ADVANCED WATER DISTRIBUTION MODELING AND MANAGEMENT

Authors
Thomas M. Walski
Donald V. Chase
Dragan A. Savic
Walter Grayman
Stephen Beckwith
Edmundo Koelle

Contributing Authors
Scott Cattran, Rick Hammond, Kevin Laptos, Steven G. Lowry,
Robert F. Mankowski, Stan Plante, John Przybyla, Barbara Schmitz

Peer Review Board
Lee Cesario (Denver Water), Robert M. Clark (U.S. EPA),
Jack Dangermond (ESRI), Allen L. Davis (CH2M Hill),
Paul DeBarry (Borton-Lawson), Frank DeFazio (Franklin G. DeFazio Corp.),
Kevin Finnan (Bristol Babcock), Wayne Hartell (Bentley Systems),
Brian Hoefer (ESRI), Bassam Kassab (Santa Clara Valley Water District),
James W. Male (University of Portland), William M. Richards
(WMR Engineering), Zheng Wu (Bentley Systems ),
and E. Benjamin Wylie (University of Michigan)

Click here to visit the Bentley Institute Press Web page for more information
Testing Water Distribution Systems

Verifying that a water distribution model replicates field conditions requires an intimate knowledge of how the system performs over a wide range of operating conditions. For example, can the model reproduce the flow patterns and pressures that occur during periods of peak summertime usage, or can the model accurately simulate chlorine decay? Collecting water distribution system data in the field provides valuable insight into system performance and is an essential part of calibration.

Data collection, the first step in the model calibration process, is discussed in depth within this chapter. The chapter begins with a brief discussion of system testing, including descriptions of some simple tests for measuring flow and pressure, as well as some of the pitfalls that may be encountered. The details of performing fire hydrant flow tests, head loss tests, pump performance tests, and water quality tests are discussed as well. The chapter concludes with a discussion of the importance of data quality, particularly when automated calibration methods are used.

5.1 TESTING FUNDAMENTALS

Pressure Measurement

Pressures are measured throughout the water distribution system to monitor the level of service and to collect data for use in model calibration. Pressure readings are commonly taken at fire hydrants (see Figure 5.1) but can also be read at hose bibs (also called spigots); home faucets; pump stations (both suction and discharge sides); tanks; reservoirs; and blow-off, air release, and other types of valves.

If the measurements are taken at a location other than a direct connection to a water main (for example, at a house hose bib), the head loss between the supply main and the site where pressure is measured must be considered. Of course, the best solution is to have no flow (and hence no head loss) between the main and the gage. To check if flow into the building is occurring, listen at the hose bib for the sound of rushing water.
When measuring pressure, slight fluctuations may be seen on the gage due to changing flows in the system. Devices such as *pressure snubbers* and liquid-filled pressure gages can be used to dampen the pressure fluctuations, unless the fluctuations themselves are a source of interest.

Pressure gages are most accurate when measuring pressures within 50 to 75 percent of the maximum value on the scale. Using several pressure gages of varying pressure ranges is advisable when working with a water distribution system. A pressure gage with a range of 0 to 100 psi (690 kPa) is commonly used; however, a pressure gage that can read up to 200 psi (1,380 kPa) may be necessary for measurements taken at a pump discharge or at a low elevation. If pressure measurements are taken on the suction side of a pump, then a pressure gage capable of reading negative pressures, called a *pressure-vacuum gage*, may be required. Remember that it is the elevation of the gage, not the elevation of the node, that is used in calculating the elevation of the HGL (see page 252).

**Flow Measurement**

Flow is measured at key locations throughout a system to provide insight into flow patterns and system performance, develop consumption data, and determine flow rates for calibration.

Many of the tests described in this chapter require measuring flow in pipes. A variety of flow meters are available for this purpose, including *Venturi meters*, *magnetic flowmeters*, and *ultrasonic meters*. Pressure and flow metering and recording equipment should be calibrated regularly and undergo routine performance checks to ensure that it is in good working order. Furthermore, even if a flow meter is accurate and calibrated, the monitoring station may use an analog gage or dial readout that has a coarse level of precision, which limits the overall precision.
The extent of flow measurement employed varies from system to system. Usually, flow is measured continuously at only a few key locations in the distribution system such as treatment plants and pump stations. Data from these sites should be used to the greatest extent possible in system calibration. Flow from higher to lower pressure zones can also be measured at pressure zone boundaries using combination pressure reducing valve/flow meters (Walski, Gangemi, Kaufman, and Malos, 2001). More rarely, systems employ in-line flow meters at key points throughout the network and transmit the flow rates back to a control center using Supervisory Control and Data Acquisition (SCADA) systems and telemetry (See Chapter 6). This type of comprehensive flow monitoring is not typically done in the United States; however, more utility managers and operators are starting to see the value of in-line flow information.

Temporary flow metering may be a cost-effective option to check pump discharges or to see if in-line flow measurements are required throughout the system. Field measurement using a Pitot rod is shown in Figure 5.2. The rod is inserted into the pipe to measure total head and pressure head, which can then be converted into velocity (Walski, 1984a). The Pitot rod should not be confused with the Pitot gage, which measures velocity head only. Clamp-on or insertion electromagnetic or ultrasonic meters may also be used.

Placement of the flow-measuring device is important. To be sure that disturbances caused by any bends or obstructions do not influence the readings, the device should be placed far enough downstream of the disturbance (usually at a distance of approximately 10 times the pipe diameter) that the effects will have completely dissipated.

In certain cases it may be desirable to isolate one end of the pipe such that all of the flow through the pipe is diverted through a hydrant for measurement. The hydrant flow can then be measured with a hydrant Pitot gage as described in Section 5.2.

Net flow in and out of a tank during a time period can be measured by monitoring water level in the tank and then calculating the flow based on cross-sectional area in the tank.
Potential Pitfalls in System Measurements

Flow measurement tests can be beneficial, but there are potential drawbacks to keep in mind. Testing may result in disruption of service to some customers. For example, fire flow tests typically cause lower than normal pressures and higher than normal velocities, particularly in residential areas. Higher velocities can entrain sediments in pipes or shear against tuberculation on pipe walls, causing customers to experience discolored water.

Customers may, either by accident or necessity, be disconnected from the system when valves are operated to facilitate flow tests. As described in the following sections, head loss tests require the operation of system valves to isolate sections of water main. Valve operation needs to be carefully planned when conducting such tests to avoid inadvertently disconnecting customers from the system. To avoid surprises, customers should be notified prior to the tests.

5.2 FIRE HYDRANT FLOW TESTS

Obtaining data for a wide range of operating conditions, including peak (high) demand periods, would be difficult without fire hydrant flow tests. These tests can be used to simulate high flow conditions (see page 218) and allow the system behavior to be analyzed under extreme conditions. Fire hydrant flow tests are primarily used to measure the fire flow capacity of the system. They also provide data on pressures within the system under static conditions (no hydrants flowing) and stressed conditions (high flows occurring at the hydrants) and can be used in conjunction with the hydraulic model to calibrate parameters such as pipe roughness (Walski, 1988). Procedures for conducting fire hydrant flow tests are described in AWWA (1989) and ISO (1963).
Two or more hydrants are required to perform a fire hydrant flow test, as illustrated in Figure 5.3. One hydrant is identified as the residual hydrant(s), where all pressure measurements are taken, and the other is identified as the flowed hydrant(s), where all flow measurements are taken. When the flowed hydrant(s) is closed, referred to as static conditions, the pressure at the residual hydrant is called the static pressure. When one or more of the flowed hydrants are open, referred to as flowed conditions, the pressure at the residual hydrant is called the residual pressure.

Conducting a fire hydrant flow test is a simple procedure, and a number of these tests can be conducted throughout the system in a day’s time. Although not essential, many utilities have a policy requiring that the residual hydrant be opened and allowed to flow prior to connecting the pressure gage. This precaution helps remove any particles that have accumulated in the hydrant lateral and barrel since it was last exercised. After that, a pressure gage is connected to the residual hydrant and a static pressure reading taken.

Next, the first of the flowed hydrants is opened and flowed. Once the readings stabilize, a reading is taken at the flowed hydrant using a hand-held or clamp-on Pitot gage (shown in Figure 5.4) or a Pitot diffuser (shown in Figure 5.5). Meanwhile, another pressure reading is taken at the residual hydrant. Once the residual pressure is taken and the discharge rate of the flowed hydrant is recorded, the same procedure can be repeated for additional hydrants if needed.

The number of hydrants that should be flowed during a test is determined by the pressure drop observed at the residual hydrant. Usually, a drop of at least 10 psi (70 kPa) is needed to give good results. In a 6- to 8-in. pipe (150 to 200 mm), flowing a single hydrant is sufficient. For larger pipes, more hydrants may need to be flowed.

**Pitot Gages and Diffusers**

Because a Pitot gage (shown in Figure 5.4) converts virtually all of the velocity head associated with the flow stream to pressure head, the Pitot gage pressure reading can be converted to a hydrant discharge rate using the orifice relationship in Equation 5.1.

\[ Q = C_d D^2 \sqrt{\frac{2}{P}} \]  

(5.1)
Figure 5.4
Hand-held Pitot gage

where

\[ Q = \text{hydrant discharge (gpm, l/s)} \]
\[ C_d = \text{discharge coefficient} \]
\[ D = \text{outlet diameter (in., cm)} \]
\[ P = \text{pressure reading from Pitot gage (psi, kPa)} \]
\[ C_f = \text{unit conversion factor (29.8 English, 0.111 SI)} \]

For a typical 2.5-in. (64 mm) outlet with a discharge coefficient of 0.9, Equation 5.1 can be reduced to:

\[ Q = 167 \sqrt{P} \]

The discharge coefficient in Equation 5.1 accounts for the decrease in the diameter of flow that occurs between the hydrant opening and the end of the Pitot gage, as well as the head losses through the opening. The coefficient depends on the geometry of the inside of the hydrant opening and can be determined by feeling the inside of the hydrant nozzle (see Figure 5.6).

The Pitot diffuser is similar to a Pitot gage except that it incorporates a nozzle that redirects the flow from the hydrant, reducing its momentum and thus the potential for erosion. Because the velocity head sensor is measuring inside the diffuser at a point where the pressure is not equal to zero, a slightly modified formula is required to compute flow. This formula varies with the manufacturer of the diffuser (Walski and Lutes, 1990; and Morin and Rajaratnam, 2000). For example, for the Pitot diffuser shown in Figure 5.5, the coefficient of 167 given previously reduces to 140.
To briefly review, the procedure for conducting a fire hydrant flow test is as follows:

1. Place a pressure gage on the residual hydrant and record static pressure.
2. Take the 2 ½-in. (64 mm) cap off of the flowed hydrant.
3. Feel the inside of the hydrant opening to determine its geometry.
4. Slowly start the flow.
5. Once readings stabilize, take a Pitot gage reading at the flowed hydrant(s).
6. Simultaneously measure the residual pressure(s) at the residual hydrant(s).
7. Slowly close the hydrants.
8. Assign the discharge coefficient according to the geometry of the hydrant opening.

9. Determine the hydrant discharge rate by using Equation 5.1 or the equation provided by the Pitot diffuser manufacturer.

Once all of the data have been collected, a table similar to Table 5.1 can be constructed to present the results of the fire hydrant flow test.

**Table 5.1** Results of fire hydrant flow test

<table>
<thead>
<tr>
<th>Number of Hydrants Flowing</th>
<th>Residual Pressure (psi)</th>
<th>Hydrant #1 Discharge (gpm)</th>
<th>Hydrant #2 Discharge (gpm)</th>
<th>Hydrant #3 Discharge (gpm)</th>
<th>Total Discharge (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>78</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>72</td>
<td>1,360</td>
<td>N/A</td>
<td>N/A</td>
<td>1360</td>
</tr>
<tr>
<td>2</td>
<td>64</td>
<td>1,150</td>
<td>975</td>
<td>N/A</td>
<td>2125</td>
</tr>
<tr>
<td>3</td>
<td>49</td>
<td>850</td>
<td>745</td>
<td>600</td>
<td>2195</td>
</tr>
</tbody>
</table>

When sufficient resources are available, additional residual pressure measurements can be taken during the fire hydrant flow test at various locations throughout the system. Taking these additional pressure readings will provide more information on how the hydraulic grade changes across the system. Depending on the nature of the water distribution system, the pressure drop may be localized to the vicinity of the flowed hydrants.

If the hydrant flow test is conducted to provide data for model calibration, it is extremely important to note the boundary conditions at the time of the test. Recall that boundary conditions reflect the water levels in tanks and reservoirs, as well as the operational status of any high-service pumps, booster pumps, or control valves (for example, pressure reducing valves) for both static and flowed conditions. As will be discussed in Chapter 7 (see page 261), these boundary conditions must also be defined in the hydraulic model.

In addition, system demands in place at the time of the test need to be replicated in the model. It is important to note the time of day and the weather conditions when the test was performed to assist in establishing the demands and boundary conditions.

**Potential Problems with Fire Flow Tests**

Fire hydrant flow tests are a useful tool. They do, however, present some areas of concern. Because the discharges from fire hydrants can be quite large, the following suggestions can reduce potential problems associated with these flows.

1. Minimize the period of time over which hydrants are flowed to limit flooding potential. (In some locations it may be necessary to dechlorinate water before it can be discharged into receiving waters.)

2. Direct the flow through the 2 ½-in. (64 mm) nozzle opening instead of the 4 ½-in. (115 mm) opening. This will help to reduce street flooding while still producing flow velocities sufficient for calibration.
### Evaluating Distribution Capacity with Hydrant Tests

The results of hydrant flow tests described in this chapter are used primarily to evaluate the distribution system's capacity to provide water for fighting fires. The standard formula for converting the test flow to the distribution capacity at some desired residual pressure—usually 20 psi (135 kPa)—was developed by the Insurance Services Office (1963), and is given in AWWA M-17 (1989) as:

\[
Q_r = Q_t \left( \frac{P_s - P_r}{P_s - P_f} \right)^{0.54}
\]

where:
- \(Q_r\) = fire flow at residual pressure \(P_r\) (gpm, l/s)
- \(Q_t\) = hydrant discharge during test (gpm, l/s)
- \(P_s\) = static pressure (psi, kPa)
- \(P_f\) = desired residual pressure (psi, kPa)
- \(P_r\) = residual pressure during test (psi, kPa)

The value of \(Q_r\) is referred to as the distribution main capacity in that location, and is used in evaluation of water systems for insurance purposes.

Assumptions made when using the above equation are as follows:
1. Head loss is negligible during static conditions.
2. Demands correspond to maximum day demands.
3. All pumps and regulating valves that would open during an actual fire are open and operating during the test.
4. There is sufficient water quantity to supply the fire throughout the duration of the fire event.
5. Tank level is at normal day low level.
6. The residual and flowed hydrants are close to one another (Walski, 1984b).

Water system models can explicitly account for these factors and are a more accurate and flexible way of assessing available fire flow at a given residual pressure. However, this equation is still widely used.

The previous equation can also be rearranged to provide a rough estimate of residual pressure for some future flow, given hydrant flow test results, according to

\[
P_r = P_s - (P_s - P_f) \left( \frac{Q_r}{Q_t} \right)^{1.85}
\]

In this case, \(Q_r\) is the estimated flow, and \(P_r\) is the pressure that will exist at that flow rate, given that all other conditions remain the same.

---

3. Use hydrant diffusors to reduce the high velocity of the hydrant stream. This will help to avoid erosion problems and damage to vegetation.

4. Conduct fire hydrant flow tests during warm weather to avoid ice problems.

5. Notify customers who may be impacted by the test beforehand. In some systems, hydrant flow tests can stir up sediments and rust, causing temporary water quality problems.

6. Make sure to open and close the hydrants gradually, as sudden changes in flow can induce dangerous pressure surges in the system.

7. Make sure that the residual and flowed hydrants are hydraulically close to one another. It is possible to have two hydrants that are near each other at the street but are fed by different mains that may not be hydraulically connected for several blocks. Ideally, the flowed and residual hydrants would be located side-by-side on the same pipeline, but because this will almost never be the case, accuracy can instead be improved by minimizing the flow between the hydrants. (The flow can often be reduced by bracketing the residual hydrant between two flowed hydrants.)
Using Fire Flow Tests for Calibration

In addition to measuring the fire protection capacity of the network, fire hydrant flow tests can provide valuable data for hydraulic model calibration. To use the results of a test, a demand equivalent to the hydrant discharge should be assigned to the junction node in the model that corresponds to the flowed hydrant. When the hydraulic simulation is conducted, the HGL at the junction node representing the residual hydrant should agree with the HGL measured in the field. Note that comparisons between field measurements and model results should be done in terms of HGL, not pressure (see page 252). Although this section refers to pressure comparisons, remember that in practice, the field pressures should be converted to the equivalent HGL before comparing them to the model results.

Consider the system shown in Figure 5.7 and the results of the fire hydrant flow test presented in Table 5.1. The top half of the figure illustrates the model representation of a series of hydrants where nodes J-23, J-24, and J-25 correspond to Hydrants 1, 2, and 3 respectively; and J-22 corresponds to the residual hydrant. The hydrant flow test results outlined in Table 5.1 can be described in four unique scenarios:

- Static conditions where none of the hydrants are flowing
- Hydrant 1 is flowing
- Hydrants 1 and 2 are flowing simultaneously
- Hydrants 1, 2, and 3 are flowing simultaneously

The scenario in which only Hydrant 1 is flowing results in a discharge of 1,360 gpm (0.086 m³/s) and a residual pressure of 72 psi (497 kPa). Therefore, a demand of 1,360 gpm will be placed at model node J-23, and when the hydraulic simulation is conducted, the pressure computed at node J-22 will be compared to the residual pressure of 72 psi measured in the field. If the pressure at J-22 is close to that figure, the model will be nearly calibrated (at least for this one condition).

On the other hand, if the pressure at J-22 is not close to the measured pressure, adjustments need to be made to the model to bring it into better agreement. Identifying the actual adjustments that need to be made depends on the cause of the discrepancy.
Chapter 7 has more information regarding reasons why differences might occur as well as details on modeling the results of flow tests. The procedure described previously is repeated for each of the flowed conditions, and the parameters are changed as necessary to obtain a suitable match between observed and computed pressures.

It is critical that the modeling nodes used to represent the hydrants are placed in exactly the same location as the hydrants in the field. Accurate placement is particularly important for calibration purposes, as illustrated in the following example.

In Figure 5.8a, the pressure measurements are taken at the residual hydrant, and the model representation of the hydrant is at J-35 (Figure 5.8b), a few hundred feet away. The modeler may have justified this simplification by reasoning that the locations of the residual hydrant and node J-35 are relatively close together, and that the pressures should be similar because the elevations are approximately the same. During calibration, the modeler then (mistakenly) compares the field-measured pressure at the residual hydrant to the modeled pressure at J-35 and adjusts the model to achieve an acceptable match.

What the modeler has failed to consider in this situation is the head loss between the two points (J-35 and the actual hydrant location) during the fire hydrant flow test. If the head loss is significant, the computed pressure at node J-35 would be higher than the computed pressure at the residual hydrant. By trying to match pressures at different locations, the modeler could introduce inaccuracies into the model. The subject of model calibration and the use of fire hydrant flow tests for that purpose are treated in greater detail in Chapter 7.

### 5.3 HEAD LOSS TESTS

The purpose of a head loss test is to directly measure the head loss and discharge through a length of pipe—information that can then be used to compute the pipe roughness. Head loss tests can be performed using either the two-gage or the parallel-pipe method. The two-gage method uses pressure readings from two standard pressure gages to determine the head loss over the pipe length, and the parallel-pipe method uses a single pressure differential gage to find the head loss.
The length of water main being tested is typically located between two fire hydrants. During a head loss test, valves are closed downstream of the length of test pipe to hydraulically isolate the test section. Thus, all flow through the section is directed to the downstream fire hydrant for measurement. Assuming that the internal pipe diameter is known, head loss, pipe length, and flow rate are then measured between the two points and used to compute the internal pipe roughness using the expressions for the Hazen-Williams C-factor and the Darcy-Weisbach friction factor (Equations 5.2 and 5.3).

\[
C = \left( \frac{C L Q}{h L D^{4.87}} \right)^{1/1.852}
\]

(5.2)

where
- \( C \) = Hazen-Williams C-factor
- \( L \) = length of test section (ft, m)
- \( Q \) = flow through test section (cfs, m\(^3\)/s)
- \( h_L \) = head loss due to friction (ft, m)
- \( D \) = diameter of test section (ft, m)
- \( C_f \) = unit conversion factor (4.73 English, 10.7 SI)

\[
f = \frac{h_i D^2 g}{L V^2}
\]

(5.3)

where
- \( f \) = Darcy-Weisbach friction factor
- \( g \) = gravitational acceleration constant (32.2 ft/s\(^2\), 9.81 m/s\(^2\))
- \( V \) = velocity through test section (ft/s, m/s)

The velocity is determined from the flow and diameter by using Equation 2.9:

\[
V = \frac{4Q}{\pi D^2}
\]

To apply the friction factor to other pipes, it is necessary to convert \( f \) to absolute roughness. Equation 5.4 is the Colebrook-White formula solved for roughness.

\[
\frac{\varepsilon}{D} = 3.7 \left[ \exp \left( \frac{1}{0.86 \sqrt{f}} \right) - \frac{2.51}{Re \sqrt{f}} \right]
\]

(5.4)

where
- \( \varepsilon \) = absolute roughness
- \( Re \) = Reynolds number

For smooth pipes, the above equation can occasionally yield negative numbers, which should be converted to zero roughness (that is, hydraulically smooth pipe).
Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15–20 psi (100 - 140 kPa) should be attained.

\[ h_L = HGL_U - HGL_D \]  \hspace{1cm} (5.5)

where \( HGL_U \) = hydraulic grade at upstream fire hydrant (ft, m)  
\( HGL_D \) = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

\[ h_L = C_f (P_U - P_D) + (Z_U - Z_D) \]  \hspace{1cm} (5.6)

where \( P_U \) = pressure at upstream fire hydrant (psi, kPa)  
\( P_D \) = pressure at downstream fire hydrant (psi, kPa)  
\( Z_U \) = elevation at upstream fire hydrant (ft, m)  
\( Z_D \) = elevation at downstream fire hydrant (ft, m)  
\( C_f \) = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-
tion, the head loss in Equation 5.6 is set to zero and the elevation difference can be expressed through the use of pressures, as shown in the following equation.

\[
Z_U - Z_D = -C_f (P_{US} - P_{DS})
\]  

(5.7)

where

- \( P_{US} \) = pressure at upstream hydrant, static conditions (psi, kPa)
- \( P_{DS} \) = pressure at downstream hydrant, static conditions (psi, kPa)
- \( C_f \) = unit conversion factor (2.31 English, 0.102 SI)

Substituting Equation 5.7 into 5.6 provides a new expression for determining the head loss between two hydrants. This expression eliminates the need to obtain the elevation of the pressure gages by using two sets of pressure readings: static and flowed.

\[
h_L = C_f [(P_{UT} - P_{DT}) - (P_{US} - P_{DS})]
\]  

(5.8)

where

- \( P_{UT} \) = pressure at upstream hydrant, flowed conditions (psi, kPa)
- \( P_{DT} \) = pressure at downstream hydrant, flowed conditions (psi, kPa)
- \( C_f \) = unit conversion factor (2.31 English, 0.102 SI)

In some situations, the test section may be located near a permanent system meter, such as at the discharge of a pump station, and thus the flow meters at the pump station can be used instead of a hydrant. A pressure gage located on the pipe just before it leaves the pump station can give the upstream pressure. The downstream pressure must be measured sufficiently far away such that the head loss will be much greater than the error associated with measuring it. It may be necessary to close valves at tees and crosses along the pipeline to obtain this long run of pipe with constant flow. Wal斯基 and O’Farrell (1994) described how head loss testing equipment can be installed with important transmission mains to assist routine head loss testing.

**Parallel-Pipe Test**

Figure 5.10 illustrates the concept of the parallel-pipe head loss test. As with the two-gage test, a test section is isolated between two hydrants by closing the downstream valves. Then a hose equipped with a differential pressure gage is connected between the two hydrants in parallel with the pipe test section. Because there is no flow, and consequently no head loss, through the hose or gage, the hydraulic grade on each side of the gage is equal to the hydraulic grade of the hydrant on that same side. Therefore, the measured pressure differential can be used in the following expression to calculate the head loss through the pipe.

\[
h_L = C_f \times \Delta P
\]  

(5.9)

where

- \( \Delta P \) = differential pressure reading (psi, kPa)
- \( C_f \) = unit conversion factor (2.31 English, 0.102 SI)

The head loss, or pressure head difference, over the test section can be found for any fluid by dividing the differential pressure (\( \Delta P \)) by the specific weight of the fluid (\( \gamma \)).
Because the pressure readings are taken at one location (at the pressure differential gage), there is no need to consider the elevation of either hydrant. However, if water in the parallel hose is allowed to change temperature from the water in the pipes, errors can occur (Walski, 1985). Accordingly, water in the hose should be kept moving whenever a reading is not being taken. This can be accomplished by opening a small valve (pit-cock) at the differential pressure gage.

The procedure for finding the discharge through the test section is similar to the one used for fire hydrant flow tests. A Pitot gage is used to measure the velocity head at the flowed hydrant, assuming the flow out of the hydrant equals the flow through the test section. The orifice formula (Equation 5.1) is then used to convert the Pitot gage reading into the discharge from the hydrant (McEnroe, Chase, and Sharp, 1989).

McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 2–3 psi (14 - 21 kPa) for the parallel-pipe method should be attained.

**Potential Problems with Head Loss Tests**

Regardless of the method used for measuring head loss, all flow that passes through the test section is directed out of a flowed hydrant by closing the valve downstream of the flowed hydrant. When working with a looped system, isolation valves on some side mains may also be closed, as shown in Figure 5.11. To ensure that no customers are taken out of service when closing valves, the utility should examine system maps to verify that alternate flow paths (loops) are available within the system. As a check, have one individual watch the pressure gage as the valve is being closed and be ready to give a signal if the pressure drops to zero.

Frequently, there will be customers connected to the test section between the two hydrants. To obtain accurate results, the customer water usage during the head loss tests should be negligible compared to the amount of water discharged through the flowed hydrant. Recall from Equations 5.2 and 5.3 that the discharge is assumed to reflect the total amount of water that passes through the test section. Therefore, if the amount of water that passes through the test section is significantly different from the
measured discharge due to withdrawals at other points in the system, a correction must be made.

**Figure 5.11**
Use of isolation valves during a head loss test

Using Head Loss Test Results for Calibration

The process of using the results of a head loss test is fairly straightforward. Head loss tests provide information on the internal roughness of a pipe; therefore, once the head loss tests are complete, the calculated roughness values can simply be supplied to the computer model. The extent of head loss testing, however, is dependent on the project budget. Some projects are planned such that a sample of mains that are representative of the system are selected for testing. Then the results are extrapolated to the rest of the system.

One way to limit the amount of head loss testing that must be done to get valid data is to perform head loss tests for a wide variety of pipe sizes, types, and ages. These data can then be placed in chart form, showing pipe roughness values as a function of pipe and size (Ormsbee and Lingireddy, 1997). Roughness values are then selected based on the age and size of the selected main.

Systems in which pipe roughness varies over a wide range usually contain a significant amount of unlined cast iron pipe. Sharp and Walski (1988) showed that equivalent sand grain roughness heights in unlined, commercial, cast iron pipe increased linearly with time. Therefore, in terms of absolute roughness:

\[
\varepsilon = \varepsilon_o + at
\]

where
- \(\varepsilon\) = roughness height at age \(t\) (in., mm)
- \(\varepsilon_o\) = roughness height when pipe was new \((t=0)\) (in., mm)
- \(a\) = rate of change in roughness height (in./year, mm/year)
- \(t\) = age of pipe (years)

Roughness height for new cast iron pipe is usually on the order of 0.008 in. (0.18 mm).
By measuring the roughness height for a few pipes in a head loss test, it is possible to determine the coefficient in Equation 5.10 (the rate of change of roughness, \(a\)) and use the value for other pipes of that type, provided that the corrosive characteristics of the water have not changed significantly. Walski, Edwards, and Hearne (1989) developed a method for adjusting values when water quality had changed during the life of a pipe.

For those using Hazen-Williams C-factor instead of equivalent sand grain roughness height, the relationship between C and age is related to the base 10 log of the roughness height and diameter.

\[
C = 18.0 - 37.2 \log \left( \frac{n_a + at}{D} \right) \tag{5.11}
\]

where \(D = \text{diameter (in., mm)}\)

Using data from Lamont (1981) and Hudson (1966), Sharp and Walski (1988) performed a regression analysis using Equation 5.10, relating the corrosivity of the water using the Langelier Index, shown in Table 5.2. It should be noted that values for any water system are specific to that system.

<table>
<thead>
<tr>
<th>Description</th>
<th>(a) (in./year)</th>
<th>(a) (mm/year)</th>
<th>Langelier Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slight attack</td>
<td>0.00098</td>
<td>0.025</td>
<td>0.0</td>
</tr>
<tr>
<td>Moderate attack</td>
<td>0.003</td>
<td>0.076</td>
<td>-1.3</td>
</tr>
<tr>
<td>Appreciable attack</td>
<td>0.0098</td>
<td>0.25</td>
<td>-2.6</td>
</tr>
<tr>
<td>Severe attack</td>
<td>0.030</td>
<td>0.76</td>
<td>-3.9</td>
</tr>
</tbody>
</table>

### 5.4 PUMP PERFORMANCE TESTS

There are four types of pump characteristic curves: head, brake horsepower, efficiency, and NPSH (see page 49). Although modelers can usually rely on pump characteristic curves that are provided by the manufacturer, it is good practice to check these curves against pump performance data collected in the field. The following section discusses how to determine points for the head characteristic curve. It is followed by a discussion of measuring efficiency, which is needed for pump energy analysis.

#### Head Characteristic Curve

As presented in Chapter 2 (see page 44), the head characteristic curve gives total dynamic head as a function of discharge through the pump. Consider the pump shown in Figure 5.12. If the energy equation is applied between the discharge (section 2) and suction (section 1) sides of the pump, the following expression is obtained.
where

\[ h_{dis} \] = pump discharge head (ft, m)

\[ V_{dis} \] = velocity at point where discharge head is measured (ft/s, m/s)

\[ g \] = gravitational acceleration constant (32.2 ft/sec^2, 9.81 m/sec^2)

\[ h_{suc} \] = pump suction head (m, ft)

\[ h_P \] = head added at pump (m, ft)

\[ h_L \] = head loss due to friction (m, ft)

\[ h_m \] = minor head losses due to fittings and appurtenances (m, ft)

\[ V_{suc} \] = velocity at point where suction head is measured (ft/s, m/s)

**Figure 5.12**
Pump performance test

Because sections 1 and 2 are close together, any head losses due to friction, \( h_L \), will usually be negligible. In addition, the minor losses that occur within the pump as a result of changing streamlines are not directly considered through the \( h_m \) term. Accordingly, the head loss terms are usually set to zero, and the minor losses within the pump are addressed through the pump head term, \( h_P \).

Assuming that sections 1 and 2 have the same elevation, Equation 5.12 can be rewritten as shown:

\[
h_p = \left( \frac{P_{dis}}{\gamma} - \frac{P_{suc}}{\gamma} \right) + \left( \frac{V_{dis}^2}{2g} - \frac{V_{suc}^2}{2g} \right) + h_L + h_m \tag{5.13}
\]

where

\[ P_{dis} \] = discharge pressure (psi, kPa)

\[ P_{suc} \] = suction pressure (psi, kPa)

A pump head characteristic curve is a plot of \( h_P \) versus flow. As shown in Equation 5.13, suction and discharge pressures and the suction and discharge velocity heads are needed to develop the curve. The velocity heads can be calculated based on the flow through the pump (most pump stations are equipped with flow meters) and the suction and discharge pipe diameters. Because the suction and discharge pipe diameters are usually not significantly different for water distribution pumps, the difference between velocity head terms is often negligible.

If the pump is equipped with pressure gages on the suction and discharge lines, the pressure information can also be easily collected. In some instances, however, a pump
Section 5.4 Pump Performance Tests

will have a pressure gage on the discharge side only. In this case, the suction head can be found by applying the energy equation to the suction side of the pump, making sure that all head losses between a hydraulic boundary condition and the pump are accounted for. If the pump does not have a discharge pressure gage, then the energy equation can be applied between the pump discharge and a point of known head (a boundary condition). Again, all head losses between the two points must be considered.

The pump head characteristic curve is developed by finding the pump heads for a series of corresponding pump flows. To do so, the operator varies pump flows through the use of a valve on the discharge side of the pump. With the discharge valve wide open, the pump is turned on and allowed to arrive at full speed. Next, the suction pressure, discharge pressure, and pump flow are measured. The result of substituting these measured values into Equation 5.13 is a point on the pump curve. Then the valve is adjusted slightly, and another set of pressure and flow data is collected. This process is repeated, closing the valve a little more each time, until the desired number of data points have been obtained. The key to developing a useful curve is to vary the discharge over the entire range, from shutoff head to maximum flow. In some cases, it may be necessary to operate hydrants or blow-offs to get sufficiently high flows.

**Pump Efficiency Testing**

Typically, only the head characteristic curve is needed for modeling; however, some models determine energy usage at pump stations as well as flow and head. To determine energy usage, the model must convert the water power produced by the pump into electric power used by the pump. This conversion is done using the efficiency relationships summarized below.

\[
e_p = \frac{\text{water power}_\text{out}}{\text{pump power}_\text{in}} \quad (5.14)
\]

\[
e_m = \frac{\text{pump power}_\text{in}}{\text{electric power}_\text{in}} \quad (5.15)
\]

where \( e_p \) = pump efficiency (%) \\
\( e_m \) = motor efficiency (%)

Pump power refers to the brake horsepower on the pump shaft, and it is difficult to measure in the field. Therefore, all that can be calculated is the overall (wire-to-water) efficiency.

\[
e_{\text{w-w}} = e_p \times e_m = \frac{\text{water power}_\text{out}}{\text{electric power}_\text{in}} \quad (5.16)
\]

where \( e_{\text{w-w}} \) = wire-to-water efficiency (%)

Although efficiency is expressed as a decimal in the above equations and in most calculations, it is generally discussed in terms of percentages. Water power is computed from the following relationship:

\[
WP = C_f Q h_f \quad (5.17)
\]

where \( WP \) = water power (hp, Watts)
\[ \begin{align*}
Q &= \text{flow rate (gpm, l/s)} \\
\h_p &= \text{head added at pump (ft, m)} \\
\gamma &= \text{specific weight of water (lb/ft}^3, \text{N/m}^3) \\
C_f &= \text{unit conversion factor (4.058 \times 10^{-6} \text{ English, 0.001 SI})}
\end{align*} \]

The measurement of electric power depends on the instrumentation available at the pump station. Large stations may have a direct readout of kilowatts, thus the wire-to-water efficiency can be easily computed by converting the water power and electric power to the same units. In other cases, it may be possible to measure the current drawn in amps. Knowing the voltage, power factor, and number of phases, the electric power drawn can be determined as

\[ \text{EP} = VI_\bar{N}(PF) \tag{5.18} \]

where

\[ \begin{align*}
\text{EP} &= \text{electrical power (watts)} \\
V &= \text{voltage (volts)} \\
I &= \text{current averaged over all legs (amps)} \\
N &= \text{number of phases} \\
PF &= \text{power factor}
\end{align*} \]

Except for the motors driving the smallest pumps, pump motors are generally three-phase. The power factor is a function of the motor size and the load for three-phase motors. Additional information can be found in WEF (1997).

At some pump stations, there may be no instrumentation available for measuring electric power, and it may be difficult for electricians to directly determine amperage. In these situations, it is necessary to measure the energy usage at the building power meter and divide the energy use by time to determine power. If the meter is being read directly, be sure to account for other sources of power consumption.

Similar to the head characteristic curve, the efficiency curve can be developed by setting a flow rate, measuring the necessary parameters, and then adjusting the flow until sufficient points to form a curve are determined.

**Potential Problems with Pump Performance Tests**

A key piece of information needed for the model representation of the pump is the shutoff head (the head at zero flow). To find this point, the discharge valve is closed and measurements are taken while the pump is operating. It is important to note that if the pump operates with the valve closed for an extended period of time, the water in the pump may begin to heat, potentially damaging the pump and seals. Thus, the measurements must be taken quickly.

Another potential area of concern is electricity billing rates. Some water utilities include an electricity demand charge in their billing structure that is typically based on the highest 15- or 30-minute peak power usage period for the pump station. This demand charge, which can be quite high (US$14/kW for example), is applied to all of the current billing period, and may be applied to subsequent billing periods for up to a year. It is important to note that pump testing may require large amounts of energy,
and care should be taken that a new and expensive demand charge is not set for the utility.

**Using Pump Performance Test Data for Calibration**

The data obtained from a pump performance test are used to generate the pump head versus discharge and efficiency curves, which are used to mathematically model the performance of the pumps. The pump test data collected are input into the model, which then uses curve-fitting techniques to create the relationships describing the pump efficiency and head curves.

### 5.5 EXTENDED-PERIOD SIMULATION DATA

Most of the testing described in Sections 5.1 to 5.4 results in static measurements of the distribution system—that is, measurements taken at a single point in time under a single set of conditions. This information is useful for estimating various parameters used in steady-state and EPS models. When an extended-period simulation model is developed, it is necessary to supplement the static field testing with field measurements taken over a period of several days. This information can be used for calibrating an EPS model (see Chapter 7) and validating that an existing EPS model adequately represents the behavior of the distribution system over time.

Two types of data that are useful for calibrating and validating an extended-period simulation model are

- Time-varying measurements of flow, pressure, and tank water levels in the distribution system
- Concentrations of a conservative tracer over time throughout the system

The following sections discuss these two types of data.

**Distribution System Time-Series Data**

Flows, pressures, tank water levels, and other characteristics vary throughout the distribution system both temporally and spatially. Seasonal variations, variations by day of the week, diurnal variations, and small time scale stochastic variations typically occur. If an extended-period model of the distribution system has been properly constructed and calibrated, the model results should approximately mimic the behavior of the system over a period of time. Such temporally and spatially varying data are frequently collected for use in calibrating an EPS model (see Chapter 7).

Frequently, time-series data is available automatically through remote telemetry that is part of a SCADA (Supervisory Control and Data Acquisition) system (see Chapter 6). This information can usually be easily downloaded or converted to a format for use in the calibration process. Though information may be transmitted at very frequent intervals, for most calibration purposes, measurements every 15 to 60 minutes are generally adequate. SCADA data can be supplemented by that from flow meters or pressure gages and data loggers installed for short-term data collection. Section 5.1 describes different types of flow meters.
Conducting a Tracer Test

In a tracer test, a conservative substance is added to the water in a distribution system over a period of time, and the movement of the tracer through the system is determined by measuring its concentration over time at stations located at key points within the system (Grayman, 2001). The resulting data may be used in conjunction with an EPS hydraulic model and a water quality model in the calibration process (see Chapter 7 for details on the use of these data for calibration).

The steps in conducting a tracer test are as follows:

1. A conservative tracer is identified for a distribution system. The tracer can be a chemical that is added to the flow at an appropriate location taking into account the study objective and location specific details or, for the situation where there are multiple sources of water, a naturally occurring difference in the water sources, such as hardness. Chemicals that are typically used include fluoride, calcium chloride, sodium chloride, and lithium chloride. Selection of the tracer generally depends on government regulations (for example, some localities will not allow the use of fluoride), the availability and cost of the chemicals, the methods for adding the chemical to the system, and the measuring and analysis devices. For example, a tracer chemical may be selected because it is inexpensive and can
be added using a water plant’s existing dry chemical feed system. The amount of tracer that is added depends on the measurement methods (that is, it must be great enough so that differences in concentration can be measured) and on regulations (resulting concentrations should not exceed allowable levels).

2. Before beginning the tests, it is recommended to simulate the test with the model to determine the likely results. Determining the results ahead of time assists in identifying the best sampling locations and times, and identifies the most likely concentrations to make sure that they are in the range of the measuring equipment.

3. A controlled field experiment is performed in which either: (1) the conservative tracer is injected into the system for a prescribed period of time; (2) a conservative substance that is normally added, such as fluoride, is shut off for a prescribed period; or (3) a naturally occurring substance that differs between sources is traced.

4. During the field experiment, the concentration of the tracer is measured at selected locations in the distribution system. It is desirable to have quick feedback on the movement of the tracer through the system so that adjustments can be made in the sampling schedule. For example, if the tracer takes more time than expected to reach a station, sampling needs to be extended beyond the originally planned period. With some tracers, such as fluoride, more accurate measurements can be made in the laboratory. To satisfy the need for quick feedback and accurate measurements, a quick method, such as use of a Hach handheld digital meter, can be employed for immediate feedback in conjunction with samples taken in bottles for later analysis in the lab. Measurements should continue until the tracer has reached the areas of the distribution system with the oldest water. In a system that contains a tank or multiple tanks, that may require a tracer test lasting many days (Clark, Grayman, Goodrich, Deininger, and Hess, 1991).

5. During the tracer test, other parameters that are required by a hydraulic model, such as tank water levels, pump operations, flows, and so on, should be collected at frequent intervals as well. This information is needed as part of the model calibration/validation process.

6. Frequently, a tracer test is conducted in conjunction with a water quality study (see Section 5.6) so that other constituents, such as chlorine residual, may be measured at the same time that tracer measurements are made.

### 5.6 WATER QUALITY SAMPLING

When extending a calibrated hydraulic model to include water quality, various physical and chemical parameters must be determined. Some tests require bench scale analyses that can easily be conducted in a modestly outfitted water quality laboratory. Other measurements can be made directly in the field. The sections that follow describe the types of tests that are performed in order to support the development of a water quality model.

A calibrated, extended-period simulation hydraulic model provides a starting point for water quality modeling. Steady-state hydraulic analysis is not adequate because it
does represent operational characteristics that vary temporally and does not account
for the effect of storage and mixing in tanks and reservoirs, a factor known to contrib-
ute to the degradation of water quality. As described in Chapter 2, transport, mixing,
and chemical reactions depend on the pipe flows, transport pathways, and residence
times of water in the network (all are network characteristics determined by the
hydraulic simulation). Therefore, a calibrated extended-period hydraulic model is a
prerequisite for any water quality modeling project. After a hydraulic model is pre-
pared, some types of water quality modeling analyses (especially water age and
source tracing) can be conducted with little additional effort whereas modeling reac-
tive constituents requires additional information on reaction rate coefficients.

Disinfectant residuals (chlorine, chloramines) decay due to reactions in the bulk water
and reactions that take place at the pipe wall. Disinfection by-products (DBP) grow
over time in the distribution system, and so a formation reaction rate is required by a
model. Bulk reaction coefficients are required for all nonconservative substances, and
wall reaction coefficients are required for disinfectant modeling. Boundary conditions
and initial conditions are needed for all substances. Determining bulk and wall reac-
tion coefficients involves laboratory analysis and field studies, as discussed in the fol-
lowing section. The determination of boundary and initial conditions is simpler and is
addressed in Section 7.5.

**Laboratory Testing**

For constituent analysis, reaction dynamics can be specified using bulk and wall reac-
tion coefficients. Bulk reaction coefficients can be associated with individual pipes
and storage tanks or applied globally. Wall reaction coefficients can be associated
with individual pipes, applied globally, or assigned to groups of pipes with similar
characteristics. Unlike bulk reaction coefficients, which can be determined through
laboratory testing, wall reaction coefficients must be measured using field tests or
determined as part of the calibration process, as discussed later in this section and in
Section 7.5.

**Bulk Reaction Coefficients.** Recall that the parameter used to express the rate
of the reaction occurring within the bulk fluid is called the bulk reaction coefficient.
Bulk reaction coefficients can be determined using a simple experimental procedure
called a *bottle test*. A bottle test allows the bulk reactions to be separated from other
processes that affect water quality, and thus the bulk reaction can be evaluated solely
as a function of time. Conceptually, the volume of water in a bottle can be thought of
as a water parcel being transported down a pipe (see page 52). A bottle test allows for
the evaluation of the impact of transport time on water quality and for an experimental
determination of the parameters necessary to model this process accurately.

Determining the length of the bottle test and the frequency of sampling is the first and
most critical decision. The duration and frequency of sampling will influence the
error associated with the experimental determination of the rate coefficient. The dura-
tion of the experiment should reflect the transport times occurring in the network. If,
for example, a water age analysis using the calibrated hydraulic model indicates that
residence times range from 5 to 7 days, conducting a 7-day test would provide bulk
reaction data over the entire range.
The frequency of the sampling should be proportional to the rate of the reaction. Typically, the sampling frequency should be more rapid at the start of the experiment (every 30 minutes for a fast reaction and once every two hours for a slow one) and can gradually decrease to a lower level (once or twice a day). After a schedule of samples is determined, bottles, reagents, and other experimental equipment can be gathered.

Bottle tests can be used to determine bulk reaction rates for different types of reactions (for example, disinfectant decay or DBP formation). The size of the bottles depends on the volume of water required by the experimental procedure. Methods for determining disinfectant concentrations can require anywhere from 20 to 100 ml. Methods for determining DBP formation typically require a smaller sample volume. In either case, the volume and number of bottles should also include any duplicates taken.

It is important for the modeler to appreciate the precision (or lack thereof) when measuring disinfectant residual concentrations. Each analytical method has its own minimum and maximum detection limits, and each person performing a method may have a bias or error associated with them as well. For example, attempting to measure concentrations of 0.08 mg/l when the analytical method is only accurate to 0.20 mg/l can produce misleading data. Duplicates and replicates can be used to quantify these types of errors.

Bottles should be washed prior to the experiment in accordance with the experimental procedure. Frequently, bottles are prepared improperly and the experiment yields worthless data. For example, if disinfectant decay is being measured, the bottle should be prepared so that it does not contribute to the decay reaction. This can be accomplished by soaking the bottles for 24 hours in a strong solution of the disinfectant.

### Bottle Test Procedure

1. **Preparation**
   - Plan the length of the experiment.
   - Collect materials needed for the experiment.
   - Wash bottles and prepare them using the chlorine demand-free procedure.
   - Prepare reagents for experimental methods and work area.
   - Prepare laboratory notebook for recording experimental conditions and results.

2. **Sample Collection**
   - Collect water from the clearwell as it enters the distribution system.
   - Fill and cap bottles headspace-free.
   - Start the master clock.

3. **Sample Testing**
   - Store samples in complete darkness with the temperature held constant (a water bath or BOD incubator may be used).
   - Pull samples at designated times and measure using experimental procedure.
   - Record time and result of the experimental procedure.

4. **Processing Data**
   - Plot data.
   - Process data to determine rate coefficient. (Summers, Hooper, Shukairy, Solarik, and Owen, 1996; Rossman, Clark, and Grayman, 1994; and Vasconcelos, Rossman, Grayman, Boulos, and Clark, 1996)
(10 mg/l) and then rinsing with laboratory-clean water (Summers, Hooper, Shukairy, Solarik, and Owen, 1996). Reagents should be gathered and prepared in accordance with the experimental procedure specific to the constituent being measured. Once the experiment has been planned and the laboratory prepared, the test can begin.

For the purposes of determining rates of reaction in the distribution system, water is typically collected as it leaves the clearwell and enters the network, though this need not always be the case. The water should be gathered and the bottles quickly filled and capped with no airspace in the bottle. The experiment starts when the last bottle is capped. At scheduled times, samples should be pulled and tested using the constituent-specific experimental procedure. Between sampling times, samples should be stored in complete darkness and at a constant temperature because reaction rates (and thus reaction coefficients) are temperature dependent, and some reactions are influenced by ambient light.

---

**Example — Bottle Test Data Analysis.** When all measurements have been taken and the experiment is over, the data will describe the constituent concentration for each of the samples as a function of time. The data can then be graphed. The constituent concentrations are charted along the y-axis (the dependent variable), and the time is charted along the x-axis (the independent variable). Figure 5.13 and Table 5.3 show an example of data collected from a bottle test for which the constituent was chlorine.

**Figure 5.13**
A best-fit straight line drawn through the charted results where the slope of the line is the bulk reaction coefficient.
### Table 5.3 Bottle test results

<table>
<thead>
<tr>
<th>Time (hours)</th>
<th>Observed Concentration (mg/l)</th>
<th>Time (hours)</th>
<th>Observed Concentration (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.2</td>
<td>54</td>
<td>0.9</td>
</tr>
<tr>
<td>6</td>
<td>2.1</td>
<td>60</td>
<td>0.9</td>
</tr>
<tr>
<td>12</td>
<td>2.0</td>
<td>66</td>
<td>0.8</td>
</tr>
<tr>
<td>18</td>
<td>1.7</td>
<td>72</td>
<td>0.7</td>
</tr>
<tr>
<td>24</td>
<td>1.4</td>
<td>78</td>
<td>0.6</td>
</tr>
<tr>
<td>30</td>
<td>1.3</td>
<td>84</td>
<td>0.5</td>
</tr>
<tr>
<td>36</td>
<td>1.2</td>
<td>90</td>
<td>0.5</td>
</tr>
<tr>
<td>42</td>
<td>1.0</td>
<td>96</td>
<td>0.5</td>
</tr>
<tr>
<td>48</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

If the substance in the bulk fluid exhibits a first-order reaction, then the reaction rate coefficient can be found using linear regression techniques. A best-fit straight line is drawn through the data collected from the bottle test, with concentration plotted on a log axis as illustrated in Figure 5.13. The slope of the line for the data in Table 5.3, 0.0165 hr⁻¹, becomes the bulk reaction coefficient. Note that the reaction coefficient is negative since the constituent concentration decays over time.

The straight line shown in Figure 5.13 produces a very nice fit with the observed data. A more likely scenario is that the data will not fit as well as that shown. In fact, there may be a few data points that are widely scattered. Outliers, as these points are called, should be carefully examined and possibly discarded if they are found to negatively influence the results of the data analysis. If there is a large amount of scatter, another bottle test may be performed in an attempt to collect more reliable data.

Bottle tests can be performed on any water sample, regardless of where the sample is collected. Samples from treatment plants and other entry points to the distribution system are particularly important because these facilities act as water sources and therefore, have a strong influence on water quality. Raw water quality influences finished water quality as well. Therefore, if a system has multiple treatment facilities each with a different raw water source, the bulk reaction rates for each of the finished waters are likely to differ. Although storage tanks are not sources of finished water, under some circumstances, the bulk decay rate can be different in tanks than in the distribution system. As a result, a separate bulk reaction coefficient should be considered for tanks and reservoirs. Booster disinfection (when disinfectant is reapplied to previously disinfected water) is another circumstance in which bulk reaction coefficients are likely to change.

Bulk reaction coefficients are associated with pipes for purposes of a simulation, and are assumed to remain constant throughout the simulation for a particular pipe. Since the bulk reaction coefficient is, in reality, associated with the fluid itself, the bulk reaction rate can change throughout the actual system as water from different sources...
becomes mixed at nodes. When assigning bulk reaction coefficients for pipes, the mixing can be considered by designing and conducting bottle tests with representative source mixtures. A source tracing analysis can assist in determining the degree of mixing in a system. Source blending can change over the course of the day for a particular pipe, thus the predominant source or mix of sources should be used in assigning the bulk reaction coefficient.

For example, suppose that a tracing analysis is performed on a system that has two treatment plants. Through the bottle tests, a bulk reaction coefficient is established for each treatment facility and each storage tank. Through a source tracing analysis of a junction node, it is found that 90 percent of the water for that node comes from a specific treatment plant. Accordingly, the bulk reaction rate coefficient for those pipes that make up the path between the plant and the node would be equal to, or nearly equal to, the rate coefficient for the plant. For a single-source network, it is useful to remember that the bulk reaction coefficient is really a function of the water passing through a pipe or stored in a tank, and not a function of the pipe or tank itself. Thus, specifying a global bulk reaction coefficient is frequently the simplest and the best method for modeling bulk reactions occurring in such networks.

Field Studies

Several different types of field studies may be performed to collect data used in calibrating a water quality model. The sections that follow describe three types of studies. The first type of study is aimed at determining actual pipe diameters — an important factor in water quality modeling. The second type of field study can be performed to determine pipe wall reaction coefficients.

Determining Actual Pipe Diameters. In the United States, friction head loss is usually predicted in network models by using the Hazen-Williams head loss equation:

\[ h_L = \frac{C_f Q^{1.852} (L)}{C^{1.852} D^{4.87}} \]  

(5.19)

where

- \( h_L \) = head loss due to friction (ft, m)
- \( C_f \) = unit conversion factor (4.73 English, 10.7 SI)
- \( Q \) = flow (cfs, m\(^3\)/s)
- \( L \) = length (ft, m)
- \( C \) = Hazen-Williams C-factor
- \( D \) = diameter (ft, m)

Most water distribution system models use pipe diameters based on the originally stated nominal diameters (see page 255). Even for new pipes these values are only approximations, and for older pipes these values can seriously overstate the effective diameter due to tuberculation. For example, a new Class 50 ductile iron pipe with a nominal diameter of 6 in. has an actual internal diameter of 6.4 in. Tuberculation is most common in older metallic pipes and can result in significant reductions in the effective pipe diameter.
For most hydraulic applications, the use of the nominal diameter is acceptable. This is especially the case if C-factor tests were performed, and the nominal diameter was used in the process of estimating C-factors. In effect, errors in both the true diameter and the C-factor offset each other and, as a result, head loss and flows are calculated to a generally accepted level of accuracy. This can be illustrated by examining the denominator in Equation 5.19 and observing that any combination of C-factor and diameter that result in the same value of the denominator will result in the same value for head loss. For example, a C-factor of 83 and a pipe diameter of 12 in.(305 mm) results in the same head loss as a C-factor of 134 and a pipe diameter of 10 in. (254 mm).

However, use of an incorrect pipe diameter can result in significant errors in predicting velocity. Because velocity is a major factor in water quality modeling both in terms of its effect on travel times and on calculation of chlorine wall demand, a premium exists on correctly estimating velocity. Therefore, one should ensure that the diameters used in a hydraulic model when used as part of water quality modeling more closely reflect actual values. Several methods are available that can be used to estimate the true diameter of a pipe:

- **In-situ direct measurement of diameters**: A specially designed set of calipers can be used to directly measure the diameter of a pipe at a particular location. Figure 5.14 depicts a particular design for these calipers. The calipers are inserted into a pipe at a corporation stop and adjusted so that they directly measure the inside diameter of the pipe. It should be noted that this method can provide relatively accurate point measurements. However, if the diameter varies significantly along the pipe due to irregular tuberculation, the point measurements may or may not be representative of the true diameter for a pipe segment.

- **Measurement of flow and velocity and calculation of diameters**: Flow can be calculated as the product of velocity and cross-sectional area. Therefore, if the flow and velocity in a pipe are known, the actual pipe diameter can be calculated. Various methods are available for measuring flow and velocity. In order to use these measurements to calculate true diameter, however, the measurements must be independent. Thus, if a flow meter is measuring velocity and using an assumed diameter to convert to flow, this measurement can be used only as a single measurement, and a second independent measurement is needed. A Pitot gage is frequently used as part of a hydrant test to determine flow. Injection of a pulsed slug of tracer at an upstream corporation stop and measurement at a downstream hydrant can provide an accurate travel time and velocity calculation (Wright and Nevins, 2002).

- **Development and use of an inventory of actual pipe diameters**: It is good practice to keep a database on pipes that have been taken out of service. The database should include the pipe material, date of installation, pipe location, nominal diameter, and notes on the condition of the pipe including the effective diameter of the pipe. This information can be used to define representative effective diameters for pipes of a particular age and material and entered into the model as an alternative to the original, nominal diameter. Also, this information can be used as a basis for analyzing pipe breaks in a system.
Measuring Chlorine Wall Demand. Wall demand can be indirectly measured in the field in a manner that is analogous to C-factor tests (Grayman, Rossman, Li, and Guastella, 2002). A homogeneous pipe segment (constant diameter, material, and age) with a length of at least 1,000 ft (305 m) is selected for analysis. The pipe segment is isolated by valving off major laterals and at the downstream end, as shown in Figure 5.15.

The downstream hydrant is then flowed at a constant rate and chlorine measurements are taken at an upstream hydrant ($C_1$) and at the flowed hydrant ($C_2$). The chlorine measurement at the downstream hydrant should be taken $TT$ minutes after the measurement at the upstream hydrant where $TT$ is the travel time between the two hydrants. The process can be repeated at different flow rates. After accounting for the bulk decay of chlorine (usually negligible in a short pipe segment), the wall demand coefficient can be calculated based on the field data in a spreadsheet or by iteratively running a water quality model of the pipe segment until the model results match the
field data. This method is most appropriate for pipes that are expected to have relatively higher wall demands such as smaller diameter, unlined cast iron pipes. Larger pipes and nonferrous pipe materials, such as PVC or cement, generally have relatively low wall demands. It is likely that little or no chlorine loss would be measured in short segments.

**Intensive Water Quality Surveys.** Intensive hydraulic and water quality surveys that extend over a multiday period can be conducted to calibrate or validate an extended-period hydraulic and/or water quality model. In such a survey, information on how the system was operated is compiled along with time-series data collected in the field or through remote telemetry. Surveys of this type can be quite expensive and involve significant personnel, equipment, and laboratory analyses. Therefore, they should be carefully planned and executed. A properly conducted survey can yield a wealth of information that is invaluable in understanding the movement of water through the study area and for calibrating and validating a model. Clark and Grayman (1998) provide a detailed description of how to prepare for and conduct an intensive water quality field survey. The information in this section draws heavily upon the description provided in their book, *Modeling Water Quality in Drinking Water Distribution Systems*.

Within the modeling context, the purpose of an intensive field survey is to collect sufficient information in order to adjust model parameters or validate model parameters by mimicking the results observed in the field. Operational information that is not collected during the study becomes unknowns, introducing uncertainty in the modeling process. Ideally, a detailed model of the study area is available prior to planning a field study. If that is the case (and even if it is not a well-calibrated model), the model can be used to predict how the system will behave and thus to determine what data to collect, and when and where, in order to make the best use of the study results to calibrate or validate the model.

In the ideal case, it is best to predefine how the distribution system should be operated during the field study in order to design an experiment that best serves the purpose of the work. For example, one objective may be to operate the system in a manner that is typical for that season. Another objective could be to operate the system in a streamlined manner (for example, avoid turning pumps on for a few minutes at a time) in order to simplify the modeling process.

Intensive surveys conducted as part of a water quality modeling effort generally focus on collection of data related to the hydraulic nature of the system, field data on disinfection residuals, and other water quality constituents such as temperature, pH, and disinfection by-products (DBP). In some studies, a tracer is used to determine the travel times and the patterns of movement of water through the distribution system. The mixing patterns within storage facilities may also be studied during an intensive survey.

A field study is divided into three phases: designing the field study, executing the field study, and analyzing and using the resulting set of data. Arguably, the most important phase is the preplanning stage. Prior to performing a sampling study, a detailed written sampling plan should be prepared. Clark and Grayman (1998) provide a list of issues that should be addressed in a sampling plan and specific actions that should be taken prior to a sampling study:
• **Sampling location:** Selection of sampling sites is typically a compromise between selecting sites that provide the greatest amount of information and sites that are most amenable to sampling. Sites should be spread throughout the study area and should reflect a variety of situations of interest, such as transmission mains and local lines, areas served directly from a source, and areas under the influence of tanks. Because sampling is usually performed around the clock, the sites must be accessible at all times. Also, sampling taps should be placed close to mains so that the samples reflect the water quality in the mains.

If automatic samplers are used, the sites may need to be secure (automatic sampling equipment is expensive), and electric power and a method for disposing of waste flow may be required. For manual sampling, the travel time for crews between sampling sites is a consideration. Dedicated sampling taps; water utility and public buildings, such as firehouses; and hydrants are frequently used as sampling sites. Pump stations and valve vaults are also good locations because it is possible to directly sample the main. Care must be taken when sampling through a hydrant or service line to minimize or account for the lag between the sample time and the time the water left the main.

• **Sampling frequency:** For automated samplers, such as chlorine monitors, pH meters, and pressure gages, the sampling frequency can generally be set and is usually performed every few minutes. In the case of manual sampling, a circuit is usually defined for a sampling crew to follow. The crew takes samples at one site, analyzes them, and then moves to the next site, and continues this circuit for their entire shift. These practical constraints and the project budget result in trade-offs between the number of samplers, the number of sites, and the sampling frequency.

Sampling frequency is usually on the order of once per hour to once every several hours. The rate at which characteristics change at a site is an important factor in choosing a sampling frequency. Thus, if the chlorine residual is expected to change gradually over the course of a day, less frequent sampling is
needed than if the residual is expected to vary rapidly over time. Dress rehearsals in which a crew drives a circuit to determine driving times and tests the sampling process to define the amount of time that must be spent at a site provides important information when designing the sampling program. In following a front through the systems (when the fluoride feed is turned off, for example), it is desirable to sample more frequently at the time that the front is expected to arrive.

- **System operation:** The sampling plan should specify the general system operation that is expected to occur during the sampling study and discuss the methodology for capturing the information on system operation. For example, the plan should indicate if any unusual operations are planned during the study period.

- **Tracer study:** If a tracer study is to be performed, details on the tracer study should be discussed. This includes the type of tracer to be used, regulatory requirements to use the tracer, the quantity of tracer needed, where the tracer will be purchased, how and at what rate the tracer will be injected, how the tracer will be measured in the field, and equipment. A meeting with the regulatory agency may be required.

- **Preparation of sampling sites:** Various activities should be planned to prepare sampling sites prior to the sampling study. These may include testing and flushing hydrants, installing sampling appurtenances, determining required flushing times, and notifying personnel located at sampling sites in buildings. Automated monitoring devices should be installed and thoroughly tested several days before the actual sampling commences.

- **Sample collection procedures:** The sampling plan should include specific procedures to be followed during the sampling program. Topics to be discussed include the flow rate and length of time that the tap should be flowed before each sample is taken, the filling and sealing of the sampling containers, preservation of samples and required reagents, labeling of sampling containers, data recording, and delivery of samples to laboratories.

- **Analytical procedures:** Specific analytical procedures should be determined and described for any analyses that are conducted in the field. This includes chlorine measurements and measurements of other water quality constituents. Procedures to be used in the laboratory should be specified. It is important for the modeler to know the accuracy and minimum detection limits for each test so that he or she can correctly assess the field results.

- **Personnel organization and schedule:** Intensive sampling surveys can involve a large number of personnel working around the clock in shifts for several days. Work schedules should be determined for each member of the survey team. This discussion should include logistical arrangements (where to meet, who will bring what equipment, emergency phone numbers, and so on).
• **Safety issues:** The safety of the survey team is of paramount importance when planning a study. Potential for accidents is very high due to the around-the-clock effort, possible bad weather, unusual surroundings, sampling locations in close proximity to traffic, and countless other factors. Crews should be clearly identified and carry proper identification cards, should wear reflective vests or other paraphernalia to make them more visible, use marked vehicles (if possible), carry flashlights, and be advised how to respond to various emergency or unusual situations. The police department and other government agencies should be notified prior to the sampling study, and public notification through newspapers and television should be considered.

• **Data recording:** The information being collected is very valuable, and a procedure for recording and safeguarding the information should be developed in the study plan. Data forms should be prepared that include the sampling location, sampling time (consider using military time to avoid ambiguity), sampler’s name, field measurements, samples taken for laboratory analysis, and any comments or observations. Forms should be filled in ink and safeguarded until they are delivered to a central location.

• **Equipment and supply needs:** A complete list of equipment and supplies should be developed along with a schedule and plan for obtaining the materials well before the start of the sampling survey.

• **Calibration and review of analytical instruments:** All instruments should be properly calibrated and thoroughly checked out prior to the study. The study plan should specify the methodology and schedule for performing this task.

• **Training requirements:** Before the start of the survey, all survey personnel should observe the sampling sites and receive hands-on training in the use of the equipment and protocols.

• **Contingency plans:** Over the course of the several-day survey, some aspect of the survey will most likely not go according to plan. The purpose of contingency planning is to be prepared for such events. The contingency plan should address equipment malfunction, severe weather, illness, changes in the operation of the system, and other potential events.

• **Communications:** During the sampling survey, it is important that sampling crews, personnel at the operations center, and the overall supervisor for the study be in communication in case of questions or changes. Using cell phones and two-way radios is recommended.

The study plan serves as a blueprint for conducting the water quality survey. During the actual survey, data and information are assembled and assessed at a central location on a near real-time basis. If some aspect of the study is not proceeding as originally intended (the tracer is spreading at a quicker rate through the system than expected, for example), modifications can be made in real-time. All aspects of the
survey should be documented during the survey and following the completion of the field work.

Although intensive water quality sampling studies can be expensive, they are valuable in developing and validating water quality parameters and ensuring that the hydraulic calibration is adequate for water quality modeling tasks.

## 5.7 SAMPLING DISTRIBUTION SYSTEM TANKS AND RESERVOIRS

The primary water quality issues in distribution system tanks and reservoirs are contamination entering the facility, long residence times, and poor mixing conditions. Monitoring can be an effective mechanism for identifying contaminants and studying the mixing and water quality behavior in existing tanks or reservoirs. There are three categories of sampling and monitoring studies:

- Water quality studies provide data on the temporal and spatial variation of water quality parameters within the storage facility and in the inflow and outflow.
- Tracer studies provide information on the mixing behavior in the tank.
- Temperature studies gather information on how the temperature may vary at different locations and depths within the tank over time.

These studies can be performed as part of an integrated study to develop a better understanding of how reservoirs and tanks behave. The sampling results can also be used to help calibrate or validate a mathematical or scale model of the storage facility. The design and implementation of monitoring programs for distribution system tanks and reservoirs was examined in a recent AWWA Research Foundation sponsored study (Grayman et al., 2000).

### Water Quality Studies

Water quality monitoring studies of tanks and reservoirs can provide data on the temporal and spatial variation of water quality parameters within the facility and in the inflow and outflow. Information on the state of the reservoir (is it filling or draining, for example) should be collected during a water quality study in order to interpret the water quality monitoring data. Internal sampling of a tank or reservoir provides information on the actual spatial variation of water quality parameters inside the facility. Water quality data in combination with information on the inflow and outflow history furthers the understanding of the water quality behavior of the tank.

Water quality studies in tanks and reservoirs can be conducted to meet many different goals. Routine and regulatory sampling is performed at storage facilities throughout a distribution system to satisfy regulatory requirements, define the general water quality in the facilities, and identify potential problems. Water quality sampling studies can also be designed specifically to identify the variations in water quality over time and location within the facility, the water quality transformations that occur during storage, or the mixing processes that occur during inflow and outflow.
In addition to sampling at the inlet and outlet, samples taken at different locations within the storage facility and at different depths provide information on spatial variability. Grab samples can be supplemented with automated monitors that perform and log analyses at a pre-set sampling frequency. Chlorine residual and temperature are typically measured. Bulk chlorine decay tests are usually performed in order to understand the kinetics of chlorine decay within the facility. The effects of wall demand in a tank are usually minimal because of the large ratio of volume to wall surface. Other water quality constituents may be sampled to meet regulatory requirements or in response to specific water quality concerns.

Internal samples are frequently taken through hatches located on the top of a tank. When sampling elevated tanks or standpipes, tank climbing and sampling issues become more substantial, and safety concerns become more of an issue.

**Tracer Studies**

Tracer studies in tanks serve a similar purpose as tracer studies in a distribution system: to define the movement of water through the vessel. In a tank tracer study, a chemical tracer is added to the inflow and its movement is monitored through analysis of the tracer within the facility or in the outflow. Water quality sampling studies are usually performed in conjunction with tracer studies.

Tracer studies of distribution system tanks and reservoirs can provide information regarding the detention time of water in the storage facility, as well as the mixing conditions as it fills and drains. The objective is to determine how influent water entering the reservoir mixes and subsequently leaves the facility. In addition, tracer study data can be used to develop, calibrate, or validate computational fluid dynamics (CFD) models (see page 358) or physical scale models. When these models exist for a given reservoir, modifications in design or operation can be tested before implementing a costly change at full scale.

Grayman et al. (2000) describe the procedures involved in planning a tracer study. They include selection and injection of the tracer chemical, logistical considerations in performing the study, and the collection of various ancillary hydraulic and water quality data. The following sections discuss these topics.

**Tracer Chemicals.** The most frequently used tracer chemicals include fluoride and salt solutions (calcium chloride, sodium chloride, lithium chloride, and potassium chloride). In the United States, acceptable chemicals for a potable water supply are usually limited to National Sanitation Foundation (NSF) approved chemicals or food grade chemicals that have been approved by the state regulatory agency. It is important to make sure that the tracer does not affect the density of the inflow water because this occurrence can give misleading results.

**Tracer Injection.** The step dose method of injection is generally used in distribution system storage facilities in which the tracer is fed into the influent over one or more fill periods at a relatively constant concentration. The injection point should be located far enough from the reservoir inlet so that the tracer has fully mixed with the influent prior to its entry into the reservoir but close enough so that the tracer does not
enter the distribution system directly. A sample tap downstream of the injection point before the water enters the reservoir is desirable so that the actual tracer concentrations entering the storage facility can be monitored.

**Tracer Dosage.** The tracer dosage should be calculated so that variations in concentration can be clearly measured in the tank. It should not result in concentrations exceeding regulatory requirements.

**Monitoring Locations and Frequency.** Monitors or grab samples should be taken at the inlet and outlet of the tank and, ideally, at locations within the facility. If stratification is suspected, sampling at varying depths is important. Internal sampling is generally limited by access points and the location of permanent sampling ports. If grab samples are taken, a sampling frequency of approximately once per hour is generally sufficient with more frequent sampling at the inlet sample tap during the fill period and less frequently during the draw period. With automated monitors, more frequent sampling is recommended. Samples should be collected over several fill and draw cycles or until the water exiting the reservoir approaches the background concentration of the tracer chemical.

**Regulatory Approval.** Policies of state agencies concerning the addition of tracers varies significantly around the country. For example, some may not allow the addition of fluoride while others may not allow normal fluoridation to be turned off. It is good practice to obtain written approval from the state regulatory agency before performing the tracer study.

**Flow Measurements.** Inflow and outflow rates are required to assess the behavior of the reservoir and to interpret the tracer results. If the reservoir operates in a fill and draw mode (as opposed to simultaneous inflow-outflow), inflow and outflow rates can be estimated from water level measurements during the study. For simultaneous fill and draw, and in situations where more accurate flow rates are needed, flow meters on the inlet and/or outlet can be used.

**Temperature Monitoring**

Temperature variations in a tank or reservoir can affect the mixing characteristics in the facility and, in extreme cases, lead to stratification. Spatial and temporal variations in temperature can result from changes in inflow temperatures, differential heating in the tank, and insufficient mixing. Flow patterns in a tank or reservoir can sometimes be affected by temperature differences of less than 1.0°C. Temperature can vary in both the vertical and horizontal directions and may change over the course of a fill and draw period, over a few days, or between seasons.

Temperature can be measured manually with a thermometer or probe, or automatically by a thermistor and data logger. Measuring temperature manually is inexpensive and easy to implement but labor intensive for longer sampling periods. Measuring temperature automatically requires equipment that costs a few thousand dollars. For either method, temperature measurements should be quite accurate (to the nearest 0.1°C, if possible) because small variations in temperature are generally observed.
With manual sampling, samples can be collected from a sampling tap or can be drawn from different locations by using a pump or sampling apparatus. Collection of samples from different depths by using a pump or depth sampler requires access from above the reservoir or tank through access hatches.

Long-term temperature measurements can be taken using an apparatus composed of a series of thermistors and a data logger. The thermistors are positioned to the required depth and attached to a data logger which can be set to take a reading at a preset frequency (generally every 15 minutes to 60 minutes is adequate). Figure 5.16 is a schematic representation of a set of thermistors and a data logger. In this case, thermistors are attached to a chain and set at specific depths. Additional thermistors are attached to floats to measure temperatures at fixed preset depths below the water surface as the water level varies. Internal sampling equipment should be removed before the winter in areas where ice can form in tanks.

### 5.8 QUALITY OF CALIBRATION DATA

Users will sometimes try to calibrate models where the velocity and head loss are very low, and thus the hydraulic grade line is essentially flat. Under such conditions, the heads in the system are essentially the same as those at the boundary conditions and virtually any value of the roughness coefficient or demand can be used to produce similar results (Walski, 2000). McBean, Al-Nassari, and Clarke (1983) used first-order analysis to determine the accuracy of pressure measurements needed for field data to be useful for model calibration.

Referring to the head loss equations presented in Chapter 2 (see page 34), the head loss depends heavily on the flow and the C-factor. Most model calibration eventually comes down to adjustments in a parameter like the C-factor, according to the equation:

$$C = \frac{k(Q \pm \Delta Q)}{(h_L \pm \Delta h_L)^{0.54}}$$  \hspace{1cm} (5.20)

where

- $C =$ Hazen-Williams C-factor
- $k =$ factor depending on units and distribution system
- $Q =$ estimated flow (gpm, m³/s)
- $\Delta Q =$ error in measuring $Q$ (gpm, m³/s)
- $h_L =$ estimated head loss due to friction (ft, m)
- $\Delta h_L =$ error in measuring head loss due to friction (ft, m)

If the flows and heads are small, errors in measuring these quantities will be on the same order of magnitude as the quantities themselves, making them of little use in the calibration process. If such data are used, the value of parameters found by calibration will be poor.
The key to successful calibration is to increase the flows and the head losses such that these values are significantly greater than errors in measurement. The best way to do this is by conducting hydrant flow tests as described in Section 5.2. If the model can match conditions under normal demands and fire flow tests, then the user can feel confident that the model can be applied to other conditions.

In larger pipes [for example, greater than 16 in. (400 mm)], hydrant flow tests will not generate significant velocities. For these larger pipes, the engineer needs to create head loss either by measuring it over very long lengths of pipe, or by artificially increasing flow by allowing tank water levels to drop significantly and then filling the tank quickly. Figure 5.17 shows a hydrant flow test.
Impact on Optimized Calibration. With the availability of powerful optimization software (see page 268) that can accurately and automatically calibrate a water distribution model, it is more important than ever to have high quality field data. This is because the optimization software is guided by the field measurements. With perfect field measurements of head loss, optimization programs can give excellent calibration results for pipe roughness, water demand, and element statuses. However, with error in the head loss measurements, the optimization programs will deliver misleading calibration results because they do not distinguish between good and bad field measurements.

The denominator in Equation 5.20 serves as the key to the basic rule for acceptance of data for use in calibration as shown in Equation 5.21. As the error in head loss approaches the actual head loss, the usefulness of an observation for calculating head loss is diminished.

\[
h_L \gg \Delta h_L
\]  

(5.21)

This rule is applicable for evaluating data for either manual or automated calibration but is especially critical for automated calibration. For manual calibration, HGL values are used as a check of results, and odd data can be discounted. However, with automated calibration optimization, every HGL observation is treated as if it is exactly true and can drive the solution to erroneous values, which the program would claim are optimal. Therefore, only HGL observations that meet the criterion stated in Equation 5.21 should be used in the calibration.

Data collected when head loss is small can be used to check if the roughness and demand are wrong but cannot be used to determine what values are correct. For example, if the measured pressure is 65 psi (448 kPa) and the model predicts 75 psi (517 kPa) during low demand, the user can be certain there is something wrong with the model (most likely not roughness or demand). But when the model predicts 65 psi (448 kPa) and the measured pressure is 65 psi (448 kPa), the user cannot conclude the model is correct because even incorrect roughness or demand may yield that result.

Compounding the problem is the fact that although model users have an appreciation for pressures or HGL values throughout the system, they usually do not think in terms of head loss between two points. For example, a modeler may know that the pressure in one part of the system is 50 psi (340 kPa) and that the HGL in that area is 960 ft (293 m), but will have very little intuitive feel for how much head loss there is between that point and the nearby tank.

Example — C-Factor Sensitivity. For a given distribution system, error in flow is usually not larger than flow measurements so Equation 5.20 can be simplified and the head loss can be related to C-factor by:

\[
C = \frac{k}{(h_L \pm \Delta h_L)^{0.54}}
\]

Obviously, as the error term becomes large with respect to the actual head loss, the confidence bound on any calculated C-factor becomes large. For example, Figure 5.18 shows that for a system with an actual C-factor of 100 and a 10 ft (or m) head loss between the boundary node and the measurement
point, the confidence bounds are wide, and it does not take a very small error in measuring head loss to make a huge difference in C-factor. For example, if the head loss is 10 ft and the error in measurement is 5 ft, one can only conclude that the C-factor is somewhere between 124 and 69. On the other hand if the head loss is 40 ft and the error in measurement is 2 ft, then the C-factor is between 103 and 97.

Figure 5.18
Impact of error in head measurement

Sources of Error in Calibration Data. The inequality provided in Equation 5.21 can be viewed as the basic law for screening data for use in automated calibration and can be satisfied by increasing the left side or decreasing the right side. Errors in the right side can be due to inaccuracies in pressure readings, elevations, or boundary conditions, as discussed in the following.

- **Pressure readings:** Pressure gages must be accurate and readable to +/- 1 psi (6.8 kPa) and preferably should have an accuracy better than that. Even good quality gages can drift out of calibration by several psi (kPa), rendering the entire calibration effort in doubt, so they must be calibrated frequently.

- **Elevation:** Elevation data are usually the largest source of error; however, unlike pressure data, once an elevation is accurately established, it does not change over time. Elevations used for normal model nodes can have a fair amount of error and still be useful; however, for calibration, elevations should be known to within 1 ft (0.3 m). This means that elevations of the pressure gage (not the ground) should be determined by surveying, using a high quality global positioning system, or using contour maps from digital
orthophotogrammetry with an accuracy on the order of 1 ft (0.3 m) (Walski, 1998). Other possible sources for elevation data are elevations surveyed from sewer manhole lids if their elevations are considered accurate and readings from a high quality altimeter that is regularly calibrated and sufficiently accurate.

- **SCADA data during flow tests:** Some engineers trust data from SCADA systems without question. However, SCADA data may be taken from inaccurate sensors at inaccurate elevations. Concerning SCADA data, Akel (2001) notes that even though they were digitally generated, they are not precluded as sources of error. SCADA data may also pick up errors in transmission and polling intervals. Most SCADA systems are not continuously wired into a sensor; they poll the sensor periodically (interval of minutes to seconds). Therefore, the pressures being displayed on the SCADA may not correspond to the pressure in the system at that time. (See Chapter 6 for more information on sources of error in SCADA data.)

- **Chart recorders during flow tests:** Chart recorders have similar problems in capturing flow test data. Because the chart speed is slow, hydrant flow tests usually show up as a vertical line. Unless the test was run for a long time, it is impossible to get an accurate flow test pressure reading from a chart recorder. Human operators viewing a gage at the hydrant, pump station, or control valve of interest are the safest means of obtaining pressure data during flow tests. If personnel are available, it is best to station an individual at a key pressure control valve or pump station to read the gages during the test. Data loggers with a fairly high speed of data capture may also be used to capture pressure and flow readings during a flow test. It is essential during flow tests to run hydrants long enough so that all transient effects have dissipated, otherwise they may mask the actual values. It may also take a while for a pressure-regulating valve to fully adjust itself during a flow test.

- **Tank water levels:** Operators are usually more interested in the fluctuations of tank water levels than the accuracy of the level. As a result, it is not uncommon to find tank level readouts off by several feet. Tank water level sensors need to be checked before calibration data are collected. In addition, the water level and pump status must be taken at exactly the moment when pressure readings are taken. Using the average level of the tank during the afternoon when data were collected can lead to errors in calibration.

If the benefits of optimal calibration are to be realized, the modeler needs to carefully plan the data collection effort and recognize instances where optimal calibration may not be the best alternative. For example, in some situations, such as larger transmission mains, it may be better to run a C-factor test on the pipe and use that value, rather than perform optimal calibration. While optimal calibration programs can greatly simplify the adjustments needed for calibration, the software does not have the capability to judge and ignore/discount questionable data. It is the responsibility of the user to ensure that the model is fed accurate and useful data.


DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See Continuing Education Units on page xxix for more information, or visit www.haestad.com/awdm-ceus/

5.1 *English Units:* Compute the HGL at each of the fire hydrants for the pressure readings presented below and complete the table.

<table>
<thead>
<tr>
<th>Location</th>
<th>Elevation (ft)</th>
<th>Pressure Reading (psi)</th>
<th>HGL (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FH-1</td>
<td>235</td>
<td>57</td>
<td></td>
</tr>
<tr>
<td>FH-5</td>
<td>321</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>FH-34</td>
<td>415</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>FH-10</td>
<td>295</td>
<td>68</td>
<td></td>
</tr>
<tr>
<td>FH-19</td>
<td>333</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>FH-39</td>
<td>412</td>
<td>27</td>
<td></td>
</tr>
</tbody>
</table>

*SI Units:* Compute the HGL at each of the fire hydrants for the pressure readings presented below and complete the table.

<table>
<thead>
<tr>
<th>Location</th>
<th>Elevation (m)</th>
<th>Pressure Reading (kPa)</th>
<th>HGL (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FH-1</td>
<td>71.6</td>
<td>393</td>
<td></td>
</tr>
<tr>
<td>FH-5</td>
<td>97.8</td>
<td>290</td>
<td></td>
</tr>
<tr>
<td>FH-34</td>
<td>126.5</td>
<td>103</td>
<td></td>
</tr>
<tr>
<td>FH-10</td>
<td>89.9</td>
<td>469</td>
<td></td>
</tr>
<tr>
<td>FH-19</td>
<td>101.5</td>
<td>310</td>
<td></td>
</tr>
<tr>
<td>FH-39</td>
<td>125.6</td>
<td>186</td>
<td></td>
</tr>
</tbody>
</table>
5.2 *English Units:* A tank is used to capture the flow from a fire hydrant as illustrated in the figure. The tank is 50 ft long, 30 ft wide, and 12 ft high. What is the average discharge from the fire hydrant if the container is filled to a depth of 10 ft in 90 minutes?

\[
\begin{align*}
\text{SI Units:} & \quad \text{A tank is used to capture the flow from a fire hydrant as illustrated in the figure. The tank is 15.2 m long, 9.1 m wide, and 3.7 m high. What is the average discharge from the fire hydrant if the container is filled to a depth of 3.0 m in 90 minutes?}
\end{align*}
\]

5.3 *English Units:* A fire flow test was conducted using the four fire hydrants shown in the following figure. Before flowing the hydrants, the static pressure at the residual hydrant was recorded as 93 psi. Given the data for the flow test in the following tables, find the discharges from each hydrant and finish filling out the tables. Flow was directed out of the 2 ½-in. nozzle, and each hydrant has a rounded entrance where the nozzle meets the hydrant barrel.

\[
\begin{align*}
\text{Residual Hydrant} & \quad \text{FH-1} \quad \text{FH-2} \quad \text{FH-3} \\
Q_{2} & = Q_{1} = Q_{3} \\
\end{align*}
\]

a) Would you consider the data collected for this fire flow test to be acceptable for use with a hydraulic simulation model? Why or why not?

b) Based on the results of the fire flow tests, do you think that the hydrants are located on a transmission line or a distribution line?

c) Would these results typically be more consistent with a test conducted near a water source (such as a storage tank) or at some distance away from a source?

d) If the needed fire flow is 3,500 gpm with a minimum residual pressure of 20 psi, is this system capable of delivering sufficient fire flows at this location?
### Discussion Topics and Problems

**SI Units:** A fire flow test was conducted using the four fire hydrants shown in the figure. Before flowing the hydrants, the static pressure at the residual hydrant was recorded as 641 kPa. Given the data for the flow test in the following tables, find the discharges from each hydrant and finish filling out the tables. Flow was directed out of the 64 mm nozzle, and each hydrant has a rounded entrance where the nozzle meets the hydrant barrel.

a) Would you consider the data collected for this fire flow test to be acceptable for use with a hydraulic simulation model? Why or why not?

b) Based on the results of the fire flow tests, do you think that the hydrants are located on a transmission line or a distribution line?

c) Would these results typically be more consistent with a test conducted near a water source (such as a storage tank), or at some distance away from a source?

d) If the needed fire flow is 220 l/s with a minimum residual pressure of 138 kPa, is this system capable of delivering sufficient fire flows at this location?

<table>
<thead>
<tr>
<th>Residual Hydrant</th>
<th>Residual Pressure (psi)</th>
<th>Pitot Reading (psi)</th>
<th>Hydrant Discharge (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual Hydrant</td>
<td>88</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>FH-1</td>
<td>N/A</td>
<td>58</td>
<td></td>
</tr>
<tr>
<td>FH-2</td>
<td>N/A</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td>FH-3</td>
<td>N/A</td>
<td>Closed</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Residual Hydrant</th>
<th>Residual Pressure (psi)</th>
<th>Pitot Reading (psi)</th>
<th>Hydrant Discharge (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual Hydrant</td>
<td>91</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>FH-1</td>
<td>N/A</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>FH-2</td>
<td>N/A</td>
<td>Closed</td>
<td></td>
</tr>
<tr>
<td>FH-3</td>
<td>N/A</td>
<td>Closed</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Residual Hydrant</th>
<th>Residual Pressure (psi)</th>
<th>Pitot Reading (psi)</th>
<th>Hydrant Discharge (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual Hydrant</td>
<td>83</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>FH-1</td>
<td>N/A</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>FH-2</td>
<td>N/A</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td>FH-3</td>
<td>N/A</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>Residual Hydrant</td>
<td>Residual Pressure (kPa)</td>
<td>Pitot Reading (kPa)</td>
<td>Hydrant Discharge (l/s)</td>
</tr>
<tr>
<td>------------------</td>
<td>-------------------------</td>
<td>--------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>Residual Hydrant</td>
<td>607</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>FH-1</td>
<td>N/A</td>
<td>400</td>
<td></td>
</tr>
<tr>
<td>FH-2</td>
<td>N/A</td>
<td>359</td>
<td></td>
</tr>
<tr>
<td>FH-3</td>
<td>N/A</td>
<td></td>
<td>Closed</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Residual Hydrant</th>
<th>Residual Pressure (kPa)</th>
<th>Pitot Reading (kPa)</th>
<th>Hydrant Discharge (l/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual Hydrant</td>
<td>572</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>FH-1</td>
<td>N/A</td>
<td>365</td>
<td></td>
</tr>
<tr>
<td>FH-2</td>
<td>N/A</td>
<td>352</td>
<td></td>
</tr>
<tr>
<td>FH-3</td>
<td>N/A</td>
<td>331</td>
<td></td>
</tr>
</tbody>
</table>

5.4 **English Units:** A two-gage head loss test was conducted over 650 ft of 8-in. PVC pipe, as shown in the figure. The pipe was installed in 1981. The discharge from the flowed hydrant was 1,050 gpm. The data obtained from the test are presented in the following table.

<table>
<thead>
<tr>
<th>Fire Hydrant 1</th>
<th>Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fire Hydrant 1</td>
<td>500</td>
</tr>
<tr>
<td>Fire Hydrant 2</td>
<td>520</td>
</tr>
</tbody>
</table>

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) If the test results cannot be used, what is most likely causing the problem?
SI Units: A two-gage head loss test was conducted over 198 m of 203-mm PVC pipe, as shown in the figure. The pipe was installed in 1981. The discharge from the flowed hydrant was 66.2 l/s. The data obtained from the test are presented in the following table.

<table>
<thead>
<tr>
<th>Elevation (m)</th>
<th>Pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fire Hydrant 1</td>
<td>152</td>
</tr>
<tr>
<td>Fire Hydrant 2</td>
<td>158</td>
</tr>
</tbody>
</table>

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) If the test results cannot be used, what is most likely causing the problem?

5.5 English Units: A different two-gage head loss test was conducted over the same 650 ft of 8-in. PVC pipe shown in Problem 5.4. In this test, the pressure at Fire Hydrant 1 was 65 psi, and the pressure at Fire Hydrant 2 was 40 psi. The discharge through the flowed hydrant was 1,350 gpm.

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) What is the Hazen-Williams C-factor for this line?

c) How can the results of this test be used to help calibrate the water distribution system?

d) Is this a realistic roughness value for PVC?

SI Units: A different two-gage head loss test was conducted over the same 198 m of 203-mm PVC pipe shown in Problem 5.4. In this test, the pressure at Fire Hydrant 1 was 448 kPa, and the pressure at Fire Hydrant 2 was 276 kPa. The discharge through the flowed hydrant was 85.2 l/s.

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) What is the Hazen-Williams C-factor for this line?

c) How can the results of this test be used to help calibrate the water distribution system?

d) Is this a realistic roughness value for PVC?
5.6 The table below presents the results of a chlorine decay bottle test. Compute the bulk reaction rate coefficient for this water sample.

<table>
<thead>
<tr>
<th>Time (hr)</th>
<th>Concentration (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>1.4</td>
</tr>
<tr>
<td>6</td>
<td>1.2</td>
</tr>
<tr>
<td>9</td>
<td>1.0</td>
</tr>
<tr>
<td>12</td>
<td>1.0</td>
</tr>
<tr>
<td>15</td>
<td>0.9</td>
</tr>
<tr>
<td>18</td>
<td>0.7</td>
</tr>
<tr>
<td>21</td>
<td>0.7</td>
</tr>
<tr>
<td>24</td>
<td>0.6</td>
</tr>
<tr>
<td>27</td>
<td>0.5</td>
</tr>
<tr>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>33</td>
<td>0.5</td>
</tr>
<tr>
<td>36</td>
<td>0.4</td>
</tr>
<tr>
<td>39</td>
<td>0.4</td>
</tr>
<tr>
<td>42</td>
<td>0.3</td>
</tr>
<tr>
<td>45</td>
<td>0.3</td>
</tr>
<tr>
<td>48</td>
<td>0.3</td>
</tr>
<tr>
<td>51</td>
<td>0.3</td>
</tr>
<tr>
<td>54</td>
<td>0.2</td>
</tr>
<tr>
<td>57</td>
<td>0.2</td>
</tr>
<tr>
<td>60</td>
<td>0.2</td>
</tr>
</tbody>
</table>
5.7  *English Units:* Data from a pump test are presented in the following table. Fortunately, this pump had a pressure tap available on both the suction and discharge sides. The diameter of the suction line is 12 in. and the diameter of the discharge line is 8 in. Plot the pump head-discharge curve for this unit.

<table>
<thead>
<tr>
<th>Suction Pressure (psi)</th>
<th>Discharge Pressure (psi)</th>
<th>Pump Discharge (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.5</td>
<td>117</td>
<td>0</td>
</tr>
<tr>
<td>10.1</td>
<td>116</td>
<td>260</td>
</tr>
<tr>
<td>9.3</td>
<td>114</td>
<td>500</td>
</tr>
<tr>
<td>8.7</td>
<td>111</td>
<td>725</td>
</tr>
<tr>
<td>7.2</td>
<td>101</td>
<td>1,250</td>
</tr>
<tr>
<td>5.7</td>
<td>93</td>
<td>1,500</td>
</tr>
<tr>
<td>4.4</td>
<td>85</td>
<td>1,725</td>
</tr>
<tr>
<td>3.0</td>
<td>76</td>
<td>2,000</td>
</tr>
<tr>
<td>1.6</td>
<td>65</td>
<td>2,300</td>
</tr>
<tr>
<td>-0.2</td>
<td>53</td>
<td>2,500</td>
</tr>
<tr>
<td>-2.0</td>
<td>41</td>
<td>2,700</td>
</tr>
</tbody>
</table>

*SI Units:* Data from a pump test are presented in the following table. Fortunately, this pump had a pressure tap available on both the suction and discharge sides. The diameter of the suction line is 300 mm, and the diameter of the discharge line is 200 mm. Plot the pump head-discharge curve for this unit.

<table>
<thead>
<tr>
<th>Suction Pressure (kPa)</th>
<th>Discharge Pressure (kPa)</th>
<th>Pump Discharge (l/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>72.4</td>
<td>803</td>
<td>0</td>
</tr>
<tr>
<td>69.6</td>
<td>798</td>
<td>16.4</td>
</tr>
<tr>
<td>64.1</td>
<td>784</td>
<td>31.5</td>
</tr>
<tr>
<td>60.0</td>
<td>764</td>
<td>45.7</td>
</tr>
<tr>
<td>49.6</td>
<td>694</td>
<td>78.9</td>
</tr>
<tr>
<td>39.3</td>
<td>644</td>
<td>94.6</td>
</tr>
<tr>
<td>30.3</td>
<td>586</td>
<td>108.8</td>
</tr>
<tr>
<td>20.7</td>
<td>522</td>
<td>126.2</td>
</tr>
<tr>
<td>11.0</td>
<td>451</td>
<td>145.1</td>
</tr>
<tr>
<td>-1.4</td>
<td>366</td>
<td>157.7</td>
</tr>
<tr>
<td>-13.8</td>
<td>283</td>
<td>170.3</td>
</tr>
</tbody>
</table>

5.8  A C-factor test is conducted in a 350 ft length of 12-in. pipe. The upstream pressure gage is at elevation 520 ft, and the downstream gage is at 524 ft.

a) The Hazen-Williams equation can be rearranged to solve for $C$ as

$$C = K Q h_L^{0.54}$$

where $C = $ Hazen-Williams roughness coefficient, $K = $ constant, $\bar{Q} = $ flow (gpm), $h_L = $ head loss due to friction (ft)

What is the expression for $K$ if length ($L$) is in feet and diameter ($D$) is in inches? All of the terms in $K$ are constant for this problem, so determine the numerical value for $K$. 

b) What is the expression for head loss between the upstream and downstream pressure gages if the head loss (h) and elevations (z₁ and z₂) are in feet, and the pressures (P₁ and P₂) are in psi?

c) The elevations are surveyed to the nearest 0.01 ft and the pressure gage is accurate to +/- 1 psi. Opening a downstream hydrant resulted in a flow of 800 gpm (accurate to +/- 50 gpm) with a measured upstream pressure of 60 psi and a measured downstream pressure of 57 psi. Determine the possible range of actual Hazen-Williams C-factors and fill in the following table.

*Hint:* For the roughest possible C-factor, use 800 – 50 gpm for flow and h + 5 ft for head loss. For the smoothest possible C-factor, use 800 + 50 gpm for flow and h – 5 ft for head loss.

<table>
<thead>
<tr>
<th></th>
<th>Roughest Possible C</th>
<th>Smoothest Possible C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q (gpm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>h (ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

d) What can you conclude about the C-factor from this test?

e) Which measurement contributed more to the error in this problem, head loss or flow?

f) What could you do to improve the results if you ran the test over again?

5.9 *English Units:* A hydrant flow test was performed on a main line where a new industrial park is to tie in. The following hydrant flow test values were obtained from a 2 ½-in. nozzle in the field. First, use Equation 5.1 to determine the hydrant discharge for a discharge coefficient of 0.90.

Determine if the existing system is able to handle 1,200 gpm of fire flow demand for the new industrial park by using the equation given in the sidebar on page 189 entitled *Evaluating Distribution Capacity with Hydrant Tests.*

<table>
<thead>
<tr>
<th>Fire Hydrant Number</th>
<th>Static Pressure</th>
<th>Residual Pressure</th>
<th>Pitot Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>48 psi</td>
<td>33 psi</td>
<td>12 psi</td>
</tr>
</tbody>
</table>

5.10 *SI Units:* A hydrant flow test was performed on a main line where a new industrial park is to tie in. The following hydrant flow test values were obtained from 64-mm nozzle in the field. First, use Equation 5.1 to determine the hydrant discharge for a discharge coefficient of 0.90.

Determine if the existing system is able to handle 75.7 l/s of fire flow demand for the new industrial park by using the equation given in the sidebar on page 189 entitled *Evaluating Distribution Capacity with Hydrant Tests.*

<table>
<thead>
<tr>
<th>Fire Hydrant Number</th>
<th>Static Pressure</th>
<th>Residual Pressure</th>
<th>Pitot Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>331 kPa</td>
<td>227.5 kPa</td>
<td>82.7 kPa</td>
</tr>
</tbody>
</table>

5.10 A utility performed a C-factor test on a pipe with a nominal diameter of 8 in. and calculated the C-factor as 40. Later, tests showed that the true diameter was 6 in. due to severe tuberculation. Using the correct diameter, what would the corrected C-factor be? If the flow in the pipe is 200 gpm (0.446 cfs), what would the velocity be using the 8-in. nominal diameter and the 6-in. actual diameter?
5.11 A chlorine field test is conducted to estimate the wall demand for a 6-in. diameter (actual diameter) 1500-ft length of pipe. The flow rate during the test is 300 gpm. The chlorine bulk decay rate was determined to be –0.2/day based on a bottle test. Chlorine residual at the upstream and downstream ends of the segment during the test was measured as 0.80 mg/l and 0.55 mg/l respectively. Calculate the following values: velocity, travel time, chlorine loss due solely to bulk demand, and chlorine loss due to wall demand (that is, the difference between observed chlorine loss and loss due to bulk decay). Then set up a model of this link as shown in the figure and iteratively run the model to find the wall demand coefficient that results in the observed chlorine loss. Assume a water temperature of 15°C.