Calibration of Hydraulic Network Models

By Lindell E. Ormsbee and Srinivasa Lingireddy (1997)

Ormsbee, L, and Lingireddy, S., "Calibrating Hydraulic Network Models," *Journal of the American Water Works Association**, Vol 89, No. 2, February 1997, pp 42-50**.**

1.0 Introduction

Computer models for analyzing and designing water distribution systems have been available since the mid 1960's. Since then, however, many advances have been made with regard to the sophistication and application of this technology. A primary reason for the growth and use of computer models has been the availability and widespread use of the microcomputer. With the advent of this technology it has been possible for water utilities and engineers to analyze the status and operations of the existing system as well as to investigate the impacts of proposed changes (Ormsbee and Chase, 1988). The validity of these models, however, depends largely on the accuracy of the input data.

1.1 Network Characterization

Before an actual water distribution system may be modeled or simulated with a computer program, the physical system must be represented in a form that can be analyzed by a computer. This will normally require that the water distribution system first be represented by using node-link characterization (see Figure 1). In this case the links represent individual pipe sections and the nodes represent points in the system where two or more pipes (links) join together or where water is being input or withdrawn from the system.

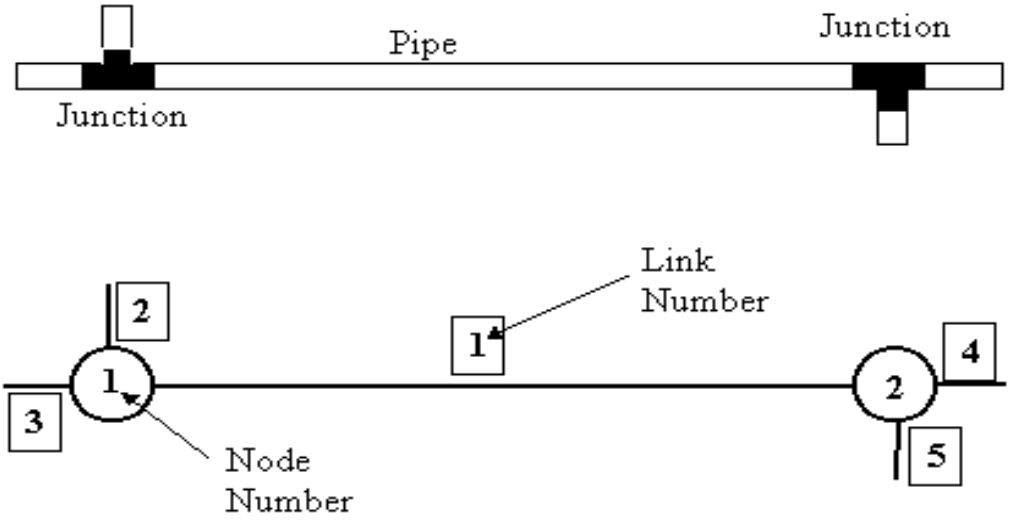


Figure 1. Node-Link Characterization

1.2 Network Data Requirements

Data associated with each link will include a pipe identification number, pipe length, pipe diameter, and pipe roughness. Data associated with each junction node will include a junction identification number, junction elevation, and junction demand. Although it is recognized that water leaves the system in a time varying fashion through various service connections along the length of a pipe segment, it is generally acceptable in modeling to lump half of the demands along a line to the upstream node and the other half of the demands to the downstream node as shown in Figure 2.

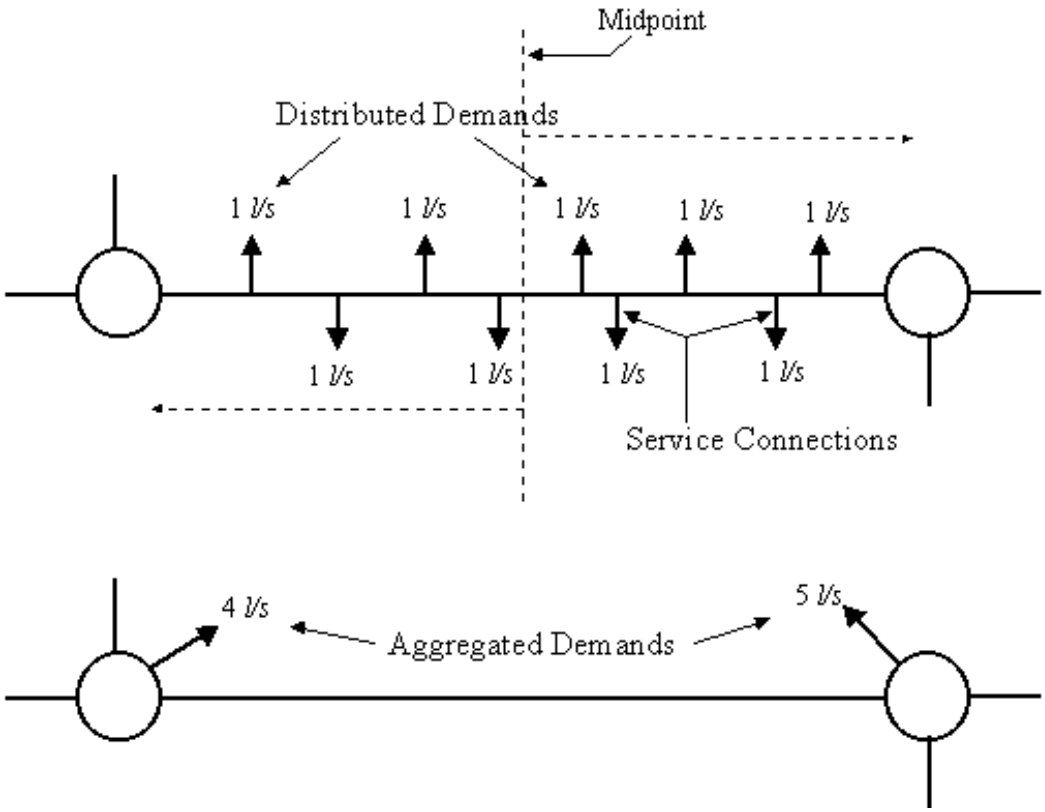


Figure 2. Demand Load Simplification

In addition to the network pipe and node data, physical data for use in describing all tanks, reservoirs, pumps, and valves must also be obtained. Physical data for all tanks and reservoirs will normally include information on tank geometry as well as the initial water levels. Physical data for all pumps will normally include either the value of the average useful horsepower, or data for use in describing the pump flow/head characteristics curve. Once this necessary data for the network model has been obtained, the data should be entered into the computer in a format compatible with the selected computer model.

1.3 Model Parameters

Once the data for the computer network model has been assembled and encoded, the associated model parameters should then be determined prior to actual model application. In general, the primary parameters associated with a hydraulic network model will include pipe roughness and nodal demands. Due to the difficulty of obtaining economic and reliable measurements of both parameters, final model values are normally determined through the process of model calibration. Model calibration involves the adjustment of the primary network model parameters (i.e. pipe roughness coefficients and nodal demands) until the model results closely approximate actual observed conditions as measured from field data. In general, a network model calibration effort should encompass seven basic steps (see Figure 3). Each of these steps is discussed in detail in the following sections.

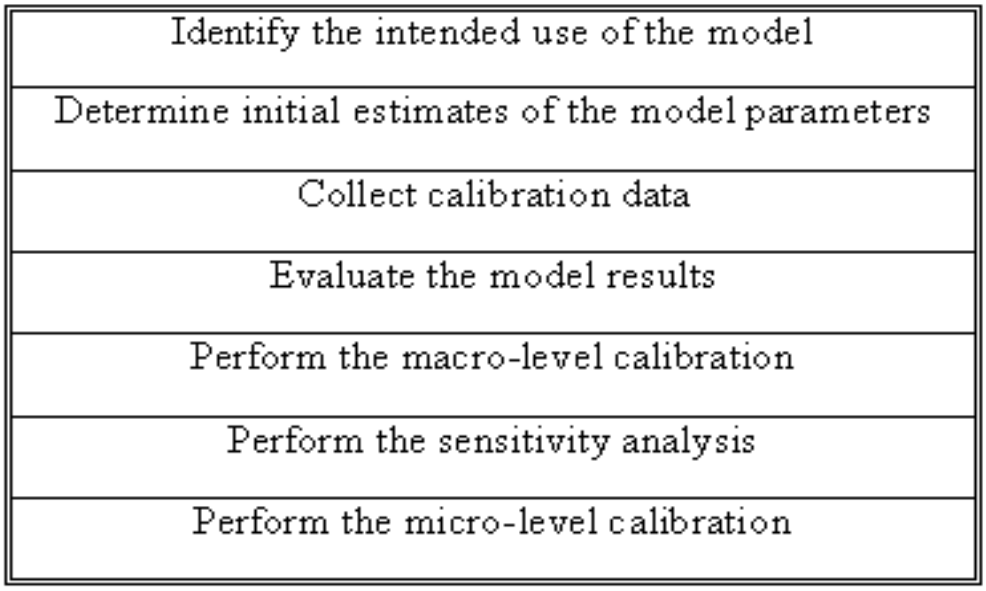


Figure 3. Seven Basic Steps for Network Model Calibration

2.0 Identify the Intended Use of the Model

Before calibrating a hydraulic network model, it is important to first identify its intended use (e.g., pipe sizing for master planning, operational studies, design projects, rehabilitation studies, water quality studies) and the associated type of hydraulic analysis (steady-state versus extended-period). Usually the type of analysis is directly related to the intended use. For example, water quality and operational studies require an extended-period analysis, whereas some planning or design studies may be performed using a steady state analysis (Walski, 1995). In the latter, the model predicts system pressures and flows at an instant in time under a specific set of operating conditions and demands (e.g., average or maximum daily demands). This is analogous to photographing the system at a specific point in time. In extended-period analysis, the model

predicts system pressures and flows over an extended period (typically 24 hours). This is analogous to developing a movie of the system performance.

Both the intended use of the model and the associated type of analysis provide some guidance about the type and quality of collected field data and the desired level of agreement between observed and predicted flows and pressures (Walski, 1995). Models for steady-state applications can be calibrated using multiple static flow and pressure observations collected at different times of day under varying operating conditions. On the other hand, models for extended-period applications require field data collected over an extended period (e.g., one to seven days).

In general, a higher level of model calibration is required for water quality analysis or an operational study than for a general planning study. For example, determining ground evaluations using a topographic map may be adequate for one type of study, whereas another type of study may require an actual field survey. This may depend on the contour interval of the map used. Such considerations obviously influence the methods used to collect the necessary model data and the subsequent calibration steps. For example, if one is working in a fairly steep terrain (e.g. greater than 20 foot contour intervals), one may decide to use a GPS unit for determining key elevations rather than simply interpolating between contours.

3.0 Determining Model Parameter Estimates

The second step in calibrating a hydraulic network model is to determine initial estimates of the primary model parameters. Although most models will have some degree of uncertainty associated with several model parameters, the two model parameters that normally have the greatest degree of uncertainty are the pipe roughness coefficients and the demands to be assigned to each junction node.

3.1 Pipe Roughness Values

Initial estimates of pipe roughness values may be obtained using average literature values or directly from field measurements. Various researchers and pipe manufacturers have developed tables that provide estimates of pipe roughness as a function of various pipe characteristics such as pipe material, pipe diameter, and pipe age (Lamont, 1981). One such typical table is shown in Table 1 (Wood, 1991). Although such tables may be useful for new pipes, their specific applicability to older pipes decreases significantly as the pipes age. This may be due to the effects of such things as tuberculation, water chemistry, etc. As a result, initial estimates of pipe roughness for all pipes other than relatively new pipes should normally come directly from field testing. Even when new pipes are being used it is helpful to verify the roughness values in the field since the roughness coefficient used in the model may actually represent a composite of several secondary factors such as fitting losses and system skeletonization.

Table 1. Typical Hazen-William Pipe Roughness Factors

Pipe Material	Age (years)	Diameter	C Factor
Cast Iron	New	All Sizes	130
	5	>380mm (15in)	120
		>100mm (4in)	118
	10	>600mm (24in)	113
		>300mm (12in)	111
		>100mm (4in)	107
	20	>600mm (24in)	100
		>300mm (12in)	96
		>100mm (4in)	89
	30	>760mm (30in)	90
		>400mm (16in)	87
		>100mm (4in)	75
	40	>760mm (30in)	83
		>400mm (16in)	80
		>100mm (4in)	64
Ductile Iron	New		140
Plastic PVC	Average		140
Asbestos Cement	Average		140
Wood Stave	Average		120

3.1.1 Pipe Roughness Chart

A customized roughness nomograph for a particular water distribution system may be developed using the process illustrated in Figure 4. To obtain initial estimates of pipe roughness through field testing, it is best to divide the water distribution system into homogeneous zones based on the age and material of the associated pipes (see Figure.4a). Next, several pipes of different diameters should be tested in each zone to obtain individual pipe roughness estimates (see Figure 4b). Once a customized roughness nomograph is constructed, (see Figure 4c), it can be used to assign values of pipe roughness for the rest of the pipes in the system.

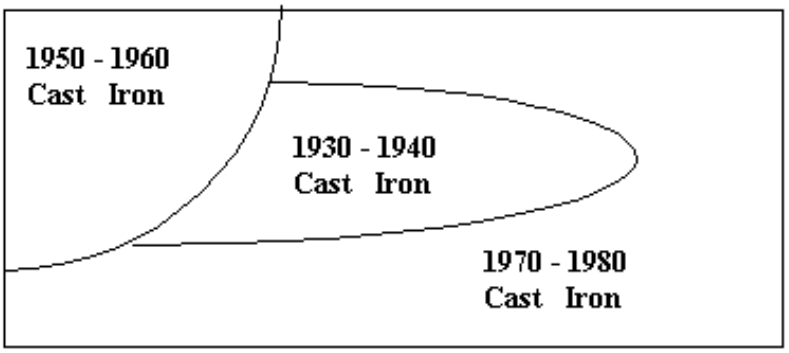


Figure 4a. Subdivide Network into Homogeneous Zones of Like Age and Material

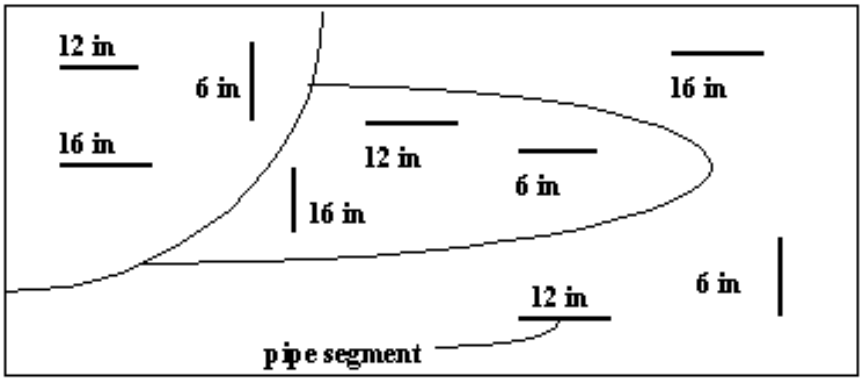


Figure 4b. Selected Representative Pipes from Each Zone

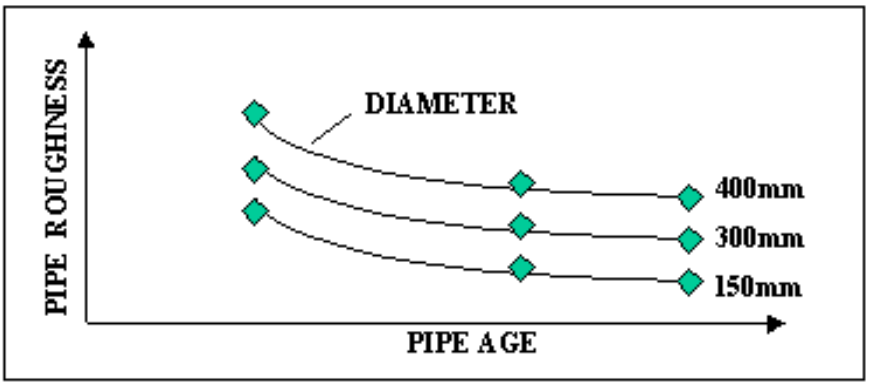


Figure 4c. Plot Associated Roughness as a Function of Pipe Diameter and Age

3.1.2 Pipe Roughness Field Estimation

Pipe roughness values may be estimated in the field by selecting a straight section of pipe that contains a minimum of three fire hydrants (see Figure 5a). When the line has been selected, pipe roughness may be estimated using one of two methods (Walski, 1984): 1) The parallel-pipe method (see Figure 5b) or 2) The two-hydrant method (see Figure 5c). In each method, the length and diameter of the test pipe are first determined. Next, the test pipe is isolated, and the flow and pressure drop are measured either through the use of a differential pressure gauge or by using two separate pressure gauges. Pipe roughness can then be approximated by a direct application of either the Hazen-Williams equation or the Darcy-Weisbach equation. In general, the parallel-pipe method is preferable for short runs and for determining minor losses around valves and fittings. For long runs of pipe, the two-gage method is generally preferred. Also, if the water in the parallel pipe heats up or if a small leak occurs in the parallel line, it can lead to errors in the associated head loss measurements (Walski, 1985).

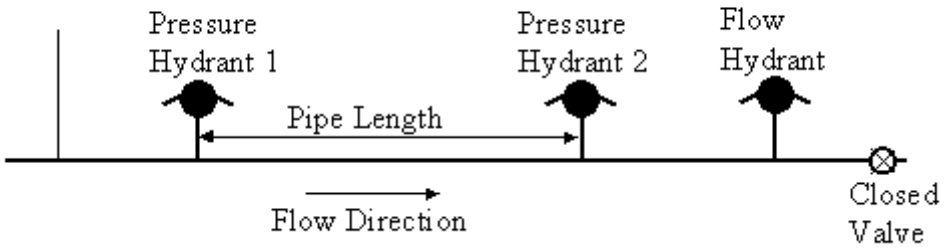


Figure 5a. Pipe Roughness Test Configuration

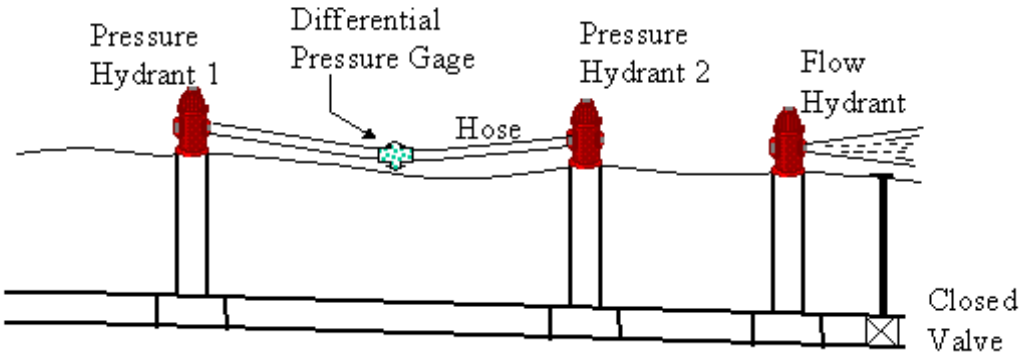


Figure 5b. Parallel Pipe Method

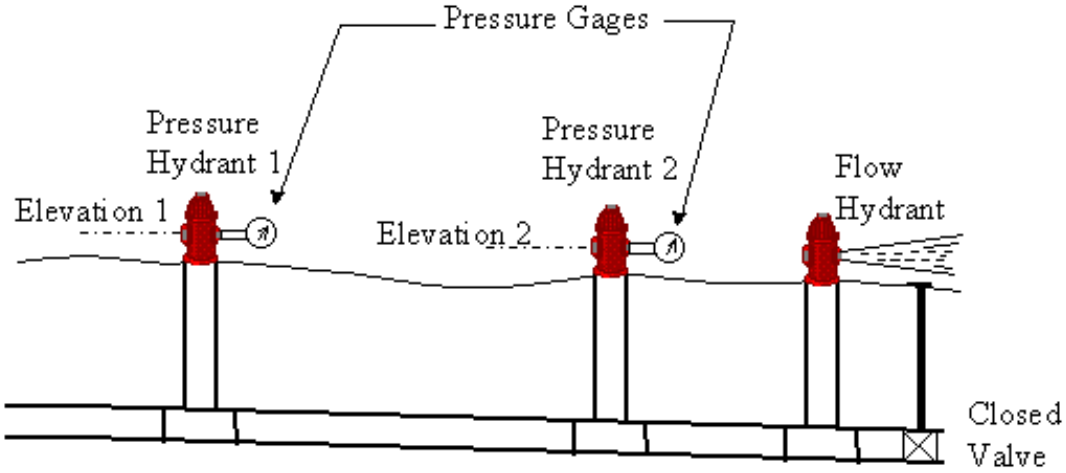


Figure 5c. Two Gage Method

3.1.2.1 The Parallel-Pipe Method

The steps involved in the application of the parallel pipe method are summarized as follows:

1) Measure the length of pipe between the two upstream hydrants (Lp) in meters.

2) Determine the diameter of the pipe (Dp) in mm. In general this should simply be the nominal diameter of the pipe. It is recognized that the actual diameter may differ from this diameter due to variations in wall thickness or the buildup of tuberculation in the pipe. However, the normal calibration practice is to incorporate the influences of variations in pipe diameter via the roughness coefficient. It should be recognized however, that although such an approach should not significantly influence the distribution of flow or headloss throughout the system it may have a significant influence on pipe velocity, which in turn could influence the results of a water quality analysis.

3) Connect the two upstream hydrants with a pair of parallel pipes, (typically a pair of fire hoses) with a differential pressure device located in between (see Figure 5b). The differential pressure device can be a differential pressure gage, an electronic transducer or a manometer. Walski (1984) recommends the use of an air filled manometer due to its simplicity, reliability, durability and low cost. (Note: When connecting the two hoses to the differential pressure device, make certain that there is no flow through the hoses. If there is any leak in the hoses the computed headloss for the pipe will be in error by an amount equal to the headloss through the hose).

4) Open both hydrants and check all connections to insure there are no leaks in the configuration.

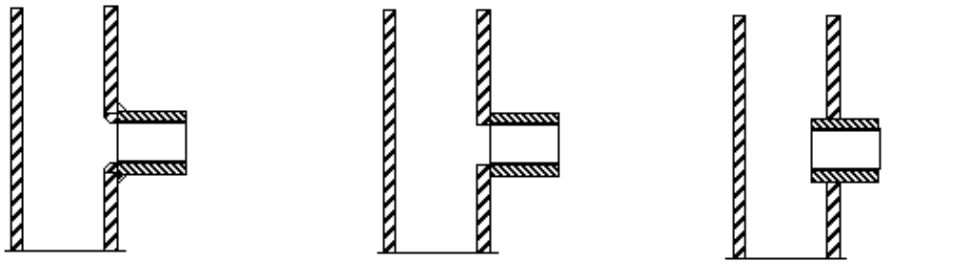
5) Close the valve downstream of the last hydrant and then open the smaller nozzle on the flow hydrant to generate a constant flow through the isolated section of pipe. Make sure the discharge has reached equilibrium condition before taking flow and pressure measurements.

6) Determine the discharge Qp (l/s) from the smaller nozzle in the downstream hydrant. This is normally accomplished by measuring the discharge pressure Pd of the stream leaving the hydrant nozzle using either a hand held or nozzle mounted pitot. Once the discharge pressure Pd (in kPa) is determined it can be converted to discharge (Qp) using following relationship:

$$Q_p = C_d D_n^2 P_d^{0.5} / 900.3$$

..... eq..1

where Dn is the nozzle diameter in mm and Cd is the nozzle discharge coefficient which is a function of the type of nozzle (see Figure 6). (Note: When working with larger mains, sometimes you can't get enough water out of the smaller nozzles to get a good pressure drop. In such cases you may need to use the larger nozzle).



Outlet Smooth and Rounded. Coefficient: 0.90

Outlet Square and Sharp. Coefficient: 0.80

Outlet Square and Projecting into Barrel. Coefficient: 0.70

Figure.6. Hydrant Nozzle Discharge Coefficients

7) After calculating the discharge, determine the in-line flow velocity Vp (m/s) where:

$$V_p = Q_p / (\pi D_p^2 / 4)^2$$

..... eq..2

8) After the flow through the hydrant has been determined, measure the pressure drop (p) through the isolated section of pipe by reading the differential pressure gage. Convert the measured pressure drop in units of meters (Hp) and divided by the pipe length Lp to yield the hydraulic gradient or friction slope Sp.

$$S_p = H_p / L_p$$

..... eq..3

9a) Once these four measured quantities have been obtained, the Hazen-Williams Roughness Factor (Cp) can then be determined using the Hazen-Williams equation as follows:

$$C_p = \frac{218 V_p}{D_p^{0.63} S_p^{0.54}}$$

..... eq..4

9b) To calculate the actual pipe roughness , it is first necessary to calculate the friction factor f using the Darcy-Weisbach equation as follows (Walski, 1984):

$$f = \frac{g S_p D_p}{500 V_p^2}$$

..... eq..5

where g = gravitational acceleration constant (9.81m/sec2)

Once the friction factor has been calculated, the Reynolds number must be determined. Assuming a standard water temperature of 20ºC (68º F), the Reynolds number is:

$$R = 993 V_p D_p$$

..... eq..6

Once the friction factor f, and the Reynolds number R have been determined, they can be inserted into the Colebrook-White formula to give the pipe roughness E (mm) as:

$$\varepsilon = 3.7 D_p \left\{ \exp(-1.16\sqrt{f}) - {}^{2.51}\!/_{R\sqrt{f}} \right\}$$

..... eq..7

3.1.2.2. The Two-hydrant Method

The two hydrant method is basically identical to the parallel pipe method with the exception that the pressure drop across the pipe is measured using a pair of static pressure gages as shown in Figure 5c. In this case the total headloss through the pipe is the difference between the hydraulic grades at both hydrants. In order to obtain the hydraulic grade at each hydrant, the observed pressure head (m) must be added to the elevation of the reference point (the hydrant nozzle). For the two hydrant method, the head loss through the test section Hp (m) can be calculated using the following equation:

$$H_p = {}^{(P_2 - P_1)}\!/_{9.81} + (Z_2 - Z_1)$$

..... eq..8

where P1 is the pressure reading at the upstream gage (kPa) , Z1 is the elevation of the upstream gage (m), P2 is the pressure reading at the downstream gage (kPa), and Z2 is the elevation of the downstream gage (m).

The elevation difference between the two gages should generally be determined using a transit or a level. As a result, one should make sure to select two upstream hydrants that can be seen from a common point. This will minimize the number of turning points required in determining the elevation differences between the nozzles of the two hydrants. As an alternative to the use of a differential survey, topographic maps can sometimes be used to obtain estimates of hydrant elevations. However, topographic maps should not generally be used to estimate the elevation differences unless the contour interval is 1m or less. One hydraulic alternative to measuring the elevations directly is to simply measure the static pressure readings at both hydrants before the test and convert the observed pressure difference to the associated elevation difference (e.g. Z1 - Z2 = 2.31*[P2(static) - P1(static)]).

3.1.2.3. General Observations and Suggestions

Hydrant pressures for use in pipe roughness tests are normally measured with a Bourdon tube gage which can be mounted to one of the discharge nozzles of the hydrant using a lightweight

hydrant cap. Bourdon tube gages come in various grades (i.e 2A, A, and B) depending upon their relative measurement error. In most cases a grade A gage (1 percent error) is sufficient for fire flow tests. For maximum accuracy one should chose a gage graded in 5kPa (1 psi) increments with a maximum reading less than 20% above the expected maximum pressure (McEnroe, et al., 1989). In addition, it is a good idea to use pressure snubbers in order to eliminate the transient effects in the pressure gages. A pressure snubber is a small valve that is placed between the pressure gage and the hydrant cap which acts as a surge inhibitor (Walski, 1984).

Before conducting a pipe roughness test, it is always a good idea to make a visual survey of the test area. When surveying the area, make sure that there is adequate drainage away from the flow hydrant. In addition, make sure you select a hydrant nozzle that will not discharge into oncoming traffic. Also, when working with hydrants that are in close proximity to traffic, it is a good idea to put up traffic signs and use traffic cones to provide a measure of safety during the test. As a further safety precaution, make sure all personnel are wearing highly visible clothing. It is also a good idea to equip testing personnel with radios or walkie-talkies to help coordinate the test.

While the methods outlined previously work fairly well with smaller lines (i.e. less than 16 inches in diameter), their efficiency decreases as you deal with larger lines. Normally, opening hydrants just doesn't generate enough flow for meaningful head-loss determination. For such larger lines you typically have to conduct the headloss tests over very much longer runs of pipe and use either plant or pump station flow meters or change in tank level to determine flow (Walski, 1999).

3.2 Nodal Demand Distribution

The second major parameter determined in calibration analysis is the average (steady-state analysis) or temporally varying (extended-period analysis) demand to be assigned to each junction node. Initial average estimates of nodal demands can be obtained by identifying a region of influence associated with each junction node, identifying the types of demand units in the service area, and multiplying the number of each type by an associated demand factor. Alternatively, the estimate can be obtained by first identifying the area associated with each type of land use in the service area and then multiplying the area of each type by an associated demand factor. In either case, the sum of these products will provide an estimate of the demand at the junction node.

3.2.1 Spatial Distribution of Demands

Initial estimates of nodal demands can be developed using various approaches depending on the nature of the data each utility has on file and how precise they want to be. One way to determine such demands is by employing the following strategy.

1. First, determine the total system demand for the day to be used in model calibration (i.e. TD). The total system demand may be obtained by performing a mass balance analysis for the system by determining the net difference between the total volume of flow which enters the system

(from both pumping stations and tanks) and the total volume that leaves the system (through pressure reducing valves (PRV) and tanks).

2. Second, using meter records for the day, try to assign all major metered demands (i.e. MDj where j = junction node number) by distributing the observed demands among the various junction nodes which serve the metered area. The remaining demand will be defined as the total residual demand (i.e. TRD) and may be obtained by subtracting the sum of the metered demands from the total system demand:

$$TRD = TD - \sum MD_j$$

..... eq.9

3. Third, determine the demand service area associated with each junction node. The most common method of influence delineation is to simply bisect each pipe connected to the reference node (see Figure 7a)..

4. Once the service areas associated with the remaining junction nodes have been determined, an initial estimate of the demand at each node should be made. This can be accomplished by first identifying the number of different types of demand units within the service area and then multiplying the number of each type by an associated demand factor (see Figure 7b). Alternatively, the estimate can be obtained by first identifying the area associated with each different type of land use within the service area and then multiplying the area of each type by an associated unit area demand factor (see Figure 7c). In either case, the sum of these products will represent an estimate of the demand at the junction node. While in theory the first approach should be more accurate the later approach can be expected to be more expedient. Estimates of unit demand factors are normally available from various water resource handbooks (Cesario, 1995). Estimates of unit area demand factors can normally be constructed for different land use categories by weighted results from repeated applications of the unit demand approach.

5. Once an initial estimate of the demand has been obtained for each junction node j (i.e. IEDj), a revised estimated demand (i.e. REDj) may be obtained using the following equation:

$$RED_j = IED_j * TRD / \quad IED_i$$

..... eq..10

6. Once the revised demands have been obtained for each junction node, the final estimate of nodal demand can be obtained by adding together both the revised demand and the metered demand (assuming there is one) associated with each junction node:

$$D_j = NED_j + MD_j$$

..... eq..11

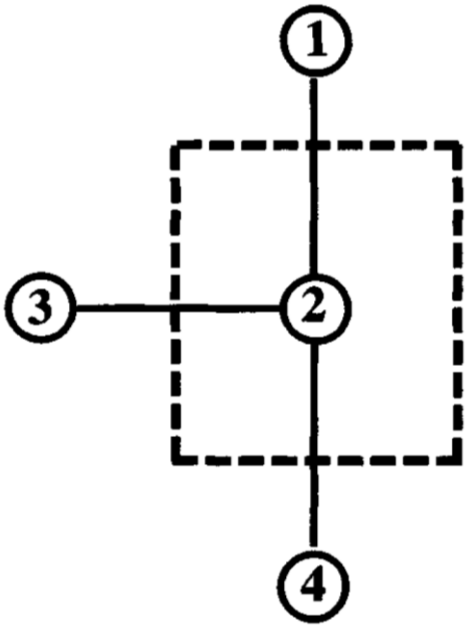
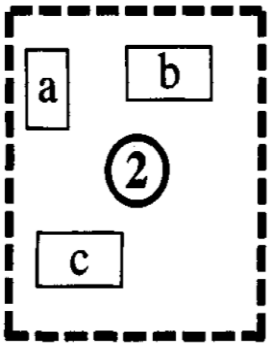
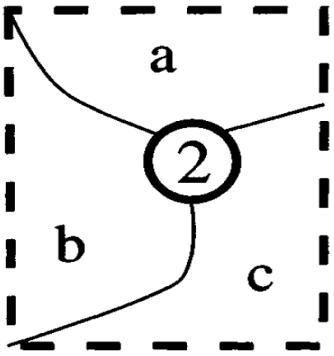


Figure 7a. Delineation of region of influence for node 2



Type of Establishment	Units	Average Annual Demand (gpd/unit)	Maximum Daily Demand (gpd/unit)
a. metered residential	gpcd	70.00	140.00
b. garden apartment	gpd/unit	213.00	272.00
c. car wash	gpd/ft^2	4.78	10.3

Figure 7b. Demand assignment using individual units



Type of Land Use	Unit Demand	Area	Total Demand
	(gpd/acre)	(acres)	(gpd)
a. metered residential	700	5	3500
b. garden apartament	600	4	2400
c. car wash	160,000	1	160000

Figure 7c. Demand assignment using land use units

3.2.2 Temporal Distribution of Demands

Time-varying estimates of model demands for use in extended-period analysis can be made in one of two ways, depending on the structure of the hydraulic model. Some models allow the user to sub-divide the demands at each junction node into different use categories, which can then be modified separately over time using demand factors for water use categories. Other models require an aggregate-use category for each node. In the latter case, spatial-temporal variations of nodal demands are obtained by lumping nodes of a given type into separate groups, which can then be modified uniformly using nodal demand factors. Initial estimates of either water use category demand factors or nodal demand factors can be obtained by examining historical meter records for various water use categories and by performing incremental mass balance calculations for the distribution system. The resulting set of temporal demand factors can then be fine tuned through subsequent model calibration.

4.0 Collect Calibration Data

After model parameters have been estimated, the accuracy of the model parameters can be assessed. This is done by executing the computer model using the estimated parameter values and observed boundary conditions and then comparing the model results with the results from actual field observations. Data from fire flow tests, pump station flowmeter readings, and tank telemetry data are most commonly used in such tests.

In collecting data for model calibration, it is very important to recognize the significant impact of measurement errors. For example, with regard to calibrating pipe roughness, the C factor may be expressed as:

$$C = k(V + \text{error})/(h + \text{error})^{0.54}$$

..... eq. 12

If the magnitude of V and h are on the same order of magnitude as the associated measurement errors (for V and h) then the collected data will be essentially useless for model calibration. That is to say, virtually any value of C will provide a "reasonable" degree of model calibration (Walski, 1986). However, one can hardly expect a model to accurately predict flows and pressures for a high stress situation (i.e. large flows and velocities) if the model was calibrated using data from times when the velocities in the pipes were less than the measurement error (e.g. less than 1 ft/s). The only way to minimize this problem is to either insure that the measurement errors are reduced or the velocity or headloss values are significantly greater than the associated measurement error. This latter condition can normally be met either using data from fire flow tests or by collecting flow or pressure readings during periods of high stress (e.g. peak hour demand periods).

4.1 Fire Flow Tests

Fire flow tests are useful for collecting both discharge and pressure data for use in calibrating hydraulic network models. Such tests are normally conducted using both a normal pressure gage (for measuring both static and dynamic heads) and a pitot gage (for use in calculating discharge). In performing a fire flow test, at least two separate hydrants are first selected for use in the data collection effort. One hydrant is identified as the pressure or residual hydrant while the remaining hydrant is identified as the flow hydrant. The general steps for performing a fire flow test may be summarized as follows (McEnroe, et al., 1989):

1. Place a pressure gage on the residual hydrant and measure the static pressure.

2. Determine which of the discharge hydrant's outlets can be flowed with the least amount of adverse impact (flooding, traffic disruption, etc.)

3. Make sure the discharge hydrant is initially closed in order to avoid injury.

4. Remove the hydrant cap from the nozzle of the discharge hydrant to be flowed.

5. Measure the inside diameter of the nozzle and determine the type of nozzle (i.e. rounded, square edge, or protruding) in order to determine the appropriate discharge coefficient. (see Figure A.6).

6. Take the necessary steps to minimize erosion or traffic impacts during the test.

7. Flow the hydrant briefly to flush sediment from the hydrant lateral and barrel.

8a. If using a clamp on pitot tube, attach the tube to the nozzle to be flowed and then slowly open the hydrant.

8b. If using a hand held pitot tube, slowly open the hydrant and then place the pitot in the center of the discharge stream being careful to align it directly into the flow.

9. Once an equilibrium flow condition has been established, make simultaneous pressure readings from both the pitot and the pressure gage at the residual hydrant.

10. Once the readings are completed, close the discharge hydrant, remove the equipment from both hydrants and replace the hydrant caps.

In order to obtain sufficient data for an adequate model calibration it is important that data from several fire flow tests be collected. Before conducting each test, it is also important that the associated system boundary condition data be collected. This includes information on tank levels, pump status, etc. In order to obtain adequate model calibration it is normally desirable that the difference between the static and dynamic pressure readings as measured from the residual hydrant be at least 35kPa (5psi) with a preferable drop of 140kpa (20psi) (Walski, 1990a). In the event that the discharge hydrant does not allow sufficient discharge to cause such a drop it may be necessary to identify, instrument, and open additional discharge hydrants. In some instances, it may also be beneficial to use more than one residual hydrant (one near the flowed hydrant and one off the major main from the source). The information gathered from such additional hydrants can sometimes be very useful in tracking down closed valves (Walski, 1999).

4.2. Telemetry Data

In addition to static test data, data collected over an extended period of time (typically 24 hours) can be very useful for use in calibrating network models. The most common type of data will include flowrate data, tank water level data, and pressure data. Depending upon the level of instrumentation and telemetry associated with the system, much of the data may be already collected as part of the normal operations. For example, most systems collect and record tank levels and average pump station discharges on an hourly basis. These data are especially useful verifying the distribution of demands among the various junction nodes. If such data are available, the data should first be checked for accuracy before use in the calibration effort. If such data are not readily available, the modeler may have to install temporary pressure gages or flowmeters in order to obtain the data. In the absence of flowmeters in lines to tanks, inflow or discharge flow rates can be inferred from incremental readings of the tank level.

4.3 Water Quality Data

In recent years, both conservative and non-conservative constituents have been used as tracers to determine the travel time through various parts of a water distribution system (Grayman, 1998, Cesario, A. L., et al., 1996, Kennedy, et. al., 1991). The most common type of tracer for such applications is fluoride. By controlling the injection rate at a source, typically the water treatment plant, a pulse can be induced into the flow that can then be monitored elsewhere in the system.

The relative travel time from the source to the sampling point can be determined. The measured travel time thus provides another data point for use in calibrating a hydraulic network model.

Alternatively, the water distribution system can also be modeled using a water quality model such as EPANET (Rosman, 1994). In this case the water quality model is used to predict tracer concentrations at various points in the system. Since all water quality models results depend on the underlying hydraulic results, deviations between the observed and predicted concentrations can thus provide a secondary means of evaluating the adequacy of the underlying hydraulic model.

5.0 Evaluate Model Results

In using fire flow data, the model is used to simulate the discharge from one or more fire hydrants by assigning the observed hydrant flows as nodal demands within the model. The flows and pressures predicted by the model are then compared with the corresponding observed values in an attempt to assess model accuracy. In using telemetry data, the model is used to simulate the variation of tank water levels and system pressures by simulating the operating conditions for the day over which the field data was collected. The predicted tank water levels are then compared with the observed values in an attempt to assess model accuracy. In using water quality data, the travel times (or constituent concentrations) are compared with model predictions in an attempt to assess model accuracy.

Model accuracy may be evaluated using various criteria. The most common criteria are absolute pressure difference (normally measured in psi) or relative pressure difference (measured as the ratio of the absolute pressure difference to the average pressure difference across the system). In most cases a relative pressure difference criteria is normally to be preferred. For extended period simulations, comparisons are normally made between the predicted and observed tank water levels. To a certain extent, the desired level of model calibration will be related to the intended use of the model. For example, a higher level of model calibration will normally be required for water quality analysis or an operational study as opposed to use of the model in a general planning study. Ultimately, the model should be calibrated to the extent that the associated application decisions will not be significantly affected. In the context of a design application, the model should normally be calibrated to such an extent that the resulting design values (e.g. pipe diameters, tank and pump sizes and/or locations, etc) will be the same as if the exact parameter values were used. Determination of such thresholds will frequently require the application of model sensitivity analysis (Walski, 1995).

Because of the issue of model application, it is difficult to derive a single set of criteria for a universal model calibration. From the authors' perspective, a maximum state variable (i.e. pressure grade, water level, flowrate) deviation of less than 10 percent will generally be satisfactory for most planning applications while a maximum deviation of less than 5 percent to be highly desirable for most design, operation, or water quality applications. Although no such general set of criteria have been officially developed for the United States, a set of "Performance

Criteria" have been developed by the Sewers and Water Mains Committee of the Water Authorities in the United Kingdom (1989). For steady state models the criteria are:

1. Flows agree to:
    a. 5% of measured flow when flows are more than 10% of total demand.
    b. 10% of measured flow when flows are less than 10% of total demand.
2. Pressures agree to:
    a. 0.5 m (1.6ft) or 5% of headloss for 85% of test measurements.
    b. 0.75 m (2.31 ft) or 7.5% of headloss for 95% of test measurements.
    c. 2 m (6.2 ft) or 15% of headloss for 100% of test measurements.

For extended period simulation, the criteria require that three separate steady state calibrations be performed for different time periods and that the average volumetric difference between measured and predicted reservoir storage be within 5%. Additional details can be obtained directly from the report.

Deviations between results of the model application and the field observations may be caused by several factors, including: 1) erroneous model parameters (i.e. pipe roughness values and nodal demand distribution), 2) erroneous network data (i.e. pipe diameters, lengths, etc), 3) incorrect network geometry (i.e. pipes connected to the wrong nodes, etc.), 4) incorrect pressure zone boundary definitions, 5) errors in boundary conditions (i.e. incorrect PRV value settings, tank water levels, pump curves, etc.), 6) errors in historical operating records (i.e. pumps starting and stopping at incorrect times), 7) measurement equipment errors (i.e. pressure gages not properly calibrated, etc.), and 8) measurement error (i.e. reading the wrong values from measurement instruments). The last two sources of errors can hopefully be eliminated or at least minimized by developing and implementing a careful data collection effort. Elimination of the remaining errors will frequently require the iterative application of the last three steps of the model calibration process - macro-level calibration, sensitivity, and micro-level calibration. Each of these steps is described in the following sections.

6.0 Perform Macro-level Model Calibration

In the event that one or more of the measured state variable values are different from the modeled values by an amount that is deemed to be excessive (i.e greater than 30 percent), it is likely that the cause for the difference may extend beyond errors in the estimates for either the pipe roughness values or the nodal demands. Possible causes for such differences are many but may include: 1) closed or partially closed valves, 2) inaccurate pump curves or tank telemetry data, 3) incorrect pipe sizes (e.g. 6 inch instead of 16, etc.), 4) incorrect pipe lengths, 5) incorrect network geometry, and 6) incorrect pressure zone boundaries, etc. (Walski, 1990a).

The only way to adequately address such errors is to systematically review the data associated with the model in order to insure its accuracy. In most cases, some data will be less reliable than other data. This observation provides a logical place to start in an attempt to identify the problem. Model sensitivity analysis provides another means of identifying the source of discrepancy. For example, if it is suspected that a valve is closed, this assumption can be modeled by simply closing the line in the model and evaluate the resulting pressures. Potential errors in pump curve data may sometimes be circumvented by simulating the pumps with negative inflows set equal to observed pumps discharges (Cruickshank, and Long, 1992). This of course assumes that the errors in the observed flow rates (and the induced head) are less than the errors introduced by using the pump curves. In any rate, only after the model results and the observed conditions are within some reasonable degree of correlation (usually less than 20% error) should the final step of micro-level calibration be attempted.

7.0 Perform Sensitivity Analysis

Before attempting a micro-level calibration, it is helpful to perform a sensitivity analysis of the model in order to help identify the most likely source of model error. This can be accomplished by varying the model parameters by different amounts and then measuring the associated effect. For example, many current network models have as an analysis option the capability to make multiple simulations in which global adjustment factors can be applied to pipe roughness values or nodal demand values. By examining such results, the user can begin to identify which parameters have the most significant impact on the model results and thereby identify potential parameters for subsequent fine tuning through micro-level calibration.

8.0 Perform Micro-level Model Calibration

After the model results and the field observations are in reasonable agreement, a micro-level model calibration should be performed. As discussed previously, the two parameters adjusted during this final calibration phase will normally include pipe roughness and nodal demands. In many cases it may be useful to break the micro calibration into two separate steps: 1) steady state calibration, and 2) extended period calibration. In performing a steady state calibration the model parameters are adjusted to match pressures and flowrates associated with static observations. The normal source for such data is from fire flow tests. In an extended period calibration, the model parameters are adjusted to match time varying pressures and flows as well as tank water level trajectories. In most cases the steady state calibration will be more sensitive to changes in pipe roughness while the extended period calibration will be more sensitive to changes in the distribution of demands. As a result, one potential calibration strategy would be to first fine tune the pipe roughness parameter values using the results from fire flow tests and then try to fine tune the distribution of demands using the flow/pressure/water level telemetry data.

Historically, most attempts at model calibration have typically employed an empirical or trial and error approach. Such an approach can prove to be extremely time consuming and frustrating when dealing with most typical water systems. The level of frustration will, of course, depend somewhat on the expertise of the modeler, the size of the system, and the quantity and quality of the field data. Some of the frustration can be minimized by breaking complicated systems into

smaller parts and then calibrating the model parameters using an incremental approach. Calibration of multi-tank systems can sometimes be facilitated by collecting multiple data sets with all but one of the tanks closed (Cruickshank, and Long, 1992). In recent years, several researchers have proposed different algorithms for use in automatically calibrating hydraulic network models. These techniques have been based on the use of analytical equations (Walski, 1983), simulation models (Rahal et al., 1980; Gofman and Rodeh, 1981; Ormsbee and Wood, 1986; and Boulos and Ormsbee, 1991), and optimization methods (Meredith, 1983; Coulbeck, 1984, Ormsbee, 1989; Lansey and Basnet, 1991; and Ormsbee, et al., 1992).

8.1 Analytical Approaches

In general, techniques based on analytical equations require significant simplification of the network through skeletonization and the use of equivalent pipes. As a result, such techniques may only get the user close to the correct results. Conversely, both simulation and optimization approaches take advantage of using a complete model.

8.2. Simulation Approaches

Simulation techniques are based on the idea of solving for one or more calibration factors through the addition of one or more network equations. The additional equation or equations are used to define an additional observed boundary condition (such as fire flow discharge head). By addition of an extra equation, an additional unknown can then be determined explicitly.

The primary disadvantage of the simulation approaches is that they can only handle one set of boundary conditions at a time. For example, in applying a simulation approach to a system with three different sets of observations (all of which were obtained under different boundary conditions, i.e. different tank levels, pump status, etc.), three different results can be expected. Attempts to obtain a single calibration result will require one of two application strategies: 1) a sequential approach, or 2) an average approach. In applying the sequential approach the system is subdivided into multiple zones whose number will correspond to the number of sets of boundary conditions. In this case the first set of observations is used to obtain calibration factors for the first zone. These factors are then fixed and another set of factors is then determined for the second zone and so on. In the average approach, final calibration factors are obtained by averaging the calibration factors for each of the individual calibration applications.

8.3 Optimization Approaches

The primary alternative to the simulation approach is to use an optimization approach. In using an optimization approach, the calibration problem is formulated as a nonlinear optimization problem consisting of a nonlinear objective function subject to both linear and nonlinear equality and inequality constraints. Using standard mathematical notation, the associated optimization problem may be expressed as follows:

Minimize:

$$f(\mathbf{X})$$

..... eq. 13

Subject To:

$$g(\mathbf{X}) = 0$$

..... eq. 14

$$L_h \leq h(\mathbf{X}) \leq U_h$$

..... eq. 15

$$L_x \leq \mathbf{X} \leq U_x$$

..... eq. 16

where X is the vector of decision variables (pipe roughness coefficients, nodal demands, etc.), f(X) is the nonlinear objective function, g(X) is a vector of implicit system constraints, h(X) is a vector of implicit bound constraints, and, Lx and Ux, are the lower and upper bounds on the explicit system constraints and the decision variables.

Normally, the objective function will be formulated so as to minimize the square of the differences between observed and predicted values of pressures and flows. Mathematically, this may be expressed as:

$$f(\mathbf{X}) = \alpha \sum_{j=1}^{J} (OP_j - PP_j)^2 + \beta \sum_{p=1}^{P} (OQ_p - PQ_q)^2$$

..... eq. 17

where OPj = the observed pressure at junction j, PPj = the predicted pressure at junction j, OQp = the observed flow in pipe p, PQp = the predicted flow in pipe p, and α and β are normalization weights.

The implicit bound constraints on the problem may include both pressure bound constraints and flowrate bound constraints. These constraints may be used to insure that the resulting calibration does not produce unrealistic pressures or flows as a result of the model calibration process. Mathematically, for a given vector of junction pressures P these constraints can be expressed as:

$$L_p \leq \mathbf{P} \leq U_p$$

..... eq.18

Likewise for a given vector of pipe flows Q these constraints can be expressed as:

$$L_Q \leq \mathbf{Q} \leq U_Q$$

..... eq. 19

The explicit bound constraints may be used to set limits on the explicit decision variables of the calibration problem. Normally, these variables will include (1) the roughness coefficient of each pipe, and (2) the demands at each node. For a given vector of pipe roughness coefficients C these constraints can be expressed as:

$$L_C \leq \mathbf{C} \leq U_C$$

..... eq. 20

Likewise for a given vector of nodal demands D, these constraints can be expressed as:

$$L_D \leq \mathbf{D} \leq U_D$$

..... eq. 21

The implicit system constraints include nodal conservation of mass and conservation of energy.

The nodal conservation of mass equation Fc (Q) requires that the sum of flows into or out of any junction node n minus any external demand Dj must be equal to zero. For each junction node j this may be expressed as:

$$F_c(Q) = \sum_{n \in j}^{N_j} Q_n - D_j = 0$$

..... eq. 22

where Nj = the number of pipes connected to junction node j and {j} is the set of pipes connected to junction node j.

The conservation of energy constraint Fe(Q) requires that the sum of the line loss (HLn) and the minor losses (HMn) over any path or loop k, minus any energy added to the liquid by a pump (EPn), minus the difference in grade between and two points of known energy (DEk) is equal to zero. For any loop or path k this may be expressed as:

$$F_e(Q) = \sum_{n \in k}^{N_k} (HL_n + HM_n - EP_n) - DE_k = 0$$

..... eq. 23

where Nk = the number of pipes associated with loop or path k, and {k} is the set of pipes associated with loop or path k. It should be emphasized that HLn, HMn, and EPn, are all nonlinear functions of the pipe discharge Q.

While both the implicit and explicit bound constraints have traditionally been incorporated directly into the nonlinear problem formulation, the implicit system constraints have been handled using one of two different approaches. In the first approach, the implicit system constraints are incorporated directly within the set of nonlinear equations and solved using

normal nonlinear programming methods. In the second approach, the equations are removed from the optimization problem and evaluated externally using mathematical simulation (Ormsbee, 1989; Lansey and Basnet, 1991). Such an approach allows for a much smaller and more tractable optimization problem, since both sets of implicit equations (which constitute linear and nonlinear equality constraints to the original problem) can now be satisfied much more efficiently using an external simulation model (see Figure 8). The basic idea behind the approach is to use an implicit optimization algorithm to generate a vector of decision variables which are then passed to a lower level simulation model for use in evaluating all implicit system constraints. Feedback from the simulation model will include numerical values for use in identifying the status of each constraint as well as numerical results for use in evaluating the associated objective function.

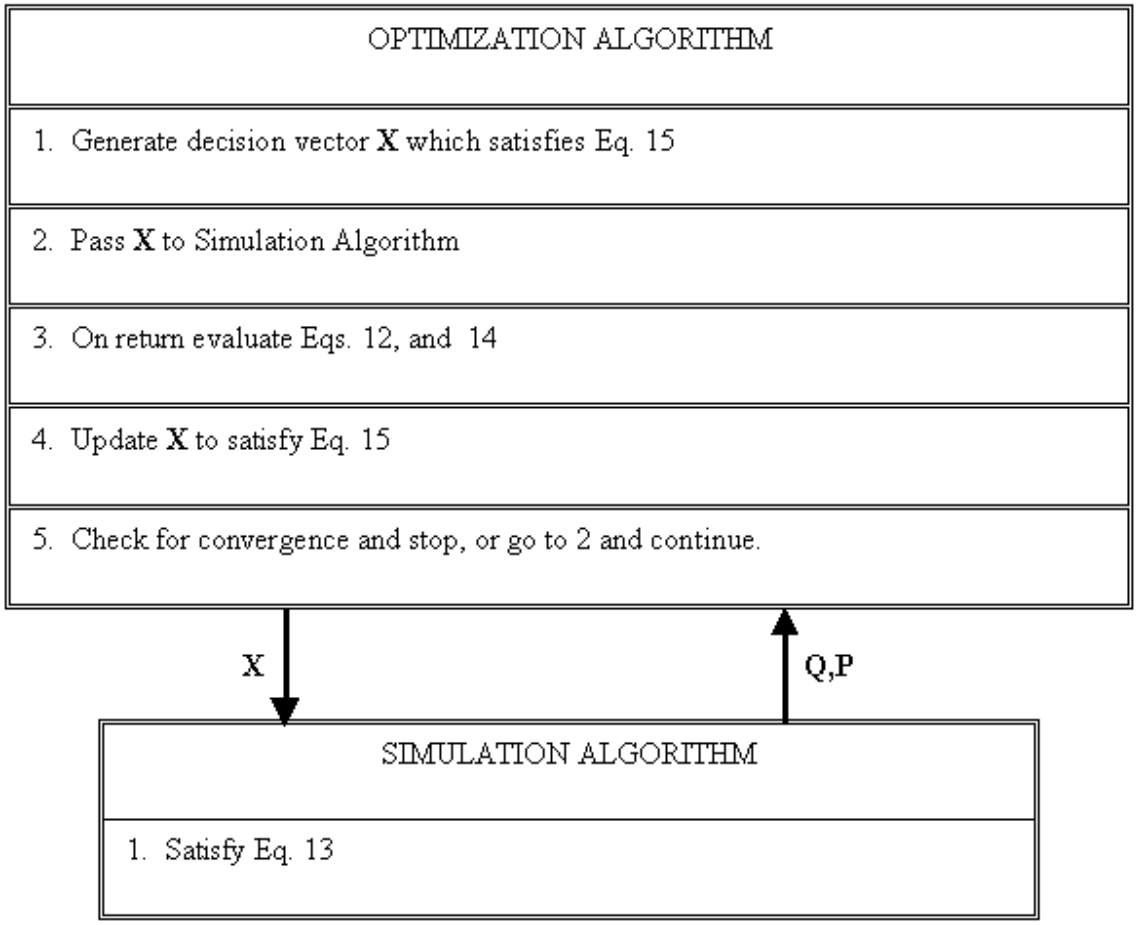


Figure 8. Bi-Level Computational Framework

Regardless of which approach is chosen, the resulting mathematical formulation must then be solved using some type of nonlinear optimization method. In general, three different approaches have been proposed and used: (1) gradient based methods, (2) pattern search methods, and (3) genetic optimization methods.

Gradient based methods require either first or second derivative information in order to produce improvements in the objective function. Traditionally, constraints are handled using either a penalty method or the Lagrange multiplier method (Edgar and Himmelblau, 1988). Pattern search methods employ a nonlinear heuristic that uses objective function values only in determining a sequential path through the region of search (Ormsbee, 1986, Ormsbee and Lingireddy, 1995). In general, when the objective function can be explicitly differentiated with respect to the decision variables the gradient methods are preferable to search methods. When the objective function is not an explicit function of the decision variables, as is normally the case with the current problem, then the relative advantage is not as great, although the required gradient information can still be determined numerically.

Recently, several researchers have begun to investigate the use of genetic optimization for solving such complex nonlinear optimization problems (Lingireddy et.al. 1995, Lingireddy and Ormsbee, 1998, and Savic and Walters 1995). Genetic optimization offers a significant advantage over more traditional optimization approaches in that it attempts to obtain an optimal solution by continuing to evaluate multiple solution vectors simultaneously (Goldberg, 1989). In addition, genetic optimization methods do not require gradient information. Finally, genetic optimization methods employ probabilistic transition rules as opposed to deterministic rules which have the advantage of insuring a robust solution methodology.

Genetic optimization starts with an initial population of randomly generated decision vectors. For an application to network calibration, each decision vector could consist of a subset of pipe roughness coefficients, nodal demands, etc. The final population of decision vectors is then determined through an iterative solution methodology that employs three sequential steps: 1) evaluation, 2) selection, and 3) reproduction. The evaluation phase involves the determination of the value of a fitness function (objective function) for each element (decision vector) in the current population. Based on these elevations, the algorithm then selects a subset of solutions for use in reproduction. The reproduction phase of the algorithm involves the generation of new offspring (additional decision vectors) using the selected pool of parent solutions. Reproduction is accomplished through the process of crossover whereby the numerical values of the new decision vector is determined by selecting elements from two parent decision vectors. The viability of the thus generated solutions is maintained by random mutations that are occasionally introduced into the resulting vectors. The resulting algorithm is thus able to generate a whole family of optimal solutions and thereby increase the probability of obtaining a successful model calibration.

Although optimizations in general and genetic optimization in particular offer very powerful algorithms for use in calibrating a water distribution model, the user should always recognize that the utility of the algorithms are very much dependent upon the accuracy of the input data. Such algorithms can be susceptible to convergence problems when the errors in the data are significant (e.g. headloss is on the same order of magnitude as the error in headloss). In addition, because most network model calibration problems are under-specified (i.e. there are usually many more unknowns than data points), many different solutions (i.e. roughness coefficients,

junction demands) can give reasonable pressures if the system is not reasonably stressed when the field data are collected.

9.0 Future Trends

With the advent and use of nonlinear optimization, it is possible to achieve some measure of success in the area of micro-level calibration. It is of course recognized that the level of success will be highly dependent upon the degree that the sources of macro-level calibration errors have first been eliminated or at least significantly reduced. While these sources of errors may not be as readily identified with conventional optimization techniques, it may be possible to develop prescriptive tools for these problems using expert system technology. In this case general calibration rules could be developed from an experiential data base that could then be used by other modelers in an attempt to identify the most likely source of model error for a given set of system characteristics and operating conditions. Such a system could also be linked with a graphical interface and a network model to provide an interactive environment for use in model calibration.

In recent years, there has been a growing advocacy for the use of both GIS technology and SCADA system databases in model calibration. GIS technology provides an efficient way to link customer billing records with network model components for use in assigning initial estimates of nodal demands (Basford and Sevier, 1995). Such technology also provides a graphical environment for examining the network database for errors. One of the more interesting possibilities with regard to network model calibration is the development and implementation of an on-line network model through linkage of the model with an on-line SCADA system. Such a configuration provides the possibility for a continuing calibration effort in which the model is continually updated as additional data is collected through the SCADA system (Schulte and Malm, 1993).

Finally, Bush and Uber (1998) have developed three sensitivity-based metrics for ranking potential sampling locations for use in model calibration. Although the documented sampling application was small, the developed approach provides a potential basis for selecting improved sampling sites for improved model calibration. It is expected that this area of research will see additional activity in future years.

10.0 Summary and Conclusion

Network model calibration should always be performed before any network analysis planning and design study. A seven-step methodology for network model calibration has been proposed. Historically, one of the most difficult steps in the process has been the final adjustment of pipe roughness values and nodal demands through the process of micro-level calibration. With the advent of recent computer technology it is now possible to achieve good model calibration with a reasonable level of success. As a result, there remains little justification for failing to develop good calibrated network models before conducting network analysis. It is expected that future

developments and applications of GIS and SCADA technology, as well as optimal sampling algorithms will lead to even more efficient tools.

11 References

Basford, C. and Sevier, C., (1995) "Automating the Maintenance of a Hydraulic Network Model Demand Database Utilizing GIS and Customer Billing Records," Proceedings of the 1995 AWWA Computer Conference, Norfolk, VA, 197-206.

Boulos, P., and Ormsbee, L., (1991) "Explicit Network Calibration for Multiple Loading Conditions, Civil Engineering Systems, Vol 8., 153-160.

Brion, L. M., and Mays, L. W., (1991) "Methodology for Optimal Operation of Pumping Stations in Water Distribution Systems," ASCE Journal of Hydraulic Engineering, 117(11).

Bush, C.A., and Uber, J.G., (1998) "Sampling Design Methods for Water Distribution Model Calibration," ASCE Journal of Water Resources Planning and Management, 124(6). 334-344.

Cesario, L., Kroon, J.R., Grayman, W.M., and Wright, G., (1996). "New Perspectives on Calibration of Treated Water Distribution System Models." Proceedings of the AWWA Annual Conference, Toronto, Canada.

Cesario, L., (1995). Modeling, Analysis and Design of Water Distribution Systems, American Water Works Association, Denver, CO.

Coulbeck, B., (1984). "An Application of Hierachial Optimization in Calibration of Large Scale Water Networks," Optimal Control Applications and Methods, 6, 31-42.

Cruickshank, J.R & Long, S.J. (1992) Calibrating Computer Model of Distribution Systems. Proc. 1992 AWWA Computer Conf., Nashville, Tenn.

Edgar, T.F., and Himmelblau, D.M., (1988) Optimization of Chemical Processes, McGraw Hill, New York, New York, 334-342.

Gofman, E. and Rodeh, M., (1981) "Loop Equations with Unknown Pipe Characteristics," ASCE Journal of the Hydraulics Division, 107(9), 1047-1060.

Goldberg, D.E., (1989) Genetic Algorithms in Search, Optimization and Machine Learning, Addison-Wesley Pub. Co., Reading, MA.

Grayman, W.M., (1998). "Use of Trace Studies and Water Quality Models to Calibrate a Network Hydraulic Model," Esstential Hydraulics and Hydrology, Haested Press

Kennedy, M., Sarikelle, S., and Suravallop, K., (1991) "Calibrating hydraulic analyses of distribution systems using fluoride tracer studies." Journal of the AWWA, 83(7), 54-59

Lamont, P.A., (1981), "Common Pipe Flow Formulas Compared with the Theory of Roughness," Journal of the AWWA, 73(5), 274.

Lansey, K, and Basnet, C., (1991) "Parameter Estimation for Water Distribution Networks," ASCE Journal of Water Resources Planning and Management, 117(1), 126-145.

Lingireddy, S., Ormsbee, L.E. and Wood, D.J.(1995) User's Manual - KYCAL, Kentucky Network Model Calibration Program, Civil Engineering Software Center, University of Kentucky.

Lingireddy, S., and Ormsbee, L.E., (1998) "Neural Networks in Optimal Calibration of Water Distribution Systems," Artificial Neural Networks for Civil Engineers: Advanced Features and Applications. Ed. I. Flood, and N. Kartam. American Society of Civil Engineers, p277.

McEnroe, B, Chase, D., and Sharp, W., (1989) "Field Testing Water Mains to Determine Carrying Capacity," Technical Paper EL-89, Environmental Laboratory of the Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi.

Meredith, D. D. (1983) "Use of optimization in calibrating water distribution models," ASCE Spring Convention, Philadelphia, Pa.

Ormsbee, L.E., (1989) "Implicit Pipe Network Calibration," ASCE Journal of Water Resources Planning and Management, 115(2), 243-257.

Ormsbee, L.E., (1986) "A nonlinear heuristic for applied problems in water resources," Proc. Seventeenth Annual Modeling and Simulation Conference, University of Pittsburgh, 1117-1121.

Ormsbee, L.E., Chase, D.V., and Grayman, W., (1992) "Network Modeling for Small Water Distribution Systems," Proceedings of the AWWA 1992 Computer Conference, Nashville, TN, 15.

Ormsbee, L., Chase and D., and Sharp, W., (1991) "Water Distribution Modeling", Proceedings, 1991 AWWA Computer Conference, Houston, TX, April 14-17, 27-35.

Ormsbee, L.E. and Chase, D.V., (1988) "Hydraulic Network Calibration Using Nonlinear Programming," Proceedings of the International Symposium on Water Distribution Modeling, Lexington, Kentucky, 31-44.

Ormsbee, L.E. and Lingireddy, S., (1995) Nonlinear Heuristic for Pump Operations, Journal of Water Resources Planning and Management, American Society of Civil Engineers, 121, 4, 302-309.

Ormsbee, L.E. and Wood, D.J., (1986) "Explicit Pipe Network Calibration," ASCE Journal of Water Resources Planning and Management, 112(2), 166-182.

Rahal, C. M., Sterling, M.J.H, and Coulbeck, B., (1980), "Parameter tuning for simulation models of water distribution Networks, Proc., Institution of Civil Engineers, London, England, 69(2), 751-762.

Rossman, L., (1994) EPANET User's Manual, Drinking Water Research Division, Risk Reduction Engineering Laboratory, Cincinnati, Ohio 45268

Savic, D.A., and Walters, G.A. (1995) Genetic Algorithm Techniques for Calibrating Network Models, Report No. 95/12, 1995, Center for Systems and Control, University of Exeter, UK.

Schulte, A. M., and Malm, A. P., (1993) "Integrating Hydraulic Modeling and SCADA Systems for System Planning and Control," Journal of the American Water Works Association, 85(7), 62-66.

Walski, T.M. (1999), Personal Communication

Walski, T. M. (1995) "Standards for model calibration," Proceedings of the 1995 AWWA Computer Conference, Norfolk, VA, 55-64.

Walski, T.M. (1990a) Sherlock Holmes Meets Hardy Cross, or Model Calibration in Austin, Texas, Jour. AWWA, 82:3:34.

Walski, T. M. (1990b) Water Distribution Systems: Simulation and Sizing, Chelsea, Mich, Lewis Publishers.

Walski, T.M., (1986) "Case Study: Pipe Network Model Calibration Issues," ASCE Journal of Water Resources Planning and Management, 112(2), 238.

Walski, T.M., (1985) "Correcting Head Loss Measurements in Water Mains," Journal of Transportation Engineering, 111(1), 75.

Walski, T, M. (1984) Analysis of Water Distribution Systems, Van Nostrand Reinhold Company, New York, New York.

Walski, T. M. (1983) "Technique for Calibrating Network Models," ASCE Journal of Water Resources Planning and Management, 109(4), 360-372.

Water Authorities Association and WRc, (1989), Network Analysis - A Code of Practice, WRc, Swindon, England.

Wood, D. J., (1991) Comprehensive Computer Modeling of Pipe Distribution Networks, Civil Engineering Software Center, College of Engineering, University of Kentucky, Lexington, Kentucky.