$

$10Q_s + 15 = 3Q_s + 150$ $Q_s = 135/7 = 19.3 \approx 20$ ML/d

3. $D_i = 11 - 8 = 3$ mg/L; with Equation 5-2, find critical time as follows:

$t_c = \{1/(0.4 - 0.1)\} \times \log\{(0.4/0.1) \times (1 - 3 \times (0.4 - 0.1)/(0.1 \times 25))\} =$

$(1/0.3) \times \log\{4 \times (1 - 0.36)\} = (1/0.3) \times \log 2.56 = 1.36$ days

Using Equation 5-3, solve for the critical oxygen deficit:

$D_c = (0.1 \times 25/0.4 - 0.1) \times \{10^{-0.1x1.36} - 10^{-0.4x1.36}\} + 3 \times 10^{-0.4x1.36}$

$= 8.33 \times 0.455 + 3 \times 0.286 = 4.6$ mg/L; $DO_{min} = 11 - 4.6 = 6.4$ mg/L

4. $BOD_5 = (6 \times 16 + 28 \times 4)/(16 + 4) = 10.4$ mg/L (Equation 5-1)

From Equation 4-2, $10.4 = BOD_L \times (1 - 10^{-0.1\ x5})$;

Solve for the ultimate BOD: $BOD_L = 10.4/0.68337 = 15.2$ mg/L

Compute the initial DO level and the DO deficit below the mixing zone:

DO = (7 x 16 + 2 x 4)/(16 + 4) = 6.0 mg/L; $DO_i = 10.0 - 6.0 = 4.0$ mg/L

Compute the critical time, using Equation 5-2:

$t_c = 1/(0.3\text{-}0.1)x\log\{(0.3/0.1)x(1\text{-}4.0x(0.3\text{-}0.1)/(0.1x15.2)\} = 0.7631$ d

4. (continued)

Compute the critical oxygen deficit, using Equation 5-3:

D_c = (0.1x15.2/(0.3-0.1)x{$10^{-0.1x0.7631}$ - $10^{-0.3x0.7631}$}

+ 4 x $10^{-0.3x0.7631}$ = 7.6 x 0.279 + 4 x 0.590 = 4.5 mg/L

Minimum DO = 10.0 - 4.5 = 5.5 mg/L

Distance downstream where the minimum DO occurs:

Distance = velocity x time = 0.1 m/s x 0.7631 d x 24 h/d x 3600 s/h
= 6593 m ≈ 6.6 km .

5. DO = (3 x 6 + 10 x 30)/(6 + 30) = 318/36 = 8.8 mg/L

6. BOD_5 = (2 x 30 + 10 x 5)/(2 + 10) = 110/12 = 9.17 mg/L Eq. 5-1

From Equation 4-2, 9.17 = BOD_L x (1 - $10^{-0.1 x5}$);

Solve for the ultimate BOD: BOD_L = 9.17/0.68337 = 13.4 mg/L

Compute the initial DO level and the DO deficit below the mixing zone:

DO = (3 x 2 + 9 x 10)/(2 + 10) = 8.0 mg/L; DO_i = 12.0 - 8.0 = 4.0 mg/L

Compute the critical time, using Equation 5-2:

t_c = 1/(0.3-0.1)xlog{(0.3/0.1)x(1-4.0x(0.3-0.1)/(0.1x13.4)} = 0.412 d
9.89 h ≈ 10h

Compute the critical oxygen deficit, using Equation 5-3:

D_c = (0.1 x13.4/(0.3-0.1)x{$10^{-0.1X0.412}$ - $10^{-0.3x0.412}$}

+ 4 x $10^{-0.3x0.412}$ = 1.053 + 3.01 = 4.1 mg/L

Minimum DO = 12.0 - 4.1 = 7.9 mg/L

Distance downstream where the minimum DO occurs:

Distance = velocity x time = 20 ft/min x 9.89 h 1/60 h/min x 3600 s/h
= 11,868 ft ≈ 2.2 miles

CHAPTER 6 DRINKING WATER PURIFICATION

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(1) 150	(9) 159	(17) 164	(25) 169	(33) 176
(2) 150	(10) 158	(18) 164	(26) 170	(34) www.idadesa.org/pdf/ABCs1.pdf
(3) *	(11) 160	(19) 166	(27) 171	(35) www.awwa.org/mstream.htm
(4) 151	(12) 160	(20) 167	(28) 175	(36) www.epa.gov/ogwdw/pwsinv.html
(5) 151	(13) 161	(21) 166	(29) 174	(37) www.envirosources.com
(6) 151	(14) 161	(22) 168	(30) 174	
(7) 155	(15) 161	(23) 168	(31) 175	
(8) 157	(16) 163	(24) 168	(32) 175	

* Community systems are public systems that serve year-round residents while noncommunity systems serve travelers or intermittent users (e.g., camping sites and motels).

Solutions to Practice Problems

1. Tank volume $V = \pi(50)^2/4 \times 9 \times 7.48$ gal/ft^3 = 132,183 gal
   Eq. 6-1: detention time $T_D = V/Q$ = 132,183 gal/15,000 gal/d ≈ 8.8 d
   (This is an unrealistic time for practical purposes. Typical detention times are about 2 h.)

2. From Eq. 6-1, $V = T_D \times Q$ = 3 h x (1 d/24 h) x 10 ML/d = 1.25 ML

   V = 1.25 ML = (1.25 x 10^6 L) x (1 m^3/1000 L) = 1250 m^3
   SWD = 1250 m^3/(10 x 25) m^2 = 5 m

3. V_O = 500 gal/d ft^2 x 1 ft^3 /7.48 gal = 66.8 ft/d

   66.8 ft/d x 1 d/24 h x 1 h/60 m x 12 in./ft ≈ 0.5 in./min

4. Eq. 6-1: $V = T_D \times Q$ = 3 h x 2 mil gal/d x 1 d/24 h = 0.25 mil gal

   V = 250,000 gal x 1 ft^3 /7.48 gal = 33,400 ft^3

   From Eq. 6-2: $A_S = Q/V$ = 2,000,000/800 = 2500 ft^2

   Since $A = \pi D^2/4$, we get diameter $D = \sqrt{4A/\pi}$ and
   $D = \sqrt{4 \times 2500/\pi}$ = 56.4 ft; SWD = 33,400/2500 = 13.4 ft