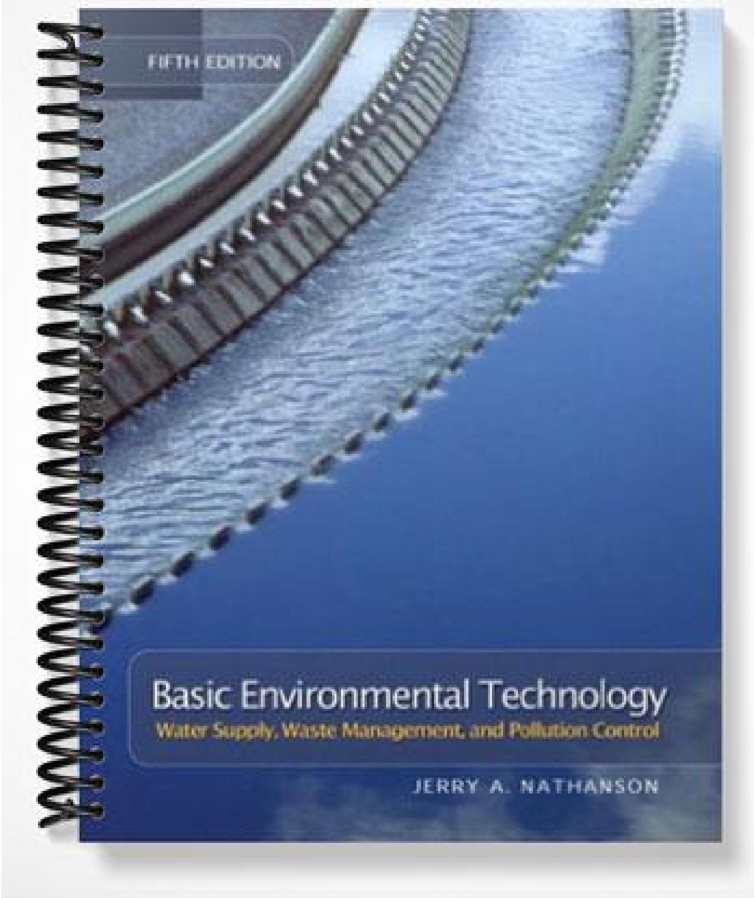
SOLUTIONS MANUAL



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

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(There are no Practice Problems for Chapter 1)

CHAPTER 2 HYDRAULICS

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(23) www.usbr.gov/wrrl (24) www.envirosources.com

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: $h = (78$ m $- 50$ m$) + 2$ 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 $V = Q/A$
   $A = \pi D^2/4 = \pi\,(0.3)^2/4 = 0.0707\ m^2$
   $100\ L/s \times 1\ m^3/1000L = 0.1\ m^3/s$
   $V = 0.1\ m^3/s\ /\ 0.707m^2 = 1.4\ m/s$

7. $Q = (500$ gal/min$) \times (1$ min/60 sec$) \times (1\ ft^3/7.5$ gal$) = 1.11$ cfs
   $A = Q/V$ (from Eq. 2-4) .
   $A = 1.11\ ft^3/sec\ /1.4\ ft/sec = 0.794\ ft^2$
   $A = \pi D^2/4$, 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 m/s) x (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= $\pi(0.2)^2/4 = 0.03142\ m^2$

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

$= 0.04712\ m^3/s$ (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)

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

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

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

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

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

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

$V_1 = 0.05/0.0707 = 0.70721$ m/sec $\quad$ $V_2 = 0.05/0.00785 = 6.369$ 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 $h_L = S \times L = 0.0013 \times 1000$ m = 1.3 m

Pressure drop $p = 9.8 \times 1.3 = 12.7 \approx 13$ kPa per km

13. $h_L = 2.3 \times 20 = 46$ 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 $Q \approx 100$ 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 $A^2 = \pi(0.075)^2/4 = 0.00442\ m^2$

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

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

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

(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