

Concrete Pressure Pipe

MANUAL OF WATER SUPPLY PRACTICES

M9



Third Edition



**American Water Works
Association**

The Authoritative Resource on Safe Water®

Advocacy
Communications
Conferences
Education and Training
Science and Technology
Sections



Documents
Document Sharing Hub



**American Water Works
Association**

Errata to
AWWA Manual M9, *Concrete Pressure Pipe*, 3rd ed.
(December 2014)

1. On pages 36, 40, 43, 47, and 48, replace the definition for the notation K with the following:

$$\begin{aligned} K &= \tan(45^\circ - \phi'/2) = \text{Rankine's active lateral pressure coefficient} \\ &= \text{ratio of active lateral pressure to vertical pressure} \\ &\text{where } \phi' = \text{friction angle of soil} \end{aligned}$$

2. On page 48, replace the definition for the notation c with:

$$c = \text{cohesion intercept of soil, lb/ft}^2$$

3. On page 49, the table heading for Table 5-2 should read "Design values of cohesion intercept."

4. On page 103, Eq 8-5 should read:

$$EI = \frac{1}{12} E_c t_l^3 + E_s t_s + \frac{1}{12} E_c t_c^3$$

Below Eq 8-5 and under "Where" delete the definition of I_s and add the definition of t_s as follows:

$$t_s = \text{steel cylinder thickness, in.}$$

Below Eq 8-5 and below the definitions, delete the paragraph within this same section that begins "For unreinforced steel cylinders, I_s would be equal . . ."

5. On page 108, Eq 8-12, replace d_{yo} with d_{yi} . The equation should read:

$$A_1 = \frac{d_{yi} + 2T_y}{\sin \Delta} (t_y - t_r)$$

Below the equation and under “Where” replace the definition of d_{yo} with the definition of d_{yi} as follows:

d_{yi} = branch cylinder inside diameter, in.

Below the equation Eq 8-14 and under “Where” add the definition of d_{yo} under the definition of w by adding “where d_{yo} = branch cylinder outside diameter, in.”

6. On page 109, in Figure 8-10, diagram (A), replace the dimension $\frac{d_{yo} + 2T_y}{2 \sin \Delta}$ with $\frac{d_{yi} + 2T_y}{2 \sin \Delta}$.

7. On pages 130, 142, and 152, the equation Eq 9-5B should read:

$$W_e = \gamma \left[\frac{D_o H}{12} + \left(1 - \frac{\pi}{4} \right) \left(\frac{D_o^2}{12^2 \times 2} \right) \right]$$

8. On page 132, at the top of the page, “ D_o = pipe outside diameter, ft” should read

“ D_o = pipe outside diameter, in.”

9. On page 133, in Fig. 9-10, reverse the direction of the moment arrow for M_o acting on the upper portion of the free-body diagram.

10. On page 135, in the first paragraph below Eq 9-11A, in the third line replace $e^{\lambda x}$ with $e^{-\lambda x}$ so that the equation reads:

$$M = \frac{V_o}{2\lambda} \left[e^{-\lambda x} (\cos \lambda x - \sin \lambda x) \right]$$

11. On page 136, in Table 9-1, for Soil Type IV, under the column for Soil Properties change the coefficient of friction between pipe and soil from $\mu = 0.5$ to $\mu = 0.4$.

12. On page 140, in the second paragraph under the section on “Effects of Temperature and Poisson’s Ratio,” the second sentence should read: “They do affect the stresses away from the bend; however, a linear reduction in required cylinder thickness is sufficiently conservative to account for the stresses in the pipe wall due to temperature and Poisson’s ratio effects (Zarghamee et al. 2004).”

13. On page 141, at the top of the page, correct the expression to read as follows:

$$\text{Dead weight resistance per foot of pipe} = (W_p + W_f + W_e) \cos \left(\alpha - \frac{\Delta}{2} \right)$$

14. On page 141, correct the current Eq 9-14 to read as follows:

$$L_{ft} = \frac{PA \sin \frac{\Delta}{2}}{(W_p + W_f + W_e) \cos \left(\alpha - \frac{\Delta}{2} \right)}$$

Change the equation number Eq 9-14 to Eq 9-12, and change Eq 9-15 to Eq 9-13. Change all text that references Eq 9-15 to Eq 9-13. (These are: page 145 last paragraph, page 150 last paragraph, and page 165 middle of the page in paragraph addressing F_{ji})

15. On page 143, Table 9-2, first column, under Bend Conditions, change “Bend length, l_b , in.” to “Half bend length, l_b , in.”

16. On page 147, the equation under “Weight of soil above pipe:” should read as follows:

$$W_e = \gamma_s \left[\frac{D_o H}{12} + \left(1 - \frac{\pi}{4} \right) \left(\frac{D_o^2}{12 \times 2} \right) \right]$$

17. On page 148: under step #5 and under “Where:” make the following corrections:

- In the first line where the notation a is defined, delete $\frac{\text{in.}}{\text{lb}}$ at the end of the bracket.
- In the second line, delete $\frac{\text{in.}}{\text{lb}^2}$ at the end of the bracket and add $\frac{\text{lb}}{\text{in.}}$ to the end of the line
(i.e., $5.253 \times 10^5 \frac{\text{lb}}{\text{in.}}$).
- In the third line where the notation b is defined, delete $\frac{\text{in.}}{\text{lb}^2}$ at the end of the bracket.
- In the fourth line, delete $\frac{\text{in.}}{\text{lb}^2}$ at the end of the bracket and add $\frac{\text{lb}^2}{\text{in.}}$ at the end of the line
(i.e., $2.517 \times 10^{11} \frac{\text{lb}^2}{\text{in.}}$).
- In the fifth line where the notation c is defined, delete $\frac{\text{in.}}{\text{lb}^3}$ at the end of the bracket.
- In the sixth line, delete $\frac{\text{in.}}{\text{lb}^3}$ at the end of the bracket and add $\frac{\text{lb}^3}{\text{in.}}$ to the end of the seventh line
(i.e., $-1.683 \times 10^{17} \frac{\text{lb}^3}{\text{in.}}$).

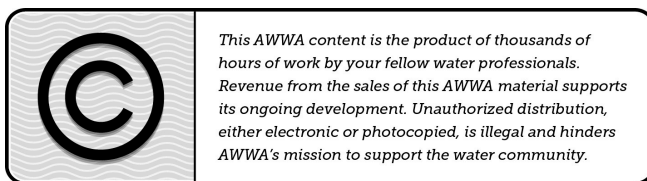
Concrete Pressure Pipe

AWWA MANUAL M9

Third Edition



**American Water Works
Association**



MANUAL OF WATER SUPPLY PRACTICES — M9, Third Edition

Concrete Pressure Pipe

Copyright © 1979, 1995, 2008 American Water Works Association

All rights reserved. No part of this publication may be reproduced or transmitted in any form or by any means, electronic or mechanical, including photocopy, recording, or any information or retrieval system, except in the form of brief excerpts or quotations for review purposes, without the written permission of the publisher.

Disclaimer

The authors, contributors, editors, and publisher do not assume responsibility for the validity of the content or any consequences of their use. In no event will AWWA be liable for direct, indirect, special, incidental, or consequential damages arising out of the use of information presented in this book. In particular, AWWA will not be responsible for any costs, including, but not limited to, those incurred as a result of lost revenue. In no event shall AWWA's liability exceed the amount paid for the purchase of this book.

Project Manager and Technical Editor: Franklin S. Kurtz

Production: Melanie Schiff

Manual Coordinator: Beth Behner

Unless otherwise noted, all photographs used in this manual are provided courtesy of the American Concrete Pressure Pipe Association.

Library of Congress Cataloging-in-Publication Data

Concrete pressure pipe. -- 3rd ed.

p. cm. -- (AWWA manual ; M9)

Includes bibliographical references and index.

ISBN 1-58321-549-2

1. Water-pipes. 2. Pipe, Concrete. 3. Pressure vessels. I. American Water Works Association.

TD491.C66 2008

628.1'5--dc22

2008010413

Printed in the United States of America

American Water Works Association

6666 West Quincy Avenue

Denver, CO 80235

ISBN 1-58321-549-2

978-1-58321-549-4

e-ISBN: 978-1-61300-071-7

Contents

List of Figures, vii

List of Tables, xi

Acknowledgments, xiii

Chapter 1 Purpose and Scope 1

Chapter 2 Description of Concrete Pressure Pipe 3

Prestressed Concrete Cylinder Pipe ([ANSI/AWWA C301-Type Pipe](#)), 3

Reinforced Concrete Cylinder Pipe ([ANSI/AWWA C300-Type Pipe](#)), 7

Reinforced Concrete Noncylinder Pipe ([ANSI/AWWA C302-Type Pipe](#)), 8

Concrete Bar-Wrapped Cylinder Pipe ([ANSI/AWWA C303-Type Pipe](#)), 10

Fittings and Special Pipe, 11

Reference, 12

Chapter 3 Hydraulics 13

Flow Formulas, 13

Effects of Aging on Carrying Capacity, 20

Head Losses, 20

Determining an Economical Pipe Diameter, 21

Air Entrapment and Release, 25

Blowoff Outlets, 25

References, 25

Chapter 4 Surge Pressure. 27

Equations and Variables, 27

Negative Pressures, 30

Causes of Surge Pressure, 30

Control of Water Hammer, 31

References, 32

Chapter 5 External Loads. 33

Major Installation Classifications, 33

Trench Conduits, 33

Embankment Conduits, 37

Positive Projection Installations, 37

Earth Loads on Large Diameter [ANSI/AWWA C303 Pipe](#), 41

Negative Projection Installations, 42	
Induced Trench Installations, 44	
Jacked or Tunneled Conduits, 47	
Determination of Live Load, 48	
References, 56	
Chapter 6 Bedding and Backfilling	57
Introduction, 57	
Rigid Pipe, 57	
Semirigid Pipe, 59	
Unstable Foundations, 60	
References, 61	
Chapter 7 Design of Reinforced Concrete Pressure Pipe	63
Information Required for Pipe Design, 63	
Design Procedure for Rigid Pipe (ANSI/AWWA C300 - and C302 -Type Pipe), 64	
Design Procedure for Semirigid Pipe (ANSI/AWWA C303 -Type Pipe), 86	
References, 93	
Chapter 8 Design of Fittings and Appurtenances.	95
Fittings, 95	
Fittings Design, 98	
Specials, 117	
References, 119	
Chapter 9 Design of Thrust Restraints for Buried Pipe.	121
Thrust Forces, 121	
Hydrostatic Thrust, 121	
Thrust Resistance, 123	
Thrust Blocks, 124	
Joints With Small Deflections, 126	
Tied Joints, 132	
Design Examples, 141	
Combination Thrust-Restraint System, 167	
Other Uses for Restraints, 170	
References, 171	
Chapter 10 Design of Pipe on Piers	173
Loads, 173	

Cylinder Pipe, 173	
Location of Piers, 177	
Design of Pipe and Supports at Piers, 179	
Protection of Exposed Concrete Pipelines, 183	
References, 185	
Chapter 11 Design of Subaqueous Installations.	187
Application, 187	
Pipe Design Features, 187	
Subaqueous Pipe Details, 190	
Installation, 191	
Testing, 194	
Chapter 12 Design Considerations for Corrosive Environments	195
History, 195	
Inherent Protective Properties, 195	
Special Environmental Conditions, 196	
Bonding Pipelines, 199	
Monitoring for Pipe Corrosion, 205	
Cathodic Protection, 213	
References, 214	
Chapter 13 Transportation of Pipe	215
Truck Transportation, 215	
Rail Transportation, 215	
Barge Transportation, 218	
Loading Procedures, 218	
Delivery and Unloading, 218	
Chapter 14 Installation by Trenching or Tunneling—Methods and Equipment	221
Trenching—General Considerations, 221	
Open Trench Construction, 223	
Bedding, 226	
Pipe Installation—General, 227	
Laying the Pipe, 228	
Backfilling, 230	
Tunnel Installations, 232	
Jacking Methods, 233	

Mining Methods, 235	
Grouting, 235	
References, 238	
Chapter 15 Hydrostatic Testing and Disinfection of Mains	239
Hydrostatic Testing, 239	
Disinfection of Mains, 241	
Reference, 242	
Chapter 16 Tapping Concrete Pressure Pipe	243
Threaded Pressure Taps Up to 2 In. (51 mm) in Diameter, 243	
Pressure Taps for Flanged Outlets 3 In. (76 mm) and Larger, 245	
Tapping Noncylinder Reinforced Concrete Pressure Pipe, 248	
Supplemental Data, 248	
Chapter 17 Guide Specifications for the Purchase and Installation of Concrete Pressure Pipe	251
Guide Specifications for Purchasing Concrete Pressure Pipe, 252	
Guide Specifications for the Installation of Concrete Pressure Pipe, 254	
Index	261
List of AWWA Manuals	269

Figures

Figure 1-1	A typical installation in rugged terrain, 1
Figure 2-1	Prestressed concrete cylinder pipe (ANSI/AWWA C301 -type pipe), 5
Figure 2-2	Fabrication of a steel cylinder on a “helical” machine, 6
Figure 2-3	Prestressing concrete embedded cylinder pipe, 6
Figure 2-4	Welding joint rings to cylinder, 7
Figure 2-5	Reinforced concrete cylinder pipe (ANSI/AWWA C300 -type pipe), 8
Figure 2-6	Reinforced concrete noncylinder pipe (ANSI/AWWA C302 -type pipe), 9
Figure 2-7	Concrete bar-wrapped cylinder pipe (ANSI/AWWA C303 -type pipe), 11
Figure 2-8	Steel cylinder being hydrostatically tested on a mandrel-type tester, 11
Figure 2-9	Installation of a large-diameter wye with reducer attached, 12
Figure 3-1	Hydraulic profile for a gravity flow system, 15
Figure 3-2	Hydraulic profile for a pumped flow system, 15
Figure 3-3	The Moody diagram for friction in pipe, 18
Figure 3-4	Approximate loss coefficients for commonly encountered flow configurations, 22
Figure 4-1	Idealized surge cycle for instantaneous pump shutdown, 28
Figure 5-1	Underground conduit installation classifications, 34
Figure 5-2	Essential features of types of installations, 35
Figure 5-3	Settlements that influence loads—positive projecting embankment installation, 38
Figure 5-4	Settlements that influence loads—negative projecting embankment installation, 38
Figure 5-5	Settlements that influence loads—induced trench installation, 39
Figure 5-6	Load coefficient for positive projection embankment condition, 43
Figure 5-7	Load coefficient for negative projection and induced trench condition ($p' = 0.5$), 45
Figure 5-8	Load coefficient for negative projection and induced trench condition ($p' = 1.0$), 45
Figure 5-9	Load coefficient for negative projection and induced trench condition ($p' = 1.5$), 46
Figure 5-10	Load coefficient for negative projection and induced trench condition ($p' = 2.0$), 46
Figure 5-11	Live load distribution, 50
Figure 5-12	Live load spacing, 50
Figure 5-13	Wheel load surface contact area, 51
Figure 5-14	Distributed load area—single dual wheel, 52
Figure 5-15	Effective supporting length of pipe, 53
Figure 5-16	Cooper E80 design load, 56
Figure 6-1	Bedding and backfill for rigid pipe, 58
Figure 6-2	Bedding and backfill for semirigid pipe, 60
Figure 7-1	Olander moment, thrust, and shear coefficients for 90° bedding angle, 72
Figure 7-2	Force diagrams for reinforced concrete pipe design, 73
Figure 7-3	Section for proportioning tensile steel, 74
Figure 7-4	Section through AWWA C303 -type pipe wall, 92

Figure 8-1	Common fittings, 96
Figure 8-2	A 90-in. (2,290-mm) bend ready to install, 97
Figure 8-3	A reducer or increaser makes a gradual change in ID of the line, 97
Figure 8-4	A tee or cross is used for 90° lateral connections, 98
Figure 8-5	Bifurcations for splitting flow, 99
Figure 8-6	Various types of reinforcing, 100
Figure 8-7	Adapters used to connect pipe to valves, couplings, or pipe of a different material, 101
Figure 8-8	Typical end configurations, 102
Figure 8-9A	Typical four-piece bend, 105
Figure 8-9B	Free body diagram of one-fourth of a miter segment and fluid, 105
Figure 8-9C	Schematic of A_{trap} , 105
Figure 8-10	Replacement of steel at openings in fabricated fittings requiring collar or wrapped type of reinforcement, 109
Figure 8-11	Assumed hydrostatic load distribution for an equal-diameter wye with two-crotch-plate reinforcement, 110
Figure 8-12	Nomograph for selecting reinforcement plate depths of equal-diameter pipes, 111
Figure 8-13	N -factor curves for branch deflection angles less than 90°, 112
Figure 8-14	Q -factor curves for branch diameter smaller than run diameter, 112
Figure 8-15	Selection of top depth, 113
Figure 8-16	Wye-branch reinforcement plan and layout, 115
Figure 8-17	Three-plate wye-branch reinforcement plan and layout, 117
Figure 8-18	Typical outlets built in to pipe, 118
Figure 9-1	Thrusts and movement in pipeline, 122
Figure 9-2	Hydrostatic thrust T applied by fluid pressure to typical fittings, 122
Figure 9-3	Typical thrust blocking of a horizontal bend, 125
Figure 9-4	Typical profile of vertical-bend thrust blocking, 126
Figure 9-5	Tee and reducer on large-diameter line. Note sandbags behind tee as forms for placement of thrust block, 127
Figure 9-6	Locking of pipe at small deflections, 128
Figure 9-7	Four sections of unrestrained 36-in. (910-mm) prestressed concrete pipe span an 80-ft (24-m) wide washout in Evansville, Ind., 128
Figure 9-8A	Restraint of thrust at deflected, nontied joints on long-radius horizontal curves, 129
Figure 9-8B	Horizontal bearing factor for sand vs. depth-to-diameter ratio (H/D_o)(adapted from O'Rourke and Liu [1999]), 129
Figure 9-9A	Restraint of uplift thrust at nontied, deflected joints on long-radius vertical curves, 131
Figure 9-9B	Earth load for restrained joint calculations, 131
Figure 9-10	Free body diagram of forces and deformations at a bend, 133
Figure 9-11	Schematic strain and stress distributions at cylinder yield design criterion (ϵ_y = steel cylinder yield strain, f_y = steel cylinder yield stress, f_s = steel cylinder stress, f_c = concrete stress), 139
Figure 9-12	Schematic strain and stress distributions at ultimate strength limit state (ϵ_y = steel cylinder yield strain, f_y = steel cylinder yield stress, f_s = steel cylinder stress, f_c = concrete

	stress, f'_c = concrete stress), 139
Figure 9-13	Interaction diagram for example 2. The axial force-moment interaction diagrams corresponding to the ultimate strength and the onset of yielding of the steel cylinder are both nonlinear, but have been approximated by a linear relationship here for ease of computation of the examples, 150
Figure 9-14	Infinite beam on elastic foundation with applied forces and moments at the ends of a beam segment, 154
Figure 9-15	Infinite beam on elastic foundation with applied force and moment at the end of a semi-infinite beam, 154
Figure 9-16	Schematic of mechanically harnessed joint with joint rotation, 155
Figure 9-17	Moment-rotation diagram, 156
Figure 9-18	Displacement along the pipe length, 161
Figure 9-19	Moment along the pipe length, 161
Figure 9-20	Rotation along the pipe length, 162
Figure 9-21	Shear along the pipe length, 162
Figure 9-22	Interaction diagram for example 3. The axial force-moment interaction diagrams corresponding to the ultimate strength and the onset of yielding of the steel cylinder are both nonlinear, but have been approximated by a linear relationship here for ease of computation of the examples, 163
Figure 9-23A	Thrust restraint requirements for 36 in. C301(LCP) bulkhead with restrained joints at $P_w = 150$ psi, $P_t = 80$ psi, and $P_f = 200$ psi with 4-ft earth cover in type III soil, 166
Figure 9-23B	Thrust restraint requirements for 54 in. C303 45° bend with welded restrained joints at $P_w = 200$ psi, $P_t = 100$ psi, and $P_f = 250$ psi with 6-ft earth cover in type II soil, 166
Figure 9-23C	Thrust restraint requirements for 42 in. C301 (ECP) 75° bend with mechanically restrained joints at $P_w = 140$ psi, $P_t = 60$ psi, and $P_f = 180$ psi with 5-ft earth cover in type III soil, 167
Figure 9-24	Typical welded pipe joints, 168
Figure 9-25	Details of typical harnessed joints, 169
Figure 9-26	Typical anchor pipe with thrust collar, 170
Figure 10-1	Pipe on piers, 174
Figure 10-2	Pipe elements assumed effective in resisting bending, 175
Figure 10-3	Configuration of pile-supported installations with joints offset from the supports, 178
Figure 10-4	Typical supports for pipe on piling, 180
Figure 10-5	Assumed load and force configuration for design of rigid pipe on piers, 181
Figure 10-6	50-ft (15-m) spans of 42-in. (1,070-mm) concrete pressure pipe were used in this aerial line crossing of a highway and river levee, 184
Figure 11-1	54-in. (1,370-mm) diameter pipe, preassembled to a 32-ft (9.75-m) unit, being lowered with a double bridge sling in a sewer outfall installation, 188
Figure 11-2	Typical subaqueous joint-engaging assembly, 191
Figure 11-3	Five 20-ft (6-m) sections of 84-in. (2,130-mm) diameter pipe were assembled on deck into a 100-ft (30-m) unit. The 100-ft (30-m) unit was then mounted on precast concrete caps and cradles and installed as a single unit on piles. Two derricks were used to lower the strongback, cradles, caps, and the 100-ft (30-m) pipe unit, 192
Figure 11-4	Large-diameter pipe being lifted with a special double bridge sling. Joint-engaging

	assemblies were installed at the ends of the pipe at the springline, 192
Figure 11-5	Rail-mounted gantry crane used for installing sections of ocean outfall near shore, 193
Figure 12-1	Typical joint bonding details for embedded cylinder (AWWA C301-type) pipe, 200
Figure 12-2	Typical joint bonding details for AWWA C303-type pipe or lined cylinder AWWA C301-type pipe, 201
Figure 13-1	Unloading a typical flatbed truck trailer loaded with 36-in. (910-mm) concrete pressure pipe, 216
Figure 13-2	Special double-drop highway truck trailer loaded with a single large-diameter pipe, 216
Figure 13-3	Flatbed piggyback truck trailers loaded with small-diameter pipe, 217
Figure 13-4	Rail flat cars loaded with 144-in. (3,660-mm) diameter concrete pressure pipe, 217
Figure 13-5	Oceangoing barge carrying approximately 18,000 ft (5,486 m) of concrete pressure pipe, 218
Figure 13-6	Specialized equipment for handling and transporting 252-in. (6,400-mm) (ID) pipe weighing 225 tons (204 metric tons) each, 219
Figure 14-1	Backhoe laying pipe, 224
Figure 14-2	Clamshell excavating inside sheeting, 225
Figure 14-3	Lowering 84-in. (2,140-mm) pipe to final bedding position in laying shield, 225
Figure 14-4	Pipe being laid on granular foundation, 227
Figure 14-5	Pipe bedding, 227
Figure 14-6	Applying lubricant to the gasket prior to installation, 229
Figure 14-7	Grouting a pipe joint, 231
Figure 14-8	Backfilling, 232
Figure 14-9	Tunnel casing pipe (132 in. [3,350 mm] in diameter) being jacked under a railroad using a metal push ring with a plywood cushion to protect the face of the pipe, 234
Figure 14-10	Concrete pressure pipe as a carrier pipe inside a casing, 236
Figure 14-11	Typical tunnel sections, 237
Figure 14-12	Special equipment, called a “tunnelmobile,” being used to install large-diameter pipe in a tunnel, 237
Figure 15-1	Temporary internal test plug, 241
Figure 16-1	Small-diameter pressure tap using saddle with straps, 244
Figure 16-2	Small-diameter pressure tap using split studs, 245
Figure 16-3	Installation of a small-diameter threaded pressure tap using a saddle with straps, 246
Figure 16-4	Large-diameter tapping assembly, 247
Figure 16-5	Pressure tapping a 12-in. (300-mm) diameter flanged outlet on a larger-diameter pipe, 247
Figure 16-6	Pressure tap of a 12-in. (300-mm) diameter flanged outlet on a 24-in. (600-mm) diameter pipe, 248
Figure 16-7	Typical pressure-tapping site requirements, 249
Figure 17-1	30-in. (750-mm) pipe ready for delivery, 255
Figure 17-2	24-in. (600-mm) lined cylinder pipe strung along a right-of-way, 256
Figure 17-3	Prestressed concrete cylinder pipe being lowered into position. Note depth of embankment cut in background, 257

Tables

Table 2-1	General description of concrete pressure pipe, 4
Table 3-1	Various forms of the Hazen–Williams formula, 14
Table 3-2	Comparison of theoretical Hazen–Williams C_h values to tested C_h values, 16
Table 3-3	Kinematic viscosity of water, 19
Table 3-4	Equivalent length formulas, 23
Table 4-1	Design values—surge wave velocities, <i>ft/sec (m/sec)</i> , 31
Table 5-1	Design values of settlement ratio, 42
Table 5-2	Design values of coefficient of cohesion, 49
Table 5-3	Impact factors for highway truck loads, 51
Table 5-4	Critical loading configurations, 51
Table 5-5	Highway loads on circular pipe (pounds per linear foot), 54
Table 5-6	Cooper E80 railroad loads on circular pipe (pounds per linear foot), 55
Table 7-1	Tabulation of strength method design for ANSI/AWWA C300 -type pipe, 77
Table 7-2	Tabulation of strength method design for ANSI/AWWA C302 -type pipe, 84
Table 7-3	Allowable external load for ANSI/AWWA C303 -type pipe of minimum class, 88
Table 9-1	Soil type selection guide, 136
Table 9-2	Summary of design examples and results of detailed calculation, 143
Table 9-3	Summary of additional design examples, 144
Table 12-1	Assumed values for required joint bonds shown in Tables 12-2, 12-3, and 12-4, 205
Table 12-2	Number of bonds for 48-in. prestressed concrete cylinder pipe, 206
Table 12-3	Number of bonds for 84-in. prestressed concrete cylinder pipe, 208
Table 12-4	Number of bonds for 144-in. prestressed concrete cylinder pipe, 210

This page intentionally blank.

Acknowledgments

Committee Personnel

The subcommittee that reviewed and developed this edition of AWWA Manual M9 had the following personnel at the time:

David P. Prosser, Chair

Sam A. Arnaout

Wayne R. Brunzell

David Marshall

Andrew E. Romer

Armand W. Tremblay

Mehdi S. Zarghamee

The Standards Committee on Concrete Pressure Pipe, which reviewed and approved this manual, had the following personnel at the time of approval:

Wayne R. Brunzell, Chair

David P. Prosser, Secretary

General Interest Members

<i>S.J. Abrera Jr.,*</i> South Pasadena, Calif.	(AWWA)
<i>D.T. Bradley,†</i> SC Liaison, Oak Lodge Water District, Milwaukie, Ore.	(AWWA)
<i>W.R. Brunzell,</i> Brunzell Associates Ltd., Skokie, Ill.	(AWWA)
<i>W.R. Dana,*</i> Corona Del Mar, Calif.	(AWWA)
<i>R.C. Edmunds,</i> Jones Edmunds and Associates, Gainesville, Fla.	(AWWA)
<i>L.B. Freese,</i> Freese & Nichols Inc., Fort Worth, Texas	(AWWA)
<i>J.K. Haney,</i> HDR Engineering Inc., Austin, Texas	(AWWA)
<i>M.M. Hicks,</i> MWH Americas Inc., Walnut Creek, Calif.	(AWWA)
<i>J.K. Jeyapalan,*</i> Pipeline Consultant, New Milford, Conn.	(AWWA)
<i>L.R. Keyser,*</i> Fort Myers Beach, Fla.	(AWWA)
<i>R.Y. Konyalian,</i> Consulting Engineer, Huntington Beach, Calif.	(AWWA)
<i>S.A. McKelvie,</i> Parsons Brinckerhoff Quade & Douglas, Boston, Mass.	(AWWA)
<i>H.L. Murphey,*</i> Colleyville, Texas	(AWWA)
<i>T. Niemann,</i> Elizabeth Niemann & Associates, Louisville, Ky.	(AWWA)
<i>P.J. Olson,†</i> Staff Engineer Liaison, AWWA, Denver, Colo.	(AWWA)
<i>R. Ortega,*</i> Lockwood Andrews & Newnam, Houston, Texas	(AWWA)
<i>J.J. Roller,</i> CTL Group, Skokie, Ill.	(AWWA)
<i>A.E. Romer,</i> Boyle Engineering Corp., Newport Beach, Calif.	(AWWA)

* Nonvoting

† Liaison, nonvoting

<i>D.M. Schultz,*</i> Schultz & Associates, Princeville, Hawaii	(AWWA)
<i>C.C. Sundberg,</i> CH2M Hill, Issaquah, Wash.	(AWWA)
<i>F.B. Yuen,*</i> Markham, Ont.	(AWWA)
<i>M.S. Zarghamee,</i> Simpson, Gumpertz, & Heger Inc., Waltham, Mass.	(AWWA)

Producer Members

<i>J.O. Alayon,</i> Atlantic Pipe Corp., San Juan, P.R.	(AWWA)
<i>S.A. Arnaout,</i> Hanson Pipe & Products Inc., Dallas, Texas	(AWWA)
<i>H. Bardakjian,</i> Ameron International, Rancho Cucamonga, Calif.	(AWWA)
<i>G. Bizien,</i> Hyprescon Inc., St. Eustache, Que.	(AWWA)
<i>D. Dechant,</i> Northwest Pipe Company, Denver, Colo.	(AWWA)
<i>S.R. Malcolm,</i> Vianini Pipe Inc., Somerville, N.J.	(AWWA)
<i>D.P. Prosser,</i> American Concrete Pressure Pipe Association, Reston, Va.	(ACPPA)
<i>A.W. Tremblay,</i> Price Brothers Company, Dayton, Ohio	(AWWA)
<i>J.P. Zicaro,*</i> CSR Hydro Conduit, Houston, Texas	(AWWA)

User Members

<i>B.M. Bradish,</i> City of Portsmouth, Portsmouth, Va.	(AWWA)
<i>J. Galleher,</i> San Diego County Water Authority, Escondido, Calif.	(AWWA)
<i>J. Keith,</i> Bureau of Reclamation, Denver, Colo.	(AWWA)
<i>D. Marshall,</i> Tarrant Regional Water District, Fort Worth, Texas	(AWWA)
<i>V. Soto,</i> Los Angeles Dept. of Water & Power, Los Angeles, Calif.	(AWWA)
<i>D.A. Wiedyke,</i> Clinton Twp., Mich.	(AWWA)

* Nonvoting



Chapter 1

Purpose and Scope

The use of concrete pressure pipe for conveying water and other liquids under pressure has dramatically increased in recent years. Its rugged construction and the natural corrosion resistance provided by embedment of the ferrous components in concrete or cement mortar offer the design engineer solutions to a wide range of structural and environmental problems (Figure 1-1).

The manufacture of four basic types of concrete pressure pipe are covered by the following American Water Works Association (AWWA) standards:

ANSI/AWWA C300 Standard for Reinforced Concrete Pressure Pipe,
Steel-Cylinder Type

ANSI/AWWA C301 Standard for Prestressed Concrete Pressure Pipe,
Steel-Cylinder Type



Figure 1-1 A typical installation in rugged terrain

[ANSI/AWWA C302](#) Standard for Reinforced Concrete Pressure Pipe,
Noncylinder Type

[ANSI/AWWA C303](#) Standard for Concrete Pressure Pipe, Bar-Wrapped,
Steel-Cylinder Type

Preparing pipeline project plans and specifications, however, requires many pipe-related decisions that are not covered by the respective AWWA standards.

This manual provides supplemental information to assist engineers and designers in achieving optimum field performance of concrete pressure pipelines. Information and guidelines are provided covering hydraulics, surge pressure, external loads, bedding, and backfilling; designing reinforced concrete pressure pipe, fittings and appurtenances, thrust restraints, pipe on piers, and subaqueous installations; design considerations for corrosive environments; transportation of pipe; trench and tunnel installation; and other pertinent subjects.

The information in this manual is not intended to supersede, nor should be regarded as superseding, any portion of any AWWA standard.

NOTE: This manual uses US customary units of measurement. Metric equivalents have been added where deemed appropriate.



Chapter 2

Description of Concrete Pressure Pipe

Several types of concrete pressure pipe are manufactured and used in North America. These include prestressed concrete cylinder pipe, reinforced concrete cylinder pipe, reinforced concrete noncylinder pipe, and concrete bar-wrapped cylinder pipe. Each type will be discussed in this chapter. Because of their construction, some types of concrete pressure pipe are made for a specific type of service condition; others are constructed for a broader range of service conditions. The general description of these pipes is based on whether or not the pipe has a full-length steel cylinder and whether it is conventionally reinforced with deformed bar, wire, or smooth bar, or prestressed with high-strength wire (Table 2-1). It should be noted that not all concrete pressure pipe manufacturers make all of these types of pipe.

PRESTRESSED CONCRETE CYLINDER PIPE ([ANSI/AWWA C301-TYPE PIPE](#))

Prestressed concrete cylinder pipe (Figure 2-1) has been manufactured in the United States since 1942 and is the most widely used type of concrete pressure pipe. Applications for this product include transmission mains, distribution feeder mains, water intake and discharge lines, pressure siphons, low-head penstocks, industrial pressure lines (including power plant cooling-water lines), sewer force mains, gravity sewer lines, subaqueous lines (into both freshwater and salt water), and spillway conduits. This high-strength rigid pipe has also been extensively used for conduits under small dams, deep highway fills, and other high-earth-cover projects.

Types of Construction

Prestressed concrete cylinder pipe has the following two general types of construction: (1) a steel cylinder lined with a concrete core, or (2) a steel cylinder embedded in a concrete core (Figure 2-1). In either type of construction, manufacturing begins with a full-length welded steel cylinder (Figure 2-2). Joint rings are attached to each end, and the pipe is hydrostatically tested to ensure watertightness. A concrete core with a minimum thickness of one-sixteenth times the pipe diameter is placed either

Table 2-1 General description of concrete pressure pipe

Type of Pipe	ANSI/AWWA Standard	Steel Cylinder	Noncylinder	Mild Reinforcing Steel	Prestressed Wire	Design Basis*
Reinforced concrete cylinder pipe	C300	X		X		Rigid
Prestressed concrete cylinder pipe	C301	X			X	Rigid
Reinforced concrete non-cylinder pipe	C302		X	X		Rigid
Concrete bar-wrapped cylinder pipe	C303	X		X		Semirigid

* The terms *rigid* and *semirigid* in this table and in the remainder of this manual are intended to differentiate between two design theories. Rigid pipe does not depend on the passive resistance of the soil adjacent to the pipe for support of vertical loads; semirigid pipe does require this passive soil resistance. The terms *rigid* and *semirigid* as used here should not be confused with the definitions stated by Marston (1930).

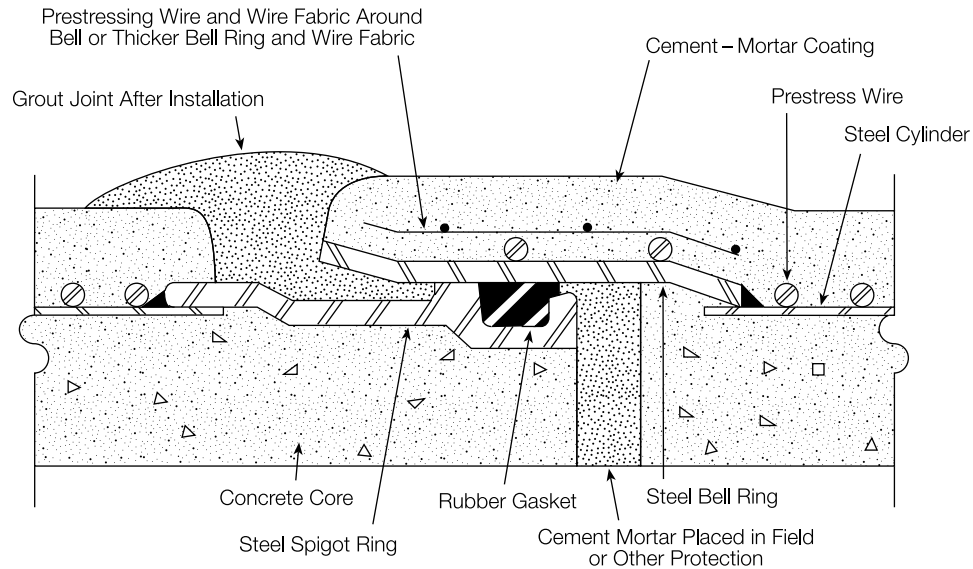
by the centrifugal process, radial compaction, or vertical casting. After the core is cured, the pipe is helically wrapped with high-strength, hard-drawn wire using a stress of 75 percent of the minimum specified tensile strength. The wrapping stress ranges between 150,000 and 189,000 psi (1,034 and 1,303 MPa) depending on the wire size and class. The wire spacing is accurately controlled to produce a predetermined residual compression in the concrete core. The wire is embedded in a thick cement slurry and coated with a dense mortar that has a high cement content.

Size Range

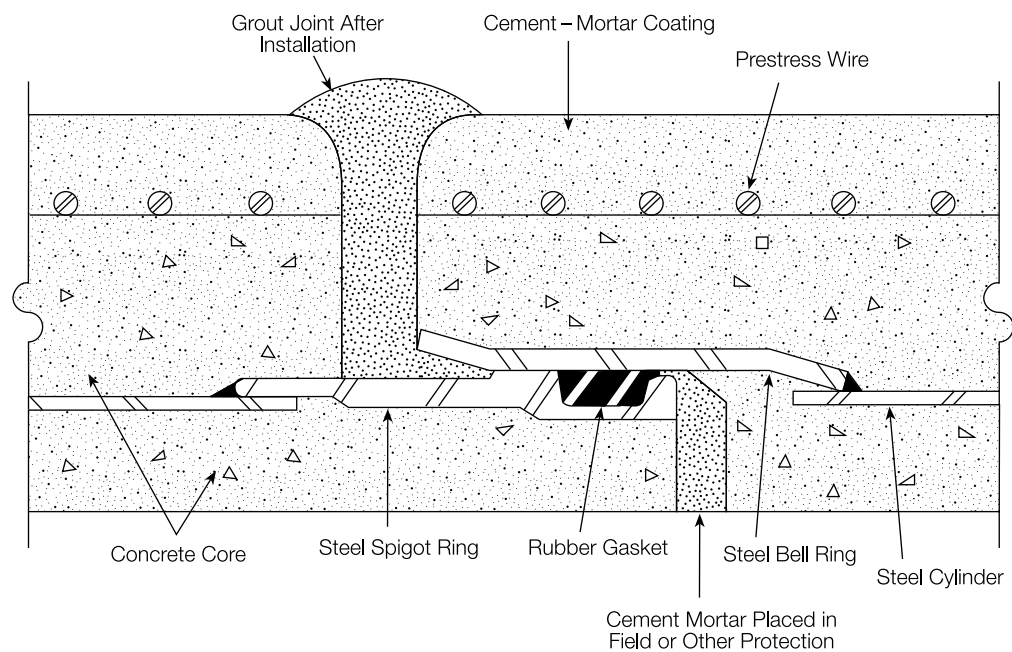
[ANSI/AWWA C301](#) covers prestressed concrete cylinder pipe 16 in. (410 mm) in inside diameter and larger. Lined cylinder pipe is commonly available with inside diameters ranging from 16 through 48 in. (410 through 1,220 mm). Sizes up to 60 in. (1,520 mm) are available from some manufacturers. Embedded cylinder pipe larger than 250 in. (6,350 mm) in diameter has been manufactured and is commonly available with inside diameters 48 in. (1,220 mm) and larger. Lengths are generally 16 to 24 ft (4.9 to 7.3 m), although longer units can be furnished. Shipping requirements usually restrict the weight and length of pipe manufactured.

Design Basis

Prestressed concrete cylinder pipe has been designed for operating pressures greater than 400 psi (2,758 kPa) and earth covers in excess of 100 ft (30 m) (Figure 2-3). The design of this pipe is covered in [ANSI/AWWA C304](#), Standard for Design of Prestressed Concrete Cylinder Pipe. The design method is based on combined loading conditions — the most critical type of loading for rigid pipe — and includes surge pressure and live loads.



A. Lined Cylinder Pipe



B. Embedded Cylinder Pipe

Figure 2-1 Prestressed concrete cylinder pipe (ANSI/AWWA C301-type pipe)

Joints

The standard joint configuration for prestressed concrete cylinder pipe is sealed with a round rubber gasket and steel joint rings, as shown in Figure 2-1. The spigot ring is a hot-rolled steel shape containing a rectangular recess that holds a continuous solid ring gasket of circular cross section. The gasket is compressed by the cylindrical

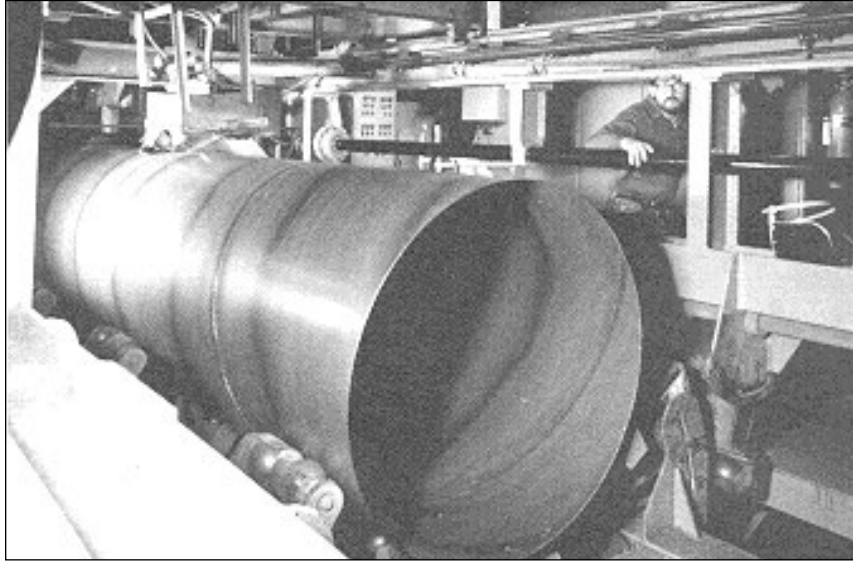


Figure 2-2 Fabrication of a steel cylinder on a “helical” machine

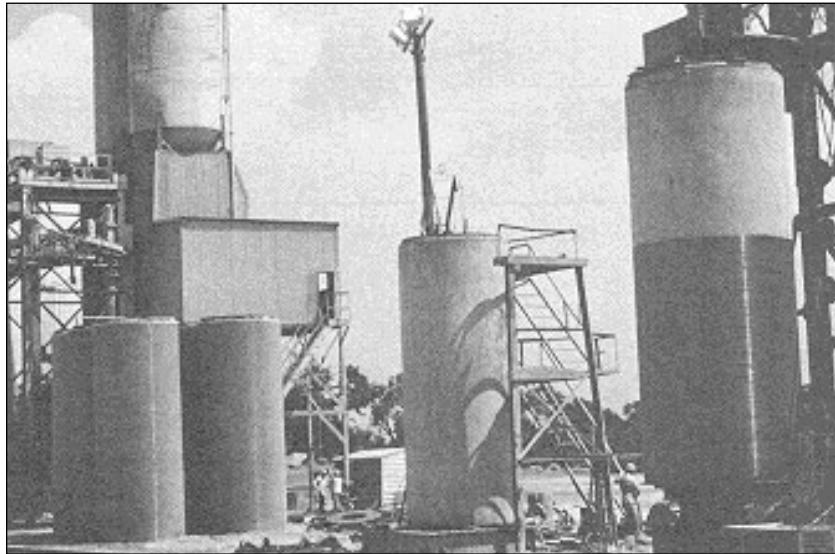


Figure 2-3 Prestressing concrete embedded cylinder pipe

portion of the steel bell ring when the spigot is pushed into the bell. Both joint rings are sized to close tolerances on a hydraulic press by expanding them beyond their elastic limit. The gasket diameter and volume are closely controlled to ensure a reliable high-pressure seal. After field assembly, the exterior joint recess is usually grouted to protect the steel joint rings, but other methods of protecting the joint rings are available. The internal joint may or may not be pointed with stiff mortar, depending on the type of liquid the pipe will carry and the type of protective coating applied to the joint rings during manufacture (Figure 2-4).



Figure 2-4 Welding joint rings to cylinder

REINFORCED CONCRETE CYLINDER PIPE ([ANSI/AWWA C300-TYPE PIPE](#))

Prior to the introduction of prestressed concrete cylinder pipe ([ANSI/AWWA C301-type pipe](#)) in the early 1940s, most of the concrete pressure pipe in the United States was reinforced concrete cylinder pipe. The construction of this pipe is similar to prestressed concrete cylinder pipe except that mild steel reinforcement is cast into the wall of the pipe instead of prestressing with high-strength wire. The minimum wall thickness is one-twelfth the inside diameter.

Manufacture

The manufacture of reinforced concrete cylinder pipe (Figure 2-5) begins with a hydrostatically tested steel cylinder and attached steel joint rings. The cylinder assembly and one or more reinforcing cages are positioned between inside and outside forms, and the concrete is placed by vertical casting.

Size Range

Reinforced concrete cylinder pipe is manufactured in diameters of 30 to 144 in. (760 to 3,660 mm), with larger sizes limited only by the restrictions of transportation to the job site. Standard lengths are 12 to 24 ft (3.7 to 7.3 m).

Design Basis

Design of reinforced concrete cylinder pipe is covered in chapter 7 of this manual. The design procedure addresses external loads and internal pressures individually and in combinations. [ANSI/AWWA C300](#) limits the reinforcing steel furnished in the cage(s) to no less than 40 percent of the total reinforcing steel in the pipe. The maximum loads and pressures for this type of pipe depend on the pipe diameter, wall thickness,

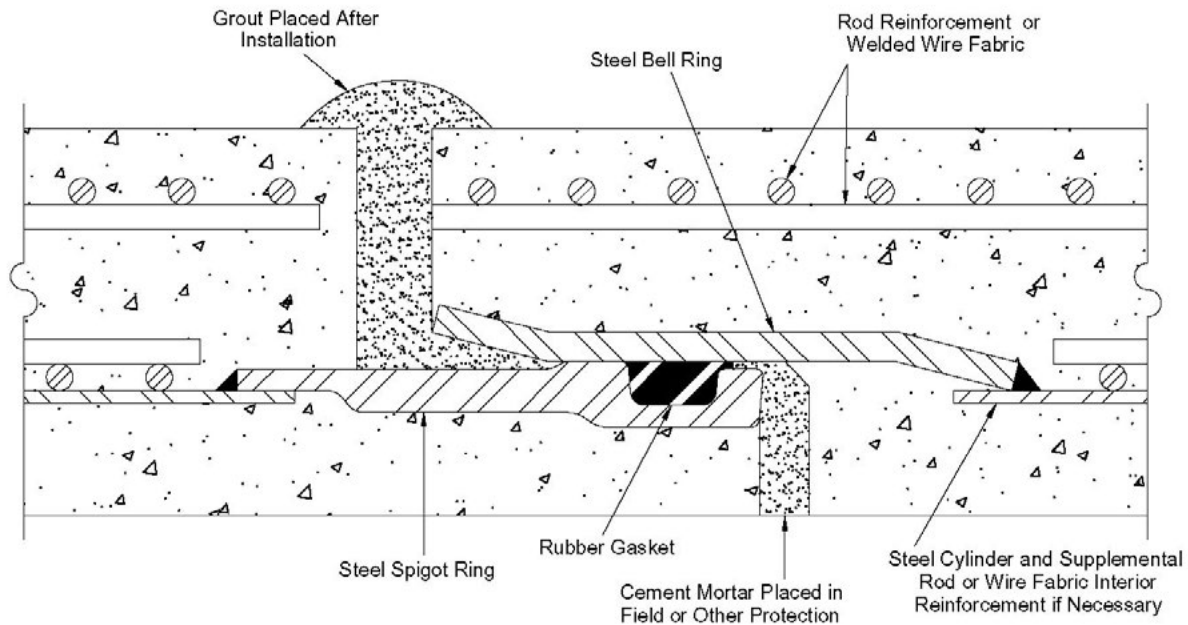


Figure 2-5 Reinforced concrete cylinder pipe ([ANSI/AWWA C300-type pipe](#))

and strength limitations of the concrete and steel. The user should be aware that this type of pipe can be designed for high internal pressure, but is limited in external load capacity.

Joints

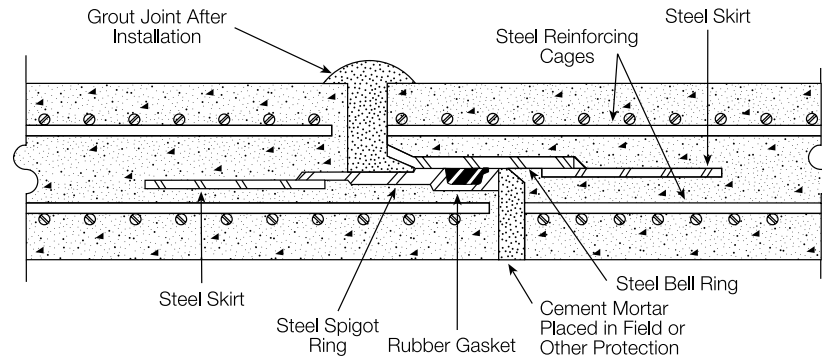
The standard joint configuration for reinforced concrete cylinder pipe, as shown in Figure 2-5, is identical to the joint for prestressed concrete cylinder pipe and consists of steel spigot and bell rings and a round rubber gasket. As with prestressed concrete cylinder pipe ([ANSI/AWWA C301-type pipe](#)), the external joint recess is normally grouted and the internal joint space may or may not be pointed with mortar, depending on the type of liquid the pipe will carry and the type of protective coating applied to the joint rings during manufacture.

REINFORCED CONCRETE NONCYLINDER PIPE ([ANSI/AWWA C302-TYPE PIPE](#))

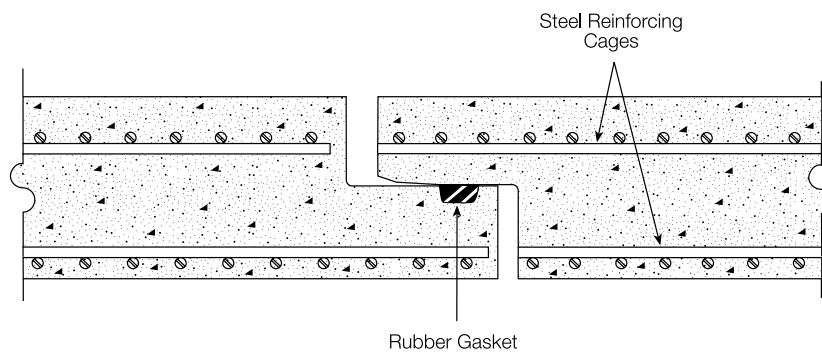
Reinforced concrete noncylinder pipe (Figure 2-6) is suitable for working pressures up to 55 psi (379 kPa). Typical applications are low-pressure transmission lines used for irrigation, industrial or domestic raw water supply and discharge lines, sanitary and storm sewers, and drainage culverts.

Size Range and Design Basis

Reinforced noncylinder pipe is commonly furnished in diameters of 12 to 144 in. (300 to 3,660 mm), but larger diameters can be furnished if shipping limitations permit. Standard lengths are 8 to 24 ft (2.4 to 7.3 m), with [ANSI/AWWA C302](#) limiting the maximum length that can be furnished for each pipe size. The wall thickness is a



A. [ANSI/AWWA C302](#)-Type Pipe With Steel Joint Rings



B. [ANSI/AWWA C302](#)-Type Pipe With Concrete Joint Rings

Figure 2-6 Reinforced concrete noncylinder pipe ([ANSI/AWWA C302](#)-type pipe)

minimum of one-twelfth the inside diameter. The design of reinforced concrete noncylinder pipe is covered in chapter 7 of this manual. The maximum working pressure is limited by [ANSI/AWWA C302](#) to 55 psi (379 kPa).

Joints

The standard joints for reinforced concrete noncylinder pipe are illustrated in Figure 2-6. The joint details are conceptual only and may vary based on design conditions and requirements. As with both reinforced and prestressed concrete cylinder pipe, these joints incorporate a continuous, round, solid rubber ring gasket placed in a spigot groove and compressed by the bell surface. However, the bell and spigot ends of reinforced concrete noncylinder pipe may be steel joint rings or formed concrete surfaces. It is recommended that the pressure sealing capacity of gasketed concrete joints be verified by testing the joint. It is also recommended that the structural capacity of concrete joints to withstand the installation and field loads be verified. If steel joint rings are used, as shown in Figure 2-6A, the grouting procedures are the same as described previously for prestressed concrete cylinder pipe and reinforced concrete cylinder pipe ([ANSI/AWWA C301](#)- and [C300](#)-type pipe, respectively). Concrete joints, as shown in Figure 2-6B, are normally not grouted.

Manufacture

Manufacture of reinforced concrete noncylinder pipe begins with the fabrication of one or more reinforcing cages (depending on pipe size and wall thickness) and the fabrication of steel joint rings if that type of joint is used. The cages may be fabricated of smooth or deformed reinforcing bars, wire, or wire fabric. When concrete is to be placed by the vertical casting method, cages and joint rings are positioned between inner and outer steel forms. When concrete is to be placed by the centrifugal process, cages are positioned with respect to the outer form, and the entire assembly is rotated at high speed while the concrete is placed.

CONCRETE BAR-WRAPPED CYLINDER PIPE ([ANSI/AWWA C303-TYPE PIPE](#))

Concrete bar-wrapped cylinder pipe (Figure 2-7) is manufactured in Canada and in the western and southwestern areas of the United States. It is normally available in diameters of 10 to 72 in. (250 to 1,830 mm), although larger diameters can be manufactured. Standard lengths are generally 24 to 40 ft (7.3 to 12.2 m). Typical applications are cross-country transmission mains, distribution feeder mains, sewer force mains, water intake and discharge lines, and plant piping.

Manufacture

Manufacture of concrete bar-wrapped cylinder pipe begins with a fabricated steel cylinder with joint rings, which is hydrostatically tested (Figure 2-8). A cement-mortar lining is placed by the centrifugal process inside the cylinder. The nominal lining thickness is $\frac{1}{2}$ in. (13 mm) for sizes up to and including 16 in. (410 mm), and $\frac{3}{4}$ in. (19 mm) for larger sizes. After the lining is cured, the cylinder is wrapped with a smooth, hot-rolled steel bar, using moderate tension in the bar. The size and spacing of the bar, as well as the thickness of the steel cylinder, are proportioned to provide the required pipe strength. The cylinder and bar wrapping are covered with a cement slurry and a dense mortar coating that is rich in cement.

Joints

The standard joint configuration shown in Figure 2-7 includes steel joint rings and a continuous, round, solid-rubber ring gasket. The exterior joint recess is normally grouted, and the internal joint space may or may not be pointed with mortar, depending on the type of liquid the pipe will carry and the type of protective coating applied to the joint rings during manufacture.

Design Basis

The design procedure for concrete bar-wrapped cylinder pipe is presented in chapter 7 of this manual. This procedure is based on flexible pipe theory in which internal pressure and external load are considered separately but not in combination. Bar-wrapped cylinder pipe has been designed for operating pressures exceeding 400 psi (2,758 kPa). Several pipelines in Arizona and Colombia (South America) have been designed for pressures greater than 400 psi (2,758 kPa). Because the theory of flexible pipe design for earth loads is based on the passive soil pressure adjacent to the sides of the pipe, the design must be closely coordinated with the installation conditions.

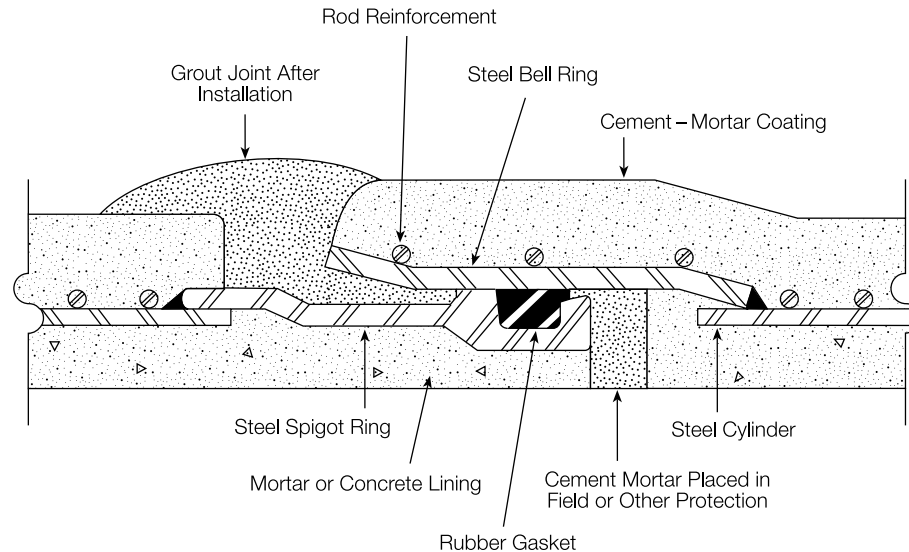


Figure 2-7 Concrete bar-wrapped cylinder pipe ([ANSI/AWWA C303](#)-type pipe)

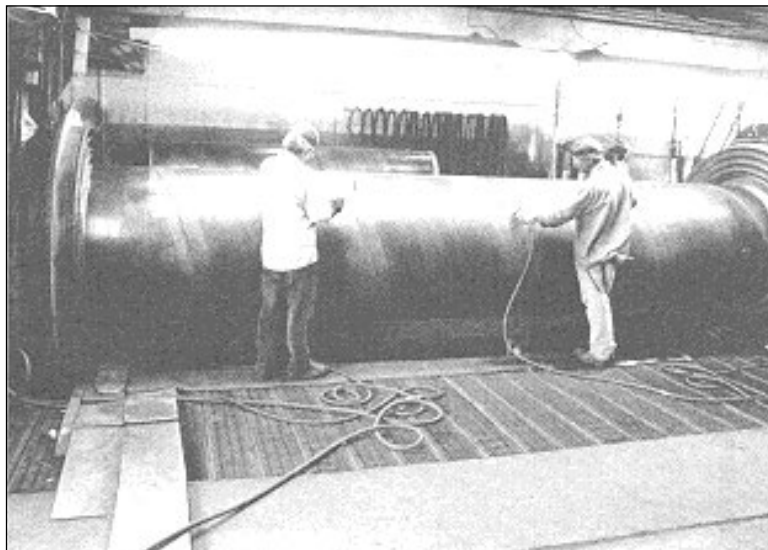


Figure 2-8 Steel cylinder being hydrostatically tested on a mandrel-type tester

FITTINGS AND SPECIAL PIPE

All concrete pressure pipe manufacturers make a wide variety of fittings and special pipe, including bends (elbows), tees, wyes, crosses, manifolds, reducers, adapters, wall sleeves, closures, bulkheads, bevel pipe, and service outlets (Figure 2-9). Adapters and outlets are available to connect to all types of joints, including flanged, threaded (standard or Mueller), plain end (for mechanical coupling or field welding to steel



Source: Hanson Pressure Pipe

Figure 2-9 Installation of a large-diameter wye with reducer attached

pipe), iron pipe (bell or spigot for leaded joints), Victaulic, and various mechanical joints. Chapter 8 of this manual addresses the typical types of fittings that are manufactured and the basis of fittings design.

Special Pipe

A special pipe is usually defined as a pipe in which the basic construction is the same as the standard pipe (prestressed cylinder, reinforced cylinder or noncylinder, or bar-wrapped cylinder), but contains some modification, such as an outlet or a beveled joint ring. Many types of outlets and beveled pipe can be furnished more economically in a special pipe than by using a fitting. Beveled pipe is manufactured with a joint ring that is deflected at a slight angle (up to 5° maximum). An individual beveled pipe or a series of beveled pipe can be used to deflect a line vertically or horizontally to avoid obstructions in the right-of-way, or a series of beveled pipe can be used to form a long sweeping curve to eliminate the need for a bend-and-thrust restraint. Most manufacturers make beveled pipe with more than one angle of deflection, which, together with standard lengths, provide many layout possibilities for offsets and curves without the use of bends. Beveled adapters (sections of beveled pipe with laying lengths generally less than 1 ft [305 mm]) are stocked by most manufacturers for contractors to use for deflections around unanticipated obstacles.

REFERENCE

- Marston, A. 1930. The Theory of External Loads on Closed Conduits in the Light of Latest Experiments. Bulletin 96, Iowa State Engineering Experiment Station. Iowa State University.



Chapter 3

Hydraulics

This chapter provides formulas and guidelines to aid in the hydraulic design of concrete pressure pipe. The three formulas typically used to determine pipeline capacity are presented and compared. Factors that contribute to the decrease with age of pipeline carrying capacity are also described. The methods used to determine minor and total head losses are presented, as well as the methodology for determining an economical pipe size. Throughout this chapter, discussions will be limited to combinations of pipe sizes and flows that provide a range of velocities and diameters most commonly used with concrete pressure pipe.

FLOW FORMULAS

The formulas commonly used to determine the capacity of pipelines are the Hazen–Williams formula, the Darcy–Weisbach formula, and the Manning formula. Many other flow formulas may be found in technical literature, but these three are most frequently used. It is impossible to say that any one of these formulas is superior to the others for all pipe under all circumstances, and the reader is cautioned not to expect identical answers to a problem from all three formulas. Judgment must be used when selecting a flow formula and the roughness coefficient for a particular hydraulics problem.

The Hazen–Williams Formula

The empirical Hazen–Williams formula is probably the most commonly used flow formula in the waterworks industry. The basic form of the equation is

$$V = 1.318 C_h R^{0.63} S^{0.54} \quad (\text{Eq 3-1})$$

Where:

- V = velocity, ft/sec
- C_h = Hazen–Williams roughness coefficient
- R = hydraulic radius, ft, which is the cross-sectional area of the pipe divided by the wetted perimeter, i.e., for circular pipe flowing full, the internal diameter in feet divided by 4
- S = slope of the hydraulic grade line, ft/ft calculated as h_L/L , where h_L equals head loss in feet occurring in a pipe of length L in ft

For circular conduits flowing full, the equation becomes

$$V = 0.550C_h d^{0.63} (h_L/L)^{0.54} \quad (\text{Eq 3-2})$$

Where:

- V = velocity, ft/sec
- C_h = Hazen–Williams roughness coefficient
- d = inside diameter of the pipe, ft
- h_L = head loss, ft
- L = pipe length, ft

Table 3-1 lists the various forms of the Hazen–Williams formula. Figures 3-1 and 3-2 show the hydraulic grade line, head losses, and static grade line for gravity and pumped flow systems.

A detailed investigation of the available flow test data for concrete pipe was performed by Swanson and Reed (1963), who concluded that the Hazen–Williams formula most closely matches the test results for the range of velocities normally encountered in water transmission. A statistical analysis of Swanson and Reed's test data led to the development of the following equation for determining C_h :

$$C_h = 139.3 + 2.028d \quad (\text{Eq 3-3})$$

Where:

- C_h = Hazen–Williams roughness coefficient
- d = inside diameter of pipe, ft

Table 3-2 compares theoretical Hazen–Williams C_h values using Equation 3-3 to tested values for 67 tests reported by Swanson and Reed.

Equation 3-3 can be used to calculate a C_h value for any size pipe; however, for design purposes the following conservative values are suggested:

Diameter, <i>in.</i>	C_h Value
16 to 48	140
54 to 108	145
114 and larger	150

Table 3-1 Various forms of the Hazen–Williams formula

In Terms of:	Velocity, <i>fps</i>	Flow Rate, <i>cfs</i>
General equation	$V = 0.550C_h d^{0.63} (h_L/L)^{0.54}$	$Q = 0.432C_h d^{2.63} (h_L/L)^{0.54}$
Head loss, <i>ft</i>	$h_L = 3.021(L/d^{1.167})(V/C_h)^{1.852}$	$h_L = 4.726(L/d^{4.870})(Q/C_h)^{1.852}$
Pipe diameter, <i>ft</i>	$d = 2.580(V/C_h)^{1.587}(L/h_L)^{0.857}$	$d = 1.376(Q/C_h)^{0.380}(L/h_L)^{0.205}$
Pipe length, <i>ft</i>	$L = 0.331h_L d^{1.167}(C_h/V)^{1.852}$	$L = 0.212h_L d^{4.870}(C_h/Q)^{1.852}$
Roughness coefficient	$C_h = 1.817(V/d^{0.63})(L/h_L)^{0.54}$	$C_h = 2.313(Q/d^{2.63})(L/h_L)^{0.54}$

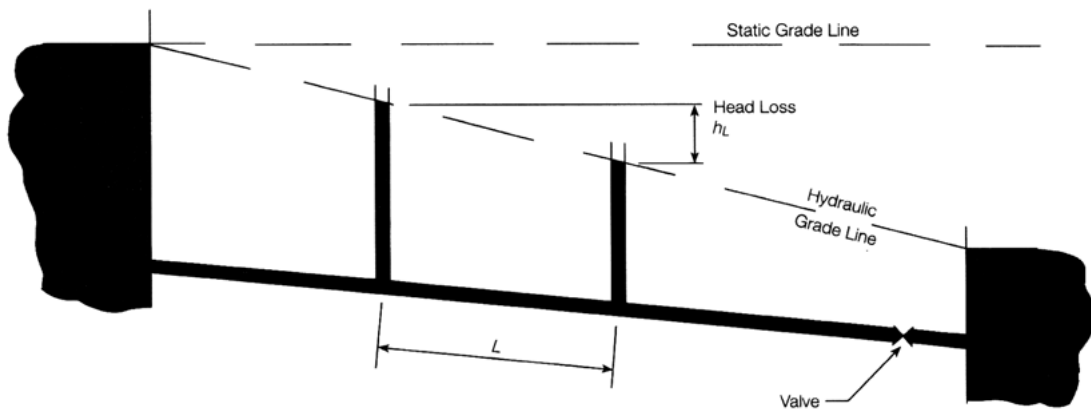


Figure 3-1 Hydraulic profile for a gravity flow system

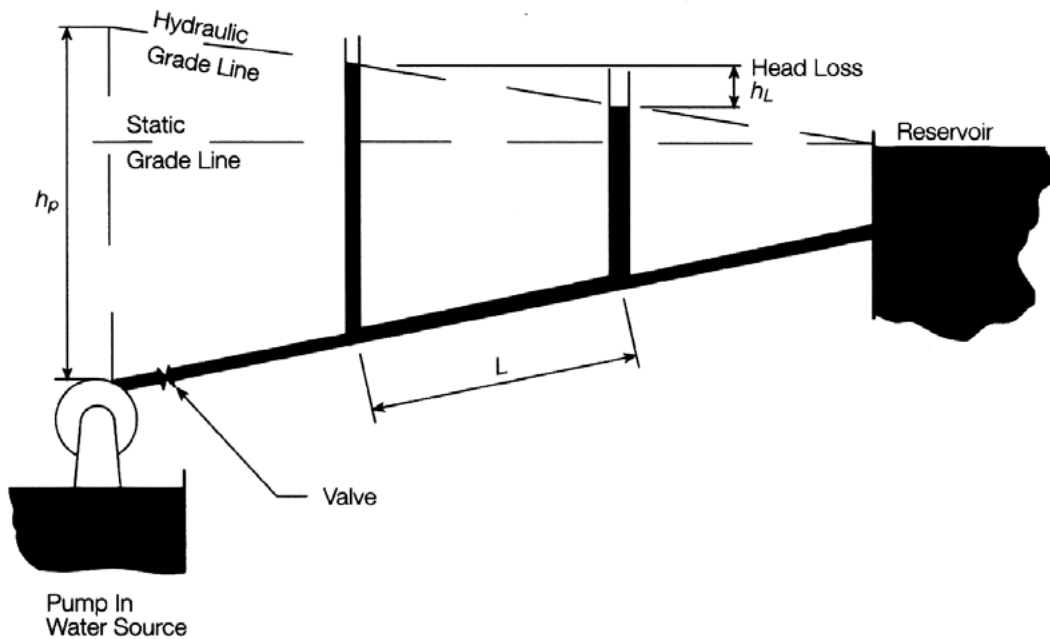


Figure 3-2 Hydraulic profile for a pumped flow system

Table 3-2 Comparison of theoretical Hazen–Williams C_h values to tested C_h values*

Sample Number	Diameter <i>in.</i>	C_h Measured	C_h Theoretical	Percent of Theoretical
1	24.0	145.0	143.4	101.1
2	30.0	147.0	144.4	101.8
3	30.0	145.0	144.4	100.4
4	31.5	147.0	144.6	101.6
5	31.8	152.0	144.7	105.1
6	36.0	143.0	145.4	98.4
7	36.0	150.0	145.4	103.2
8	36.0	141.5	145.4	97.3
9	36.0	142.5	145.4	98.0
10	36.0	150.0	145.4	103.2
11	39.0	147.0	145.9	100.8
12	42.0	147.5	146.4	100.8
13	42.0	142.0	146.4	97.0
14	42.0	149.0	146.4	101.8
15	42.0	149.0	146.4	101.8
16	42.0	142.0	146.4	97.0
17	42.0	147.0	146.4	100.4
18	46.0	144.0	147.1	97.9
19	48.0	146.0	147.4	99.0
20	48.0	150.5	147.4	102.1
21	48.0	144.0	147.4	97.7
22	51.0	142.0	147.9	96.0
23	54.0	152.0	148.4	102.4
24	54.0	141.0	148.4	95.0
25	54.0	140.5	148.4	94.7
26	60.0	145.5	149.4	97.4
27	60.0	152.5	149.4	102.0
28	46.0	148.5	147.1	101.0
29	54.0	150.0	148.4	101.1
30	36.0	138.0	145.4	94.9
31	36.0	137.0	145.4	94.2
32	36.0	142.0	145.4	97.7
33	48.0	149.0	147.4	101.1
34	54.0	151.5	148.4	102.1
35	54.0	145.0	148.4	97.7
36	54.0	146.0	148.4	98.4
37	54.0	147.5	148.4	99.4
38	60.0	147.0	149.4	98.4
39	60.0	154.0	149.4	103.1
40	60.0	156.0	149.4	104.4
41	60.0	143.5	149.4	96.0
42	66.0	142.0	150.5	94.4
43	48.0	156.5	147.4	106.2
44	48.0	152.0	147.4	103.1
45	48.0	149.5	147.4	101.4
46	48.0	153.5	147.4	104.1
47	54.0	154.0	148.4	103.8
48	54.0	155.5	148.4	104.8
49	54.0	151.0	148.4	101.7
50	54.0	152.0	148.4	102.4

Table continued next page.

Table 3-2 Comparison of theoretical Hazen–Williams C_h values to tested C_h values* (continued)

Sample Number	Diameter <i>in.</i>	C_h Measured	C_h Theoretical	Percent of Theoretical
51	54.0	143.0	148.4	96.3
52	72.0	154.0	151.5	101.7
53	12.0	145.0	141.3	102.6
54	18.0	147.0	142.3	103.3
55	20.0	134.0	142.7	93.9
56	24.0	143.5	143.4	100.1
57	30.0	141.0	144.4	97.7
58	30.0	143.0	144.4	99.1
59	33.0	147.0	144.9	101.5
60	33.0	146.0	144.9	100.8
61	33.0	146.0	144.9	100.8
62	36.0	134.0	145.4	92.2
63	31.4	138.0	144.6	95.4
64	36.0	147.0	145.4	101.1
65	36.0	150.0	145.4	103.2
66	41.9	150.0	146.4	102.5
67	42.0	150.0	146.4	102.5

* The theoretical values were calculated using Eq 3-3 and the test values are from Swanson and Reed (1963).

These values are applicable to concrete pipelines in which the fitting losses are a minor part of the total loss and to pipelines free from deposits or organic growths that can materially affect the pipe's carrying capacity.

The Darcy–Weisbach Formula

An alternative to the Hazen–Williams formula is the Darcy–Weisbach formula, which is

$$h_L = \frac{fLV^2}{d2g} \quad (\text{Eq 3-4})$$

Where:

- h_L = head loss, in feet of water
- f = Darcy friction factor
- L = pipeline length, ft
- V = velocity, ft/sec
- d = inside diameter of the pipe, ft
- g = gravitational constant, 32.2 ft/sec²

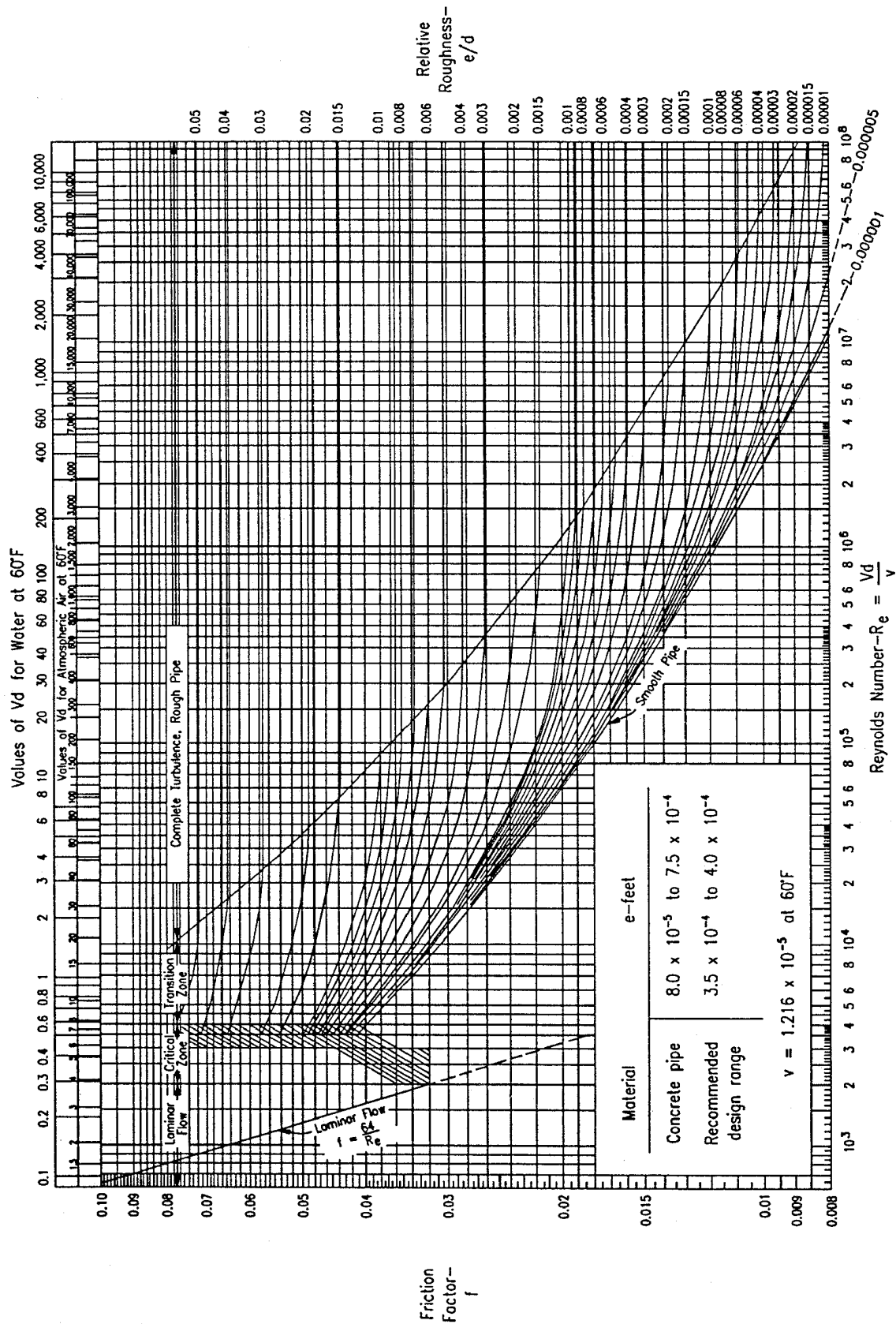


Figure 3-3 The Moody diagram for friction in pipe

The Darcy friction factor (f) can be determined from the Moody diagram, presented in Figure 3-3. To use this diagram, the Reynolds number (R_e) and the relative roughness (e/d) must first be calculated. Both terms are defined in the paragraphs that follow.

The Reynolds number is a function of the flow in the pipe and may be calculated as

$$R_e = \frac{dV}{\nu} \quad (\text{Eq 3-5})$$

Where:

- R_e = Reynolds number
- d = inside diameter of the pipe, ft
- V = velocity, ft/sec
- ν = kinematic viscosity of the fluid, ft²/sec

The kinematic viscosity of water at various temperatures from freezing to boiling is presented in Table 3-3.

The relative roughness (e/d) of a pipe is a function of the absolute roughness (e) of the interior surface of the pipe and the pipe diameter (d). Values of the absolute roughness (e) for concrete pipe range from 8.0×10^{-5} to 7.5×10^{-4} , and the recommended design range is 3.5×10^{-4} to 4.0×10^{-4} . Once the Reynolds number and the relative roughness are determined, the Moody diagram (Figure 3-3) can be used to determine values of the Darcy friction factor (f), which may then be used to solve the Darcy–Weisbach formula.

If an analytical solution for the Darcy friction factor (f) is preferred, it may be obtained by iteration from the Colebrook–White equation:

$$\frac{1}{\sqrt{f}} = -2 \log_{10} \left[\frac{e}{3.7d} + \frac{2.51}{R_e \sqrt{f}} \right] \quad (\text{Eq 3-6})$$

Where:

- f = Darcy friction factor
- e = absolute roughness, ft
- d = inside diameter of the pipe, ft
- R_e = Reynolds number

Table 3-3 Kinematic viscosity of water

Temperature, °F	Kinematic Viscosity (ν), ft ² /s
32	1.931 E-05
40	1.664 E-05
50	1.410 E-05
60	1.216 E-05
70	1.059 E-05
80	09.30 E-06
90	08.23 E-06
100	07.36 E-06
120	06.10 E-06
150	04.76 E-06
180	03.85 E-06
212	03.19 E-06

The Manning Formula

The Manning formula is more commonly used to determine the flow in partially filled gravity lines; however, it can be used for fully developed flow in conduits. The basic form of the equation is

$$V = 1.486R^{0.67}S^{0.5}/n_M \quad (\text{Eq 3-7})$$

Where:

- V = velocity, ft/sec
- R = hydraulic radius, in feet, which is the cross-sectional area of the pipe divided by the wetted perimeter (e.g., for circular pipe flowing full, the internal diameter divided by 4)
- S = slope of the hydraulic grade line, ft/ft
- n_M = Manning roughness coefficient

For pressure flow in round conduit, this equation becomes

$$V = (0.590d^{0.67}/n_M)(h_L/L)^{0.5} \quad (\text{Eq 3-8})$$

Where:

- V = velocity, ft/sec
- n_M = Manning roughness coefficient
- d = inside diameter of the pipe, ft
- h_L = head loss, in feet, occurring in a pipe of length L , ft

For concrete pressure pipe, the value for the Manning roughness coefficient n_M should be approximately 0.011 when the velocity is 3 ft/s and 0.010 when the velocity is 5 ft/s.

EFFECTS OF AGING ON CARRYING CAPACITY

The carrying capacity of concrete pressure pipe is usually not affected by age. However, some aggressive waters can roughen the lining surface, and there are several organic growths and some types of deposits that reduce the carrying capacity of pipe of any material, including concrete pressure pipe. These conditions are often associated with raw water transmission mains and can generally be prevented by chemical pretreatment. If left unchecked, however, excessive growths or deposits can develop. When this occurs, chemical treatment may not be adequate to remove the deposits if they have achieved a substantial buildup. In these cases, the growths or deposits can be removed by scraping or pigging the line. Industry has not witnessed deterioration of the mortar or concrete lining as a result of growths or deposits.

HEAD LOSSES

The total head loss in a pipeline is the sum of minor losses due to changes in flow geometry added to the head loss created due to the friction caused by flow through the pipe. The head loss caused by pipe friction may be determined from the formulas already presented. The calculation procedure for determining minor losses is presented in the following section. A method to easily include minor losses with frictional losses to simplify pipe selection calculations is also presented.

Minor Losses

Minor losses in pipelines are caused by turbulence resulting from changes in flow geometry. Minor losses, which are generally expressed as a function of the velocity head, will occur at entrances, outlets, contractions, enlargements, bends, and other fittings. In long pipelines, the minor losses are usually small compared to losses from pipe friction and may be neglected; however, in shorter lines or plant piping, these minor losses may become significant. The formula for calculating minor losses is

$$h_L = C_L V^2 / (2g) \quad (\text{Eq 3-9})$$

Where:

- h_L = head loss, ft
- C_L = a dimensionless coefficient
- V = velocity, ft/sec
- g = gravitational constant, 32.2 ft/sec²

Figure 3-4 presents values of C_L for common flow configurations.

Equivalent Length Method

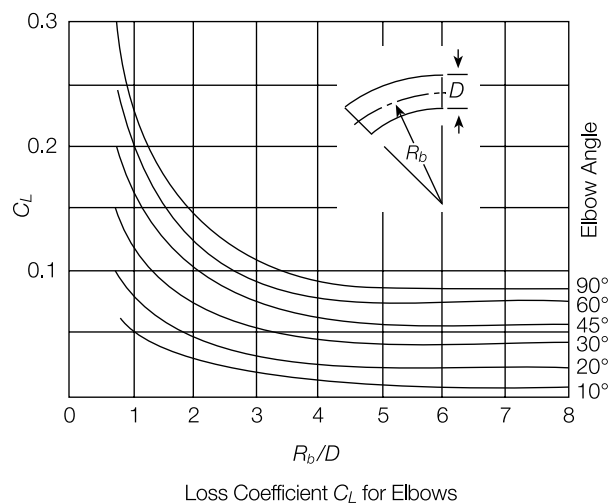
Each minor head loss in a piping system can be expressed in terms of an equivalent length of pipe L_e . This equivalent length of pipe is the number of feet of straight pipe that would have friction head loss equal to the minor losses created by the fitting. Table 3-4 lists the equivalent length formulas that correspond to the flow formulas presented earlier.

DETERMINING AN ECONOMICAL PIPE DIAMETER

To determine the most economical pipe diameter, the first cost (purchase cost plus installation cost) must be compared to the pumping cost incurred during the design life of the pipe. While a greater diameter pipe will result in a greater first cost, the reduced head loss will lead to a lower pumping cost and provide capacity for subsequent increased flow. These costs should be determined for a number of pipe sizes and the results compared in order to select the pipe size that provides the lowest total cost during the life of the line.

Experience has shown the most cost-effective water pipe diameter will usually yield design flow velocities in the range of 3 to 6 ft/s (0.9 to 1.8 m/s). Therefore, it is generally best to restrict pipe size evaluations to diameters that will deliver the required volume of water at a velocity between 2 and 7 ft/s (0.6 and 2.1 m/s), although in some rare circumstances involving short pipeline length, it may be cost effective to use velocities approaching 10 ft/s (3.0 m/s). The inside diameter of pipe for various flow velocities can be determined from

$$d = (1.27Q/V)^{0.5} \quad (\text{Eq 3-10})$$



Flow Configurations		C_L
Elbow		See figure above
Reentrant Entrance		0.80
Square-Edged Entrance		0.50
Slightly Rounded Entrance		0.23
Well-Rounded Entrance		0.04
Flow Into Reservoir		1.00
Reducer With $\alpha \leq 15^\circ$		0.04
Enlarger: $\beta = 10^\circ$ $\beta = 20^\circ$ $\beta = 30^\circ$		0.20 0.40 0.70
Tee, Through Run		0.60
Tee, Through Side Outlet		1.80
Tee Into Side Outlet		1.50
45° Wye		1.30
Gate Valve, Fully Open		0.20
Swing Check Valve, Fully Open		2.50
Butterfly Valve, Fully Open		0.25

Figure 3-4 Approximate loss coefficients for commonly encountered flow configurations

Table 3-4 Equivalent length formulas

Friction Formula	Equivalent Length Formula
Hazen–Williams	$L_e = 0.331 C_L \frac{V^{0.148} d^{1.167} C_h^{1.852}}{2g}$
Darcy–Weisbach	$L_e = C_L (d/f)$
Manning	$L_e = 0.348 C_L \frac{d^{1.333}}{2g (n_M)^2}$

NOTE: Definition of variables:

- L_e = length of pipe that would have a frictional head loss equal to the minor loss created by fitting, ft
- C_L = loss coefficient
- V = velocity, ft/sec
- d = inside diameter of the pipe, ft
- C_h = Hazen–Williams roughness coefficient
- g = gravitational constant, 32.2 ft/sec²
- f = Darcy friction factor
- n_M = Manning roughness coefficient

Where:

- d = inside diameter of the pipe, ft
- Q = flow rate, ft³/sec
- V = flow velocity, ft/sec

For each pipeline diameter to be evaluated, minor losses should be converted to equivalent lengths of pipe as outlined in the preceding section, and the friction head should be calculated from one of the flow equations presented earlier. As an example, the Hazen–Williams formula, in the following form, can be used to calculate total head loss for the sum of actual pipe lengths and equivalent lengths:

$$h_f = (4.726 L' / d^{4.870}) (Q / C_h)^{1.852} \quad (\text{Eq 3-11})$$

Where:

- h_f = total head loss, ft
- d = pipe diameter, ft
- L' = the actual length of the pipeline (L) plus the sum of the calculated equivalent lengths (L_e), ft
- Q = flow rate, ft³/sec
- C_h = Hazen–Williams roughness coefficient

The power required to overcome the total head loss may be determined from

$$P_p = (Qh_f\gamma)/550 \quad (\text{Eq 3-12})$$

Where:

- P_p = pump power requirements, hp
- Q = flow rate, ft³/sec
- h_f = total head loss, ft
- γ = unit weight of the fluid, lb/ft³; for water at 50°F, $\gamma = 62.41$ lb/ft³

The preceding equations for total head loss and power requirements may be combined to yield the following equation for the annual pumping power cost relative to frictional head:

$$\text{Cost/year} = \frac{(0.745 \times P_p \times \text{power cost} \times \text{hours/year})}{(\text{pump eff.} \times \text{motor eff.})} \quad (\text{Eq 3-13})$$

Where:

- Cost/year = pumping power cost per year, \$
- P_p = pump power requirements, hp
- power cost = power cost, \$/kW·h
- hours/year = number of hours per year of pump operation
- pump eff. = the pump efficiency, expressed as a decimal
- motor eff. = the motor efficiency, expressed as a decimal

It should be noted that the cost calculated using the preceding formulas is the cost of power required to overcome total head loss and does not include the cost of pumping required to overcome changes in elevation. Since the elevation change will be the same for all pipe diameters being evaluated, it can be ignored in the comparison of pipe diameters. If the designer desires to graphically present the hydraulic grade line for one or more of the pipe diameters to calculate total head or to calculate the total annual power cost, then the static head should be combined with the frictional head, and the sum of the two should be used in the preceding equations.

When selecting pipe diameters based on the preceding formulas, it is important that the designer consider both average annual pumping conditions and peak pumping conditions. The annual power cost equation, if used in conjunction with the peak pumping rate, will lead the designer to select a diameter that is larger than needed. In addition, the designer must consider that power factors or demand charges affect the rates paid for power and that power costs will increase with time. Therefore, it may be prudent to select an escalation rate so that a realistic comparison between the first cost of the pipe and the long-term pumping costs may be equitably made.

AIR ENTRAPMENT AND RELEASE

Air entrapment in poorly vented pipelines can reduce pipeline carrying capacity, cause unexpected pressure surges, and produce objectionable “white water.” The following three conditions must exist before air binding presents a serious problem:

- There must be a source of air in excess of that normally held in solution in the flow.
- The air must separate from the water and collect at high points or descending legs.
- The collected air must remain near the high points or in the descending legs.

It is desirable to vent high points and/or descending legs on transmission mains with automatic air-release valves to relieve this condition. Improperly sized air-release valves can contribute to surge pressure conditions as described in chapter 4.

BLOWOFF OUTLETS

Blowoffs may be located at low points in the profile of a pipeline. They are mainly used to drain lines for maintenance or inspection. Occasionally, blowoffs are used to remove silt or sediment from a line. If there is the possibility of backsiphoning contaminated water into water transmission mains, the outlet for blowoff discharge piping should be elevated. Blowoffs should generally be sized so the main pipeline velocity change is limited to 1 ft/s (0.3 m/s) or less during blowoff operation.

REFERENCES

- | | |
|---|---|
| <p>Albertson, M.L., J.R. Barton, and D.B. Simons. 1960. <i>Fluid Mechanics for Engineers</i>. Englewood Cliffs, N.J.: Prentice-Hall Inc.</p> <p>American Society of Civil Engineers (ASCE). 1975. <i>Pipeline Design for Water and Wastewater</i>. Report of the Task Committee on Engineering Practice in the Design of Pipelines. New York: ASCE.</p> <p>Bradley, J.N., and L.K. Thompson. 1951. <i>Friction Factors of Large Conduits Flowing Full</i>. Engineering Monograph 7. US Bureau of Reclamation.</p> | <p>Edmunds, R.C. 1979. Air Binding in Pipes. <i>Jour. AWWA</i>, 71(5):272–277.</p> <p>Russel, G.E. 1952. <i>Hydraulics</i>, 5th ed. New York: Henry Holt & Company.</p> <p>Swanson, H.V., and M.D. Reed. 1963. Comparison of Flow Formulas and Friction Factors for Concrete Pressure Pipe. <i>Jour. AWWA</i>, 55(1):67–80.</p> |
|---|---|

This page intentionally blank.



Chapter 4

Surge Pressure

Surge pressure is the result of a change in fluid velocity in a closed conduit. This change in flow causes waves to travel upstream and downstream from the point of origin. The waves, in turn, cause increases or decreases in pressure as they travel along the line. These pressure changes are variously referred to as water hammer, surge, or transient pressure.

Surge pressure or water hammer is independent of the internal working pressure created by the fluid in the line and is proportional to the velocity change and wave speed. Therefore, surge pressure must be added to the internal working pressure to determine the total pressure experienced by the pipe when surges occur.

The phenomenon of water hammer is extremely complex and cannot be covered in depth in this manual. Only the fundamentals of elastic wave theory and specific data pertaining to the properties of concrete pipe are discussed. For a more detailed understanding of water hammer, the references listed at the end of this chapter should be consulted.

EQUATIONS AND VARIABLES

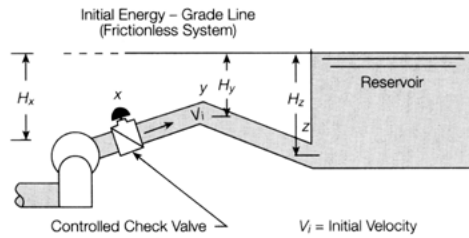
The critical time period T_c for a piping system is the time required for a transient wave to travel in the piping system to a point of reflection and return to the point of origin. The value of T_c may be determined from

$$T_c = 2L/a \quad (\text{Eq 4-1})$$

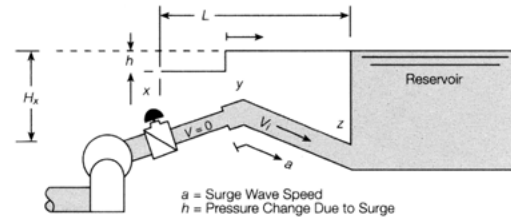
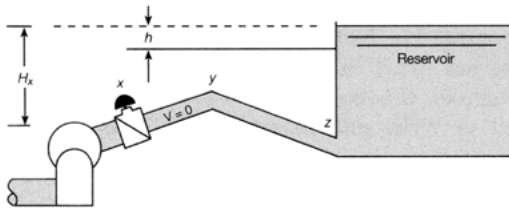
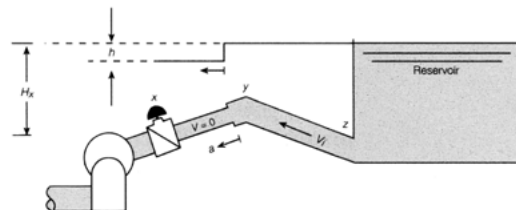
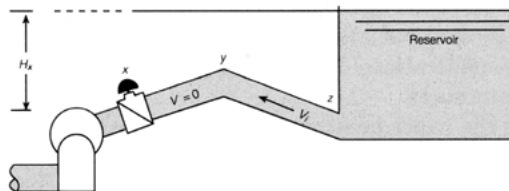
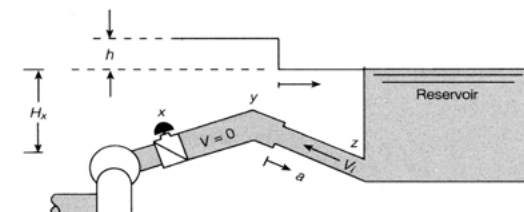
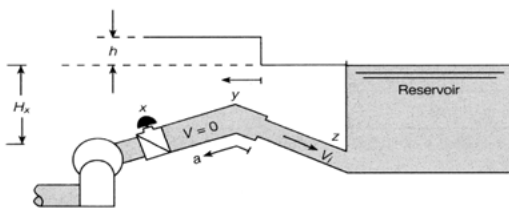
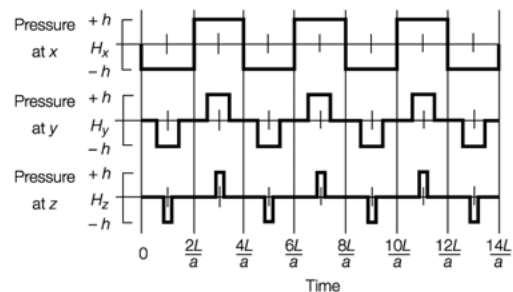
Where:

- T_c = critical time period, sec
- L = the distance that a transient wave travels in the piping system between its origin and reflection point, ft
- a = wave speed, ft/sec

The term *critical period* is used because the change in flow velocity, which occurs in a time period equal to or less than T_c , will cause the maximum pressure rise for a particular surge event. The pressure rise from a fluid velocity change occurring within the critical period is illustrated in Figure 4-1. Pressure waves from changes



A. One-Pipe System at Pump Shutdown

B. Negative Pressure Wave Travels Down Pipeline Until L/a SecondsC. System Condition at L/a SecondsD. System Condition Between L/a and $2L/a$ SecondsE. System Condition at $2L/a$ Seconds—Assume Valve Closes InstantaneouslyF. System Condition at $2L/a$ and $3L/a$ SecondsG. System Condition Between $3L/a$ and $4L/a$ Seconds

H. Pressure Versus Time For Pipeline

NOTE: While the surge wave duration is shorter near the reservoir, the magnitude of the surge wave is the same throughout the line. The magnitude of the surge wave at any station on the line is not related to the operating pressure at that station when the surge occurs.

Figure 4-1 Idealized surge cycle for instantaneous pump shutdown

in velocity occurring over longer time periods will be reduced by molecular frictional losses and by overlapping with the lower pressure troughs of returning surge waves.

The pressure rise from a change in fluid velocity occurring within the critical time period is calculated from

$$H_s = a(V_i - V_f) / g \quad (\text{Eq 4-2})$$

Where:

- H_s = pressure rise due to surge, ft
- a = wave speed, ft/sec
- V_i = initial fluid velocity, ft/sec
- V_f = fluid velocity at time $t = T_c$, ft/sec
- g = gravitational constant, 32.2 ft/sec²

If flow is completely stopped in $t < T_c$ seconds, V_f is equal to zero and the formula reduces to

$$H_s = \frac{aV}{g} \quad (\text{Eq 4-3})$$

Where:

- V = initial fluid velocity, ft/sec

and the other variables are as previously defined.

The maximum surge pressure that will be encountered by a line is a function of the maximum velocity that can be achieved by the system and the wave speed associated with the piping material. The maximum pressure surge will occur in situations in which flow is moving at the maximum velocity and is stopped by the closing of a quick-acting valve, the failure of the pump, or other actions that could stop the flow in a time that is less than the critical time T_c . The speed of the surge wave in a pipeline is determined by the type and thickness of the material from which the pipeline is constructed, the pipeline diameter, and the liquid being transported by the line.

The wave speed a of a hydraulic transient may be determined from the following formula:

$$a = \left[\frac{(144E_L / \rho)}{1 + \frac{E_L D}{E_p t}} \right]^{0.5} \quad (\text{Eq 4-4})$$

Where:

- a = wave speed, ft/sec
- E_L = bulk modulus of the liquid, psi (300,000 psi for water)
- ρ = mass density of the fluid, lb-s²/ft⁴ (1.94 lb-s²/ft⁴ for water)
- D = inside diameter of the pipe, in.
- E_p = modulus of elasticity of the pipe wall, psi
- t = transformed pipe wall thickness, in.

For water, $(144E_L/\rho)^{0.5} = 4,720$ ft/s, and Eq. 4-4 becomes

$$a = \frac{4,720}{\left[1 + \frac{E_L D}{E_p t}\right]^{0.5}} \quad (\text{Eq 4-5})$$

with variables as previously defined.

Because concrete pressure pipe is a composite structure, the solution of this equation is not straightforward. The modulus of elasticity of the pipe wall E_p is derived from the slope of the stress-strain curve of pipe wall material. These curves for concrete pressure pipe change in slope over the possible ranges of pressure the pipe may encounter. Kennison (1955) published a detailed explanation of this theory along with theoretical values and experimental results.

Calculated values of the surge wave speed for various types and classes of concrete pressure pipe range from 2,950 to 4,000 ft/sec (899 to 1,219 m/sec). Design values for different types of concrete pressure pipe are listed in Table 4-1. In piping systems where gas bubbles are dispersed throughout the liquid, the wave speed may be greatly reduced. The reader is referred to Wylie and Streeter (1983) for a more detailed explanation of this phenomenon.

NEGATIVE PRESSURES

The conditions that produce pressure rises from water hammer also cause a reduction in pipeline pressure when the pressure wave reflects at a boundary. These pressure reductions, if large enough in magnitude, will result in negative pressures in all or part of the piping system. However, because of the inherent structural qualities of the concrete pipe wall, concrete pressure pipe does not collapse from negative pressures and does not require the special design considerations necessary to guard flexible pipe against collapse as a result of negative pressures.

If the pressure becomes lower than the vapor pressure of water, extensive vapor cavities may result. The subsequent positive surge pressures that result from the collapse of the vapor cavity can be extremely high. Pipeline high points are particularly susceptible to vapor cavity formation, and, therefore, vacuum-release valves are frequently installed at such locations. A surge tank or hydropneumatic chamber can also be used to prevent vapor cavity formation.

CAUSES OF SURGE PRESSURE

Water hammer can result from any condition that causes a sudden change in the flow rate of the fluid in the pipeline. The following is a partial list of the causes of water hammer:

- normal pump startup and shutdown
- pump power interruption
- valve operations
- air venting from pipeline during filling
- improper operation of pressure-relief valves
- operation of large-orifice air and vacuum valves
- collapse of vapor cavities caused by negative pressures

Table 4-1 Design values—surge wave velocities, *ft/sec (m/sec)*

AWWA Description		Pipe Class, <i>psi (kPa)</i>					
		100	(689)	150	(1,034)	200	(1,379)
Prestressed concrete pressure pipe, steel cylinder type (ANSI/AWWA C301-type pipe)	Lined cylinder	3,500–3,750	(1,067–1,143)	3,400–3,775	(1,036–1,151)	3,350–3,850	(1,021–1,173)
	Embedded cylinder	3,025–3,625	(922–1,105)	3,000–3,650	(914–1,113)	2,975–3,675	(907–1,120)
Concrete water pipe, bar-wrapped steel cylinder type (ANSI/AWWA C303-type pipe)		3,175–3,875	(968–1,181)	3,325–3,925	(1,013–1,196)	3,450–3,975	(1,052–1,211)

Pump startup and shutdown, pump power failure, and valve operations are sometimes the only operational situations investigated to determine the magnitude of water hammer. Pipeline filling and operation of its pressure-relief valves are occasionally overlooked water hammer sources that should be considered.

During pipeline filling, air in the line may escape through some type of a control structure (e.g., an air-release or discharge valve). The viscosity of air is much less than water, so air can escape at a high rate through the air-release valves, allowing the water to enter the pipeline at an equal rate. Air-release valves generally have a float valve that snaps shut instantaneously when all of the air has been released. Depending on the physical characteristics of the system, the magnitude of the velocity change can result in very high surge pressure. In most cases, pipeline surge-pressure damage will not occur if line filling is performed at a rate of flow equivalent to a full pipe velocity of 1 ft/sec (0.3 m/sec) or less.

The operation of a pressure-relief valve may cause water hammer. When the system becomes overpressured, the valve is designed to open and relieve line pressure by releasing fluid from the pipeline. Most relief valves are quick opening but close at a controlled rate. If the size of the valve is large in comparison to the pipeline size and is improperly adjusted, or if the valve closure rate setting is not correct (especially in pipelines during the testing phase), the quick action of the relief valve may cause a sudden and extreme change in fluid velocity, thus creating water hammer.

CONTROL OF WATER HAMMER

Studies of surges should be conducted during the pipeline design stage. Once the general layout of the system has been completed, the length, diameter, thickness, material, and capacity of the pipe and the type and size of pumps can be established.

When analyzing surges, the designer generally first computes the hydraulic grade line for the ultimate operating mode of the system. The maximum working pressure at any point along the line is determined as the difference in elevation between the hydraulic grade line and the pipeline. Then, using the maximum volume of flow the pumping system can discharge or is expected to be able to discharge in the future, the maximum velocity is computed for the selected pipe diameter. Calculating surge pressure based on instantaneous stoppage of this flow will yield the maximum surge the system can expect to experience if the pipeline is designed without surge controls and if vapor cavity formation is prevented.

Concrete pressure pipe designs provide an allowance for water hammer that is generally adequate, but the design should be checked to be sure the pipe design provides an adequate allowance for surge. If not, either the pipe design should be adjusted to provide enough surge allowance to withstand the conditions that might be

encountered or suitable remedial or control devices should be provided. The latter method is almost always less costly. It is important to note that there is no single device that will cure all surge difficulties. Only by a study of both normal operating conditions and possible emergency conditions can methods be determined to provide proper control.

It is not feasible to make general recommendations on the type, size, and application of surge-control equipment for all systems. Several possible solutions should be considered for any individual installation, and the one selected should be the one that gives the most protection for the least expenditure. Surges can often be reduced substantially by using bypasses around check valves, by cushioning check valves for the last 15–20 percent of the stroke, or by adopting a two-speed rate of valve stroke. Water hammer resulting from power failure to centrifugal pumps can sometimes be held to safe limits by providing flywheels or by allowing the pumps to run backward, as long as (1) the reverse pump flow does not produce motor backspin speed greater than recommended by the motor manufacturer, and (2) the motor cannot be reenergized during backspin. Otherwise, the electric motor coupling should be designed to automatically disengage if pump backspin is permitted. Air-inlet valves may be needed, or the preferred solution may be to use a surge tank, a surge damper, or a hydropneumatic chamber.

It is essential to coordinate all the elements of a system properly and to ascertain that operating practices conform to the requirements for safety. As changes take place in the system demand, it may be necessary to review and revise the surge conditions, particularly if the capacity is increased, additional pumpage or storage is added, or booster stations are planned.

The surge wave theory has been proven in actual practice, and design engineers should take the initiative in making surge studies and installing surge-control devices without waiting for serious failures to occur. The time and effort spent on a surge study in advance of the final design is the least expensive means of avoiding problems from excessive surge pressures.

REFERENCES

- | | |
|---|---|
| <p>Angus, R.W. 1937. Water Hammer in Pipes, Including Those Supplied by Centrifugal Pumps; Graphical Treatment. In <i>Proc. of the Inst. Mech. Engrs.</i>, 136:245.</p> <p>Chaundhry, M.G. 1979. <i>Applied Hydraulic Transients</i>. New York: Van Nostrand Reinhold Company.</p> <p>Kennison, H.G. 1955. Surge-Wave Velocity — Concrete Pressure Pipe. Paper presented at the ASME Annual Meeting in Chicago, Ill.</p> <p>Kerr, S.L. 1948. Practical Aspects of Water Hammer. <i>Jour. AWWA</i>, 40(6):599–605.</p> | <p>———. 1951. Water Hammer Control. <i>Jour. AWWA</i>, 43(12):985–1000.</p> <p>Parmakian, J. 1963. <i>Water Hammer Analysis</i>. New York: Dover Publications.</p> <p>Rich, G.R. 1951. <i>Hydraulic Transients</i> (Engineering Societies Monographs). New York: McGraw-Hill Book Company.</p> <p>Wylie, E.B., and V.L. Streeter. 1983. <i>Fluid Transients</i>. Ann Arbor, Mich.: Feb Press.</p> |
|---|---|