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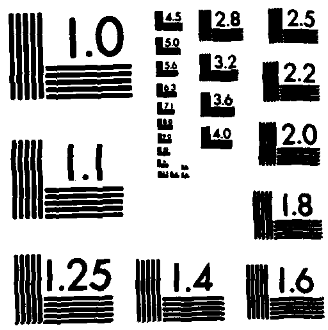

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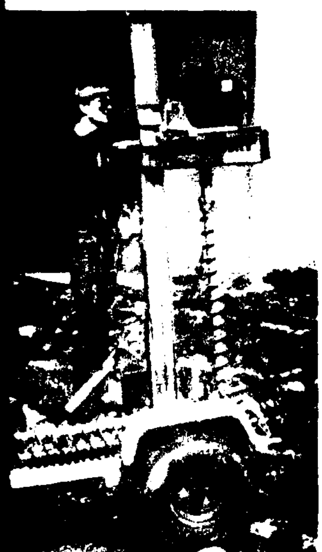

TECHNICAL REPORT EL-84-5

(2)

APPLICATION OF PROCEDURES FOR TESTING AND EVALUATING WATER DISTRIBUTION SYSTEMS

by

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Final Report

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This report is made up of eight individual papers describing solutions to specific problems in evaluating water distribution systems. The topics range from general discussions on deterioration of water systems and the need for pipe network model calibration to examples of actual reports on pipe break analysis and model calibration. Other papers give examples of how pipe roughness tests and gage calibration can be performed. Two additional papers

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20. **ABSTRACT (Continued).**

describe criteria for selecting differential pressure measuring devices for use with pitot tubes and a computer program for determining flow rates given these differential pressure readings. Subtopics include [illegible], [illegible], [illegible] and pipe flow.

PREFACE

The work described in this report was performed to demonstrate the techniques of field testing and evaluation, as presented in Engineer Technical Letter (ETL) 1110-2-278 "Evaluation of Existing Water Distribution Systems." Both the ETL and the report presented herein were prepared at the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., under Work Unit No. 31794 (Water System Operation, Maintenance, and Rehabilitation) of the Office, Chief of Engineers, Water Supply and Conservation Research Program.

The principal investigator for this work was Dr. Thomas M. Walski of the Water Resources Engineering Group (WREG), Environmental Engineering Division (EED), Environmental Laboratory (EL), WES. Mr. Sidney (Buddy) Ragsdale of the Sedimentation Branch, Estuaries Division, Hydraulics Laboratory, WES, assisted in the field testing and equipment development. Mr. Ragsdale's field testing capabilities were substantially responsible for the success of that portion of the work. Dr. Walski and Mr. Ragsdale were assisted by Mr. Michael Evans, WREG. Reviews of portions of the report were provided by Drs. Roger W. Burke and Joe Miller Morgan, WREG.

Work performed with the Washington Aqueduct Division (WAD) of the Baltimore District of the Corps was done under the purview of Mr. Harry C. Ways, Chief, WAD, and Mr. C. C. Peterson, Chief, Engineering Branch, WAD. Mr. Keith Copeland of WAD also provided valuable assistance in this study.

Testing conducted in Vicksburg, Miss., was conducted under the purview of Mr. John Kelly of the Vicksburg Water Department. The work unit technical monitor at the Office, Chief of Engineers, was Mr. James Ballif (DAEN-ECE-BU).

Part VI is based on a paper "Why Calibrate Water Distribution System Models?" which appeared in the October 1983 issue of Water Engineering and Management. Part IX is based on a paper "How Water Systems Age" which appeared in the July-August 1983 issue of The Military Engineer. Both were written by Dr. Walski.

The study was conducted under the direct supervision of Mr. Michael R. Palermo, Chief, WREG, and under the general supervision of Mr. Andrew J. Green, Chief, EED, and Dr. John Harrison, Chief, EL.

The Commander and Director of WES was COL Tilford C. Creel, CE. The Technical Director of WES was Mr. F. R. Brown.

This report should be cited as follows:

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CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
feet per second	0.3048	metres per second
feet per second per second	0.3048	metres per second per second
gallons per minute	3.785412	cubic decimetres per minute
inches	25.4	millimetres
miles (U. S. statute)	1.609347	kilometres
pounds (force) per square inch	6894.757	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres

APPLICATION OF PROCEDURES FOR TESTING AND EVALUATING WATER DISTRIBUTION SYSTEMS

PART I: INTRODUCTION

Background and Purpose

1. With the publication of Engineer Technical Letter (ETL) 1110-2-278 (Headquarters, Department of the Army 1983), the Corps of Engineers has guidance on testing, making decisions concerning rehabilitation, and developing and calibrating computer models for water distribution systems. While this ETL provided engineers with tools for testing and evaluating water distribution systems, it also raised more specific questions. Some examples of the questions left unanswered included: what kind of equipment should be used to conduct head loss tests, how do water systems deteriorate, how can flow measuring devices be calibrated, and why is it important to calibrate pipe network models? Similarly, there was the need to demonstrate application of the techniques presented in the ETL on analysis of pipe breaks and network model calibration.

2. During fiscal year 1983, the U. S. Army Engineer Waterways Experiment Station (WES) addressed many of the above issues under the Office, Chief of Engineers, Water Supply and Conservation Research Program. The purpose of this report is to document the field testing and other analyses conducted as part of this work.

3. Unlike most WES reports which address a single issue, this report is actually a compendium of eight separate reports, each with its own purpose, target audience, and level of detail. In general, the target audience consists of engineers conducting water distribution studies. While Part VI describes the need to calibrate pipe network models and is oriented toward study managers, Part III illustrates the calibration procedure in an actual system and is oriented toward the engineer actually performing the work.

4. The primary lesson learned in this work was that there is no substitute for collection of substantial, accurate data. It is hoped that the description of the collection and use of such data in water distribution system evaluation will serve as a model for good data collection practices for

future studies. This report contains not only a description of successful test results but also highlights the pitfalls awaiting those who might want to use haphazard methods for system evaluation.

Overview

5. Each part of this report addresses individual issues involved with evaluation of water distribution system. The respective issues are listed in the following paragraphs.

6. Part II gives the results of an analysis of pipe breaks in the Federally Owned Water Main (FOWM) System operated by the Washington Aqueduct Division of the U. S. Army Engineer District, Baltimore. The process used to determine whether to replace or rehabilitate water mains, based on an analysis of prior breakage, is illustrated.

7. Part III illustrates the development and calibration of a water distribution system model for the FOWM System. Special emphasis is placed on collection and use of model calibration data.

8. In Part IV the problem of calibrating a meter using a pitot tube is addressed for a situation in which a pitot tube was used to calibrate an orifice plate.

9. Loss-of-head tests are conducted to measure pipe roughness and carrying capacity. In Part V, the techniques for conducting these tests are demonstrated in six field tests.

10. Some engineers and utility managers question whether the precise model calibration methods described in EM 1110-2-278 and demonstrated in Part III are necessary. Part VI discusses the need for accurate model calibration.

11. In measuring flow rate using a pitot tube, it is necessary to convert the differential pressure readings made using a manometer or differential gage into point velocities and then integrate the velocity distribution to determine flow. Part VII describes an easy-to-use computer program which can be used to solve this problem.

12. As mentioned above, pitot tubes must be attached to some differential pressure measuring device. Part VIII discusses the relative merits of manometers, differential gages, and transducers for making these readings and concludes an air-filled manometer is best for field use. Instructions for

constructing and using such a manometer are also presented.

13. Part IX is a general description of how water distribution systems deteriorate with time. It discusses in fairly broad terms how to analyze and correct these problems.

PART II: PIPE BREAK ANALYSIS

Introduction

14. The Federally Owned Water Main System has been serving Federal facilities in the northern part of Arlington County, Virginia, since the 1940's. As with any water main, these mains are subject to breaking, and there is a cost associated with each break for repair, water loss, damages, and inconvenience. If the cause of the breaks can be identified and corrected by replacement or repair of the main, the rate of breakage for those mains can be reduced to virtually zero with the elimination of costs associated with breaks. The purpose of this analysis is to determine whether it is economical to replace water mains or take other remedial action in the FOWM System due to frequent main breakage.

15. In this part, the historical record of breaks is reviewed to identify problem areas and breakage trends. The results of soil corrosivity tests are presented and the costs of breaks and remedial actions are estimated. Finally, the costs and break projections are combined to develop recommendations for remedial actions.

Historical Records

Problem areas

16. To evaluate the economics of pipe replacement, it was necessary to assemble records on main breaks in recent years. These records were grouped into two data files: a pipe inventory and a break record file. The pipe inventory consists of the pipe identification number, length, diameter, type, and a brief description of the pipe as shown in Table 2-1. Since all mains in Table 2-1 are cast iron except for the one 30-in.* steel main, the type of pipe is not listed. Table 2-2 gives data on breaks during the period 1970-1981. The data in Tables 2-1 and 2-2 were assembled by the Washington Aqueduct Division (WAD) and stored in data files on the Boeing Computer Service (BCS) computer by WES personnel.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) is presented on page 5.

Table 2-1

Pipe Inventory

Pipe Number	Length ft	Diameter in.	Description
1	6,000	30	Main--Key Bridge to Pentagon
2	14,200	16	Main--Key Bridge to V291
3	500	16	Main--Marshall Drive
4	4,400	24	Main--Pentagon Loop to 15th and Eads Sts.
5	3,200	16	Main--15th and Eads to V306 Airport
6	1,200	10	Main--V229 to V235 Navy Annex
7	1,400	8	Fire hydrant loop-Navy Annex
8	1,500	8	Main--V233 to pump, Henderson Hall
9	1,600	12	Main--from V291 to V103 Pentagon South Park
10	600	18	Main--V108 to V104A Pentagon Loop
11	3,910	16	Main--V121 to 107 Pentagon Loop
12	1,600	24	Main--V121 to V108 Pentagon Loop
13	1,600	8	Fire hydrant feed from V113A (Pantagon Loop)
14	1,000	8	Supply to Pentagon heating plant
15	700	6	Supply to heating plant from Arlington County
16	2,600	8	Fire hydrant line V131 to V123
17	60	10	Line--D.C. side Key Bridge
18	1,400	6	Line--feed to Columbia Island and sewage plant
19	1,000	8	Fire hydrant loop around Pentagon heating plant
20	800	6	Fire hydrant feed to FH1, FH2, FH3, Pentagon

Note: V = valve, FH = fire hydrant.

Table 2-2

Break Record

Pipe No.	Month/Year of Break	Type of Break*	Description
11	7/70	4	Corrosion hole
17	2/71	1	Shear-approx. 15 ft of pipe abandoned-blank flange installed
15	3/71	1	Line sheared
11	7/71	4	Corrosion hole
4	8/72	2	Lead joint repair
11	9/72	4	Hole in pipe
15	11/72	1	Line sheared where it was resting on concrete footer
15	1/73	1	
4	7/73	2	Lead joint repair
16	2/74	1	Lead joint on fire hydrant
15	8/75	1	Sleeve over shear
11	6/76	4	Hole in pipe
5	8/76	2	Lead joint
7	12/76	5	
4	2/77	2	Lead joint
6	8/77	5	
2	8/77	5	
15	9/77	5	
4	6/78	2	Lead joint Army Navy and Eads
14	8/78	5	
11	8/78	4	
7	3/79	1	Sheared, sleeve install
2	6/79	1	Rupture in cemetery
1	8/79	4	Hole in pipe near Key Bridge
15	9/79	5	Line sheared where resting footer short piece and two dressers installed
8	12/79	1	Shear split sleeve installed
2	12/79	1	Ord and Wetzel Gates in cemetery
2	2/80	2	D.C. end of Key Bridge
11	8/80	4	Hole in pipe
4	8/80	1	Light pole bearing on pipe
4	9/80	2	Lead joints
18	10/80	1	Sheared repaired with dressers
14	12/80	1	
11	12/80	4	Hole near FH33
19	12/80	1	Loop Heating Plant between FH21 and FH22
20	1/81	1	
15	2/81	1	
2	2/81	1	Split in pipe near Rt 50 and George Washington Parkway
11	11/81	4	Two patches welded

* Break type: 1 = break in pipe; 2 = joint leak; 3 = hydrant leak; 4 = other; 5 = unknown.

Breakage trends

17. Before analyzing the economics of breaks, it is important to identify trends in break rates. Evaluation of these trends is necessary since the projection of future breaks depends on identification of break causes and determination of how these causes are affected by time. For example, if breaks are caused by corrosion, the break rate will increase with time, while for breaks due to improper bedding, the rate may remain constant or even decrease with time.

18. Data were provided by WAD for 20 pipes with a length of 9.33 miles. In the 12-year period under consideration, 39 breaks were reported giving an overall break rate of 0.35 breaks/year/mile. With this rate the FOWM would have the third highest break rate out of 15 cities for which O'Day (1982) reported break rates. This high rate is probably due to the large amount of construction activity which has taken place around the FOWM mains and the presence of corrosive soil in the area. Some mains have been subjected to loads in excess of design loads during construction and operation of the George Washington Parkway, I-95, and the Washington Metro.

19. One obvious trend in the breakage rate data is the increase in break rates after 1976. For the years 1970-1975 the break rate was 0.196 breaks/year/mile, while for 1976-1981, the rate was 0.500 breaks/year/mile. The apparent dramatic increase in break rate may be due simply to better record keeping after 1976. However, if the quality of the records is consistent, then the break rate in any year can be given by (ETL 1110-2-278):

$$J = J_o \exp\left[b(t - 1970)\right] \simeq J_o(1 + b)^{t-1970}$$

$$J = 0.28 \exp\left[0.13(t - 1970)\right] \simeq 0.28(1.13)^{t-1970} \qquad (2\text{-}1)$$

where

J = break rate, breaks/year/mile

J_o = break rate in 1970, breaks/year/mile

b = rate constant, 1/year

t = year

It is doubtful that such a steep increase in break rates as shown above would continue into the future. However, if it did, the break rate in the year 2000 would be 13.8 breaks/year/mile, which is higher than any rates reported in the literature. While an experimental growth rate b of 0.13/year may be inordinately high, the data show that breaks are increasing. If the data set is accurate and complete, the most likely explanation is the heavy construction that has taken place in the area in recent years. Some of the notes in Table 2-2 indicate that breaks have been caused by contact between pipes and other structures. With new construction there are simply more opportunities for contact to occur. Despite the possible explanation for the increase in breaks given above, the increased break rate may be misleading or exaggerated since a few missing entries in the break data for a system with a fairly small number of breaks may drastically change the calculated break rate. This appears to be the case here.

20. Another phenomenon shown by the data is that breaks tended to be repaired in August (25 percent) while no breaks were repaired in April and May. Most of the joint leaks were detected and repaired in the summer. This appears to be primarily due to the dry weather in the summer which makes it much easier to locate these breaks. Breaks are most likely to be more evenly spread throughout the year than indicated by the repair dates.

21. While the temporal trends associated with pipe breaks are interesting, the primary purpose of this analysis is to identify bad sections of pipe to be replaced or repaired. This was done using break rate data files prepared early in this work. The break rates are summarized by pipe and type of break in Table 2-3.

22. From Table 2-3 it is clear that a small number of pipes accounted for most of the breaks. Two thirds of the breaks can be assigned to pipes 2, 4, 11, and 15 which have overall break rates of 0.17, 0.65, 0.98, and 4.80 breaks/year/mile, respectively. Note that pipes 7, 17, and 20 also have high break rates but this is primarily because they are fairly short (e.g. pipe 17 is 60 ft long and had only one break). The next step in the analysis was to examine each pipe with a significant number of breaks or break rate to identify the causes of the breaks.

23. Pipe 2 is the 14,200-ft-long, 16-in. pipe from Key Bridge to Valve 291. It broke five times, which is not unduly high for such a length. There are no significant patterns to the breakage except that all of the breaks have occurred since 1977.

24. Pipe 4 is the 24-in. pipe from the Pentagon to Eads St. Five of its six breaks have been lead joint leaks, possibly indicating poor thrust

Table 2-3

Summary of Breaks by Pipe and Type of Break

Pipe Number	Pipe Length ft	Pipe Diameter in.	Pipe Breaks Number 1970-81	Pipe Breaks Rate break/year/mile	Joint Leaks Number 1970-81	Joint Leaks Rate break/year/mile	Unknown and Other Number 1970-81	Unknown and Other Rate break/year/mile	Overall Break Rate break/year/mile
1	6,000	30	0	0.00	0	0.00	1	0.08	0.08
2	14,200	16	3	0.10	1	0.03	1	0.03	0.17
3	500	16	0	0.00	0	0.00	0	0.00	0.00
4	4,400	24	1	0.11	5	0.55	0	0.00	0.65
5	3,200	16	0	0.00	1	0.15	0	0.00	0.15
6	1,200	10	0	0.00	0	0.00	1	0.40	0.40
7	1,400	8	1	0.34	0	0.00	1	0.34	0.68
8	1,500	8	1	0.32	0	0.00	0	0.00	0.32
9	1,600	12	0	0.00	0	0.00	0	0.00	0.00
10	600	18	0	0.00	0	0.00	0	0.00	0.00
11	3,910	16	0	0.00	0	0.00	8	0.98	0.98
12	1,600	24	0	0.00	0	0.00	0	0.00	0.00
13	1,600	8	0	0.00	0	0.00	0	0.00	0.00
14	1,000	8	1	0.48	0	0.00	1	0.48	0.96
15	700	6	5	3.43	0	0.00	2	1.37	4.80
16	2,600	8	1	0.18	0	0.00	0	0.00	0.18
17	60	10	1	8.00	0	0.00	0	0.00	8.00
18	1,400	6	1	0.34	0	0.00	0	0.00	0.34
19	1,000	8	1	0.48	0	0.00	0	0.00	0.48
20	800	6	1	0.60	0	0.00	0	0.00	0.60

restraint or backfill combined with waterhammer events in the area. If the breaks were in the same general area, then it may be economical to reinforce thrust blocks or install tie rods to restrain the pipe in critical areas.

25. Pipe 11, which is the 16-in. pipe around the east side of the Pentagon, had eight breaks. Most were corrosion related, indicating corrosive soil or stray current. The soil survey described in the next section indicates that corrosive soil is the problem in this area.

26. Pipe 15 has the worst break record with seven breaks in 12 years over a 700-ft length (i.e. 4.8 breaks/year/mile). This 6-in. pipe lies directly under I-395 and connects the Arlington system to the Pentagon heating plant. The primary problem with this pipe is line shear which accounted for all of the breaks for which the causes were reported. These breaks appear to be caused by the loads from the interstate highway.

Soil Corrosivity Tests

27. The corrosivity of soil to metal pipe can be determined by taking a sample of the soil and performing tests such as redox potential, pH, soil resistivity, and sulfide which indicate the corrosivity. The Ductile Iron Pipe Research Association (DIPRA) was asked to perform these soil tests for the WAD system. They provided the auger shown in Figure 2-1 which was used to obtain samples from the level of the pipe as opposed to taking samples at the surface.

28. A copy of DIPRA's test results is presented as Appendix A. In summary, pipe 11, located on the east side of the Pentagon, was found to lie in a corrosive soil. A ball of this organic clay soil which has low resistivity and negative redox potential is shown in Figure 2-2. This explains the large number of corrosion-related breaks along that pipe. Samples taken along pipes 2, 4, and 15 indicated that the soil was a fairly noncorrosive fill material. Potential corrosion problems exist in this fill material in that some fill material can contain flyash which results in a fairly corrosive soil.

29. The implication of these tests on the pipe break rate is that the rate should increase with time along pipe 11 which lies in the highly corrosive clay soil. Any replacement pipe should be made of inert material or wrapped to protect the pipe.

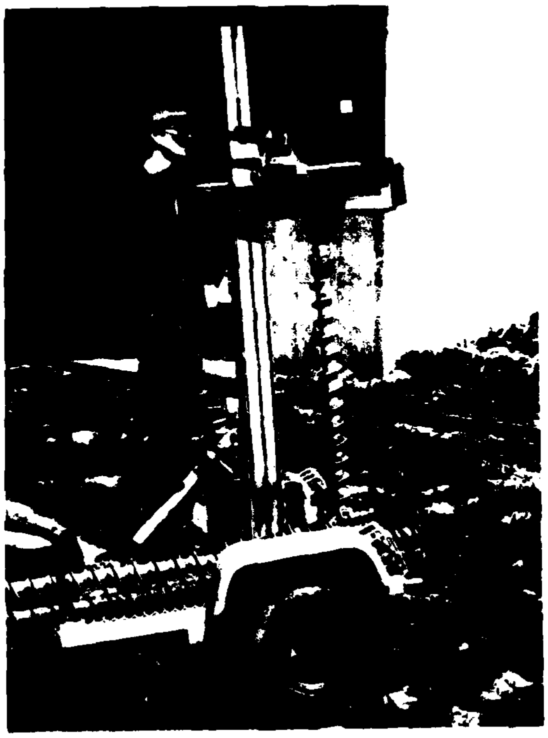


Figure 2-1. Auger used to take soil samples

Figure 2-2. Organic clay found at pipe 11

Costs of Breaks and Remedial Measures

30. Before an economic analysis can be performed, it is necessary to develop cost estimates for repair of breaks and installation of new pipe or some other remedial measure. The cost of remedial measures is described separately for each pipe. Those estimates are very rough "planning level" estimates. Before any contracts are let for work, detailed estimates must be made. The cost of breaks (cost of repair plus indirect costs) is discussed below.

31. The WAD provided WES with work orders from five breaks repaired since 1981. While these work orders only contain estimates of the cost, they can at least be regarded as approximations of the actual cost. The average cost was $2848 which will be rounded up to $3000 to account for inflation and uncertainty for use in economic analysis.

32. The cost to repair a break is only one portion of the cost associated with a break. There are also indirect costs due to interruption of service, flooding, disruption of traffic, and ice, all of which are difficult to quantify. In some cases these costs are negligible, while, on the other hand, if a pipe break should flood a Metro station or cut off water to the Pentagon, the costs can be enormous. For the purpose of this study, the cost to repair a break will be doubled (to $6000) to account for the indirect cost.

Economic Analysis of Breaks

Data required

33. The economic analysis of breaks consists of comparing the cost of repairing breaks with the cost of remedial action such as replacement of pipe, improvement of supports and restraints, and cathodic protection. The data required to perform the analysis include: existing break rate, cost to repair a break (plus damage/inconvenience factor), cost to replace pipe (or institute other remedial measure), interest rate, and fraction of pipe to be replaced. Three pipes in the FOWM System are candidates for this analysis--pipe 11, the 16-in. main around the Pentagon; pipe 4, the 24-in. main to the east of the Pentagon; and pipe 15, the 6-in. main connecting the Pentagon heating plant to the Arlington County system. Because each pipe has different problems, a slightly different analysis is required in each case.

Formulas

34. The formula for deciding when to replace the mains (or take other remedial action) is essentially:

$$\text{Replace if} \quad J > J^* \tag{2-2}$$

where

J = actual break rate, breaks/year/mile

J^* = critical break rate at which it is economical to replace or correct weak pipes, breaks/year/mile

35. In the case in which the corrective action involves a unit cost in dollars per length, J^* can be given by (ETL 1110-2-278):

$$J^* = \frac{5280\ C_r L \ln \left[(1 + b)/(1 + R)\right]}{C_b\left(\frac{1 + b}{1 + R}\right)^m - 1} \tag{2-3}$$

where

C_r = cost to replace pipe or correct problem, \$/ft

L = fraction of pipe to be replaced

b = rate constant for increase in breaks, 1/year

R = interest rate, %

C_b = cost to repair break, \$

m = study period, years

For this study, a time frame of 20 years was used with an interest rate of 7-7/8 percent, which is the interest rate for Federal water resources projects for fiscal year 1983.

36. In the case in which the cost was a single amount rather than a dollar per foot cost, J^* can be given by

$$J^* = \frac{C_r \ln \left[(1 + b)/(1 + R)\right]}{\ell C_b\left[\left(\frac{1 + b}{1 + R}\right)^m - 1\right]} \tag{2-4}$$

where

C_r = single amount to correct problem, \$

ℓ = length of pipe, miles

An analysis of each problem pipe is presented below.

Pipe analyses

37. Pipe 11. Pipe 11 has primarily corrosion-related problems. The two most likely solutions are replacement and cathodic protection. Equation 2-3 is first used to determine whether it is economical to replace pipe 11. The cost of break is given as \$6000 and the cost to replace the pipe with polyethylene encased pipe is \$80/ft. It is assumed that approximately one half of the pipe is in corrosive soil and will need replacement (i.e. L = 0.5). The primary uncertainty is the value of b (0 to 0.13/year). J* can be given by

$$J^* = \frac{5280(80)(0.5)\ \ln \left[(1 + b)/1.07875\right]}{(6000)\left[\left(\frac{1 + b}{1.07875}\right)^{20} - 1\right]} \qquad (2\text{-}5)$$

where

C_b = \$6000

C_r = \$80/ft

L = 0.5

R = 7-7/8%

m = 20 years

ℓ = 0.74 mile

C_b = \$6000

C_r = \$30,000

Values of J* for different values of b are given below:

b 1/yr	J* breaks/year/mile
0	0.66
0.04	0.46
0.13	0.20

These results indicate that cathodic protection is economical if it can be provided for \$30,000.

38. Cathodic protection is only useful for protecting pipes which are in still good condition. If the pipe has already deteriorated significantly,

cathodic protection cannot restore its strength. Deterioration shows up most dramatically as pitting. The next time pipe 11 is excavated due to a break or new construction, it should be examined for pits. If the pits have penetrated significantly into the pipe, then cathodic protection can be eliminated as a remedial measure.

39. Pipe 4. Pipe 4 is the 24-in. main west of the Pentagon to Eads St. Most (five out of six) of its breaks are lead joint leaks, which indicates poor jointing or bedding or inadequate restraints. These problems can be corrected by identifying specific locations where the breaks are concentrated and replacing tie rods or reinforcing thrust blocks in those areas. Unfortunately, there is no easy way to identify joints or bends which may need additional restraints. The only way to permanently correct the problem is to install new pipe. The cost to install new 24-in. pipe in a congested area is \$120/ft. Assuming all of the pipe will be replaced ($L = 1$) and the break rate due to leakage is not increasing, Equation 2-3 becomes (for large m)

$$J^* = \frac{5280\ C_r \ln (1 + R)}{C_b} \tag{2-6}$$

Substituting the actual values of cost and interest rate gives

$$J^* = \frac{5280\ (120) \ln (1.07875)}{6000} = 8.0 \tag{2-7}$$

Since the current break rate is only 0.55 breaks/mile/year, it is clearly not economical to replace the entire 24-in. pipe. This does not mean that the problem of joint leaks in this pipe can be ignored. Joint leaks tend to erode bedding material and result in major breaks. In a high value district, this can be catastrophic.

40. The WAD needs to pinpoint the cause and location of the joint leak and take remedial action to prevent fixture leaks. One way to prevent the problem from becoming worse is to conduct a sonic leak detection survey in the area. In this way the leaks can be detected and repaired in one pass at lower cost than waiting until they reach the surface. If the leaks are concentrated in a small area, it can be economical to install tie rods or bell joint clamps to adequately prevent the joints from moving.

Four values of b were inserted into Equation 2-5 to determine possible values of J* . They were 0 (no change in breaks), 0.014/year (observed in Binghamton, N.Y. (Walski and Pelliccia 1981), 0.04/year (observed in Buffalo, N.Y. (U.S. Army Engineer District, Buffalo 1981)), and 0.13/year as determined by Equation 2-1. The critical break rates determined using Equation 2-5 are summarized below:

b 1/yr	J* breaks/year/mile
0	3.42
0.014	3.06
0.040	2.46
0.130	0.96

Comparisons of these results with the calculated break rate for pipe 11 of 0.98 breaks/year/mile indicate that only at the highest value of b is it economical to replace the main.

41. The present worth cost of cathodic protection program for pipe 11 is $30,000 based on 50 anodes, 1 sacrificial anode placed every 40 ft at a cost of $100 per anode, with installation cost of $400 per anode plus $5000 for miscellaneous and contingencies (i.e. (100 + 400) × 50 + 5000 = $30,000). Equation 2-4 gives J* as

$$J^* = \frac{\$30{,}000 \ln \left[(1 + b)/1.07875\right]}{(0.74)\ 6000\left[\left(\frac{1 + b}{1.07875}\right)^{20} - 1\right]} \qquad (2\text{-}8)$$

42. <u>Pipe 15.</u> The pipe with the worst break record is pipe 15, a 6-in. main connecting the Arlington County system to the Pentagon heating plant. It is only 700 ft long but has had seven breaks in the past 12 years, which is a break rate of 4.8 breaks/year/mile. The breaks are primarily line shear. Such breaks are usually due to beam loading on a pipe with poor bedding or insufficient wall thickness. The only remedy is to replace the section of the pipe with new pipe. Using a value of b = 0 since this type of break usually does not increase with time and a replacement cost of $50/ft gives:

$$J^* = \frac{(5280)(50)(1.0)\ \ln\ (1.07875)}{6000} = 3.3 \qquad (2\text{-}9)$$

The above value of J^* is based on replacing the entire 700-ft main ($L = 1$). If, for example, closer investigation shows that the breaks were confined to a 200-ft section, then $J^* = 3.3\ (200/700) = 0.94$ and it would be economical to replace that section. Furthermore, if $b = 0.04$, a conservative value, then:

$$J^* = \frac{(5280)(50)(1.0)\ \ln\left(\frac{1.07875}{1.04}\right)}{(6000)} = 1.6 \qquad (2\text{-}10)$$

which indicates that the entire pipe could be economically replaced.

43. The breaks in pipe 15 may be due to construction of I-395. Breaks in the line may decrease now that the highway has been in place for some time. If breaks continue at their current rate, the weak sections should be pinpointed and replaced.

Recommendations

44. Pipe 11 (east side of Pentagon) is the most likely candidate for replacement or cathodic protection. During the first opportunity to visually inspect the pipe, it should be examined for pitting. If pitting is significant, the pipe should be replaced. If pitting is not significant, a firm specializing in corrosion protection should be hired to design a cathodic protection system.

45. If breaks continue along pipe 15 (heating plant), that pipe should be replaced. Ideally, the replacement will be limited to the portion responsible for the break rather than the entire pipe.

46. The WAD should conduct a sonic leak detection survey along pipe 4 (24-in. pipe between the Pentagon and the airport). If leak problems are confined to a small section of pipe, then the feasibility of remedial measures in that section should be investigated.

47. The WAD should keep better records of breaks and leaks in the FOWM System. A first step is to designate a map of the system as a map of breaks and use colored pins or some symbol to identify precise location of each break (e.g. red pins for main breaks, green for joint leaks, yellow for hydrant or hydrant lateral leaks). With such a map, it will be easy to identify "hot spots" in the system needing additional investigation to determine the feasibility of remedial measures. A second step is to identify a leak reporting system. Figure 2-3 shows a sample of a leak repair report. Such records will make it easy to make decisions as to remedial measures.

SANTA ROSA WATER UTILITY LEAK REPAIR REPORT

LOCATION:		MAP NO.:
DATE & TIME REPAIRED:	W.O. NO.	FOREMAN:

—DESCRIPTION OF DAMAGE—

WHAT PART WAS DAMAGED ?

☐ Pipe barrel ☐ Flange nuts, bolts, tie rods
☐ Joint ☐ Other –(describe): ____
☐ Valve ____

IN YOUR OPINION, WHAT CAUSED THE DAMAGE ? ____

☐ WATER MAIN	☐ SERVICE LATERAL	SIZE:	DEPTH TO TOP OF PIPE:

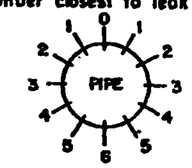
TYPE OF BREAK:

☐ Split ☐ Crushed Pipe
☐ Hole ☐ Cracked bell
☐ Circumferential split
☐ Broken coupling
☐ Service pulled
☐ Cracked at corporation stop
☐ Gasket blown
☐ Other (describe) ____

PIPE MATERIAL:

☐ Galv. iron ☐ Ductile iron ☐ Copper
☐ Black iron ☐ Cast Iron ☐ P.V.C.
☐ Steel ☐ A.C.P. ☐ Polybutylene
☐ Other ____

LOCATION OF LEAK: (circle number closest to leak)



EXAMINE BROKEN EDGE OF CAST OR DUCTILE IRON PIPE:

Original thickness:	Min. thickness of good grey metal remaining:	Deterioration is on: ☐ Outside ☐ Inside

SIZE OF LEAK: (circle one) A B C

Is there evidence of previous leak repairs in same general area ? ☐ YES ☐ NO	No of previous leak repair clamps present	Last repair date (if known)

TYPE OF SOIL:	EXISTING BEDDING:	IN YOUR OPINION, SHOULD PIPE BE REPLACED ?
☐ Rocky ☐ Sandy ☐ Adobe	☐ Gravel / Sand ☐ Native soil	☐ Yes ☐ No ☐ Do not know
☐ Clay ☐ Hard pan ☐ Loam	☐ Pea gravel	IF YES, EXPLAIN EXTENT ON REVERSE SIDE.

—DESCRIPTION OF REPAIR—

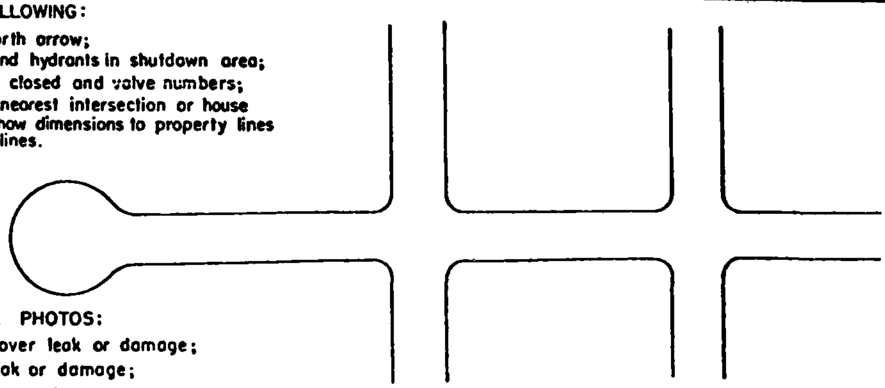
DAMAGED PART WAS: ☐ Repaired ☐ Replaced	If replaced, what material was used ?

IF REPAIRED, WHAT REPAIRS WERE MADE ?

☐ Leak clamp ☐ Repacked valve ☐ Other (describe) ____
☐ Welded ☐ Recaulked joint ____

FILL IN THE FOLLOWING:

1. Street name, north arrow;
2. Draw in main and hydrants in shutdown area;
3. Show all valves closed and valve numbers;
4. Locate leak to nearest intersection or house with address. Show dimensions to property lines or street centerlines.



ATTACH THREE PHOTOS:

1. Straight down over leak or damage;
2. Close up of leak or damage;
3. Any other photo which you feel will help.

NOTE: If possible, tag and label any replaced part which you feel describes the general level of corrosion or deterioration and deliver part to water field office with this form.

WORK ORDER, LEAK REPORT, & PICTURES MUST BE SUBMITTED AS SOON AS POSSIBLE !

Figure 2-3. Example of a leak repair report

48. Another potential problem area exists where the 30-in. steel main lies close to the metro tracks. There is often stray dc current associated with such a system, and this current can cause serious corrosion problems. Because of the importance of the 30-in. main, it is recommended that an "electrolosis survey" be conducted to determine the seriousness of the problem.

Summary

49. In general, the FOWM System is in good condition from the standpoint of main breaks. There are a few mains, however, for which the present worth of the cost of break repairs is comparable to cost of taking corrective action to prevent breaks. The effectiveness of the remedial measures depends on the precision with which the measures can be applied. For example, if only 200 ft of pipe 15 needs to be replaced, it is economical to replace the pipe, but if breaks are spread over the entire pipe, replacement is marginal. Better record keeping will make it much easier to make replacement decisions.

PART III: DEVELOPMENT OF A WATER DISTRIBUTION SYSTEM COMPUTER MODEL

Introduction

50. The Federally Owned Water Mains (FOWM), operated by the Washington Aqueduct Division (WAD), serve many important facilities in north Virginia, including the Pentagon, Washington National Airport, the Navy Annex, Fort Myer, and Arlington National Cemetery. There is some concern over the reliability of the system in an emergency situation such as a fire or failure of a line. Testing the response of the system to emergencies by shutting down part of the system or discharging water in the amount needed to fight fires would interfere with service. Therefore, in order to study these events, it is necessary to simulate them with a computer model.

51. With a model of the system it is possible to quickly and easily simulate such events as valving off segments of pipe, changes in water use, fire events, and new piping. Of course, for the model to be valid it is necessary to calibrate it carefully with observations from the field. Calibration is achieved by adjusting water use magnitude and distribution and pipe roughness (Hazen-Williams C-factor) until the heads predicted by the model agree with those observed in the field. In the following sections, the overall model is described, and the calibration process is documented.

System Description

52. The area to be modeled extends from the Dalecarlia Water Treatment Plant and Foxhall Tank in the District of Columbia to Washington National Airport, and is part of the "First High" service area served by the WAD. Pipe sizes range from 48 in. near the source to 6 in. in some of the mains near the extremity of the system. The flow in the First High service area is divided between the FOWM System and the remainder of First High system on the east side of the Key Bridge. A 30-in. and a 16-in. main cross the Key Bridge to serve the FOWM System. The 30-in. and 16-in. mains follow roughly parallel paths from the Key Bridge to the Pentagon. Past the Pentagon, the most important main is a 24-in. main which reduces to a 16-in. main as it approaches National Airport.

53. In order to study the behavior of the distribution system, it is

not necessary to include every pipe in the system. Rather, it is possible to analyze a skeletal system which includes only the major mains and still produces accurate results since the mains not included in the skeletized system do not carry a great deal of the flow. A map of the skeletized system to be modeled was prepared on a 1:7200 scale. It contained circled three-digit numbers corresponding to node numbers in black and elevations of the nodes in green. The one- or two-digit circled numbers referred to test numbers for model calibration, the distance between nodes (in feet) in red and pipe diameter (in inches) in red.

54. Pipes with diameters less than 12 in. were not included in the model of the skeletized system. There were a few exceptions, such as the pipes to the Navy Annex which were 8-in. pipes. The skeletal system did not include many of the small pipes in Arlington National Cemetery. The model also could account for opening of interconnections between the Arlington County system and the FOWM System.

55. The computer program used to calculate flows and pressures in the system is the MAPDIST program, which is the water distribution system analysis portion of MAPS (Methodology for Areawide Planning Studies)--a program developed at the U. S. Army Engineer Waterways Experiment Station. It uses the Hardy-Cross method as applied to loop equations. The user's guide and documentation for the program are provided in Chapter 17 of Engineer Manual (EM) 1110-2-502, Change 1. The program can be run on both the Boeing Computer Services system and the CDC Cybernet system.

56. A schematic drawing of the network showing the location of nodes is given in Figure 3-1. The characteristics of each line and node in the

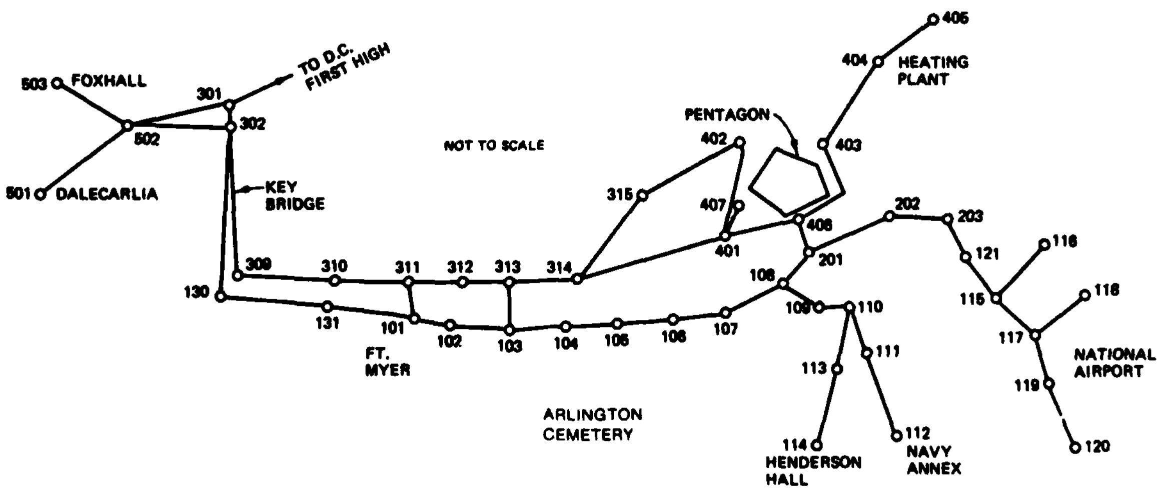


Figure 3-1. Skeletal pipe network for FOWM System

system are given in Table 3-1, which is a listing of a data file for a run of the MAPS computer program. There is one entry in the table for each pipe in the network as designated by a "LINE" card. Each of these cards contains numbers of the nodes connected by the pipes, the pipe diameter in inches, and the length in feet. The "ELEV" cards contain the elevation of each node in the system. The "TANK" card contains the water level above the bottom of the tank in feet. The "COEF" card contains the Hazen-Williams C-factor. The "OUTPUT" cards contain the water use rate in gallons per minute for each node.

Table 3-1

Example of Data for FOWM Model

Keyword	Node		Pipe Diameter in.	Pipe Length ft
LINE	501	502	48	11900
LINE	502	503	48	1200
LINE	502	301	48	7420
LINE	502	302	30	7320
LINE	302	130	16	1820
LINE	302	309	30	1580
LINE	309	310	30	2590
LINE	310	311	30	3340
LINE	311	312	30	340
LINE	312	313	30	1890
LINE	313	314	30	1960
LINE	314	315	8	1210
LINE	315	402	8	1600
LINE	131	102	16	3450
LINE	130	131	16	2360
LINE	311	101	16	530
LINE	101	102	16	63
LINE	102	103	16	1900
LINE	103	104	16	1390
LINE	104	105	16	1260
LINE	105	106	16	880
LINE	106	107	16	670
LINE	107	108	16	240
LINE	108	109	10	420
LINE	109	110	8	66
LINE	110	113	8	1460
LINE	113	114	8	1150
LINE	110	111	8	720
LINE	111	112	8	1400
LINE	108	201	12	1610
LINE	201	406	18	700
LINE	406	401	24	1700
LINE	401	402	16	1220
LINE	314	401	30	1830
LINE	403	404	8	1120
LINE	401	407	16	1225
LINE	404	405	6	1170
LINE	403	406	16	1940
LINE	201	202	24	1860
LINE	202	203	24	2800
LINE	203	121	16	1730
LINE	121	115	16	1540
LINE	115	116	8	1250
LINE	115	117	16	1650
LINE	117	118	8	1100
LINE	117	119	16	570
LINE	119	120	8	1330

Keyword	Node	Elevation, ft
ELEV	101	65
ELEV	102	65
ELEV	103	60
ELEV	104	50
ELEV	105	60
ELEV	106	60
ELEV	107	65
ELEV	108	65
ELEV	109	65
ELEV	110	85
ELEV	111	150
ELEV	112	150
ELEV	114	150
ELEV	115	30
ELEV	116	30

(Continued)

Table 3-1 (Concluded)

Keyword	Node	Elevation, ft
ELEV	117	40
ELEV	118	39
ELEV	119	40
ELEV	120	18
ELEV	121	30
ELEV	130	50
ELEV	131	10
ELEV	201	45
ELEV	202	45
ELEV	203	50
ELEV	301	75
ELEV	302	75
ELEV	309	15
ELEV	310	10
ELEV	311	35
ELEV	312	20
ELEV	313	30
ELEV	314	30
ELEV	315	12
ELEV	401	30
ELEV	402	30
ELEV	403	30
ELEV	404	10
ELEV	405	10
ELEV	407	50
ELEV	406	30

Keyword	Node	Elevation, ft
ELEV	501	150
ELEV	502	155
ELEV	503	235

Keyword	Node	Water Level, ft
TANK	503	10

Keyword	C-factor
COEF	70

Keyword	Node	Water Use Rate, gpm/node
OUTPUT	403	200
OUTPUT	406	600
OUTPUT	101	200
OUTPUT	111	100
OUTPUT	112	100
OUTPUT	117	100
OUTPUT	114	100
OUTPUT	116	100
OUTPUT	118	100
OUTPUT	119	100
OUTPUT	120	100
OUTPUT	103	50
OUTPUT	104	50
OUTPUT	404	100
OUTPUT	301	5000

Table 3-2

Water Use in FOWM System

User	Node Number	Percent of Water Use
Pentagon	406,403	40
Washington National Airport	116, 117, 118, 119, 110	25
Fort Myer	101	10
Arlington National Cemetery	103, 104	5
Pentagon Heating Plant	404	5
Navy Annex	111, 112	10
Henderson Hall	114	5
		100

57. Assigning water use to the nodes in the network is quite difficult in this study because of the lack of metering. Table 3-2 shows how water use was divided among major users served by the FOWM System. For model calibration, flow of 5000 gpm between the Foxhall Reservoir and the District of Columbia portion of the First High service area was used.

58. In actually running the program, the data were divided into two computer files: MAIN, which contained a description of the pipes and nodes in the system; and USE, which contained water use data. These files were merged along with data on C-factors and fire flows to make up data for a run of the program.

Calibration Process

Data

59. In order to calibrate the model, it is necessary to collect data on pressures and flows in the system during a period of time in which the operation of pumps, tanks, and valves is constant (i.e. pumps are not switched on and off; tank levels do not fluctuate widely; and valve settings are not changed). Such data were collected on 18 November 1982 between 10:20 a.m. and 1:45 p.m. and throughout the day and on 12-13 April 1983. On 18 November the pumps at the Dalecarlia Pumping Station to the First High service area were off, no valve settings were changed, and the water level at Dalecarlia Tank changed only from 245.8 ft to 244.7 ft. That set of data is used for calibration while the set collected on 12-13 April is used for later verification.

60. One problem with the data collection for this work was that it was not possible to meter the flow to the FOWM System as opposed to the remainder of the First High service area. It was, however, possible to determine the drawdown on the Foxhall Tank over a known period of time. Given that the water level dropped by 1.9 ft from 10:00 a.m. to 2:00 p.m. on 18 November and there was a change in storage of 0.877 MG per foot of elevation change, the outflow from the tank was 1.66 MG/4 hr or 6917 gpm during the time testing was conducted. Since it was highly unlikely that any flow was occurring from the other First High reservoir (near U.S. Soldiers Home) to Key Bridge, this flow represents an upper limit of the flow to the FOWM System.

61. Pressure readings were taken at numerous locations in the FOWM System on 18 November 1983. At several of these locations fire flow tests

were also conducted. The results of the pressure readings and fire flow tests are given in Table 3-3. Some additional pressure readings were made on 12-13 April 1983, but these were not included in the table since they correspond to a different set of water levels in reservoirs and pump operation. The data shown in Table 3-3 correspond to the Foxhall Road Tank being filled to elevation 245 ft and the pumps at Dalecarlia, which serve the First High Service Area (to which the FOWM System is connected), being off.

62. During the testing conducted on 13 April 83, the flow measured at the meter on the 30-in. mains in the vault under Key Bridge was compared with flow measured using a pitot tube. Over a 21.6-min period, the meter recorded a flow of 40,000 gal, which corresponds to an average flow rate of 1850 gpm. A pitot traverse of the 30-in. main, which is actually 24 in. in diameter in the vault immediately upstream of a gate valve and meter, indicated that the flow was 1680 gpm. The 10 percent difference in readings was most likely due to the fact that the pitot tube had to be inserted into the pipe very close to a bend. The meter reading in this case was probably more accurate than the pitot tube reading. Since the flow measurement was made during a weekday, the reading of 1850 gpm (plus roughly 300 gpm, which is the flow in the parallel 16-in. line) can be considered a typical daytime water use in the FOWM System.

63. Another type of measurement conducted during the 12 April 83 field work was head loss. This measurement was conducted by connecting a manometer between two hydrants along a major transmission main. The hydraulic gradient in feet/1000 feet could then be measured by reading the manometer to determine head loss and dividing by the length of pipe over which the head loss occurred in thousands of feet. The tests were conducted along the west side of the Pentagon between hydrants 13 and 12 (210 ft of 24-in. pipe) and between hydrants 57 and 58 along the George Washington Parkway in front of Washington National Airport (720 ft of 16-in. pipe). The results showed that the hydraulic grade line had a slope of 1.0 ft/1000 ft on the west side of the Pentagon and 0.4 ft/1000 ft in front of Washington National Airport.

Results

64. Calibration of the model of the FOWM System presented some unusual problems in that there was no metering of individual water users; the quality of available elevation data was poor; and there was very little head loss in the system at normal flows. The lack of metering meant that the distribution

Table 3-3
Summary of Pressure Test Results

Test Number	Location and Pipe Size	Time	Hydrant No.	Pressure psi	Elevation ft	Head ft	Hydrant Pressure psi	Test Head ft	Hydrant Discharge gpm
1	Vault below Key Bridge (30 in.)	10:21	--	100	9	240	--	--	--
2	Park Service hose bib (16 in.)	10:30	--	92	30	242	--	--	--
3	Along Metro (30 in.)	11:00	136A	93	25	240	--	--	--
4	Pentagon north parking lot (8 in.)	11:18	15	83	48	240	78	228	1300
5	South side (16 in.) Pentagon	11:43	5	92	34	246	85	230	1140
6	Riverside parking lot (8 in.)	12:00	36	95	12	231	65	162	1620
7	Heating plant (6 in.)	12:20	22	98	12	238	68	169	800
8	National Airport (16 in.) entrance	12:40	56	95	20	239	80	205	1140
9	National Airport south terminal (8 in.)	1:00	41	89	18	224	--	--	--
10	Navy Annex (8 in.)*	1:15	46	62	150	293	45	254	650

(Continued)

* Connected to Arlington System.

Table 3-3 (Concluded)

Test Number	Location and Pipe Size	Time	Hydrant No.	Pressure psi	Elevation ft	Head ft	Hydrant Pressure psi	Test Head ft	Hydrant Discharge gpm
11	H&S Headquarters (past Henderson Hall) (8 in.)	1:30	--	60-80**	150	289-335	--	--	--
12	Arlington Cemetery Visitors Center (16 in.)	1:45	--	82	48	239	--	--	--
13	Lee Mansion (4 in.)†	2:10	--	47	210	318	25	267	440

** Booster pump cycling on/off.
† Downstream of Fort Myer booster pumps.

of water use had to be estimated based on size of the installations and water use had to be adjusted during calibration.

65. The head loss across the FOWM System during normal afternoon use was on the order of 0.3 ft/1000 ft, which is extremely low. This meant that pressure readings taken during normal use were not very helpful for calibration since head loss was of the same order of magnitude as errors in the elevation data. Therefore, it was not possible to use the method of model calibration described in ETL 1110-2-278, Inclosure 5, for calibration. (At one step in that procedure it is necessary to divide by the head loss at normal use, but dividing by a number such as 4 ± 10 is meaningless.) Therefore, calibration calculations had to be based primarily on the comparison between observed and predicted heads during fire flow tests.

66. The results of the calibration are shown in Table 3-4. The first two columns give the water use and C-factor used in a particular set of model runs. Columns 3 through 7 give the heads predicted at Key Bridge, along the Metro about 2000 ft north of Memorial Bridge, at the north entrance to the Pentagon, at the entrance to Washington National Airport, and at the Pentagon heating plant, respectively. Columns 8 through 11 give the drawdown in the hydraulic grade line during fire flow tests at the North Pentagon entrance, the airport entrance, Pentagon heating plant, and Riverside parking lot (Pentagon), respectively. The final two columns give the hydraulic gradient predicted by the model along the west side of the Pentagon (between nodes 401 and 406) and along the George Washington Parkway at the airport entrances (between nodes 115 and 117).

Table 3-4

Summary of Calibration Results

		Head at Nodes at Normal Flow, ft*					Drop In Head During Fire Flow Test, ft**				Hydraulic Gradient ft/1000 ft	
C-Factor	Water Use in FOWM System gpm	Key Bridge Vault (309)	Along Metro (312)	North Pentagon (407)	Airport Entrance (115)	Heating Plant (404)	North Pentagon (Q=1300)	Airport Entrance (Q=1140)	Heating (Q=800)	Riverside Parking (Q=1620)	West Pentagon (24 in.)	Airport Entrance (16 in.)
120	1000	245	245	244	244	244	5	9	--	--	0.1	0.1
120	2000	243	243	243	242	242	4	7	--	--	0.2	0.1
80	1000	244	244	244	243	243	4	14	--	--	0.2	0.2
80	2000	242	241	240	239	239	6	18	39	51	0.4	0.3
70	2000	242	240	239	237	238	9	23	50	65	0.5	0.4
60	2000	241	238	238	235	235	17	34	66	84	0.7	0.5
Actual Measured		240	240	240	239	238	12	34	69	69	1.0	0.4

* Parens indicate node number.
** Q is the hydrant discharge, gpm.

67. The two parameters to be adjusted during calibration are the water use in the FOWM System and the Hazen-Williams C-factor. Each row in Table 3-4 corresponds to a pair of values of C-factor and water use tested using the model, except for the final row which contains the measured values of head or change in head. In some of the early test runs both the water use and C-factor were adjusted. Since water use has less effect on head, especially at high flow, than C-factor, and field data showed that water use should be approximately 2000 gpm, use was set at 2000 gpm and final adjustments were made on the C-factor only.

68. The heads recorded at normal flow do not provide much guidance on which parameter to adjust, and by how much, since Table 3-4 shows the correlation between model and field results is good for virtually any reasonable value of C-factor and water use. The decrease in head during a fire flow test gives a much better indication of the effect of C-factor and water use on the model results. The hydraulic gradient tests do give some guidance on accuracy of calibration at lower flow rates.

69. In general, the best correlations between observed and predicted heads occurred at a C-factor of 70 although in a few cases a C-factor of 60 produced better results. The only value that was not in agreement was the hydraulic gradient along the west side of the Pentagon. It was twice the value predicted by the model. Rearranging the Hazen-Williams equation shows that, for the hydraulic gradient to be 1.0 ft/1000 ft in a 24-in.-diam pipe, the flow and C-factor must be related by

$$\frac{Q}{C} = 0.0067(h/L)^{0.54}\ D^{2.63} = 0.0067(1)^{0.54}(24)^{2.63} = 28.6 \qquad (3\text{-}1)$$

where

Q = flow, gpm

C = Hazen-Williams C-factor

h/L = hydraulic gradient

D = pipe diameter, in.

The model predicts a flow in that pipe of 1365 gpm which would require a C-factor of 48 to give a head loss of 1.0 ft/1000 ft. Similarly, a C-factor of 70 would require a flow of 1995 gpm to yield calibration. Another more likely explanation is that valve number V120 may not be completely open and is therefore causing minor head loss of approximately 0.5 ft. Nevertheless, since other indicators show that the best calibration occurs for a C-factor of 70 and for water use of 2000 gpm, the model can be considered calibrated with those values.

70. A C-factor of 70 indicates that the pipes in the FOWM System are quite rough. This would correspond to an appreciable amount of corrosion in an unlined, 40-year-old pipe. This indicates that some of the critical lines in the system may need cleaning and lining, and that the quality of the water produced at the Dalecarlia should be checked to ensure that it is not corrosive.

Summary

71. Good quality data are required for developing a water distribution system model. Even with the extensive data collected for this model, calibration is not completely accurate because of the lack of data on water use distribution and a combination of poor elevation data and very low head loss. Nevertheless, the techniques for model development described in this section can serve as a guide for others faced with the problems involved with developing models.

PART IV: USING PITOT TUBES TO CALIBRATE AND TEST GAGES

Introduction

72. Calibrating or testing a gage or meter used to determine flow in a closed conduit requires comparing the reading from the gage or meter with the actual flow rate calculated using another method. For small flow rates, a "bucket and stopwatch" technique can be used to determine the actual flow rate. Tracer methods are also available which involve feeding a tracer into the flow at a known rate and then measuring the concentration downstream. If the pipe characteristics are known, ultrasonic flowmeters can be used, or small propeller or magnetic velocity meters can be inserted into the flow.

73. Generally, the easiest and most reliable method to measure flow in closed conduits (except possibly the "bucket and stopwatch" method) is to insert a pitot tube into the flow. The pitot tube senses velocity at points in the fluid. The readings can be integrated across the conduit to give the flow rate and average velocity.

74. The following paragraphs describe how a pitot tube was used to calibrate an orifice plate used to measure flow through a machine designed to test concrete for cavitation. A schematic diagram of the apparatus is shown in Figure 4-1. During production runs of the cavitation machine, the orifice plate will be used to measure flow. However, the orifice plate must be calibrated against measurements made using the pitot tube before production runs can be made.

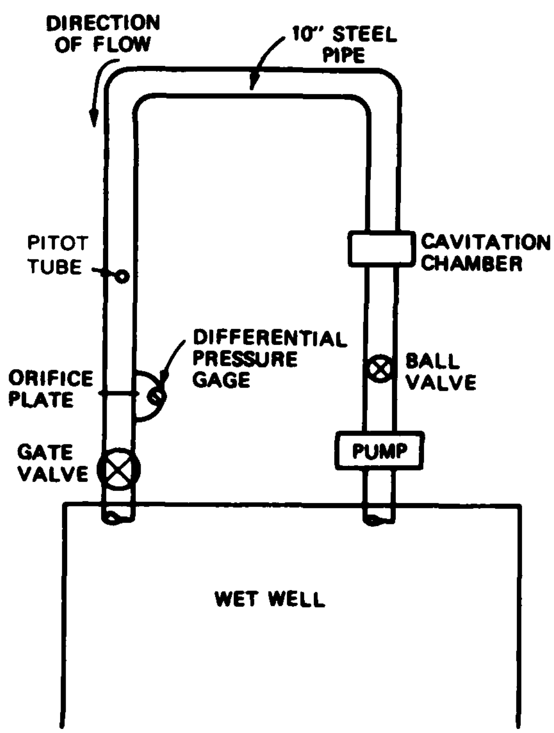


Figure 4-1. Apparatus used to calibrate an orifice plate

Calibration

Relevant equations

75. Average velocity in the pipe can be related to the head drop across the orifice by:

$$v = \frac{A_o}{A_1} C \sqrt{\frac{2g\Delta P(144)}{\gamma}} \tag{4-1}$$

where

v = velocity in pipe, ft/sec

A_o = area of orifice, ft^2

A_1 = area of pipe, ft^2

C = orifice coefficient

g = 32.2 ft/sec^2

ΔP = pressure drop across orifice, psi

γ = specific weight of water, 62.4 pcf

Since $A_o/A_1 = (D_o/D_1)^2$, Equation 4-1 can be rewritten for $D_o = 5$ in. (the opening of the orifice) and $D_1 = 10$ in. (the full pipe diameter) as

$$v = 3.05C \sqrt{\Delta P} \tag{4-2}$$

In order to calibrate the orifice, it is now necessary to determine the velocity in the 10-in. pipe for an array of values for ΔP, and calculate the average value of C from

$$C = \frac{0.328v}{\sqrt{\Delta P}} \tag{4-3}$$

The ΔP value can be determined by connecting a differential pressure gage to taps on each side of the orifice. Velocity v can be determined using a pitot tube.

76. The formula for determining point velocity from the pressure difference detected by a pitot tube at a point in the flow is

$$v_p = Ak\sqrt{\Delta P} \tag{4-4}$$

where

v_p = velocity at pitot tube tip, ft/sec

A = constant (12.2 psi or 2.32 in. H_2O)

k = pitot tube constant

ΔP = difference in pressure between legs of pitot tube, psi or in. H_2O

In this study, differential pressure was measured in two ways: (a) an air-water manometer for which the readings were made in inches of water, and (b) a differential pressure gage reporting in pounds per square inch. These point velocities were integrated using the method described in Part VII to give average velocity.

Procedure

77. Unlike a typical water distribution system which is always under pressure, flow in the cavitation equipment can be turned off except during testing, so it was possible to tap the pipe when it was empty. The pitot tube was inserted into the pipe through a 1-in. corporation cock which was specially drilled and threaded to accept the pitot tube.

78. The procedure for testing consisted of: (a) turning on the pump, (b) filling the manometer with water and bleeding air from the lines to the manometer and differential pressure gage, (c) recording the differential pressure from the gage connected across the orifice plate, (d) recording the differential pressure from the manometer or differential gage connected to the pitot tube at 11 points from the bottom of the pipe to the top of the pipe at 1-in. increments, and (e) changing the flow rate by adjusting the values and repeating steps (c), (d), and (e) until a wide range of flows had been measured.

Results

79. Once the measurements were made, the velocity in the 10-in. pipe was calculated using the computer program PITOT (developed at the U. S. Army Engineer Waterways Experiment Station) which converts the differential pressure readings from the pitot tube into point velocities and integrates the velocity profile to determine flow and average velocity.

80. The results of the tests are summarized in Table 4-1. For fairly large velocities the orifice flow coefficient C was fairly constant. As the velocity dropped, the value of C decreased. This was caused by the fact that in the later runs at low velocities the ball valve was throttled substantially and the gate valve was open. Therefore, it was doubtful that flow extended across the entire pipe downstream of the orifice plate in runs 7, 10, and 11. Therefore, the results from these runs were not used in determining an average C.

Discussion

81. The C value of 0.49 is lower than typical values for C

Table 4-1

Results of Calibration of Orifice Plate

Run No.	Velocity by Pitot Tube v ft/sec	Differential Pressure at Orifice Plate ΔP psi	$C = \frac{0.328v}{\sqrt{P}}$
1	4.2	7.1	0.52
2	3.7	6.9	0.46
3	3.5	5.4	0.49
4	2.5	2.8	0.49
5	4.3 (p)	7.7	0.51
6	3.3 (p)	4.7	0.50
7	2.3 (p)	3.1	0.43*
8	3.7	7.7	0.44
9	3.2	4.3	0.50
10	1.3	1.3	0.34*
11	2.0	2.0	0.46*
		Ave.	0.49

Note: (p) indicates readings made with differential pressure gage; other readings made with manometer.

* Indicates value was not used in averaging.

(approximately 0.62) for a $(D_o/D) = 0.5$ and a Reynolds number greater than 10^4. This is primarily due to the fact that the ports at which the pressure drop across the orifice was measured were located a significant distance upstream and downstream of the orifice. In a standard orifice the ports are located at the orifice.

82. Some problems were encountered in measuring the velocities with the pitot tube as the velocities would change somewhat over time. This was apparently caused by the ball valve moving slightly during the course of the test. Some additional problems occurred when the cavitation device would leak and suck air into the lines, thus producing unpredictable readings from the pitot tube. These readings were not used.

83. The most significant possible error was caused by the pitot tube and orifice plate being preceded by 10 and 15 ft of straight pipe, respectively. This was significantly less than the 30 pipe diameters of straight

pipe usually recommended for such installations. This proximity to a bend resulted in velocity profiles which deviated from what would be expected in a smooth, straight pipe in that maximum velocity was not at the center of the pipe. This was apparently due to secondary currents caused by the bend and could not be avoided because of the configuration of the apparatus. This error was probably small since the measured flows agreed fairly well with flows determined independently using head produced by the pump and the pump head curve for that pump.

84. Overall, the air-water manometer and differential pressure gage produced the same results when attached to the pitot tube. The manometer is superior for field use because: (a) it is less expensive and delicate; (b) it is slightly more precise (manometer could be read to 0.01 in. H_2O, while gage could be read to 0.03 in.); (c) it is more rugged; and (d) it works if the pitot tube is rotated 180 deg without reconnecting any pipe (i.e. works for flows in either direction).

Summary

85. Velocity in a 10-in. line can be determined from the pressure drop across the orifice plate using

$$v = 1.49\sqrt{\Delta P} \qquad (4\text{-}5)$$

Equation 4-5 is plotted in Figure 4-2 showing all data points. At low velocities when the gate valve is open, Equation 4-5 may overestimate velocity slightly. This should not be a problem since the cavitation apparatus will not be operated for this range of velocities.

86. In terms of flow rate QS , Equation 4-5 can be rewritten

$$Q(\text{cfs}) = 0.81\sqrt{\Delta P} \qquad (4\text{-}6a)$$

$$Q(\text{gpm}) = 364\sqrt{\Delta P} \qquad (4\text{-}6b)$$

for flow in cubic feet per second and gallons per minute, respectively.

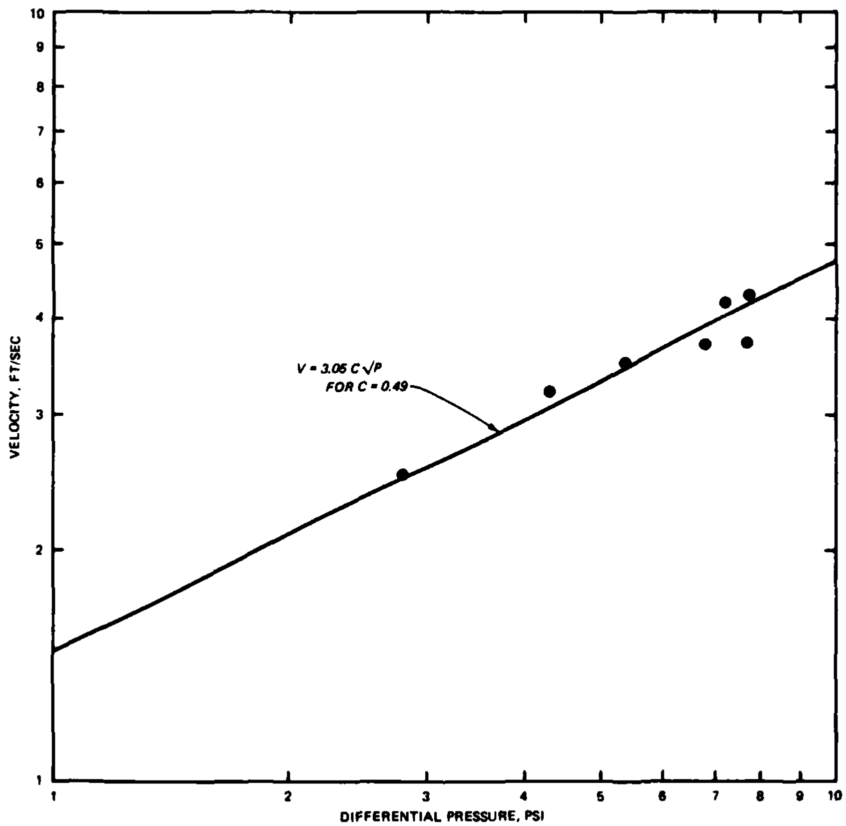


Figure 4-2. Plot of relationship of velocity and differential pressure

PART V: DETERMINING PIPE ROUGHNESS AND HEAD LOSS

Background

87. Water utility engineers need to know the internal roughness of pipes in order to analyze flows and pressures in their systems. The Hazen-Williams C-factor (Williams and Hazen 1920) is the most commonly used indicator of pipe roughness. Even though it actually depends slightly on the Reynolds number (i.e. velocity and diameter) (Walski 1983b), the C-factor is primarily a function of pipe roughness and is a value for which water engineers have considerable experience.

88. Typical values for C for a wide variety of pipes are available from many sources. One of the best summaries is Lamont (1981). There is, however, a great deal of variance about these typical values (e.g. while a typical pipe in a given category may have a C of 90, C-factors for an individual pipe may range from 60 to 120). It is, therefore, necessary to measure C-factors in a few pipes in a system before a detailed hydraulic analysis of the system can be performed.

89. Measuring this roughness is not easy since an engineer cannot see into a pipe, and even if this were possible, relating the size of randomly spaced and sized roughness elements to C is far from simple. The engineer must measure C indirectly by measuring the other quantities described by the Hazen-Williams equation and solving for C as

$$C = 3.56QL^{0.54}h^{-0.54}D^{-2.63} \qquad (5\text{-}1a)$$

or

$$C = 8.70VL^{0.54}h^{-0.54}D^{-0.63} \qquad (5\text{-}1b)$$

where

Q = flow, gpm
L = length, ft
h = head loss, ft
D = diameter, in.
V = velocity, ft/sec

90. In this part, procedures for conducting head loss tests are

presented including a description of measurements that must be made in the field and techniques for making these measurements (including techniques which do not involve tapping of pipes). The procedures are illustrated using the results of actual field tests conducted by the U. S. Army Engin r Waterways Experiment Station (WES) and potential problems are discussed.

Conducting Head Loss Tests

Measurements

91. Conducting a test to determine C involves measuring each of the unknowns on the right side of Equation 5-1. Since, in most cases, there is more than one way to measure each parameter, the implications of the measuring techniques must be considered. The decisions that must be made are described below.

92. Selection of test section. The line selected for testing should have a constant diameter pipe with uniform internal characteristics (e.g. all 8 in. unlined cast iron). There should be no change in flow along the pipe due to junctions. It is best not to have a significant loss of head along the pipe due to valves and bends since this makes interpretation of the test results difficult.

93. Head loss. There are two approaches for measuring head loss: (a) parallel pipe method, and (c) two gage method. In the parallel pipe method, the head loss is measured directly using a differential pressure gage (or manometer) that is connected to both ends of the section of pipe to be tested as shown in Figure 5-1. It is critical that the head detected by each side of the differential gage be exactly the same as the head at each end of the pipe test section. That means there must be no leakage or use along the parallel pipe (actually a hose is usually used instead of a rigid pipe). For greatest accuracy, the range of the differential pressure gage should be only slightly greater than the expected reading. Pressure snubbers may be required on the gage to dampen vibration due to surges in the system.

94. In the two gage method, pressure is measured at each end of the test section along with the difference in elevation, and head loss is calculated as

$$h = (P_1 - P_2)2.31 + z_1 - z_2 \qquad (5\text{-}2)$$

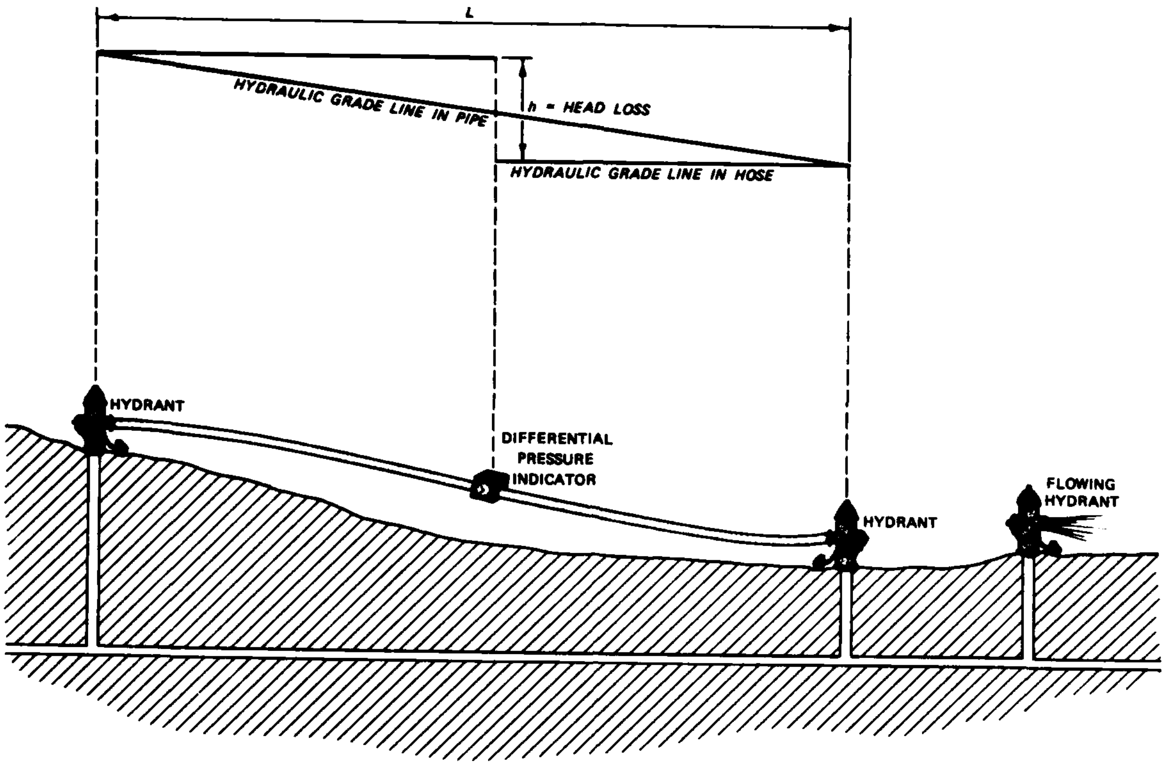


Figure 5-1. Head loss measured by parallel pipe method

where

h = head loss, ft

P_1 = pressure at beginning of test section, psi

P_2 = pressure at end of test section, psi

z_1 = elevation of gage at beginning of test section, ft

z_2 = elevation of gage at end of test section, ft

The two gage method can produce results accurate to ± 2 ft, and as such is significantly less accurate than the parallel pipe method (which is accurate to ± 0.1 ft or better depending on the differential gage). Therefore, the two gage method is only applicable to test sections where there is very large head loss (>20 ft). The elevation difference (z_1 - z_2) should be measured using surveying equipment as opposed to being read from a contour map.

95. Length. The length to be used in determining C is the length over which the head loss occurs. This is not necessarily the length of the parallel pipe. If minor losses are not negligible, the C-factor calculated based on actual length will be lower than the C-factor in the pipe alone (as it will

include loss in bends and fittings). To determine the C-factor for the pipe alone, an equivalent length for bends and fittings which accounts for minor losses should be added to the actual length.

96. Diameter. Since, in many instances, the actual diameter of the pipe is not necessarily the same as the nominal diameter, significant errors can result in using a C-factor based on actual diameter to predict head loss based on nominal diameter. Since in most cases engineers will be using the values for C in situations in which they will only know nominal diameter, it is best to calculate and report C using nominal diameter.

97. Velocity. The most straightforward way to measure velocity or flow is to insert a flow-measuring device in the pipe. On very rare occasions, there may be a flow-measuring device (e.g. venturi meter) permanently mounted at one end of the test section. Usually a pitot tube is inserted into the pipe to measure velocity while head loss is being recorded. If the characteristics of the pipe (in particular wall thickness) are known, a clamp-on ultrasonic gage can also be used. There are also some propeller type flowmeters on the market that can be inserted directly into the pipe through a corporation stop. Regardless of the exact method used, directly measuring flow or velocity involves excavating the pipe with the associated costs for shoring, repaving, and traffic control. Directly measuring flow does however provide the most accurate results.

98. Velocity from hydrant discharge. When the pipe test section can be isolated from the remainder of the system such that virtually all of the flow through the pipe is discharged from one or more metered hydrants, the total discharge from the hydrant can be used in the formula for C . This can be done on dead-end lines or pipes which can be valved off from the remainder of the system except for one end. (In Figure 5-2, this would mean closing valve B.) This method is only workable for fairly small pipes (<18 in.) in which discharge from a hydrant(s) alone can create measurable head loss.

Indirect method for determining C

99. In many cases a large pipe cannot be valved off without adversely affecting service, and the cost of excavating and tapping a pipe are prohibitive. For these cases, an indirect method for calculating C and flow, described below, can be used.

100. The problem is one of how to measure C and Q without having to excavate or tap the pipe. In order to do this it is necessary to know the

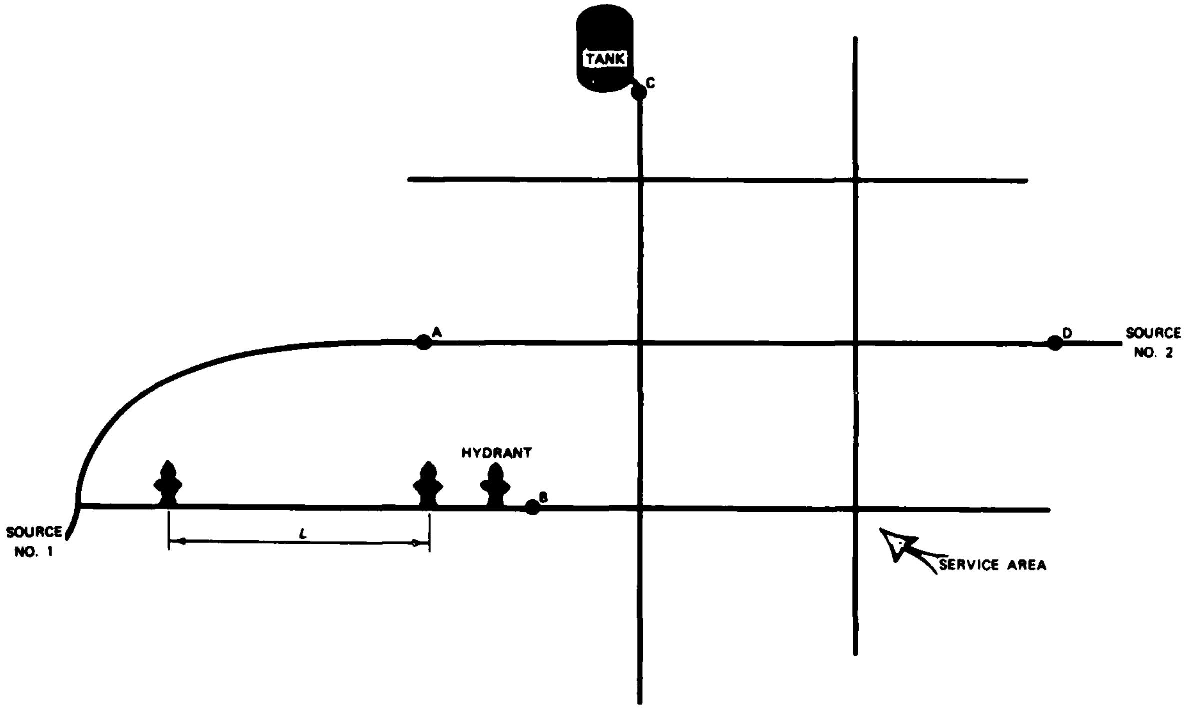


Figure 5-2. Methods for isolating pipe for testing

head loss in the line both at a flow rate Q and when that flow rate is changed by a known amount (say by opening a hydrant).

101. The key to this procedure is that the only way water can get to the service area downstream of the test pipe is through the test pipe. Storage tanks, other sources, and other paths must be shut off. This can be accomplished in one of two ways as shown for the sample service area in Figure 5-2. Either valve B can be shut and Q can be measured at the hydrant as described in the previous section or valves A, C, and D can be shut in which case Q is the actual water use in the service area. The bottom line is that there should be no "back door" through which water could get to the flowing hydrant(s).

102. The indirect procedure for determining C is based on measuring head loss at two flow rates and the difference in flow rate (caused by opening a hydrant), and then simultaneously solving the head loss equations for the test section at both flow rates to yield C and the flow rate in the pipe. The information that will be known is:

a. Diameter D of pipe (diameters if pipes are connected in series).

b. Length L of pipe(s).

c. Discharge from a hydrant(s) Q_f in the service area.

d. Head loss in length L at background flow h_1 and while hydrant is open h_2.

e. A pressure reading in the service area at background flow P_1 and while hydrant is open P_2. The term "background flow" refers to the flow in the pipe when no hydrants are open.

103. The head loss in a single pipe for background and hydrant flow can be determined by rearranging Equation 5-1 as

$$\frac{h_1}{L} = \frac{10.4}{D^m}\left(\frac{Q_1}{C}\right)^n \qquad (5\text{-}3)$$

$$\frac{h_2}{L} = \frac{10.4}{D^m}\left(\frac{Q_2 + Q_f}{C}\right)^n \qquad (5\text{-}4)$$

where

h_1 = head loss background flow, ft

L = length of pipe (including equivalent pipe for minor losses), ft

D = diameter of pipe, in.

m = exponent on diameter in head loss equation (4.87)

Q_1 = flow when hydrant(s) closed, gpm

C = Hazen-Williams C-factor

n = exponent on flow in head loss equation (1.85)

h_2 = head loss when hydrant(s) open, ft

Q_2 = flow to service area when hydrant(s) open, gpm

Q_f = hydrant discharge, gpm

The decision on whether to include equivalent length of pipe for minor losses in L depends on whether the C to be determined will be for the pipe or for the pipe plus losses. If it is for the pipe only, L must include the minor losses. The unknowns to be determined are Q_1 (and Q_2) and C.

104. Solving Equations 5-3 and 5-4 for Q_1 and C gives

$$C = \left(\frac{10.4L}{D^m h_2}\right)^{1/n} Q_f \left[\frac{1}{1 - \left(h_1/h_2\right)^{1/n} Q_r}\right] \qquad (5\text{-}5)$$

and

$$Q_1 = \left(\frac{h_1 D^m}{10.4L}\right)^{1/n} C \qquad (5\text{-}6)$$

where

$$Q_r = \frac{Q_2}{Q_1}$$

105. The term Q_r in Equation 5-5 reflects the fact that, when the hydrant is opened, flow out of orifices and nozzles in the service area decreases. In some cases the effect is offset simply by individuals opening valves on faucets wider, so that the flow is unchanged ($Q_r = 1$). The biggest change in flow occurs if no faucet settings are changed during the test and flow can be given by the orifice equation

$$Q = \sum q_i = \sum c_i \sqrt{P_i} \qquad (5\text{-}7)$$

where

q_i = flow out of i-th orifice, gpm

c_i = discharge coefficient of i-th orifice

P_i = pressure at i-th orifice, psi

Using an effective pressure for the service area, Equation 5-7 can be approximated by $Q = \sum c_i \sqrt{P}$

and

$$Q_r = \sqrt{\frac{P_2}{P_1}} \qquad (5\text{-}8)$$

This is the lower limit on Q_r, except for some special cases in hilly terrain. It represents the maximum impact of the test on water use. The upper limit is the case with no impact. Therefore,

$$\sqrt{\frac{P_2}{P_1}} \le Q_r \le 1 \qquad (5\text{-}9)$$

There is no way to know Q_r exactly but in most cases P_2 is not too far from P_1, so that the error involved with this uncertainty is small. Since the period during which the hydrant is open is short and hence not many water users will adjust valves to maintain flow rate, Q_r will be closer to $\sqrt{P_2/P_1}$ than to 1. In general, Q_r can be given by

$$Q_r = W\sqrt{\frac{P_2}{P_1}} + 1 - W \qquad (5\text{-}10)$$

where W = weighting factor ($0 \leq W \leq 1$).

106. To provide the reader with an idea of the sensitivity of W to other values, consider a typical hydrant test when P_1 = 70 psi and P_2 = 55 psi: 65/80 = 0.90. If W = 1, Q_r = 0.90 ; if W = 0, Q_r = 1 ; and if W = 0.5, Q_r = 0.95 . It is also important to note that $(h_2/h_1)^{1/n}$ is on the order of 3. So an error of 0.05 in Q_r will not result in a large error in C . In general, as h_1/h_2 becomes large in Equation 5-5, the uncertainty in C caused by uncertainty in Q_r becomes small. In most instances, W will be near 1 for large pipes for which the indirect method is appropriate and will decrease with pipe size.

Illustrative Example

107. The procedures described above were applied to several test problems. In each case head loss was measured using the parallel pipe method; that is, a differential pressure gage was connected between the two ends of the test section. Gages with 2 psi (4.6 ft) and 10 psi (23.1 ft) range were used in this study. The differential pressure gage is shown in Figure 5-3 and the connection with a fire hydrant is shown in 5-4. The pressure gage connected to the hydrant in Figure 5-5 measures line pressure for the indirect method. Discharge from hydrants was measured using a pitot gage. Figure 5-5 shows how the gage is mounted in the hydrant while Figure 5-6 shows the tip of the pitot gage which actually detects the velocity.

108. Ideally, the differential pressure gage can be located near any flowing hydrant so that both devices can be read simultaneously by a single individual as shown in Figure 5-7. In some of the tests, velocity in the pipe

Figure 5-3. Differential pressure gage

Figure 5-4. Pressure gage and one end of parallel pipe

Figure 5-5. Pitot gage connected to hydrant

Figure 5-6. Tip of pitot gage

Figure 5-7. Measuring hydrant discharge and head loss

was measured using a pitot tube inserted into the pipe. Figure 5-8 shows a pitot tube inserted into a clear pipe.

109. The following paragraphs describe the results of six tests. Each one presented different problems in applying Equations 5-1 and 5-5 to determine C. These illustrative tests should also give the reader insight into how the tests can be conducted in still other situations.

Test 1

110. Test 1 was conducted for a 6-in. nominal diameter, 345-ft-long pipe located behind the WES Structures Laboratory. A plan view is shown in Figure 5-9. The differential pressure gage is connected between a fire hydrant and a 2-in. hose bib. This test is complicated by the fact that the head loss measured when the hydrant is not flowing occurs primarily in the 2-in.-diam line. This head loss of 0.3 psi (0.69 ft) was substracted from the head loss measured when the hydrant was flowing as this was the head loss attributed to the 2-in. line.

111. The 10-year-old pipe was a lined cast iron pipe; therefore, an internal diameter of 6.4 in. was used in calculating C . Head loss and hydrant discharge were measured for a wide range of discharges and the C-factor was calculated using Equation 5-1; the results are shown in Table 5-1. The average value of 118 is based on a diameter of 6.4 in. If a nominal diameter of

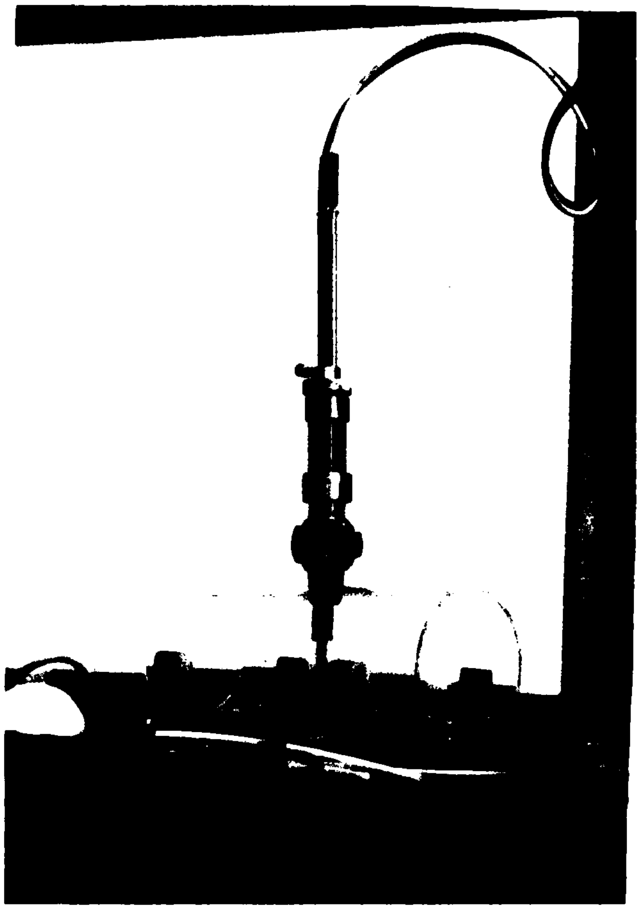


Figure 5-8. Pitot tube inserted into pipe

6 in. was used, C would be 140. This illustrates that care must be exercised in deciding whether to use actual or nominal diameter to calculate C in cases where the two differ.

112. An important lesson learned during the test is that head loss in the piping connecting the differential pressure gage with the test section can be significant. If this is not taken into account, a lower C will be determined because head loss that actually occurred in the 2-in. line would have been attributed to the 6-in. line.

Test 2

113. Test 2 was conducted in a 262-ft length of nomimal 6-in.-diam. pipe (actual diameter approximately 6.4 in.) located alongside the Headquarters Building at WES as shown in Figure 5-10. The tests were conducted on two

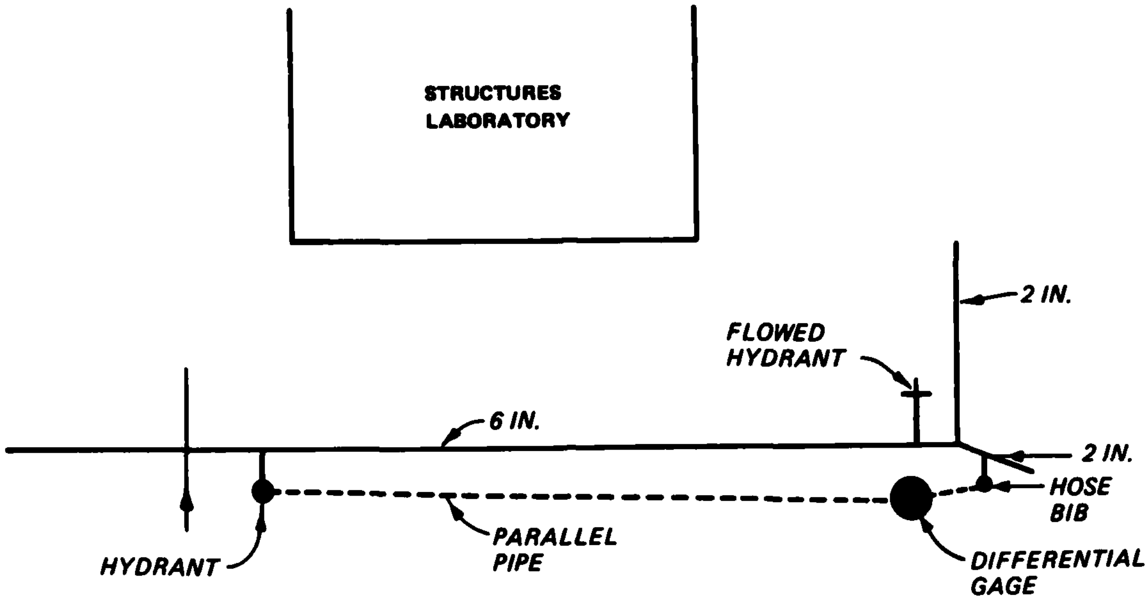


Figure 5-9. Plan view for Test 1

days--5 May 83 and 8 June 83. One or both of the hydrants were flowed during the tests so that head loss could be observed over a wide range of flows. There was a head loss of 0.1 psi (0.23 ft) when no hydrants were open. This value is used as h_1. The line pressure when the hydrants were closed was 90 psi.

114. The results of the tests are shown in Table 5-2. Since h_1 is

Table 5-1

Results of Test 1

Head Loss ft	Hydrant Discharge gpm		C
9.5	1010		121
8.9	980		121
5.4	710		114
1.8	410		120
10.3	1100		125
3.9	570		110
0.4	175		116
		Ave.	118

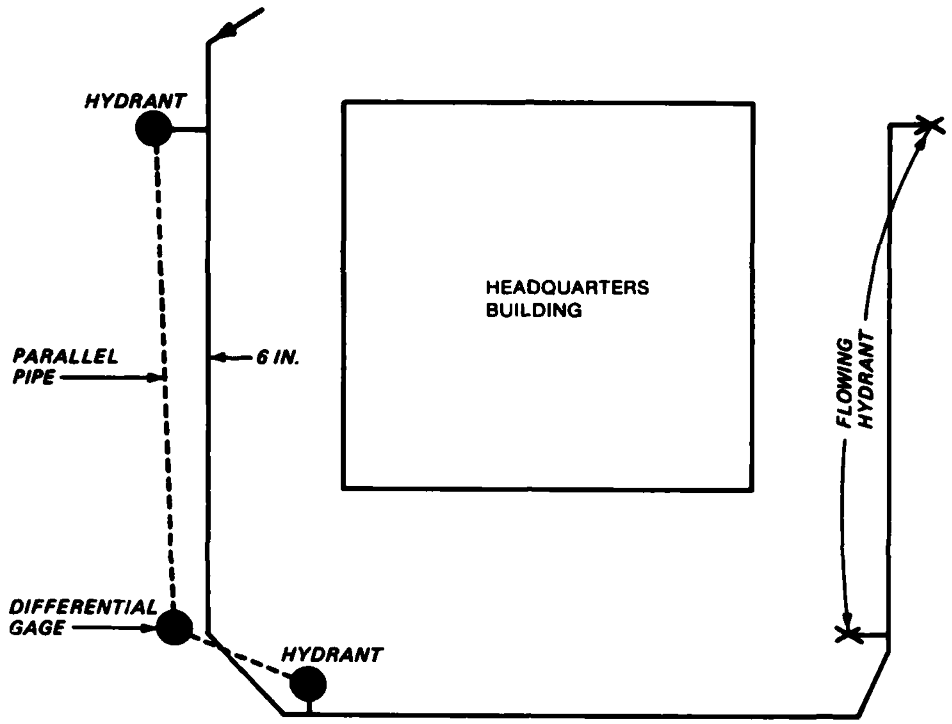


Figure 5-10. Plan view of Test 2

Table 5-2

Results of Test 2

Head Loss ft	Line Pressure psi	Hydrant Discharge gpm		C (Eq 5-1)	C (Eq 5-5)
14.0	45	820		107	113
7.2	57	556		104	115
5.5	75	500		108	126
14.5	35	960		123	127
13.6	50	820		108	116
21.4	30	1105		114	118
15.0	35	960		121	126
5.7	74	530		112	130
13.6	42	920		123	130
12.2	45	840		118	125
5.5	75	480		103	121
20.5	37	1060		112	117
			Ave.	113	122

fairly small, one is tempted to ignore that head loss and use Equation 5-1. The C-factor was calculated with Equation 5-1 which ignores h_1 and Equation 5-5 which accounted for h_1 and the results, shown in Table 5-2, indicate that neglecting the head loss caused by flows other than the hydrant, even if those flows are small, can cause a significant error. Using an average C of 122 and Equation 5-6 shows that a flow as small as 106 gpm (1 ft/sec) caused the difference. Being able to calculate Q_1 using Equation 5-6 is an extra benefit if using the indirect method for determining C.

115. The spread in calculated C values is due to both measurement error and variation in water use other than through the hydrants. Hydrant discharge readings from a pitot gage are the least accurate of the data values as the needle vibrates considerably (on the order of 100 gpm) so that obtaining an accurate reading is difficult. Since most of the parameters were measured to two significant digits, it is improbable that C values would be reproducible beyond the first two digits. The second source of error is due to the fact that it is impossible to control other water users on the line and changes in their use will change head loss and flow during the test. Since a constant value was used for h_1 in calculating C (when actually h_1 varied slightly), some error will occur.

116. The importance of using accurate gages was illustrated during this test as the C values calculated varied greatly depending on which of two pitot gages was used to measure hydrant discharge. When the gages were tested, one was shown to read low. Readings taken with this gage were adjusted and the calculated C's became consistent.

<u>Test 3</u>

117. Test 3 was conducted for a 2-in. line feeding some hose bibs in a grassy area at WES. The 120-ft-long test section is shown in Figure 5-11. Since the flows from the hose were small, the discharge was measured with a calibrated bucket and stopwatch. In this test, there was no flow in the line except the controlled discharge through hose 1 and hose 2, so there was no need to use Equation 5-5 or correct the head loss. The results are shown in Table 5-3.

118. Because of the simplicity of the problem, it is possible to illustrate how Q_1 and C could be determined using Equation 5-5. Suppose the flow (Q_1 = 17.0 gpm) was not known. Instead, all that was known was that when an additional 16.3 gpm was discharged, the head loss changed from 0.85 ft to 2.93 ft. Equation 5-5 gives

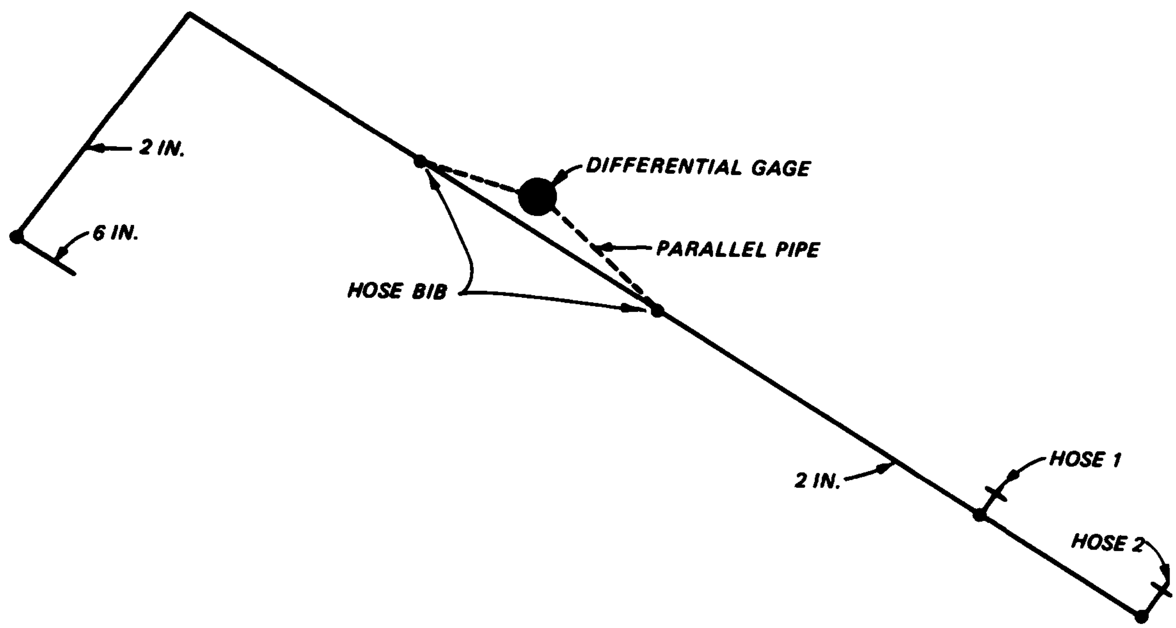


Figure 5-11. Plan view of Test 3

Table 5-3

Results of Test 3

Head Loss ft	Hydrant Discharge gpm	C
0.85	17.0	141
2.93	32.5	139

$$C = \left[\frac{(10.4)\ (120)}{(2.93)2^{4.87}}\right]^{0.54} \quad 16.3 \quad \left[\frac{1}{1 - Q_r\ (0.85/2.93)^{0.54}}\right] \qquad (5\text{-}5,\ \text{bis})$$

$$= 69.2 \left[\frac{1}{1 - Q_r\ (0.513)}\right]$$

In this case Q_r is 0.96 (i.e. 16.3/17.0) which would give a value for $C = 137$ and $Q_1 = 16.6$ gpm. This demonstrates that the indirect method for determining C and C_2, as embodied in Equation 5-5, will produce accurate results when Q_r is estimated accurately. A more important point is that

Q_r is usually not known. If it was estimated incorrectly (say 1.0, 0.9, or 0.8), the values of C would be 142, 128, and 117 which shows that, even for small errors in Q_r (<0.05), C is reasonably accurate. Usually one would not apply this method to cases with Q_r less than 0.9 since reducing pressure by this amount can adversely affect customers.

119. The C-factors observed for this pipe were fairly high. This indicates that the pipes are quite smooth and/or the actual diameter is slightly greater than nominal diameter.

Test 4

120. Test 4 was conducted on a 229-ft length of old, 6-in. cast iron pipe located on Security St. in Vicksburg, Miss. A plan view of the test is shown in Figure 5-12. This test differed from earlier tests in that: (a) velocity was measured using a pitot tube inserted into the pipe and (b) the downstream end of the parallel pipe was connected to the test section through a meter box instead of a hydrant or hose bib.

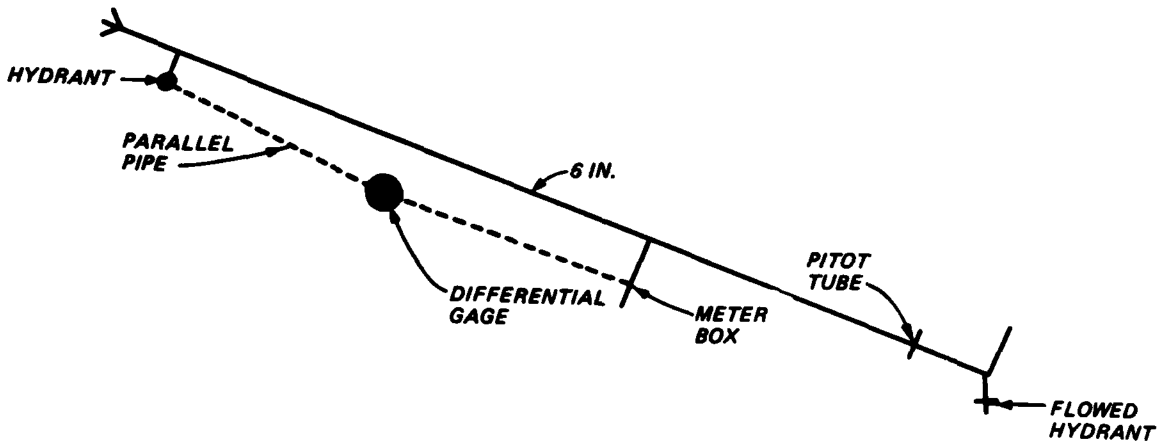


Figure 5-12. Plan view of Test 4

121. Since the pipe was a dead-end line with virtually no flow, it was necessary to open a hydrant on the downstream end to cause sufficient velocity and head loss to produce reasonable readings. In order not to waste water or cause any flooding, the hydrant was only flowed for a short period of time. Because the high velocity was maintained only briefly, a four point velocity traverse was made instead of ten points as is customary. An average velocity of 5.25 ft/sec was observed when the head loss was 5.54 ft. Substituting this information into Equation 5-1a gives a C-factor of

$$C = (8.70)(5.25)(229)^{0.54}\ (5.54)^{-0.54}(6)^{-0.63} \qquad \text{(5-1a, bis)}$$

$$= 110$$

122. This C-factor is fairly high for an old, small diameter pipe. This may be due to the fact that the test was not run for a sufficiently long period of time for the differential pressure gage with pressure snubbers to reach steady-state. Hence, the pressure loss recorded may be lower than actual and the C-factor may be too high. This points out the need (a) to run head loss tests for a long enough period of time for gages to respond, and (b) to select snubbers that do not excessively dampen the response of the gage.

123. Another lesson learned during this test was the need to use high-pressure hoses and tubing. The line pressure of 100 psi ruptured two hoses connecting the pitot tube and manometer. Only when these hoses were replaced by braided tubing could the test be completed.

Test 5

124. Test 5 was conducted on a 424-ft length of 16-in. cast iron pipe located on Lee St. in Vicksburg, Miss. The line is a dead end and serves a mill as shown in Figure 5-13. The test was originally conducted with the parallel pipe connected between the hydrant and the hose bib, located downstream of the meter on the mill property. The head loss in the 6-in. hydrant lateral, meter, and 2-in. service line for the mill was so large it caused the differential pressure gage to read full scale even when there was virtually no head loss in the 16-in. pipe. Since the 16-in. pipe was the one to be tested, a tap was made in the 16-in. pipe immediately downstream of the pitot tap so that the head loss readings would not be affected by losses in the 2-in. line and meter. In this way only head loss in the 16-in. pipe would be measured.

125. A pipe caliper was used to measure the internal diameter of the pipe which was found to be exactly 16 in. The flow was measured both with the pitot tube and at the discharge from the hydrant. Because the line pressure in this part of town is in excess of 100 psi and the hydrant is tied in to a 16-in. line, it was possible to produce significant velocity and head loss by only opening one hydrant.

126. When the hydrant was fully opened, the head loss was 2.08 ft and the velocity was 2.7 ft/sec measured with the pitot tube (which corresponds to a flow of 1680 gpm in the pipe). Discharge from the hydrant was measured as

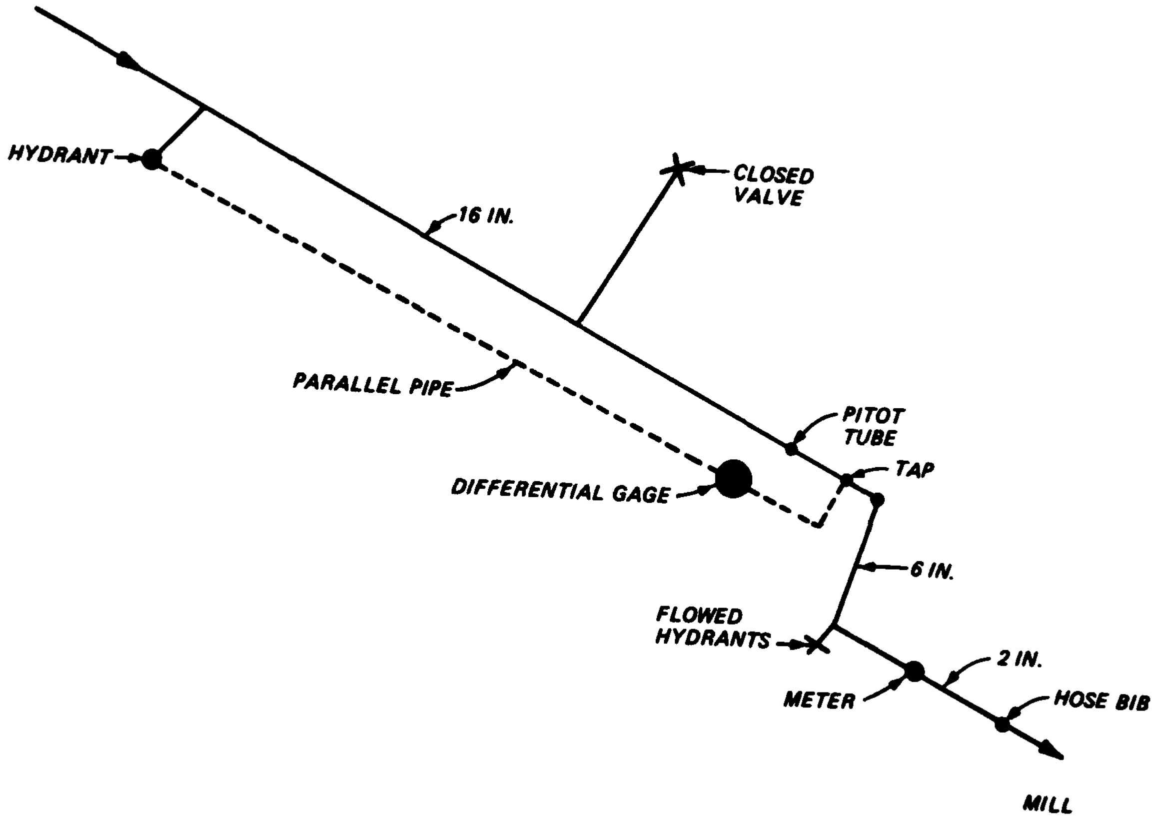


Figure 5-13. Plan view Test 5

1720 gpm which, when added to the 60-gpm use at the mill, gives a pipe flow of 1780 gpm, a 5 percent difference. The C-factor was calculated using Equation 5-1 to be 76 based on 1780 gpm and 71.5 based on 1680 gpm (use 73.7 based on 1730 gpm).

127. As a check of the results, the test was repeated with the differential gage replaced by a manometer. Because of the limited range of the manometer, the discharge from the hydrant was limited to 1185 (1245 gpm in pipe) which resulted in a head loss of 1.0 ft. Substituting this into Equation 5-1 gives a C of 79.

128. This test points out the fact that, for rough pipe flow, the C-factor will depend slightly on the velocity (or flow) at which it was measured. Walski (1983b) stated that C is inversely proportional to velocity to the 0.15 power for hydraulicly rough pipes. That is

$$\frac{C_1}{C_2} = \left(\frac{V_2}{V_1}\right)^{0.15} = \left(\frac{Q_2}{Q_1}\right)^{0.15} \tag{5-11}$$

Using C = 73.7 at 1730 gpm for the high flow test and C = 79 at 1185 gpm, gives $C_1/C_2 = 73.7/79 = 0.93$ and $(Q_2/Q_1)^{0.15} = (1185/1730)^{0.15} = 0.94$. These data tend to support Equation 5-11. The other pipes tested in this work could be categorized as being in the transition zone between hydraulically rough and smooth flow. In those situations C would decrease with increasing velocity but not as dramatically as predicted by Equation 5-11.

Test 6

129. Test 6 was conducted on a 700-ft section of 16-in. cast iron pipe along Eisenhower Drive in the Arlington National Cemetery (Figure 5-14). Head loss was measured by the parallel pipe method with a manometer rather than a differential gage. One valve had to be closed so that the pipe could be made a dead-end line. The flow fluctuated during the test because of a booster pump and hydropneumatic tank which were located downstream of the test section. Since it was not possible to control flow in the test section, the indirect method for calculating C was used.

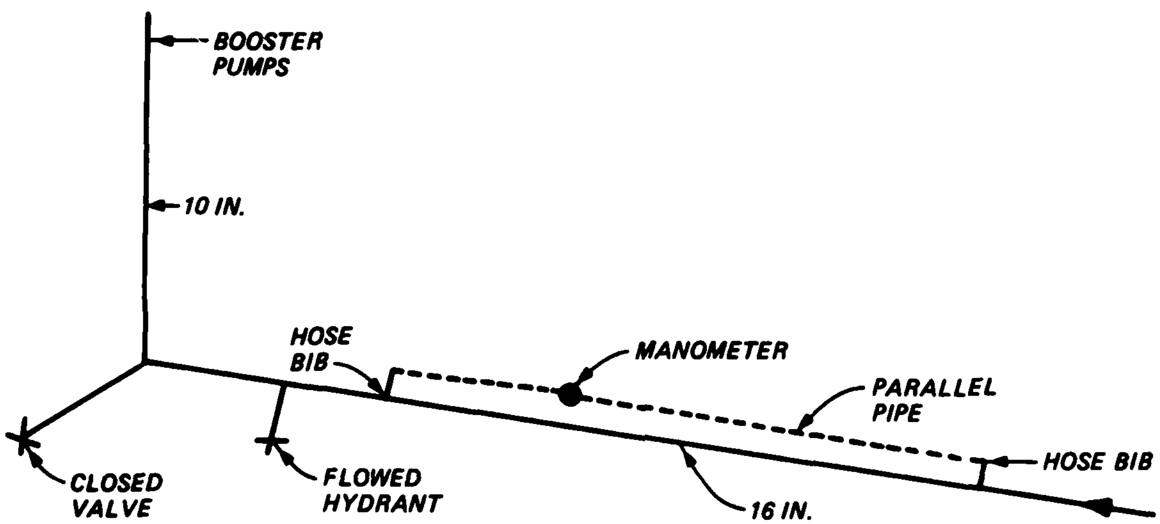


Figure 5-14. Plan view of Test 6

130. A significant problem in measuring head loss resulted from a leak in the hose bib at the downstream end of the test section. This meant that pressure detected at the downstream side of the manometer was lower than the

actual pressure at the downstream end of the line. It was necessary to correct the manometer readings for this leak. Flow in the line could only occur in one direction so the lowest flow rate was zero. Where flow was zero the head loss would be zero as the manometer reading would be due only to leakage. This would correspond to maximum negative reading on the manometer which was 12 in. (1 ft) of water column. Therefore, 1 ft was added to each manometer reading to correct for this leakage.

131. Manometer readings fluctuated from 0 to 0.73 ft when the hydrant was not open. The average was 0.45 ft which is used as h_1. When the hydrant was opened, its discharge was 1090 gpm and head loss increased to 1.97 ft. The C-factors (and flow when hydrant is closed) determined for Q_r = 1.0, 0.95, and 0.90 are shown in Table 5-4.

132. Because there is considerable uncertainty concerning the magnitude of the correction due to leakage in the parallel pipe, a sensitivity analysis was conducted to determine the effect of different corrections. The correction was changed from 1 ft to 1.2 ft and 0.8 ft and it was found that this estimated value significantly affects C and Q_1 as shown in Table 5-4. Therefore, very little confidence can be placed on the results of this test. The lesson learned is that no leakage can be tolerated in the parallel pipe.

Table 5-4

Results of Test 6

Correction to Manometer Reading, ft	C and (Flow Q, gpm)		
	$Q_r = 1$	$Q_r = 0.95$	$Q_r = 0.90$
1.0	114(890)	110(859)	106(827)
1.2	102(580)	99(563)	97(551)
0.8	124(1181)	118(1124)	112(1067)

Common Pitfalls

133. While the procedure for measuring C-factors appears fairly simple, there are a number of problems that can occur to reduce the accuracy of the results. Some of these pitfalls include: (a) leak in parallel pipe, (b) other sources of head loss, (c) fluctuations due to surges, (d) selecting differential pressure gage with wrong scale, (e) insufficient change in flow, and (f) lack of correct fittings.

Leak in parallel pipe

134. If there is a leak in the parallel pipe used in the parallel pipe method, the head loss measured by the differential pressure gage will be inaccurate. Since a fairly small parallel pipe is usually used, a small leak can produce a significant error. No leaks should be tolerated along the parallel pipe.

Other sources of head loss

135. Usually the ends of the parallel pipe are connected to the pipe test section at hydrants, and it is a safe assumption that there is no head loss between the test pipe and the hydrant. In some cases, however, hydrant laterals are used as service lines and may have flow (with its associated head loss). This problem is even more significant if head is measured at a point along a small service line. Any flow in that line will produce head loss that will make the differential pressure measurements useless. This can be avoided by making certain there is no flow in these lines.

Fluctuations due to surges

136. The differential pressure gage or manometer readings will fluctuate fairly wildly in most systems due to surges which regularly occur in most systems. This makes obtaining an accurate reading of head loss difficult. These fluctuations can be dampened out by installing pressure snubbers on both sides of the gage. The snubbers should be selected so that the gage can respond to a change in pressure in about 10 sec. Since flowing a hydrant wastes water and poses potential flooding problems, a gage should be able to respond to changes in flow in a fairly short period of time so that hydrants need not be left open long.

Selecting differential pressure gage with wrong scale

137. The pipe diameter is fixed, the length of the test section is determined by the layout of the site, and the flow rate can only be controlled to a limited extent during the test. Selection of a gage with the correct scale is therefore essential to obtaining accurate results. To select the scale of the gage, solve Equation 5-3 for head loss as

$$h = \frac{10.4\ L}{D^{4.87}} \left(\frac{Q}{C}\right)^{1.85} \qquad (5\text{-}12a)$$

$$h = \frac{54.7\ L}{D^{1.16}} \left(\frac{V}{C}\right)^{1.85} \qquad (5\text{-}12b)$$

Using a high estimate for Q and V and a low estimate for C will give an upper limit for head loss and, hence, the differential pressure gage that can be used. It is advisable to have several differential pressure gages on hand, each with a different range (e.g. 0-2 psi and 0-10 psi).

Insufficient change in flow

138. For the indirect method of determining (Equation 5-5) C to be accurate, the hydrant discharge Q_f should be significant compared with Q_1. This will make the h_2/h_1 term in the denominator of the formula large, which reduces the effect of error in estimating Q_r. For small pipes, it is adequate to open one hydrant, but for larger pipes (>18 in.), two or more hydrants may need to be opened.

Lack of correct fittings

139. When conducting a test in the field, one encounters a wide variety of fittings (hydrant outlet, hose bib, meter box, tubing, etc.), threads (machine, hose, hydrant - male and female), and sizes (from 4 in. to 1/4 in.). An entire test can be ruined if one connection is not available. Careful planning before a field trip is necessary to ensure that the right fittings are available.

Summary

140. The procedure for conducting head loss tests must be tailored to fit the conditions under which the test is being run. Head loss should be measured using the parallel pipe method. Flow in the pipe can be most accurately measured using a pitot tube, although this means the pipe must be excavated and tapped. If the pipe can be isolated properly, the flow can be measured at a hydrant. If this is not possible, some indirect methods exist to measure flow and hence C.

141. Flow measurement is usually the largest source of error in the tests, although there can be significant problems in measuring head loss if

the measurement is not made carefully. Confusion can arise in using C-factors if the individual reporting the test results does not specify whether the results are based on nominal or actual diameter (if available) and equivalent or actual length. In general, nomimal diameter should be used since this may be all that will be known when the C-factor is used, and test sections should be selected so that minor losses are negligible.

142. Head loss tests can be conducted accurately at a reasonable cost if care is exercised in planning, conducting, and reporting the tests.

PART VI: WHY CALIBRATE WATER DISTRIBUTION SYSTEM MODELS?

Introduction

143. Water distribution system models, whether they depend upon the traditional Hardy-Cross method or more sophisticated techniques, provide essential information on pressures and flows that may be used in evaluating existing systems, or designing new or enlarged systems. In applying these, or any other models to a specific system, it is necessary to make certain that the model used actually describes the behavior of the existing system before using it to analyze different conditions. The process of checking flows and pressures predicted by the model against field observations and adjusting model values to make the results agree is known as "calibration."

144. Distribution system models are fairly easy to calibrate by adjusting pipe roughness (or Hazen-Williams C-factor) and water use estimates to make the model behave like the real system. There is, however, a temptation to use haphazardly gathered sets of pressure readings or, worse yet, "literature values" for C-factors or water use, and to then claim the model is calibrated. This type of "calibration" can lead to serious problems when the model is used to design pipes and pumps. These problems are illustrated below and are followed by a discussion of how to avoid common pitfalls and how to answer someone who claims calibration with good data is unnecessary.

Scenario (Can This Happen in Your System?)

145. Suppose a new subdivision is being built at the end of Elm St. in Town X, so an expansion to the water distribution system at Elm St. is needed. Town X has a model developed to study the expansion and some other problems. Instead of gathering data to calibrate the model, the consulting engineer Z asks the town engineer, "What is the pressure at the end of Elm St.?" and is told, "about 65 psi." Assuming a Hazen-Williams C-factor of 100 and the fact that the nearest storage tank is half full when the pressure is 65 psi, engineer Z uses the model to predict a pressure of 70 psi at Elm St. He then concludes, "The model works," and begins sizing the expansion. (Do not laugh, readers; this really happens.)

146. If engineer Z had actually measured the pressure at the end of

Elm St., the results would have shown the pressure to be 60 psi. Furthermore, if engineer Z had opened up a hydrant there and measured pressure, the C-factor could have been back-calculated to be 80. But engineer Z does not want to bother opening up hydrants and measuring pressures (after all, the consulting firm has a computer program to calculate pressures), so engineer Z specifies 3000 ft of 10-in. pipe and says "the pressure at peak use will be 62.1 psi."

147. A decade later, the 10-in. line is in the ground, the subdivision is built, on a good day the pressure is 40 psi, and the fire insurance inspectors are talking about raising insurance premiums in the subdivision. The town or facility engineer loses sleep and can only say, "I do not understand what happened, the consultants used a computer model to size that line."

148. What happened? There is a saying among computer professionals--"Garbage in, garbage out." There was probably nothing wrong with the computer program that was used. Engineer Z simply did a haphazard job calibrating the model. To avoid the problem, engineer Z could have (a) specified an oversized pipe to be on the "safe side" even if it did cost the utility twice as much, or (b) carefully calibrated the model over a range of flows using good quality data.

Calibration Data (How Good is Good Data?)

149. The data collected for calibration must be accurate and should be gathered in an organized manner. Some rules are given below.

150. Flows and pressures in a water distribution system change over time, but in order to back-calculate C-factors and water use rates for a model, it is necessary to get a snapshot of the system at one point in time (i.e. one set of boundary conditions), knowing which pumps were running and the current water level in the tanks. This means that data for calibration should be collected over a short period of time, and settings of pumps and valves should not be changed during the data collection period.

151. A model must accurately predict pressure (actually static head) at specific points in the system. The observed heads must be accurate if the model is to be of any value. Pressures should be read with a type A, or better, gage (i.e. corresponds to American National Standards Institute B40.1-1974, which specifies an accuracy of ± 1 percent of full scale for

type A gages). For the observed head to be meaningful, the elevation at which the reading is taken must be accurately known. If the best available contour map has contour intervals of greater than 5 ft, then it may be worthwhile to survey the elevations of the point where the pressure reading was made.

152. If only static pressures are measured, it is virtually impossible to determine whether C-factors or water use estimates should be adjusted to achieve calibration. (It is like trying to decide which knob to turn to adjust the color on a TV set.) Engineers can argue at length about the merits of adjusting C-factors and water use rates, but actually the correct parameter to adjust is the one in error in the initial runs. This problem can be circumvented by dramatically changing the flow rate and observing how the pressures change. The easiest way of doing this is to open up a nearby fire hydrant and measure the discharge with a pitot gage while, at the same time, observing the pressure.

153. If the model can predict pressures at high and low flow rates, considerable confidence can be placed in its capability to simulate future uses, shutdowns, or expansions. On the other hand, by failing to calibrate the model over a range of flow rates, a careless individual can make the model appear to be calibrated when actually errors in C-factors and water use rates are merely compensating for one another.

Adjusting the Model (How Do You Use the Data?)

154. It is not unreasonable to expect the model to predict pressures to within ±3 psi (±7 ft). Eggener and Polkowski (1976) showed this to be possible when the data are of good quality. The obvious question is, "By how much should each parameter be adjusted?" This can be determined using some basic hydraulics, common sense, and trial and error, but Meredith (1982) and Walski (1983a and 1983c) give some procedures for zeroing in on the right C-factors. Simply stated, if the error in calibration is the same at high flow and low flow, the water use estimates need to be adjusted. If the error is worse at high flow, the C-factor should be adjusted.

Excuses, Excuses, Excuses....

155. If it's so simple to calibrate a water distribution system model

properly, why don't more people do it? This author does not understand why, but here are a few excuses, and some suggested replies.

156. "It takes a lot of time and costs the client a lot of money." Actually, it takes less than a day for a small system. A larger system can be subdivided into several smaller ones and studied over a couple of days. Considering that a modeling study takes 1 to 3 man-months, this extra time in data collection is a small price to pay for correct results. The testing provides the engineer with the opportunity of seeing the system first-hand. The only problem is that the engineer's shoes may get wet when measuring hydrant discharge.

157. "We don't have any pressure gages or pitot gages." All of the gages needed to collect this type of data will cost only a few hundred dollars. If the gages are handled properly, they will last almost forever.

158. "Literature values for C-factors are good enough." Definitely not. The California American Water Works Association Committee on Loss of Capacity in Water Mains (1962) found 30-year-old, 6-in. cast iron mains to have C-factors ranging from 40 to 75. Lamont (1981) states that 60-year-old, 24-in. cast iron mains can have C-factors of from 56 to 107. This amount of uncertainty is unacceptable. Remember also that head loss varies as a function of C to the 1.85 power, meaning that an error of 20 percent in a C-factor will result in an error of almost 40 percent in head loss.

159. "The utility has data in its records." Even the best run utilities only have a rough idea of the pressures and flows in the system. They cannot provide the modeler with the "snapshot" of the system needed for model calibration. If given precise guidance, utilities can collect the data, but the engineer should be wary of gages provided by the utility if they have not been recalibrated recently. These gages may have been mishandled (e.g. tossed into the back of a pickup truck), changing the accuracy from ± 1 percent to $\pm$ (you name it). Be cautious about accepting data from the utility without checking its validity. The utility may say, "That pressure reducing valve is set at 60 psi" and it may actually be set at 50 psi, or "The gate valve at First St. is always closed," and it may have been open for the last 3 years. This author has found some "surprises" in virtually every study with which he has been involved. These surprises can only be discovered by field observations.

160. When a utility is hiring a consultant to do modeling work, the

utility should get assurances that the model will be calibrated over a range of flows with good quality data. If the consultant gives excuses instead, the utility should look for another consultant.

Summary

161. Mathematical models of water distribution systems can only be considered calibrated when they can predict pressures and flows over a range of water use rates. The pressure data used must consist of accurate information collected over a short period of time, and not a set of haphazard observations.

PART VII: PROGRAM FOR REDUCING PITOT TRAVERSE DATA

Purpose

162. In measuring flow through a pipe using a pitot tube, the engineer must read the difference in pressure between the two tips of the pitot tube at a number of points in the pipe, convert these readings to velocities, and then integrate the velocity over the pipe cross section to obtain the flow. These calculations are fairly tedious and have therefore been computerized in a program called PITOT written in Fortran IV. The program is described in the following paragraphs. Those wishing to use the program should contact the author of this report at (601) 634-3931 or FTS 542-3931.

163. This program calculates the flow in a pipe given differential pressure readings and the location of the reading as measured from the bottom of the pipe. The program can accept up to 25 readings from a differential pressure gage (psi) or manometer (in.), and converts velocity to flow using both a 10 point equal area method and integration of either a power law or normal (semi-log) law velocity profiles.

Input

164. The data and format required by the program include:

a. Title of run (25A1).

b. Type of differential gage (IUNIT = 0, pressure gage; IUNIT = 1, liquid manometer; IUNIT = 2, air-filled manometer); if liquid manometer is specified, the second value on the data line is specific gravity of manometer fluid, while if air is specified, the second value is the line pressure in pounds per square inch (I1, F5.0);

c. Diameter of pipe (inches) and pitot tube coefficient (2F5.2);

d. Distance from bottom (inches) and reading (pounds per square inch or inches) (2F5.0) for each point in the traverse, terminating with negative number.

Calculating Velocity

165. The velocity can be determined from differential pressure using the following formula

$$v = k\sqrt{2g\Delta h} \qquad (7\text{-}1)$$

where

v = velocity, ft/sec

k = factor for pitot tube

g = acceleration due to gravity (32 ft/sec^2)

Δh = difference in pressure head, ft

For a differential pressure gage in which the readings are in pounds per square inch (1 psi = 2.31 ft), Equation 7-1 becomes for the pressure gage:

$$v = 12.2\ k\sqrt{\Delta P} \qquad (7\text{-}2)$$

where ΔP = differential pressure, psi. For a liquid manometer, the head is expressed in inches of displacement of the water columns d, so Equation 7-1 becomes (for a heavy liquid (7-3a) then for air (7-3b)):

$$v = k\sqrt{\frac{2gR(SG-1)}{12}} = 2.32\ k\sqrt{R(SG-1)} \qquad (7\text{-}3a)$$

$$v = 1.86\ k\sqrt{R\left[1 - 0.0012\left(1 + \frac{P}{14.7}\right)\right]} \qquad (7\text{-}3b)$$

where

R = displacement of manometer column, in.

SG = specific gravity of manometer fluid

P = line pressure, psi

166. The program can be modified to accept different types of data by adding new values of IUNIT to accept data in metric units for example. This part of the program produces a velocity value for every differential pressure reading.

Determining Flow

167. The next step is to convert the individual point velocities into the flow and average velocity. This is done by integrating the velocity over the area using

$$Q = \int_A v\,dA \qquad (7\text{-}4)$$

where

Q = flow, ft^3/sec

A = pipe cross-sectional area, ft^2

There are two overall approaches to determining Q. The first is to approximate the integral in Equation 7-4 with a summation over small annular areas. The second is to convert the velocities into a continuous function and integrate the function analytically.

168. In the following sections, the approximation techniques are first used to numerically integrate Equation 7-4 followed by the analytical integration for two velocity functions: power rule and normal rule.

Numerical integration

169. The integral in Equation 7-4 can be replaced by

$$Q = \sum_{i=1}^{n} v_i \pi \left(R_i^2 - R_{i-1}^2\right) = \sum_{i=1}^{n} v_i \Delta A_i \qquad (7\text{-}5)$$

where

v_i = average velocity in i-th annular area, ft/sec

R_i = outer radius of i-th annular area, ft

R_{i-1} = inner radius of i-th annular area, ft

ΔA_i = area of i-th annular area, ft^2

If the R_i values can be selected in such a way that all of the ΔA_i values are identical, then it is possible to replace Equation 7-5 by

$$Q = \frac{A}{n} \sum_{i=1}^{n} v_i \qquad (7\text{-}6)$$

In order to use Equation 7-6, the v_i values must be measured at specific locations in the pipe. When n = 10, these dimensionless locations can be given as follows:

i	Fractional Distance from Bottom of Pipe	Fractional Distance from Center of Pipe
1	0.021	0.958
2	0.082	0.838
3	0.148	0.704
4	0.228	0.544
5	0.349	0.302
6	0.651	0.302
7	0.772	0.544
8	0.852	0.704
9	0.919	0.838
10	0.979	0.958

170. Since the velocity readings are not taken at precisely the locations given above, it is necessary to interpolate using velocities at nearby points v_j . This is done with the following formula

$$v_i = v_j + (v_{j+1} - v_j) \left(\frac{d_i - d_j}{d_{j+1} - d_j} \right) \tag{7-7}$$

where

v_i = velocity at i-th plotting position, ft/sec

v_j = velocity for j-th observation, ft/sec

v_{j+1} = velocity for j+1-th observation, ft/sec

d_i = distance from bottom of pipe for i-th plotting position, in.

d_j = distance from bottom of pipe for j-th observation, in.

d_{j+1} = distance from bottom of pipe for j+1-th observation, in.

and

d_i is close to d_j and d_{j+1}

Note that the j subscripts refer to the actual observations while the i subscripts refer to the interpolated values for velocity based on Equation 7-7. Once the v_i terms are calculated, they can be substituted into Equation 7-6 to give flow.

Integrating power rule

171. The velocity profile in a pipe in fully developed turbulent flow can be approximated by a power function of the form

$$v(r) = a(R - r)^b \tag{7-8}$$

where

r = distance from center, ft

a,b = regression constants

R = pipe radius, ft

The a and b can be determined by plotting $(R - r)$ versus v on log-log graph paper. Such a plot is a straight line and, for any two points on the line, a and b can be given by

$$b = \frac{\log\left({}^{v}2/{}^{v}1\right)}{\log\left[(R - r_2)/(R - r_1)\right]} \tag{7-9a}$$

$$a = v_1\ (R - r_1)^{-b} \tag{7-9b}$$

In the program, a and b are determined in subroutine CURVE, given a table of log (R - r) and log v from the actual velocity profile, using a linear regression formula.

172. To determine flow, it is necessary to insert Equation 7-8 into Equation 7-4 and integrate

$$Q = \int_0^{2\pi} \int_0^{R} a\ (R - r)^b\ r\ dr\ d\theta \tag{7-10}$$

This yields

$$Q = \int_0^{2\pi} a \left[\frac{(R - r)^{b+2}}{(b + 2)} - \frac{R(R - r)^{b+1}}{(b + 1)}\right]_0^R d\theta \tag{7-11}$$

which gives

$$Q = 2\pi\ R^{b+2}\ a \left[\frac{1}{b + 1} - \frac{1}{b + 2}\right] \tag{7-12}$$

Equation 7-12 is evaluated in the program to give Q .

Integrating normal rule

173. Another way of describing the velocity profile in the pipe is a normal velocity profile which can be expressed as

$$v(r) = a + b \log (R - r) \tag{7-13}$$

The a and b can be determined by plotting $(R - r)$ versus v on semi-log graph paper ($R - r$ on log scale). From any two points on the straight line, a and b can be determined as

$$b = \frac{v_2 - v_1}{\log\left[(R - r_2)/(R - r_1)\right]} \qquad (7\text{-}14a)$$

$$a = v_1 - b \log (R - r_1) \qquad (7\text{-}14b)$$

In the program, a and b are determined using regression from the subroutine CURVE given log $(R - r)$ and v.

174. To determine flow, it is necessary to insert Equation 7-13 into Equation 7-4 and integrate

$$Q = \int_0^{2\pi} \int_0^{R-\varepsilon} \left[a + b \log (R - r) \, r dr d\theta\right] \qquad (7\text{-}15)$$

where ε is very small.
The upper limit is $R - \varepsilon$ since Equation 7-13 is undefined for $r = R (0 < \varepsilon < < R)$. This yields

$$Q = \int_0^{2\pi} \left[\frac{ar^2}{2} + b\left(\frac{1}{2} r^2 - (-R)^2 \log (R - r) - \frac{1}{2} (-R)^2 \left[\frac{-r}{R} + \frac{1}{2} (- \frac{r}{R})^2\right]\right)\right]_0^{R-\varepsilon} d\theta \qquad (7\text{-}16)$$

which gives, taking limit as ε approaches 0,

$$Q = \pi R^2 \left[b \log R + a - \frac{3}{2} b\right] \qquad (7\text{-}17)$$

which is the formula included in the program. Note that the logarithms in the program are base e logarithms.

<u>Conversion factors</u>

175. Flow is determined in cubic feet per second in the above formulas. It is converted to million gallons per day using 1.54 $(ft^3/sec)/MGD$, which is converted to gallons per minute using 694 gpm/MGD. The average velocity in

feet per second is determined by dividing the flow in cubic feet per second by the pipe area in square feet.

Goodness of fit

176. The subroutine CURVE prints two indicators of goodness-of-fit, the correlation coefficient (R) and the standard error of the estimate (SE). These pertain to the linear transformation of the original velocity and location data. For the goodness-of-fit to be acceptable for either the power or normal laws, R should be greater than 0.9 and SE should be less than 0.1.

177. The methods for determining flow which rely on curve fitting should only be applied when the equation actually fits the observed velocity profile. If the model is not very good (e.g. near an obstruction or bend, or simply because of unusually shaped roughness in the pipe), then only the solutions corresponding to interpolation of the actual velocities should be used.

Program and Variable Listing

178. A listing of the PITOT program is given in Table 7-1 while a listing of all the variables in the program along with definitions and units is given in Table 7-2.

Example Problems

Input

179. Two example problems are provided to illustrate use of the program (Figures 7-1 and 7-2). The first consists of six observations taken in a 20-in.-diam pipe with a differential pressure gage attached to the pitot tube. Note that the velocity profile is symmetric since the input data is symmetric. Note that there are some differences in the average velocity because of the way the three methods interpolate the data.

180. The second example uses actual data from a 16-in. pipe taken with a liquid manometer with a specific gravity of the indicating liquid of 1.27. Because there are more readings, the results from the three methods are in closer agreement.

Output

181. The output from the program, shown in the example problems, consists of several tables which are described as follows:

a. Title.

b. Pipe diameter and pitot coefficient.

c. Type of manometer or gage and specific gravity or line pressure.

d. Table with the distance from bottom, center, and pipe wall, and velocity for each observation.

e. Table with velocity and distance from bottom corresponding to interpolated points.

f. Average velocity and flow by interpolation and summation.

g. Coefficients (a and b), correlation coefficient (R), and standard error (SE) for power law approximation.

h. Average velocity and flow by integrating power law.

i. Coefficients (a and b), correlation coefficient (R), and standard error (SE) for normal law approximation.

j. Average velocity and flow by integrating normal law.

Table 7-1

Listing of Computer Program PITOT

```
      PROGRAM PITOT(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C THIS PROGRAM CALCULATES FLOW IN A PIPE FROM PITOT TESTT DATA
C THE FLOW IS CALCULATED USING A TEN POINT EQUAL AREA METHOD,
C AN INTEGRATION OF THE POWER LAW, AND THE NORMAL LAW OF THE WALL.
C THE DATA REQUIRED INCLUDES:
C 1. A TITLE, 2.A 0 IF DIFFERENTIAL PRESSURE GAGE IS USED AND A 1 AND
C THE SPECIFIC GRAVITY OF THE MANOMETER LIQUID,
C AND A 2 IF AND AIR/WATER MANOMETER IS USED
C 3. THE DIAMETER INTERNAL OF THE PIPE (IN.),
C 4. ONE LINE FOR EACH READING GIVING THE DISTANCE FROM THE BOTTOM (IN)
C AND PRESSURE (PSI) OR MANOMETER READING (IN), TERMINATED WITH A
C NEGATIVE NUMBER. THERE MUST BE AT LEAST 4 AND NO MORE THAT 25 READINGS.
C IT IS POSSIBLE TO RUN SEVERAL DATA SETS AT A TIME.
      DIMENSION RX(25),DX(25),PD(10),VX(25),V(10),PRES(25)
     1,XLV(25),XLS(25),D(10),S(25),ITITLE(25)
      DATA PD/.021,.082,.146,.228,.349,.651,.772,.852,.919,.979/
      PI2=2.*3.1416
    5 READ(5,6)ITITLE
    6 FORMAT(25A1)
      IF(EOF(5))7,8
    7 STOP
    8 WRITE(6,6)ITITLE
      READ(5,1)IUNIT,SG
    1 FORMAT(I1,F5.0)
      READ(5,2)DIAM,XK
    2 FORMAT(2F5.2)
```

(Continued)

(Sheet 1 of 5)

Table 7-1 (Continued)

```
      WRITE(6,3)DIAM
    3 FORMAT(10H DIAMETER=,F6.2,4H IN.)
      P=SG
      AREA=.7865*(DIAM/12)**2
      WRITE(6,13)XK
   13 FORMAT(19H PITOT COEFFICIENT ,F6.3)
      IF(IUNIT.EQ.0)WRITE(6,14)
   14 FORMAT(18H DIFFERENTIAL GAGE )
      IF(IUNIT.EQ.2)WRITE(6,15)P
   15 FORMAT(18H AIR MANOMETER, P= ,F6.0,4H PSI)
      IF(IUNIT.EQ.1)WRITE(6,4)SG
    4 FORMAT(22H LIQUID MANOMETER, SG=,F6.2)
      WRITE(6,9)
    9 FORMAT(5X,25H DX,IN RX,FT S,FT  VX,FPS)
      RAD=DIAM/2
C
C     RAW DATA MANIPULATIONS
C
      DO 10 I=1,25
      READ(5,11)DX(I),PRES(I)
   11 FORMAT(2F5.0)
      IF(DX(I).LT.0)GO TO 19
      IF(IUNIT.EQ.0)VX(I)=12.2*XK*SQRT(PRES(I))
      IF(IUNIT.EQ.1)VX(I)=2.31*XK*SQRT(PRES(I)*(SG-1))
      IF(IUNIT.EQ.2)VX(I)=2.31*XK*SQRT(PRES(I)*(1-.0012*(1+P/14.7)))
      RX(I)=RAD-DX(I)
      IF(DX(I).GT.RAD)RX(I)=DX(I)-RAD
      RX(I)=RX(I)/12.
      XLV(I)=ALOG(VX(I))
      S(I)=(RAD/12)-RX(I)
      IF(S(I).LT.0.001)S(I)=.001
```

(Continued)

(Sheet 2 of 5)

Table 7-1 (Continued)

```
      XLS(I)=ALOG(S(I))
      WRITE(6,12)I,DX(I),RX(I),S(I),VX(I)
   12 FORMAT(I4,4F6.2)
   10 CONTINUE
      I=I+1
C
C     VELOCITIES AT PLOTTING POSITIONS
C
   19 NR=I-1
      NR1=NR-1
      IF(NR.LT.4)GO TO 31
      WRITE(6,18)
   18 FORMAT(/5X,14H POS,IN   V,FPS)
      AVE=0
      DO 20 I=1,10
      D(I)=PD(I)*DIAM
      IF(D(I).LT.DX(1))GO TO 21
      IF(D(I).GT.DX(NR))GO TO 22
      DO 23 J=1,NR1
      IF(D(I).GT.DX(J+1))GO TO 23
   25 V(I)=VX(J)+(VX(J+1)-VX(J))*(D(I)-DX(J))/(DX(J+1)-DX(J))
      GO TO 27
  23  CONTINUE
  22   V(I)=VX(NR)+(VX(NR)-VX(NR-1))*(D(I)-DX(NR))/(DX(NR)-DX(NR-1))
      GO TO 27
  21   V(I)=VX(I)+(VX(1)-VX(2))*(D(I)-DX(I))/(DX(1)-DX(2))
   27 WRITE(6,26)I,D(I),V(I)
  26   FORMAT(I4,2F6.2)
      AVE=AVE+V(I)
  20   CONTINUE
      AVE=AVE/10.
      QCFS=AVE*AREA
      QMGD=QCFS/1.54
```

(Continued)

(Sheet 3 of 5)

Table 7-1 (Continued)

```
      QGPM=QMGD*694.
      WRITE(6,29)AVE,QCFS,QMGD,QGPM
   29  FORMAT(8H AVE VEL ,F8.2,4H FPS/5H FLOW/F8.4,4H CFS/
     1F8.4,4H MGD/F8.2,4H GPM)
C
C INTEGRATING METHOD
C
   31  WRITE(6,32)
   32  FORMAT(11H POWER RULE )
      RF=RAD/12.
      CALL CURVE(XLS,XLV,A,B,NR)
      A=EXP(A)
      WRITE(6,35)A
  35  FORMAT(7H EXP(A),F8.2)
      QCFS=PI2*A*(DIAM/24)**(B+2)*((1/(B+1))-(1/(B+2)))
      QMGD=QCFS/1.54
      QGPM=QMGD*694.
      AVE=QCFS/AREA
      VR=AVE/(A*RF**B)
      WRITE(6,29)AVE,QCFS,QMGD,QGPM
      WRITE(6,34)VR
   34 FORMAT(8H A/P VEL ,F6.4///)
      WRITE(6,33)
   33  FORMAT(11H NORMAL LAW )
      CALL CURVE(XLS,VX,A,B,NR)
      QCFS=PI2*RF**2*(A*.5+(-.75+ALOG(RF)*.5)*B)
      QMGD=QCFS/1.54
      QGPM=QMGD*694.
      AVE=QCFS/AREA
      WRITE(6,29)AVE,QCFS,QMGD,QGPM
      VR=AVE/(A+B*ALOG(RF))
      WRITE(6,34)VR
      GO TO 5
      END
```

(Continued)

(Sheet 4 of 5)

Table 7-1 (Concluded)

```
      SUBROUTINE CURVE(X,Y,A,B,N)
      DIMENSION X(25),Y(25)
      SY=0
      SYX=0
          SX=0
      SX2=0
      SY2=0
      DO 1 I=1,N
      SY=SY+Y(I)
      SYX=SYX+Y(I)*X(I)
      SX=SX+X(I)
      SX2=SX2+X(I)**2
      SY2=SY2+Y(I)**2
    1 CONTINUE
      EN=N
      A=(SY*SX2-SX*SYX)/(N*SX2-SX**2)
      B=(N*SYX-SX*SY)/(N*SX2-SX**2)
      R=(N*SYX-SX*SY)/SQRT((N*SX2-SX**2)*(N*SY2-SY**2))
      SE=SQRT((SY2-A*SY-B*SYX)/EN)
      WRITE(6,2)A,B,R,SE
    2 FORMAT(3H A=,F8.2/3H B=,F8.2/3H R=,F8.4/4H SE=,F8.4)
      RETURN
      END
```

(Sheet 5 of 5)

Table 7-2

List of Variables in Computer Program PITOT

Variable	Definition	Units
A	Regression coefficient in A + Bx	ft/sec
AREA	Pipe cross-sectional area	ft^2
AVE	Average velocity in pipe	ft/sec
B	Regression coefficient in A + Bx	--
DIAM	Pipe diameter	in.
DX	Distance from bottom of pipe	in.
EN	Floating point value of NR	--
I	Counter on loops	--
ITITLE	Title of run	--
IUNIT	0 if differential pressure	psi
	1 if liquid manometer reading	in.
	2 if air manometer reading	in.
NR	Number of observations	--
NR1	NR-1	--
P	Line pressure	psi
PD	Dimensionless plotting point for 10-point profile expressed in distance from bottom	--
P12	$2 * \pi$	--
PRES	Differential pressure	psi if IUNIT = 0
		in. if IUNIT = 1 or 2
QCFS	Flow	ft^3/sec
QGPM	Flow	gpm
QMGD	Flow	MG/day
R	Correlation coefficient	--

(Continued)

(Sheet 1 of 3)

Table 7-2 (Continued)

Variable	Definition	Units
RAD	Pipe radius	in.
RF	RAD/12	ft
RX	Distance from center to observation	ft
S	Distance from wall to observation	ft
SE	Standard error of the estimate	--
SG	Specific gravity of manometer fluid	--
SX	$\sum_{i=1}^{N} X_i$	--
SX2	$\sum_{i=1}^{N} x_i^2$	--
SY	$\sum_{i=1}^{N} Y_i$	--
SY2	$\sum_{i=1}^{N} Y_i^2$	--
SYX	$\sum_{i=1}^{N} X_i Y_i$	--
V	Velocity at plotting point	ft/sec
VX	Observed velocity	ft/sec
X	Transformed distance from wall in curve	--

(Continued)

(Sheet 2 of 3)

Table 7-2 (Concluded)

Variable	Definition	Units
XK	Manometer constant	--
XLS	ALOG (R - r)	--
XLV	ALOG (VX)	--
Y	Transformed velocity in curve	--

(Sheet 3 of 3)

```
TEST DATA SET #1
0
24.  0.83
2.   .389
6.   .508
9.   .566
15.  .566
18.  .508
22.  .389
-99.
```

a. Input

```
TEST DATA SET #1
 DIAMETER= 24.00 IN.
 PITOT COEFFICIENT    .830
 DIFFERENTIAL GAGE
      DX,IN RX,FT S,FT   VX,FPS
   1  2.00   .83   .17   6.32
   2  6.00   .50   .50   7.22
   3  9.00   .25   .75   7.62
   4 15.00   .25   .75   7.62
   5 18.00   .50   .50   7.22
   6 22.00   .83   .17   6.32
```

```
      POS,IN   V,FPS
 1      .50   5.98
 2     1.97   6.31
 3     3.55   6.67
 4     5.47   7.10
 5     8.38   7.53
 6    15.62   7.53
 7    18.53   7.10
 8    20.45   6.67
 9    22.06   6.30
10    23.50   5.98
AVE VEL      6.72 FPS
FLOW
21.1300 CFS
13.7208 MGD
9522.22 GPM
POWER RULE
A=     2.06
B=      .12
R=    .9998
SE=    .0016
EXP(A)     7.88
AVE VEL      6.59 FPS
FLOW
20.7447 CFS
13.4706 MGD
9348.59 GPM
A/P VEL .8366


NORMAL LAW
A=     7.34
B=      .86
R=    .9991
SE=    .0227
AVE VEL      6.55 FPS
FLOW
20.5998 CFS
13.3765 MGD
9283.30 GPM
A/P VEL .8350
```

b. Output

Figure 7-1. Example 1 input and output

```
LIQUID MANOMETER DATA SET
1 1.27
18.  0.83
.375 1.15
.75  1.5
2.25 2.0
2.8  2.27
3.75 2.35
6.0  2.72
6.1  2.8
8.5  3.
9.   3.
9.5  3.1
12.  2.8
14.  2.62
15.  2.35
16.2 2.
16.8  1.75
17.2 1.5
-99.
```

a. Input

```
LIGUID MANOMETER DATA SET
 DIAMETER= 18.00 IN.
 PITOT COEFFICIENT    .830
 LIGUID MANOMETER, SG=   1.27
      DX,IN RX,FT S,FT   VX,FPS
   1    .38    .72    .03  1.07
   2    .75    .69    .06  1.22
   3  2.25    .56    .19  1.41
   4  2.80    .52    .23  1.50
   5  3.75    .44    .31  1.53
   6  6.00    .25    .50  1.64
   7  6.10    .24    .51  1.67
   8  8.50    .04    .71  1.73
   9  9.00  0.00    .75  1.73
  10  9.50    .04    .71  1.75
  11 12.00    .25    .50  1.67
  12 14.00    .42    .33  1.61
  13 15.00    .50    .25  1.53
  14 16.20    .60    .15  1.41
  15 16.80    .65    .10  1.32
  16 17.20    .68    .07  1.26
```

```
      POS,IN   V,FPS
  1     .38  1.07
  2   1.48  1.31
  3   2.66  1.48
  4   4.10  1.55
  5   6.28  1.67
  6 11.72  1.68
  7 13.90  1.62
  8 15.34  1.49
  9 16.54  1.36
 10 17.62  1.20
AVE VEL     1.44 FPS
FLOW
 2.5516 CFS
 1.6569 MGD
1149.89 GPM
POWER RULE
A=       .61
B=       .15
R=    .9928
SE=    .0167
EXP(A)     1.84
AVE VEL     1.43 FPS
FLOW
 2.5280 CFS
 1.6415 MGD
1139.23 GPM
A/F VEL .8104


NORMAL LAW
A=      1.80
B=       .21
R=    .9947
SE=    .0203
AVE VEL     1.43 FPS
FLOW
 2.5239 CFS
 1.6369 MGD
1137.37 GPM
A/F VEL .8162
```

b. Output

Figure 7-2. Example 2 input and output

PART VIII: SELECTING DIFFERENTIAL PRESSURE DEVICES FOR USE WITH PITOT TUBE

Introduction

182. Velocity in water distribution systems is routinely measured by using pitot tubes which are inserted into the pipe as shown in Figure 5-8. The velocity can be determined from the difference between the total pressure and static pressure as detected by the pitot tube. This difference in pressure ranges between 0.1 in. and 20 in. of water for velocities normally encountered in water distribution systems with line pressures of roughly 80 psi. Many ways of measuring this differential pressure exist, but each has certain undesirable characteristics. In this paper, each of the individual methods for determining differential pressures is evaluated, and an inexpensive, easy-to-use manometer developed for this purpose is described.

Alternative Pressure Measuring Devices

183. Several types of devices can be connected to pitot tubes for measurement of differential pressure. These range from manometers in which the difference in pressure is balanced by different heights of fluid columns, to bellows-type differential pressure gages in which the change in position of the bellows is converted into a dial reading, to electronic pressure transducers where differential pressure is converted into an electrical signal. (Differential pressure indicators of the spring or bellows-type are referred to as "gages" in this paper whether they are connected to a pointer or chart recorder.) Manometers to be used with water can be further subdivided into heavy and light liquid manometers, depending on whether the manometer liquid is heavier or lighter than water, and air-filled manometers in which air is used as the manometer fluid.

184. Differential pressures can also be obtained by measuring the pressure using two pressure gages and subtracting the readings. For water distribution systems, gage pressure is usually much higher than the differential pressure and, therefore, this two gage approach is not sufficiently accurate.

Evaluation of Differential Pressure Measuring Devices

185. The differential pressure measuring devices described above are evaluated below based on criteria which are important in applications with a pitot tube. The evaluations are oriented toward use of the gage for field testing rather than in a permanent installation.

Accuracy and range

186. Devices to be used to measure differential pressures for pitot tubes should be able to sense velocities in the range of 0.65 to 8.20 ft/sec. The equation for velocity as measured by a pitot tube is

$$v = k \sqrt{2g\ \Delta h} \qquad (8\text{-}1)$$

where

v = velocity, L/T

k = pitot tube coefficient (usually about 0.8)

g = acceleration due to gravity, L/T^2

Δh = difference in pressure head, L

For the desired range of velocities, the differential pressure gage must measure Δh values of 0.12 to 19.6 in. in terms of water column height. In terms of differential pressure, this corresponds to a range of 0.0043 to 0.708 psi. This accuracy is within reach of pressure transducers and manometers and is at the lower limit of accuracy for differential pressure gages.

187. One advantage of manometers is that they can record flow in either direction. Gages and transducers can be equipped with positive and negative scales with some loss in accuracy. (Accuracy is a function of range on these devices.) This bi-directional feature of manometers enables a user with a symmetric pitot tube to reverse the direction of the tube and repeat the measurements as a check of the readings. Manometers have the additional advantage of giving more accurate readings at low differential pressure if they are inclined. If the manometer is tilted, the Δh in Equation 8-1 should be multiplied by $\cos(\alpha)$ where α is the angle between the manometer column and the vertical.

Durability

188. Pressure gages with ranges of 19.2 in. H_2O tend to be fairly delicate. Pressure transducers are fairly rugged, but the electronic equipment

required to display and record the readings is considerably more delicate and must be kept dry. Pressure gages and transducers can be mounted in portable cases (several firms make cases for their gages as stock items) but these cases significantly increase both cost and weight. Glass manometers tend to be too delicate for use in the field, but plastic manometers are virtually indestructible.

Overpressure

189. There are three pressures of interest in selecting a differential pressure device: the range of the instrument, the maximum line pressure (i.e., gage pressure in the distribution system), and the maximum overpressure. All components (i.e., hoses, fittings, valves, as well as the gage itself) must be able to withstand line pressures of at least 145 psi for use within water distribution systems. Overpressures (i.e., pressures in excess of the range of the gage) arise from two sources: velocities in the line higher than the gage can measure, and inadvertent opening of one side of the gage to the atmosphere. The later can result from opening a valve or from a break in the hose from the pitot tube to the differential pressure device.

190. Since pressure transducers can normally withstand overpressure to only about twice the pressure scale, a single mistake in the field can destroy the transducer. Good quality pressure gages have overpressure protection at pressures up to line pressure.

Safety

191. If a hose should break on a manometer, water and indicating liquid are sprayed on the ground (and workers), which is messy but does not destroy the manometer. A more significant problem occurs if the differential pressure exceeds the maximum which can be recorded by the manometer, or if the manometer should fall over onto its side. In either case, manometer fluid is sucked into the water lines. Since mercury and halogenated hydrocarbons (e.g., carbon tetrachloride, bromoform) are commonly used heavy manometer fluids, these toxic chemicals, which in some cases also cause color, taste, and odor problems, can be introduced into drinking water supplies. Since for drinking water systems it is best to avoid using potentially toxic chemicals, an air-filled manometer becomes attractive.

Power requirement

192. Most commercially available transducers and associated equipment to produce a digital readout generally require an a-c power source, although

some transducers can operate on d-c power from batteries. Gages and manometers require no power. Even the recorders available with some gages can be equipped with spring-wound or battery-powered drives.

Type of output

193. Manometers must be read directly in the field since there is no convenient way to record manometer readings. Gages can be attached to chart recorders to measure differential pressure over time. Since they produce an electrical signal, transducers can be connected to a digital readout, an electronic recording device, or a transmitter that sends the signal to some distant location.

Cost

194. Transducer systems are generally the highest priced of the alternatives. The cost is not so much for the transducer itself but for the associated electronic equipment required to produce a readout, plus a manifold and a carrying case. Gages are slightly less expensive than transducers, and several manufacturers make complete kits including gage, manifold, bleed valves, and carrying case. With transducers, a user must assemble the system from individual components.

195. Manometers are much less expensive and can usually be constructed from materials found in any hardware store or laboratory. High pressure fittings and hoses should be used and the manometer tube itself should be at least 0.25 in. in diameter so that air bubbles will not be trapped in the manometer. A manometer can be constructed from materials costing only a few dollars.

Size and weight

196. Gages are by far the heaviest of the devices with some weighing as much as 10 lb, including the case and manifold. The size of a transducer system depends on how efficiently the components can be interconnected. Since the size of a manometer depends on the manometer fluid, a mercury manometer can be fairly compact while a light oil manometer must be fairly tall. An air-filled manometer must be about 2 ft tall. Manometers and transducers can be fairly light.

Simplicity

197. Gages are the easiest of the above devices to operate, and transducers, once the power source is connected, are not much more difficult. Liquid-filled manometers are somewhat trickier to operate properly since it is very important to purge air from the manometer. Air-filled manometers are

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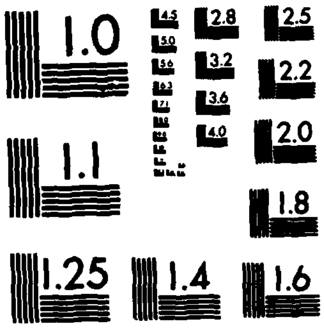

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

somewhat easier to operate in that they only require use of some extra piping to store air which is compressed during use on high pressure lines, and they do not require any special manometer liquids.

Summary of evaluation

198. In general, transducers are the best choice for permanent installations where power is available, a recorder is used, and the unit is housed. Gages are best for more remote locations where power may not be available and a record of differential pressure is required. For field installation without recording, manometers are best. Light fluid manometers tend to be extremely large but can be filled with nontoxic fluid. Heavy fluid manometers pose the threat of contamination of drinking water. Air-filled manometers circumvent these problems, are easy to use, and are inexpensive.

199. This author has found air-filled manometers to be ideal for field applications with pitot tubes for velocities in excess of 1 ft/sec. The remainder of this paper describes how to construct and operate an air-filled manometer, and how to calculate differential pressure using such an apparatus.

Construction of Air-Filled Manometers

Construction

200. Figure 8-1 is a picture of an air-filled manometer and Figures 8-2 and 8-3 are a schematic of the manometer and a close-up of the required valves. The vertical tubes in the manometer are 0.5-in.-diam, clear, rigid polyvinyl chloride (PVC) tubing with a height h_{max} of 24 in. Valves A, C, D, E, and G are flow-through petcock valves while valves B and F are three-way bleed valves. Container H is an air chamber used for storing air as it is compressed by line pressure. It is not needed for line gage pressures less than 1 atmosphere (i.e., 14.7 psi) . Its volume should be at least

$$(P_{max} - 1)V \qquad (8\text{-}2)$$

where

P_{max} = maximum line pressure in atmosphere

V = internal volume of the vertical tubes in the manometer

The junction at the top of the manometer can be made using a group of fittings, a block of plexiglass, or other material which has been drilled and threaded.

Figure 8-1. An air-filled manometer

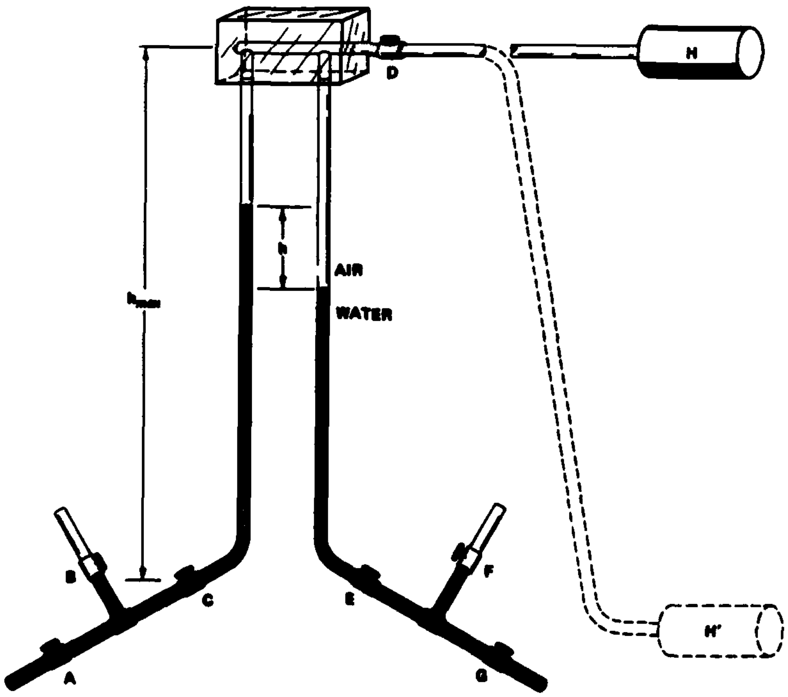

Figure 8-2. Schematic of a manometer

Operation

201. Before using the manometer, the user should test the valves under line pressure to understand how the valves operate and to ensure that there are no leaks. The manometer need not be connected to a pitot tube for this testing. The procedure to use the manometer is given as a step-by-step procedure below:

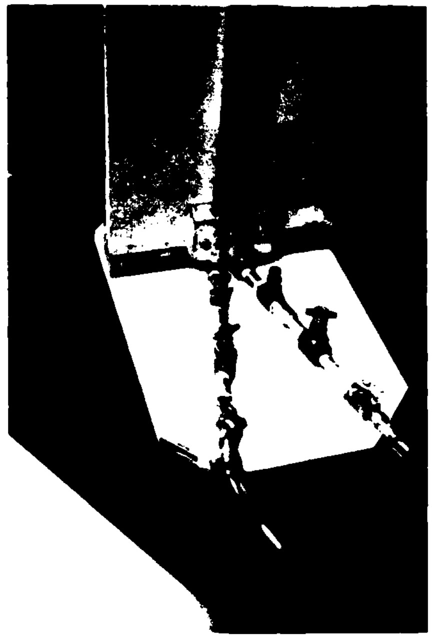

Figure 8-3. Close-up of valves for operating a manometer

a. All of the valves should be closed and air chamber H should be placed at an elevation higher than valve D.

b. The pitot tube should be inserted into the line to be tested.

c. Valves A, G, B, and F should be opened until all the lines from the pitot tube to the manometer have been purged of air.

d. When this is done, valves A and G should be closed, and, optionally, a few drops of nonstaining food dye can be placed into the manometer through valves B and F.

e. Valves B and F should be closed and valves A and G opened.

f. Valves C, E, and then D should be opened to compress the air in the manometer into air chamber H.

g. When flow has stopped, valves C and E should be closed and the level of air chamber H lowered to allow compressed air to flow back into the manometer. It may be necessary to tilt the manometer to allow water to flow into the air chamber.

h. When the vertical tubes of the manometer are roughly half full of water, valve D should be shut. (The air chamber may be disconnected at this time.) If line pressure is expected to drop significantly during the test, the manometer should be more than half full of water or the air will expand as the pressure drops.

i. Valves C and E should be opened and the water levels in each leg of the manometer should equalize at one half of the height of the column when the pitot tube is oriented perpendicular to direction of flow. If this does not happen, check for closed valves or trapped air between the manometer and pitot tube.

j. The line pressure should be recorded.

k. The pitot tube is ready for operation.

Interpretation of results

202. The manometer measures a difference between the static and total pressure sensed by the pitot tube. This difference in pressure can be determined as

$$\Delta p = \Delta h \gamma_w = h(\gamma_w - \gamma_a) \tag{8-3}$$

where

Δp = difference in pressure, F/L^2

Δh = pressure difference in terms of water column, L

γ_w = specific weight of water, F/L^3

h = difference in water level between legs of manometer, L

γ_a = specific weight of air, F/L^3

Dividing by γ_w gives

$$\Delta h = h(1 - r) \tag{8-4}$$

where $r = \gamma_a/\gamma_w$. Since the density of air is much less than the density of water, the term r can be ignored for most applications with less than a 1 percent error. In these cases, $\Delta h \approx h$ and the manometer reading corresponds directly to the difference in head.

203. For greater accuracy, the weight of air can be accounted for in Equation 8-4 by treating r as a function of temperature and pressure. Since r varies only slightly with temperature (i.e., in the fourth decimal place), temperature effects can be ignored for all practical purposes. Similarly, the density of water only changes very slightly with pressure. By far, the most significant impact on r is caused by the compressibility of air. Since air is practically an ideal gas, the relationship between r and pressure can be given by the ideal gas law

$$r = \gamma_a/\gamma_w = P_{abs}\ \gamma_{a1}/\gamma_w \tag{8-5}$$

where

P_{abs} = absolute pressure, atm

γ_{a1} = specific weight of air at 1 atm, F/L^3

At 20°C, $\gamma_{al}/\gamma_w = 0.00120$, so Equation 8-4 can be given, for pressure in gage pressures, as

$$r = 0.00120\ (1 + P_g/U) \tag{8-6}$$

where

P_g = gage pressure, F/L^2

$$U = \begin{cases} 1 \text{ for pressure in atmospheres} \\ 101.3 \text{ for pressure in kilopascals} \\ 14.7 \text{ for pressure in pounds per square inch} \\ 407 \text{ for pressure in inches of water} \end{cases}$$

Therefore Equation 8-4 becomes

$$\Delta h = h[1 - 0.0012\ (1 + P_g/U)] \tag{8-7}$$

Table 8-1 provides insight into the magnitude of the correction that must be applied to h (i.e., the quantity in brackets in Equation 8-7) to account for the compressibility of air in determining Δh . Values for r and $\Delta h/h$ corresponding to several pressures are shown.

Table 8-1

Effect of Gage Pressure on Correction for Compressibility of Air

Pressure			
KPa	psi	r	Δh/h
0	0	0.0012	0.9988
200	29.0	0.0036	0.9964
500	72.5	0.0071	0.9929
1000	145.0	0.0130	0.9870
2000	290.0	0.0249	0.9751

204. Table 8-1 clearly shows that the corrections are quite small for pressures typically encountered in water distribution systems.

Summary

205. An air-filled manometer is the best choice for use with a pitot tube in the field for measuring velocity in water supply pipes because of safety, cost, durability, and accuracy.

PART IX: PREVENTING DETERIORATION OF WATER DISTRIBUTION SYSTEMS

Introduction

206. Out of sight, out of mind--water distribution systems may be the most easily forgotten component of America's infrastructure. The average American knows that when a water tap is opened, water flows out, and concludes, sometimes incorrectly, that all's well with the water distribution system. Yet, these systems, many of which have been in use for over 100 years, are deteriorating, and if nothing is done, Americans may not be able to count on having water when they need it. The nearly 300,000 residents in Jersey City, N.J., learned this lesson in the summer of 1982 when the combination of a main break and faulty valves shut off water to that system for several days. Deterioration of water distribution systems is not limited to urban systems, but is occurring at military installations as well.

207. It is rarely economical to simply rebuild entire systems. Therefore, a wise engineer or water system manager must be able to pinpoint the weak links in a system. Furthermore, since funds for infrastructure rehabilitation are limited, it is necessary to focus primary attention on those components for which rehabilitation or replacement will more than pay for themselves.

208. There are three basic types of problems that arise in water distribution systems as a result of aging: frequent breaks and leaks, loss of carrying capacity, and malfunctioning of appurtenances. The purpose of this paper is to describe these problems, present some methods by which they may be detected and corrected, and discuss how to decide when it is economical to take corrective action.

Pipe Breaks

Problem

209. The term "pipe breaks" is commonly used to describe several different conditions by which water may be lost from a system including: major pipe ruptures, joint leaks, and service leaks. Pipe breaks usually result from external causes such as contact with other structures, improper bedding, expansive soils, frost loads, brittle failure, live loads, and accidents.

Joint leaks generally result from motion of the pipe due to inadequate restraints or poor bedding. Unprotected metal pipes become more susceptible to these problems as they remain in the ground longer and are weakened by corrosion. The rate at which corrosion occurs is, however, highly variable, depending upon pipe material, nature of the soil, location of the water table, and presence of stray electrical currents. O'Day (1982) and Morris (1967) describe the causes of breaks in some detail.

Detection

210. Large main breaks are easy to locate as water will usually reach the surface. For example, a break in a 72-in.-diam conduit in St. Louis in 1973 resulted in the loss of 40 million gallons of water in only a few hours and washed out a rail spur (Fletcher 1982). Most leaks are, however, considerably smaller and not as dramatic. Some fairly large leaks can go undetected if the water can find its way to a sewer without reaching the surface. Leaks can be located using sound detection devices ranging from simple old fashioned telephone earpieces to sophisticated electronic devices. Cole (1979), Heim (1979), and Laverty (1979) describe several different methods for leak detection. One especially useful approach is to conduct a water audit (Cole 1979) in which all of the water flowing into an area is monitored for a 24-hr period. If nighttime flows are more than 50 percent of average flows, there is reason to suspect significant leakage in the area.

Correction

211. Repair of leaks usually involves placing a sleeve or clamp around the leak. In some instances, however, it is more economical to replace a section of the pipe. In congested urban areas, it is important to schedule "advanced replacement" of water mains to coincide with road repair or sewer installation so that inconvenience to citizens is minimized. In any case, it is also necessary to correct the cause of the leak (corrosion, poor bedding, inadequate restraints, etc.) or else the problem will simply reoccur.

Evaluation

212. When considering a leak detection/repair program, there are two types of decisions the water system engineer/manager must make. The first regards whether or not to conduct a leak detection survey, consisting of a water audit, systematic use of leak detection devices, or both. Guidance on this is provided by Beckwith (1964), Brainard (1979), and Moyer (1983). Assuming that a survey is made and some bad sections of pipe are located, the next

decision is whether it is more economical to repair or replace the pipe. This decision should be based on cost (including damages and interruption of service), past break history of the pipe, and a projection of the change in break rates in the future. Useful guidance on decisionmaking is provided by Shamir and Howard (1979) and Walski and Pelliccia (1982).

Loss of Carrying Capacity

Problem

213. New pipes are generally fairly smooth inside, but as they remain in place they become rougher because scale or layers of corrosion form in the pipe. New pipes often have Hazen-Williams C-factors, an indicator of the pipe's carrying capacity, around 140, but as pipes age, this number can drop to less than half of its original value. This means that, for a given hydraulic gradient, the pipe can carry less than half of the flow that it did when it was new. If water use does not decrease, the result is low pressures in the system, especially during high use periods, or dramatically increased energy costs to maintain an acceptable pressure.

Detection

214. Loss of carrying capacity becomes evident with complaints by users of low pressure, poor results in fire flow tests (American Water Works Association (AWWA) 1962), or, if early indications are not heeded, inadequate water to fight a fire. One way in which low values for the Hazen-Williams C-factor can be detected is during the process of calibrating a model of the water distribution system (Walski 1983a). If the model can only be calibrated for C-factors of 60 to 80, then the system has lost much of its carrying capacity. To precisely determine a C-factor, it is necessary to measure flow, diameter, length, and head loss in a given section of pipe. Head losses can be readily determined using a pitot tube and pressure gages (Hudson 1954; AWWA 1962).

Correction

215. If a pipe is found to have low carrying capacity, it can be rehabilitated by cleaning and lining. This process consists of scraping the inside of the pipes with mechanical scrapers or hydraulic pigs, and spraying a thin cement mortar lining in the pipe. In most cases, this can restore the C-factor to approximately 120. For smaller pipes, especially in developing areas, it is more economical to install a parallel pipe than to clean and line an old pipe.

Evaluation

216. To determine the most economical approach to solving the problem, it is necessary to compare the cost of cleaning and lining (or installation of a parallel pipe) with the costs of the alternatives of additional pumping energy and equipment to deliver the required flows. A useful comparison procedure has been described by Walski (1982). In general, cleaning and lining will be most economical for large mains which carry water at fairly high velocities.

Malfunctioning of Appurtenances

Problem

217. Most valves and hydrants are only used in emergency situations, and, since it may be many years between uses of these appurtenances, they are often neglected. Therefore, when called upon to perform in an emergency, valve and hydrant stems may be frozen, valves may not seat adequately, and valve boxes may have been paved over, or be impossible to find for other reasons.

Detection

218. Valves and hydrants should be inspected and tested regularly, and records of the condition of these appurtenances should be kept (AWWA 1961, 1980). Annual inspection, maintenance, and testing will prevent most problems and identify the few that do occur. A file should be kept on the results of this inspection and testing. Kuranz and Barrett (1982) and Franklin (1982) describe how such programs can be set up.

Correction

219. Once defective valves and hydrants have been located, they should be repaired or replaced as soon as possible. Valves that have been paved over can be located with metal detectors and the valve box cover can be raised. Valves are often found in the wrong position (e.g., closed when they should be open) and should be set properly. Gate valves should be either completely open or closed (i.e., not used for throttling which usually results in damage to the valve).

Evaluation

220. Determining the frequency with which valves and hydrants should be inspected and tested involves consideration of the trade-off between the

cost of the program and the benefits of being assured that the system is operating properly. The American Water Works Association (1961) recommends large valves (>12 in.) be tested annually while smaller valves (<12 in.) be tested every 3 years. While such a program may at first appear costly, it is inexpensive compared with shutting down the system because valves did not operate properly.

Summary

221. Water distribution systems, like many other infrastructure components, are gradually deteriorating. In most cases, it is less expensive to detect and correct problems early than to allow the problem to continue, or to replace major sections of systems.

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APPENDIX A: DIPRA SOIL INVESTIGATION REPORT

SOIL INVESTIGATION REPORT

WASHINGTON AQUEDUCT DIVISION

U.S. ARMY CORPS OF ENGINEERS

WASHINGTON, D. C.

APRIL 29, 1983

BY:

J. Richard Page, P.E.
Regional Engineer

APPROVED BY:

Michael R. Redmon
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INDEX

EXHIBITS

SYNOPSIS

This soil investigation was conducted on existing water mains operated by the Washington Aqueduct Division of the U.S. Army Corps of Engineers. Soil Samples #1 and #2, along the Jefferson-Davis Highway near the Pentagon, are potentially corrosive to ductile iron pipe. Should a replacement line to the existing 16" diameter steel main be installed, it is recommended to polyethylene encase through this area.

Soil Samples #3, #4 and #5 were found to be non-aggressive to cast iron and ductile iron pipe. As all of these samples showed evidence of fill material, replacement of existing water mains through these areas should be polyethylene encased if large quantities of cinders or slag material is encountered.

Introduction

Requested by: Dr. Thomas M. Walski, P.E.
Waterways Experiment Station
U.S. Army Corps of Engineers
P. O. Box 631
Vicksburg, Mississippi 39180
601/634-3931

The survey was conducted in a spirit of service and cooperation for the purpose of identifying potentially corrosive conditions relative to ductile iron piping systems.

Project Data

Location: This investigation was conducted on existing water mains in the vicinity of Pentagon and Arlington National Cemetery in Washington, D.C..

Pipe Quantities: Several miles of 6" through 16" diameter water mains.

Date of Survey: April 26 & 27, 1983

Funding: Federal

Conducted by: J. Richard Page, P.E. and Allen H. Cox, P.E., DIPRA Regional Engineers.

Procedures

Representative test locations were selected along the route of proposed pipe installation. Each was assigned a number which corresponds to a number appearing in the soil analysis listing and on the project map, Exhibit I.

A small diameter boring to approximate proposed pipe depth at each location facilitated field testing and removal of soil specimens for laboratory analysis. All field and laboratory procedures were completed in accordance with Appendix A of ANSI/AWWA Standard C105.

Test Results

Specific soil analysis results are listed in Exhibit II of this report.

Observations

No major oil and gas pipeline crossings were observed.

The areas between the Potomac River and the Pentagon were originally marshland, and have been filled in for the construction of parks and roadways.

Conclusions

Due to a positive sulfides reaction, a neutral pH, and a low or negative redox reading, Soil Samples #1 and #2 are potentially corrosive to ductile iron pipe. Again, it should be noted that the existing uncoated steel water main does not have the corrosion resistant properties of either cast iron or ductile iron pipe.

Soil Samples #3, #4 and #5 had negative sulfides reaction, high positive oxidation reduction potential, and moderately high resistivities. These conditions present an environment that should be non-aggressive to cast iron and ductile iron pipe.

Recommendations

Replacement ductile iron water mains for the existing 16" diameter steel mains should be polyethylene encased through the gray clays found along the Jefferson-Davis Highway, north of the Pentagon, contained in Soil Samples #1 and #2.

Although Soil Samples #3, #4 and #5 were found to be non-aggressive to ductile iron pipe, they did show evidence of containing some fill material. Upon replacing water mains through these areas, the new water mains should be polyethylene encased through areas of cinders and slag fill material, should they be encountered.

The installation procedures and material specifications for polyethylene encasement are outlined in ANSI/AWWA Standard

The foregoing report and recommendations are based upon examinations and tests which were made in accordance with generally accepted professional engineering standards and considered necessary in the circumstances.

BY:

J. Richard Page, P.E.
Regional Engineer

APPROVED BY:

Michael R. Redmon
Director of Regional Engineers

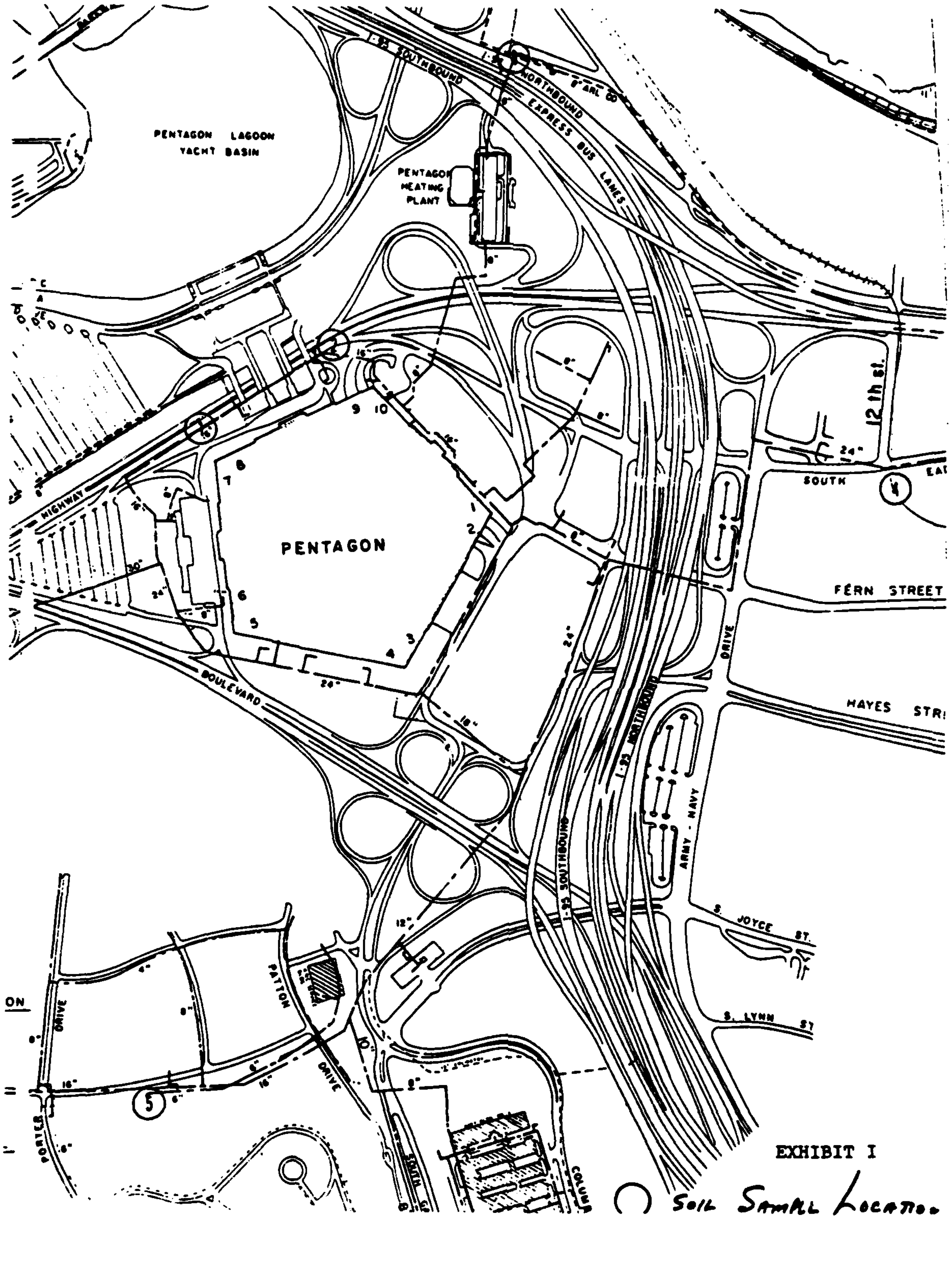
PENTAGON LAGOON
YACHT BASIN
PENTAGON
HEATING
PLANT
I-95 SOUTHBOUND
NORTHBOUND
EXPRESS BUS LANES
PENTAGON
HIGHWAY
BOULEVARD
12th st.
SOUTH
FERN STREET
HAYES STR
DRIVE
ARMY - NAVY
I-95 NORTHBOUND
I-95 SOUTHBOUND
S. JOYCE ST.
S. LYNN ST
PATTON
DRIVE
PORTER
COLUM
EXHIBIT I
Soil Sampl Locatio

EXHIBIT II

TEST RESULTS

Sample No.	Location	Resis-tivity ohm-cm	Redox mv	pH	Sulfides	Soil Description
1	Jefferson-Davis Highway Near Pentagon	2,800	- 20	6.8	Positive	Gray Clay-Saturated
2	Jefferson-Davis Highway Near Pentagon	3,360	+ 20	7.0	Positive	Brown Silty Clay W/Fill-Saturated
3	Power Plant Drive	3,320	+200	6.7	Negative	Light Brown Clayey Sand W/Fill Material-Moist
4	South Eads Street	2,720	+220	6.8	Negative	Brown Clay W/Fill Material-Moist
5	Arlington Cemetery	2,880	+180	6.3	Negative	Brown Clay W/Fill Material-Moist

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