AUTOMATED CALIBRATION
OF
WATER DISTRIBUTION NETWORKS

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ABSTRACT

AUTOMATED CALIBRATION OF WATER DISTRIBUTION NETWORKS

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Water distribution network models are widely used for various purposes such as long-range planning, design, operation and water quality management. Before these models are used for a specific study, they should be calibrated by adjusting model parameters such as pipe roughness values and nodal demands so that models can yield compatible results with site observations (basically, pressure readings). Many methods have been developed to calibrate water distribution networks. In this study, Darwin Calibrator, a computer software that uses genetic algorithm, is used to calibrate N8.3 pressure zone model of Ankara water distribution network; in this case study the network is calibrated on the basis of roughness parameter, Hazen Williams coefficient for the sake of simplicity. It is understood that there are various parameters that contribute to the uncertainties in water distribution network modelling and the calibration process. Besides, computer software's are valuable tools to solve water distribution network problems and to calibrate network models in an accurate and fast way using automated calibration technique. Furthermore, there are many important aspects that should be considered during automated calibration such as pipe roughness grouping. In this study, influence of flow velocity on pipe roughness grouping is examined. Roughness coefficients of pipes have been estimated in the range of 70-140.

Keywords: Water Distribution, Hydraulic Network Model, Calibration, Automated Calibration, Water Distribution Network of Ankara, Calibration Case Study

ÖZ

SU DAĞITIM ŞEBEKELERİNİN OTOMATİK KALİBRASYONU

Apaydın, Öncü

Yüksek Lisans, İnşaat Mühendisliği Bölümü

Tez Yöneticisi: Doç. Dr. Nuri Merzi

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Günümüzde, su dağıtım şebeke modelleri; uzun vadeli planlama, tasarım, işletme ve su kalitesi yönetimi gibi birçok alanda sıklıkla kullanılmaktadır. Bu modeller, herhangi bir çalışmada kullanılmadan önce, ürettikleri sonuçların saha ölçümleri (genellikle basınç ölçümleri) ile tutarlı olmasını sağlamak amacı ile, boru pürüzlülük katsayısı ve düğüm noktaları su ihtiyaçları gibi şebeke parametreleri ayarlanarak, kalibre edilmelidir. Kalibrasyon hesaplamaları için bugüne kadar birçok metot geliştirilmiştir. Bu çalışmada, Ankara N8.3 basınç bölgesi şebekesinin kalibrasyonu, genetik algoritma kullanan Darwin Calibrator isimli bir bilgisayar programının yardımı ile yapılmıştır; bu çalışmada dağıtım şebekesi, Hazen Williams pürüzlülük katsayısı baz alınarak kalibre edilmiştir. Bu çalışma sırasında, su dağıtım şebeke modellemesi ve bu modellerin kalibrasyonu sürecinin birçok belirsizliği barındırdığı görülmüştür. Ayrıca, bilgisayar programlarının, su dağıtım şebeke problemlerinin çözülmesi ve otomatik kalibrasyon tekniği ile şebeke kalibrasyonu için hızlı ve güvenilir çözüm ürettikleri sonucuna varılmıştır. Ayrıca ,otomatik kalibrasyon sırasında boru gruplaması gibi birçok husus göz önünde bulundurulmalıdır. Çalışmada boru hızlarının boru pürüzlülük gruplarının oluşturulmasına etkileri de analiz edilmiştir. Çalışmada, kalibre edilmiş pürüzlülük katsayıları 70 ile 140 arasında bulunmuştur.

Anahtar Kelimeler: Su Dağıtım Şebekesi, Şebeke Hidrolik Modeli, Kalibrasyon, Otomatik Kalibrasyon, Ankara Su Dağıtım Şebekesi, Kalibrasyon Durum Çalışması

To Serap & My Family...

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LIST OF ABBREVIATIONS

ASKI	Ankara Water and Sanitation Administration
AWWA	American Water Works Association
DDC	Daily demand curve
DMA	District metered area
ECAC	Engineering Computer Applications Committee
EPS	Extended period simulation
HGL	Hydraulic grade line
GA	Genetic algorithm

CHAPTER 1

INTRODUCTION

Municipalities have to spend high budgets for water supply and distribution systems to provide water to communities. All of the related operations should be implemented in a cost-effective manner. To achieve this goal, water distribution networks should be planned, designed, operated, maintained and rehabilitated appropriately. At the end, system should be able to deliver sufficient quality of service to the customers now and in the long-term. Over the years; long after the new system is designed and constructed it is possible that there will be water quantity and quality problems besides, high and low pressure problems. These problems may arise due to the unexpected demand increases, aging of pipes, aging of pumps, leakage, incorrect operation of pumps etc. Operators should be capable of identifying these problems and making correct interventions to solve these problems in order to keep the system in service and operate the system efficiently. These interventions may include regular arrangements such as correct pump operations and also the required rehabilitation works such as cleaning and/or replacing pipes and system expansions (Walski 2003).

To provide solutions for water distribution system problems, a mathematical model of the system should be constructed; then, hydraulic parameters of the system (basically, nodal demand values and pipe roughness values) should be calculated periodically throughout the economic lifetime through calibration process. Mathematical equations and numerical approximations are used to analyse hydraulics of the system. Today, computer-based hydraulic network simulators are widely used by engineers. A water distribution computer model, representing the real network, is a practical and effective tool to make required calculations concerning system hydraulics (Wu, 2002). It provides time-effective solutions with high accuracy. It is for sure that the accuracy of the generated results is highly dependent on the quality of the provided data (Walski, 2000).

Therefore, it is important that the model should reveal the real situation of the system to provide adequate solutions for rehabilitation works and operational revisions. Since the system parameters (demands, physical situation of pipes etc.) hold high uncertainty; engineers should be confident that the constructed model is a tolerable representation of the real world. A precise model can be developed after collecting real data about the system by means of continuous monitoring or field data tests; then, the water distribution model should be calibrated in order to illustrate the actual condition of the system.

Calibration refers to the procedure that is applied to construct an adjusted network model that is capable of producing hydraulic results, which agree with the measured field data sets (Bhave, 1988). As the measured field data sets are sensitive to real system parameters, the calibration will provide a sensivity level such that the water distribution model will be consistent with the real system. Of course, the uncertainty level of the calibrated network is dependent on the accuracy of the field data sets and configuration of the network (location, pipe diameters, pipe lengths, elevations, status of valves and pumps).

There are many methods that are developed to calibrate water distribution network models. Regardless of the method applied, there are some certain steps that should be followed carefully. The first step is constructing the mathematical hydraulic model. This step includes obtaining service maps generated during the design of the system, which will allow constructing the configuration of the network (pipe diameters, pipe lengths, valve locations, tanks locations etc.). Still, there is a possibility that the construction may not have been implemented according to the design drawings. Therefore, it is better to check the validity of the data for the physical components of the system by intense field investigations. Then, following steps should be realized: collecting measurements (flows, pressures) and calibrating the network parameters.

The aim of this study is to calibrate an actual water distribution system by an automated calibration software named Darwin Calibrator (Haestad Methods, 2003), which uses a genetic algorithm technique. There are many methods that have been developed for calibration. Calibration attracted interests of many researchers studying in water distribution area. Literature review and the methods developed so far are discussed in Chapter 2. Next, information about genetic algorithm methodology and automated calibration will be explained in Chapter 3. In Chapter 4, case study conducted at the N8.3 distribution zone located in Keçiören, Ankara will be presented. Finally, conclusion and recommendations will be discussed in Chapter 5. In this study, not only an automated calibration case study for the N8.3 pressure zone of Ankara is executed but also the performance of the automated calibration system is studied.

CHAPTER 2

LITERATURE REVIEW

Computer models for water distribution systems have been available for a long time. For these models, it is important to reflect the real situation of the network; in other words, a model should be calibrated. Many advances have been made to develop calibration methods and procedures.

2.1. Calibration of Water Distribution Networks

Many definitions have been proposed for calibration of water distribution networks. Shamir and Howard (1977) described the calibration as a process of both modelling and its engineering applications: (i) modelling problem refers to the determination of the physical characteristics (basically, configuration of network) and (ii) operational characteristics of an existing system and engineering applications refer to determining the data that when input to the computer model, will yield realistic results. Walski (1983) proposed that a water distribution network model is assumed as calibrated if it can predict the flows and pressures with reasonable agreement with the observed values. Bhave (1988) emphasized that the calibration process should ensure that the hydraulic model would predict the behaviour of the network with a reasonable accuracy. Cesario and Davis (1984) indicated that calibration is fine-tuning a model till it simulates the field conditions to a degree of accuracy. In conclusion, all these definitions agree that the calibration process at the end should lead to a more accurate network model that reflects the actual characteristics of the water distribution network.

2.2. Hydraulic Model Calibration Methods

Many methods have been developed since 1970s. Savic et al. (2009) grouped the methods for calibration of water distribution systems under three main titles as follows: iterative methods (trial-and-error methods), explicit methods and implicit methods.

2.2.1. Iterative Methods

Conventional methods for calibration have been a process of trial and error (Walski et al., 2003). Modelers had to change roughness value and demands till the observed values and simulated values converge. Walski (1983) and Bhave (1988) proposed methods based on trial-and-error procedure. Walski (1983) developed an iterative procedure to estimate roughness value for the pipes and the rate of nodal flows by collecting pressure data for the low (normal use) and high flow (fire flow case) conditions. In this method, the total inflow into the system is also an unknown parameter and adjusted in parallel with pipe roughness value (C-factor). By using the field observations and results of hydraulic simulation, correction factors (equation 2.1 and 2.1) are calculated to calibrate the demands and C-factor values.

$$A = \frac{F_f}{\left(\frac{b}{a}\right)(\mathrm{D_e}+\mathrm{F_f}) - D_e} \tag{2.1}$$

$$B = \frac{F_f}{(b)(Q_e+F_f) - aD_e} \tag{2.2}$$

where;

A = correction factor for demands,

B = correction factor for C-factors,

F_f = fire flow (m^3/s),

$a = (h_1/h_3)^{0.54}$,

$b = (h_2/h_4)^{0.54}$,

D_e = estimated demand in the surrounding of the test (m^3/s),

h_1 = observed head loss along the test section at low flow condition (m),

h_2 = observed head loss along the test section at high flow condition (m),

h_3 = simulated head loss along the test section at low flow condition (m),

h_4 = simulated head loss along the test section at high flow condition (m).

Then, these correction factors are applied to the estimated C-factor and demand to improve estimated values.

$$D_c = AD_e \tag{2.3}$$

$$C_c = BC_e \tag{2.4}$$

here;

D_c= corrected value for demands (m3/s),

C_c= corrected value for C-factors,

C_e= initial estimated value for C-factors.

An example problem is solved to illustrate the Walski's method. Assume that there is a tank with a head of 60 m. C-factor is estimated for the existing system as 115. Total demand in the surrounding of the test area is 200 l/s and the fire flow is 150 l/s. Simulated and observed hydraulic grade lines (HGL) are given on Table 2.1.

Table 2.1. Data for Walski's Method Example

Condition	**Observed HGL (m)**	**Simulated HGL(m)**
Low flow	48.90m	50.50m
Fire flow	35.20m	19.90m

$$a = \left(\frac{60.00 - 48.90}{60.00 - 50.50}\right)^{0.54} = 1.08$$

$$b = \left(\frac{60.00 - 7.20}{60.00 - 19.90}\right)^{0.54} = 1.16$$

$$A = \frac{250}{\left(\frac{1.16}{1.08}\right)(200 + 150) - 200} = 1.42$$

$$B = \frac{150}{(1.16)(200 + 150) - 1.08(200)} = 0.78$$

$$D_c = 1.42x200 = 284$$

$$C_c = 0.78 * 115 = 89.7$$

Accordingly, above values will be used in the next run and iterations will be done till the C-factor converges.

Bhave (1988) used a technique to adjust network parameters similar to Walski (1983). However, Bhave assumed that rate of flow into the system can be accurately measured which is almost the case in practical applications. Furthermore, this method enables to group pipes so that adjustment factors for pipe resistance coefficients and nodal demands are not generalized as a single global factor for the whole network. Bhave used the example model in Figure 2.1 to illustrate his method.

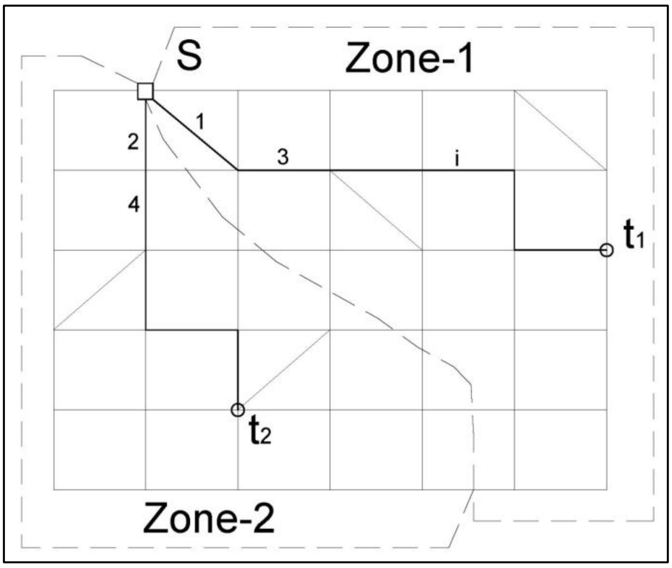


Figure 2.1. Illustrative Figure of Bhave's Method (1988)

Here, S is the source node whereas t_1 and t_2 are tests nodes. Bhave (1988) derived the following equations.

For path-1 (from node S to test node t_1);

$$B\left(H_s - H_{t1p}\right) + \frac{n\left(H_s - H_{t1p}\right)}{Q_{1p}} \Delta Q_1 = H_s - H_{t1} \tag{2.5}$$

And similarly for path-2 (from node S to test node t_2);

$$B\left(H_s - H_{t2p}\right) - \frac{n\left(H_s - H_{t2p}\right)}{Q_{2p}} \Delta Q_1 = H_s - H_{t2} \tag{2.6}$$

here;

Q_{ip}= discharge in pipes for the estimated nodal demands and pipe resistance coefficients,

H_s = head at source S,

H_{tip} = predicted head at node t_i.

B = global adjustment factor for pipe resistance constants for path s-t1.

ΔQ_1= discharge adjustment factor

When equations 2.5 and 2.6 are solved simultaneously, adjusted C-factor is founded as:

$$C_{ia} = \frac{1}{B^{0.54}} C_{ip} \tag{2.7}$$

Correction factor for the nodal demands can be calculated from equation 2.8:

$$q_{ja} = q_{jp}\left(1 + \frac{\Delta Q_z}{\sum_{j \in Nz} q_{jp}}\right) \tag{2.8}$$

where;

q_{ja} =correction factor for nodal demands

q_{jp} = predicted demand at node j,

ΔQ_z = total nodal flow adjustment for zone z,

N_z = set of demand nodes in zone z.

An example is created below in Figure 2.2 (Bhave, 1988) to illustrate an example for the method. Head at source node (1) is 60 m. Fire flows are 160 l/s and 75 l/s respectively for nodes 4 and 7. Pipe data for the example is given on Table 2.2.

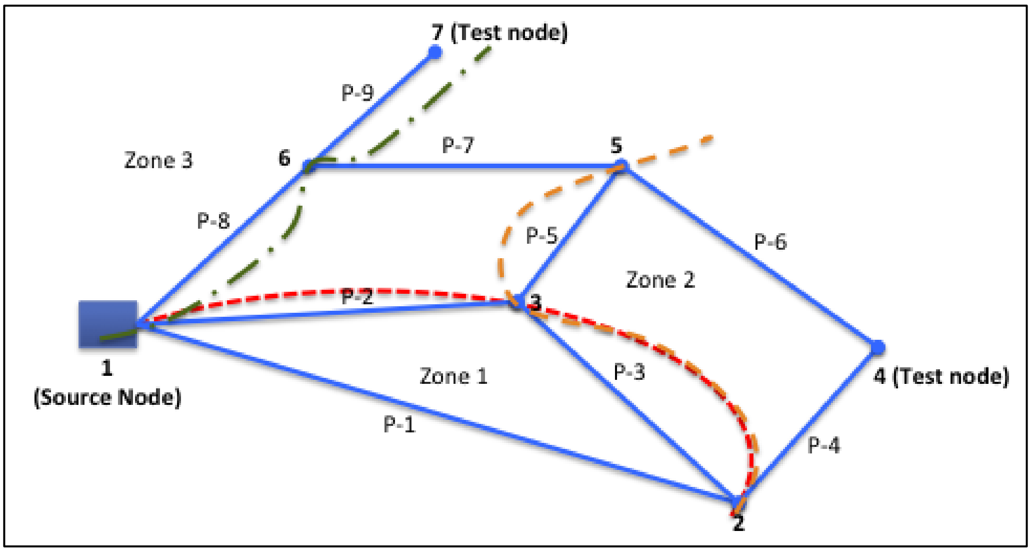


Figure 2.2. Network for Bhave's Method Example

Table 2.2. Pipe Data for Bhave's Method Example

Pipe	Actual HW Coefficient	HW Coefficient	Predicted Condition			
			Pipe Flow, L/s		Pipe Head Loss, m	
			Low Flow	High Flow	Low Flow	High Flow
P-1	100	115	51.17	90.23	5.66	16.19
P-2	130	115	183.46	293.99	4.89	11.71
P-3	120	115	27.10	69.89	0.78	4.49
P-4	110	115	38.32	120.13	0.41	3.4
P-5	120	115	56.39	124.12	1.4	6.01
P-6	110	115	-18.37	59.84	-0.21	1.87
P-7	120	115	5.07	15.69	0.02	0.16
P-8	110	115	114.98	200.64	6.26	17.57
P-9	90	115	40.00	115.00	3.19	22.57

Network is divided into three zones. Pipe adjustment factors are B_1, B_2 and B_3.

Considering path 1-3-2-4 (pipes 2,3,4):

For low flow

$$(4.89+0.78)B_1+0.41B_2+\frac{1.852x(60-53.93)}{183.46}\Delta Q=60-54.10$$

For high flow

$$(11.71+4.49)B_1+3.40B_2+\frac{1.852x(60-40.41)}{293.99}\Delta Q=60-40.47$$

Similarly for path 1-6-7 (pipes 8,9):

For low flow

$$(6.26+3.19)B_3-\frac{1.852x(60-50.54)}{114.98}\Delta Q=60-48.88$$

For high flow

$$(17.57+22.57)B_3-\frac{1.852x(60-19.87)}{200.64}\Delta Q=60-7.15$$

Solving the above equations simultaneously;

B1=0.701, B2=1.669, B3=1.504

ΔF=20.3 l/s

Hence the adjusted C-factors calculated in the first iteration for zones 1,2 and 3 are respectively:

$(1/0.701)^{0.54}$=139.4

$(1/1.669)^{0.54}$=87.2

$(1/1.504)^{0.54}$=92.3

ΔF is distributed to nodes 2,3,4,5 by using the equation 2.8. Above iterations are carried on till the HW coefficient values converges.

Main benefit gained from the iterative procedures is the creation of guidelines and procedures for the hydraulic model calibration (Savic et al., 2009). As a general result, these methods are said to be relatively slow procedures concerning converging.

2.2.2. Explicit Methods

Ormsbee and Wood (1986) suggested an explicit methodology by describing an additional continuity equation, which will allow solving an extra variable such as the global head loss adjustment. Additional equation is derived from the available field measurements (flow or pressure measurement). Each field measurement allows defining one additional equation so pipe roughness coefficients should be grouped according to the available number of field measurements.

2.2.3. Implicit Methods

Implicit calibration uses optimization techniques to minimize the objective function that is defined as the discrepancy between the measured and predicted values. Optimization tool cooperates with a hydraulic solver so that calculated hydraulic results are passed to optimization tool and updated variable parameters are passed to hydraulic solver in cycles. System equations (energy equations etc.) and limits of the calibration parameters are defined as constraints to the objective function (Savic et al., 2009). Implicit methods can be classified as evolutionary and non-evolutionary techniques.

Ormsbee (1989) developed an implicit algorithm with a non-evolutionary technique (box method) to calibrate hydraulic models for both steady state and extended period loading conditions.

The latest tendency in calibration methods is using evolutionary techniques, especially the genetic algorithms (GA). Since hydraulic models are complex with respect to size and non-linearity, many simplifications is essential to solve the problems with conventional optimization practices and analytical approaches (Savic and Walters, 1995). GA's continuously generate potential solutions based on the theory of genetics, evaluate the fitness of each potential solution, replicate and evolve into offspring solutions (Walski et al., 2003). Savic and Walters (1995) introduced the use of GA technique for calibration of a hydraulic network model. Lingireddy and Ormsbee (1999) developed a nonlinear optimization model that uses a search technique based on GA model.

Evolutionary methodologies have some distinct advantages over non-evolutionary methods in that: (1) GA calibration is conceptually simple because it does not need complex mathematical apparatus to evaluate sensitivities or invert matrices; (2) GAs can handle large calibration problems, i.e., real-life size networks; (3) GAs permit easy incorporation of additional calibration parameter types and constraints into the optimization process (Savic et al., 2009).

Since the GA has been developed so far, many computer applications grounded on this technique has been developed. Wu et al. (2002) developed automated calibration software, Darwin Calibrator, which used a competent GA technique (Wu and Simpson, 2001) for the optimized calibration.

2.3. Source of Errors

One may think that calibration is achieved by just adjusting the internal pipe roughness values or estimated nodal demands till the agreement between simulated and observed results matches. However, various factors contribute to such deviations between simulated and observed results (Walski, 1990). Therefore, possible source of errors that contribute to the discrepancy between observed field results and simulated computer model results should be investigated carefully during the calibration process.

2.3.1. Errors in Input Data

There are two types of errors that can be directly related with input data; typing errors and measurement errors. Although typing errors are much easier to identify rather than the measurement errors, they may also be difficult to discover. Entering a pipe length of 30m instead of 300m is an example for such errors. Fortunately, today's graphically based network modeling software's reduce this possibility. On the other hand measurement errors, might arise due to the imprecisions of measuring devices (Walski et al., 2003).

2.3.2. Initial Pipe Roughness Values

Initial estimate of the pipe roughness values is important as it limits the search space for the optimal solution. There are various tables produced to estimate pipe roughness values as a function of pipe material, size and age. It is important to have a fair good enough initial estimate for pipe roughness values in order to find optimal solution easily. Also, if the initial estimate is so rough, calibration process may end with failure.

2.3.3. System Demands

In water distribution modeling, it is assumed that the water is withdrawn from junctions. This is known as spatial demand allocation. However, it is distributed along the entire length of pipe in the real case as illustrated in Figure 2.3. Spatial demand allocation provides model simplification (Walski et al., 2003). If the demand that is assigned to a specific junction node is not far away from the node, then the error is relatively minor (Engineering Computer Application Committee (ECAC), 1999).

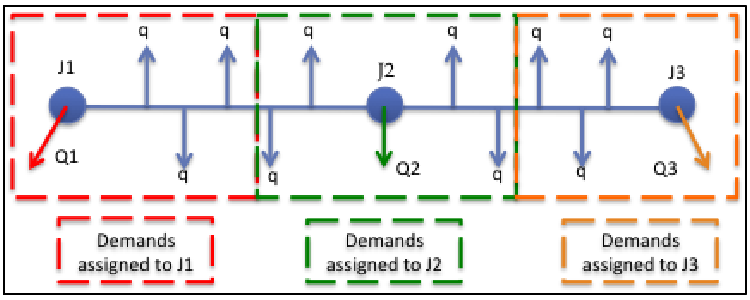


Figure 2.3. Spatial Demand Distribution

In this study demands are allocated to the nodes in proportion to the half of the length of the pipes connected to that specific node. To do this first the demand per meter pipe in the network is determined as follows (Şendil, 2013):

$$D_x = D_t \,/\sum L_i \tag{2.9}$$

here;

D_x = demand per meter pipe,

D_t = average demand measured at district metered area (DMA) inlet node,

L_i = pipe length.

For instance, if demand at J2 in Figure 2.3 is to be determined:

$$D_{J2} = \left(\frac{L_{J1-J2}}{2} + \frac{L_{J2-J3}}{2}\right) \times D_X$$

Water usage in a network should be monitored for 24 hours and daily demand curves (DDC) should be prepared. DDCs, constructed by Şendil (2013), for the N8.3 Pressure Zone that will be studied in Chapter 4, are given in Figure 2.4.

2.3.4. System Maps

System maps provide information especially about the configuration of network (location, diameter and length of pipes, location of valves, tanks, elevations etc.). These maps can be founded in various formats. Recent ones are usually very accurate and generated using Cad or GIS software whereas the older system maps can come as rolled-up plans. In either case, maps should reflect the modifications made in the system (ECAC, 1999). In this study, GIS data

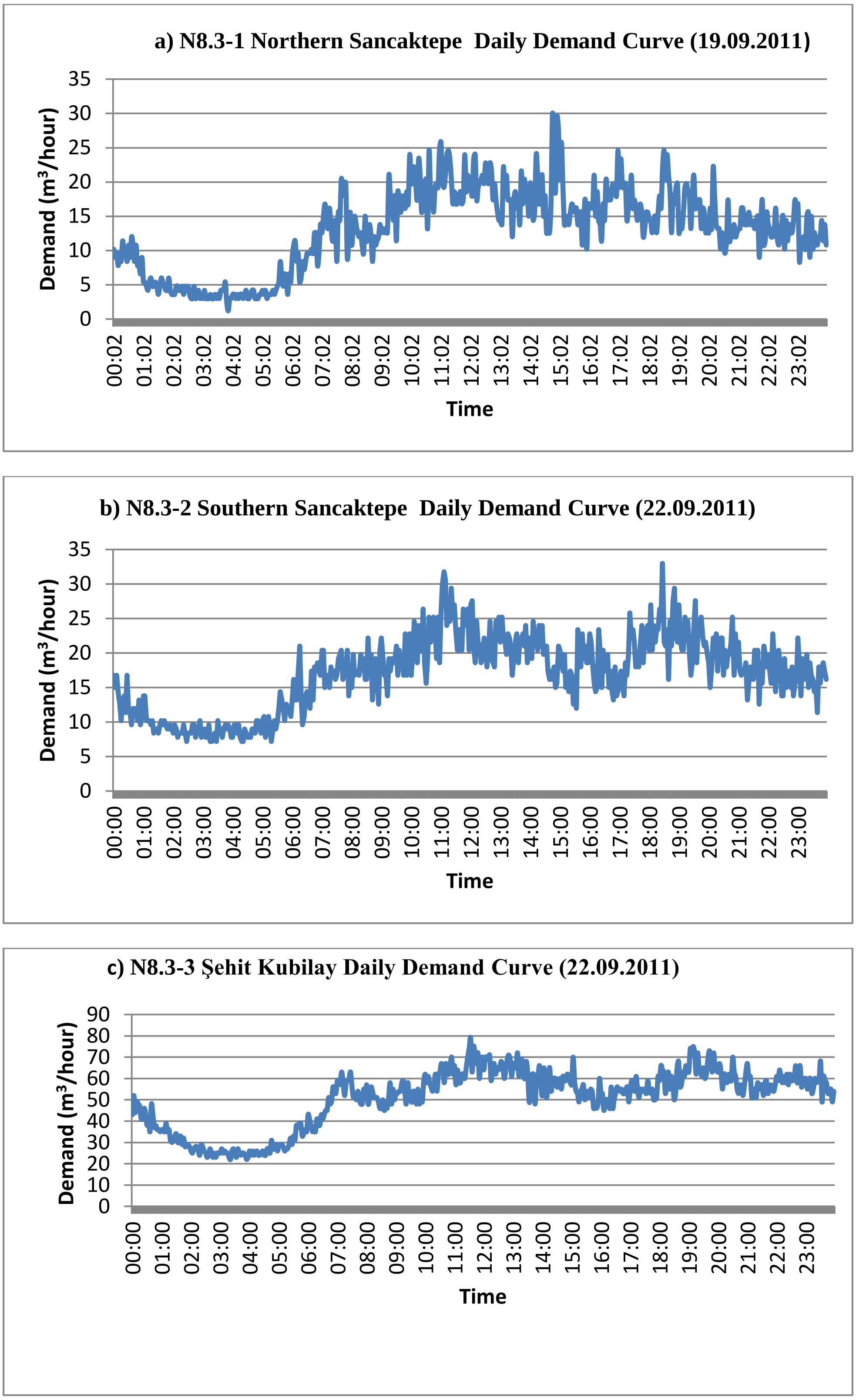
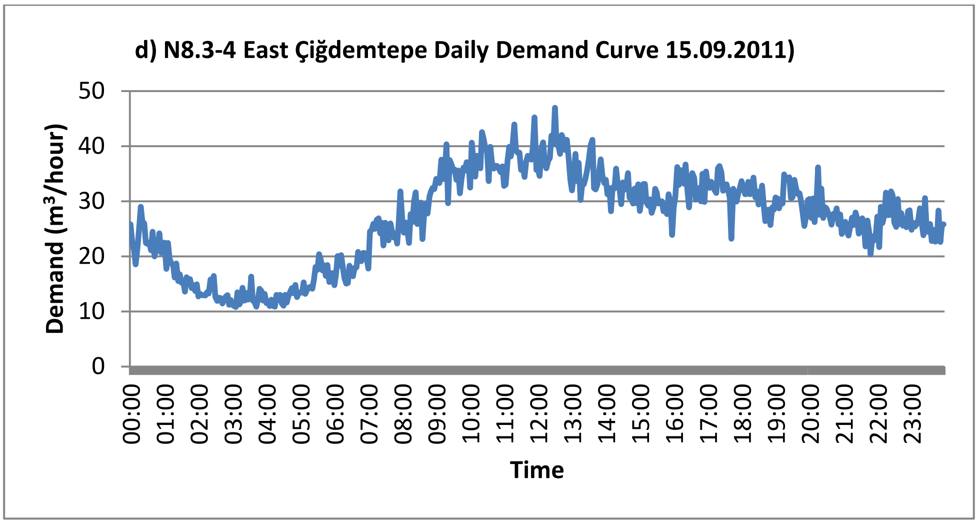
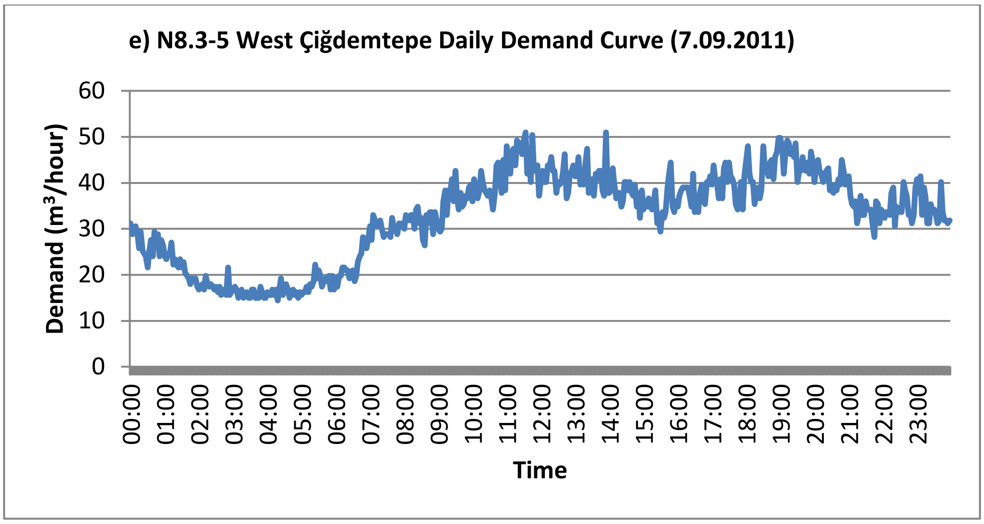


Figure 2.4. Daily Demand Curves for N8.3 Pressure Zone DMAs (Şendil, 2013)





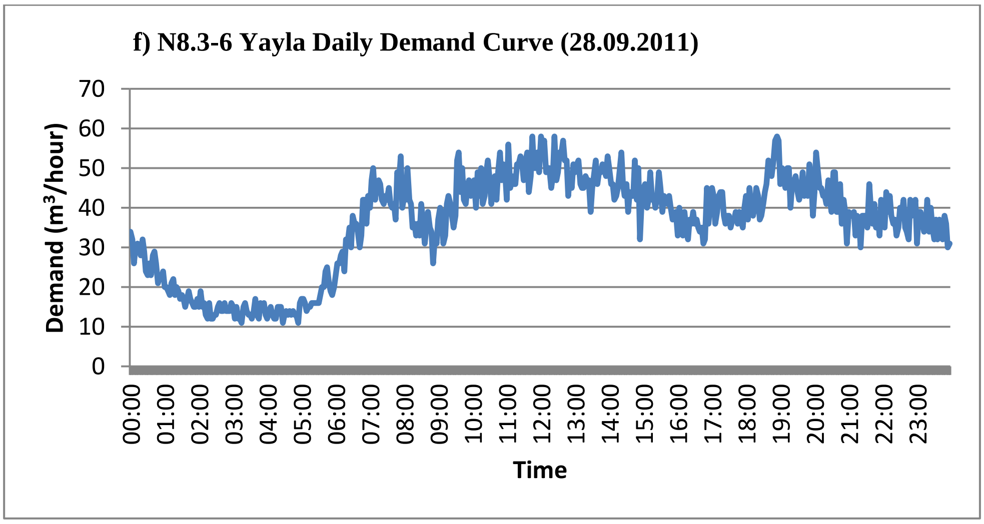


Figure 2.4. Daily Demand Curves for N8.3 Pressure Zone DMAs (continued) (Şendil, 2013)

2.3.5. Node Elevations

Pressure measurements are usually taken very close to the hydrants. The elevation of the pressure gauge at hydrant is usually higher than the ground elevation. Therefore the elevation of the pressure gauge should be determined and used in the calibration model (ECAC, 1999). In this study, GPS helped for accurate measurement of elevation of nodes as shown in Figure 2.5.

Figure 2.5. Surveying Works with GPS

2.3.6. Effect of Time

The effect of time can have a significant impact in calibration process since many parameters are time-dependent (Walski et al., 2003). If the hydraulic model is an extended period simulation (EPS) then the calibration model should consider time-varying conditions (ECAC, 1999). In this study, only steady state analysis cases are considered.

2.3.7. Model Detail

There are various applications of mathematical network models. Types of applications can be categorized, with respect to the purpose of use, as follows (Walski et al., 2003):

- Master planning
- Fire protection
- Water quality
- Energy management
- Design
- Daily operational uses

Purpose of the model determines network parts that should be included in the model or not. In other words, it determines the degree of detail of a mathematical model. Energy operation studies usually require minimal detail, on the other hand fire flow; water quality and design works require maximum details.

Reducing the size and details of the hydraulic models is known as skeletonization. Usually, in computer models, a skeletonized version of the system is used. A skeletonized system is the one that does not include pipes in small diameters or even the lines that do not have significant influence over system hydraulics (ECAC, 1999). It is possible to over-skeletonize a model, leaving out critical links. Excluding a dense grid of small-diameter main may be inappropriate if the group has a considerable effect on the system hydraulics. In such cases, removed details may need to be added back (Walski et al., 2003). In this study (as far as skeletonization is concerned), the model to be built is ready to use even for water quality studies; all the pipes are kept except customer connections.

2.3.8. Geometric Inconsistencies

Even if good quality information is supplied on the physical attributes of the system and modeler can estimate initial conditions appropriately, there can still be differences between predicted and observed performance. One can face a situation that two pipe lines seem to be connected however the cross-section view is view would show the otherwise. Another issue related with geometric inconsistencies result from the state of the valves. If a modeler is achieving unrealistically low roughness values, it may be due to a closed or partially closed valve (ECAC, 1999).

2.3.9. Pump Characteristic Curves

In most hydraulic models, three or more points from actual pump head-discharge relationship are used to reproduce curve fits. Errors in numerical fits can lead to discrepancies. However, most likely reason of errors for pumps is due to outdate pump curves since the mechanical properties of pumps can change over years (ECAC, 1999).

2.3.10. Boundary Elements

Errors in boundary elements can also cause variances in calibration. Boundary elements consist of tank levels, pressure zone boundaries, regulating valve settings.

2.3.11. Measurement Equipment

Before a measurement is taken, device used for the measurement should be calibrated. If possible digital measurement tools that have recording abilities should be used. In Figure 2.6, digital manometer that is used in this study is shown.

Figure 2.6. Digital Manometer

2.4. Calibration Procedures

It is helpful to benefit from the previous experiences for young engineers for calibrating models. But it is almost impossible to develop a cookbook procedure for model calibration (Walski, 1990). Every calibration study is unique as every distribution system has its own characteristics. Efforts to calibrate a network system is summarized in the following steps (Ormsbee and Lingireddy, 1997):

1. Identify the Intended Use of Model

Intended use of the model directly affects the type of analysis required. For instance, if water quality and operational studies are required, an extended period analysis should be performed. Whereas planning and design analysis requires a steady-state analysis (Walski, 1995).

2. Determine the Initial Estimates of the Parameters

The most important parameters that have to be estimated in advance are roughness of pipes and nodal demands. Initial pipe roughness values can be obtained using tables available in the literature. These tables provide estimates of pipe roughness value with respect to material, diameter and age. Distributing water along a length of pipe spatially is known as demand allocation. In reality, water is withdrawn along the entire pipe from several nodes but in modeling it is simplified as junctions representing several demand nodes sourcing water in total (Walski et al., 2003). It is possible to allocate demands to the junctions by using a method that usually identifies the influence area of the junctions (Ormsbee and Lingireddy, 1997).

3. Collect Calibration Data

With the estimation of the model parameters, computer model is tested and compared with the data obtained by field observations. Data from the flushing tests, measurement readings at pump-station or tanks and telemetric data are used as field observations generally (Ormsbee and Lingireddy, 1997). In Figure 2.7, field test setup for the case study (Chapter 4) is shown. Pressure measurements are performed to measure the level of service and to collect data for calibration usually at fire hydrants but it can also be read at hose bibs, home faucets, pump stations and tanks. Whereas, flow is quantified to provide intuition for flow patterns, develop consumption data and define rate of flow for calibration at strategic locations of the system (Walski et al., 2003).

Figure 2.7. Field Test Setup for Data Collection

Data quality is an important issue during calibration. The model is not appropriately calibrated when a few pressures are measured and compared with the model results. The data collected should be assessed carefully. Walski (2000) defines three different degrees of usefulness for the collected data. Good data are the kind of data to be used and collected when significant amount of head loss occurs during the test. Walski (2000) states that head loss at a fire test should be at least five times larger than the error in head loss measurements. Bad data contain errors because of misread measurements, uncalibrated instruments, incorrect elevations and lack of information about boundary conditions. Bad data should be spotted and discarded. Useless data are collected when the head loss in the system is too low that the head loss is of a similar magnitude as the error in measurements.

Walski (2000) defined a guideline to promote proper collection and processing of field data as follows:

- Maximize head loss
- Use good pressure gauges
- Use accurate elevation data
- Know boundary condition at the time of the readings
- Measure pressure far from the boundary head
- Understand water demand patterns during the test
- Use HGL units to compare field data and model results

4. Sampling Design

Sampling design is known as a planning practice to determine the time, location of the data collection and under what conditions the data will be collected to deliver best results for model calibration (Walski, 2003). Walski (1983) suggested observing pressures and flows near the high-demand locations and on the perimeter of the skeletonized network. Lansey et al. (2001) designed data collection experiments by examining the change in the assessment measure under different measurement conditions. Meier and Barkdoll (2000) did a sampling design solution via genetic algorithm to find the combination of open hydrants that causes water to flow at non-negligible velocities. Kapelan et al. (2003) formulate the sampling design problem for the calibration of water distribution system hydraulic models as a constrained two-objective optimization problem. The objectives are as follows: maximization of the calibrated model accuracy by minimization of the relevant uncertainties; and minimization of total sampling design costs.

5. Evaluate the Model Results

Accuracy of a model can be assessed using criteria available in the literature. The desired level of calibration is directly affected by the intended use of the model. Eventually, calibration should be achieved to an extent that the related decisions will not be affected considerably (Ormsbee and Lingireddy, 1997). Calibration criteria in the literature will be discussed in the following sections.

6. Perform Macro-Level Calibration

There may be some situations that field observations and the simulated results may differ from each other excessively. It can be due to errors that are stated in the previous discussions. To identify such errors, the model should be investigated systematically (Ormsbee and Lingireddy, 1997). It would be thought that model calibration is a straightforward procedure. There may be too many errors associated with the initial, uncalibrated model and the many errors with field data. Some discrepancies can be just solved by the help of operation staff (Walski, 1990).

7. Perform Micro-Level Calibration

After large discrepancies are improved by performing a macro calibration, a micro calibration or fine-tuning is made to adjust pipe roughness values and nodal demand allocation. This is the final step of the calibration process (Walski et al., 2003).

Apart from this eight-step procedure, Engineering Computer Applications Committee (1999) produced a calibration guideline for water distribution system modeling which aims to provide a background for sources of errors; proposes some calibration guidelines and attempts to establish some criteria.

Moreover, Environmental Protection Agency (2005) released a reference guide for utilities covering many topics in water distribution system analysis including calibration of the models.

As a recent study, Speight and Khanal (2009) introduced model planning matrix developed to assist utilities in understanding the range of options for data collection and calibration for models in several categories: master planning, water quality, and advanced applications.

2.5. Calibration Accuracy

Regardless of the method used for calibration, a realistic model should achieve some level of performance criteria. The model should predict in general HGL values within 1.5-3.0 m, tank levels within 1-2 m, flows within 10-20 percent depending on the intended use and the size of the system (Walski et al., 2003).

In United Kingdom, a certain calibration criteria guideline has been established by Water Association Authorities and WRc (1989) on Table 2.3.

Table 2.3. Calibration Criteria in UK

Flow criteria	
a)	5% of measured flow when flows are more than 10% of total demand (transmission lines)
b)	10% of measured flow when flows are less than 10% of total demand (distribution lines).
Pressure Criteria	
a)	0.5 m (1.6 ft) or 5% of head loss for 85% of test measurements,
b)	0.75 m (2.31 ft) or 7.5% of head loss for 95% of test measurements
c)	2 m (6.2 ft) or 15% of head loss for 100% of test measurements

ECAC (1999) also developed a set of draft criteria for modeling purposes that are summarized below on Table 2.4 and Table 2.5. These are not definite standards; however they are published to start discussions on modeling needs (Environmental Protection Agency, 2005).

Table 2.4. ECAC Calibration Criteria for Modeling (a)

Intended Use	**Level of Detail**	**Type of Simulation**	**Number of Pressure Readings[1]**	**Accuracy of Pressure Readings**	**Number of Flow Readings**	**Accuracy of Flow Readings**
Long-Range Planning	Low	Steady-State or EPS	10% of Nodes	±5 psi for 100% Readings	1% of Pipes	± 10%
Design	Moderate to High	Steady-State or EPS	5% - 2% of Nodes	±2 psi for 90% Readings	3% of Pipes	± 5%
Operations	Low to High	Steady-State or EPS	10% - 2% of Nodes	±2 psi for 90% Readings	2% of Pipes	± 5%
Water Quality	High	EPS	2% of Nodes	±3 psi for 70% Readings	5% of Pipes	± 2%

[1] The number of pressure readings is related to the level of detail as illustrated on Table 2.5.

Table 2.5. ECAC Calibration Criteria for Modeling (b)

Level of Detail	Number of Pressure Readings
Low	10% of Nodes
Moderate	5% of Nodes
High	2% of Nodes

2.6. Automated Calibration

Darwin Calibrator uses genetic algorithm developed by Wu and Simpson (2001). GA first generates a population of trial solutions of the model parameters. A hydraulic model solver (Haestad Methods, 2003) then simulates each trial solution by predicting the HGL and flow values in the network. This information is passed back to the calibration module and the module evaluates how closely the model simulation is to the observed data by computing a fitness value, which is the difference between the observed data and the model predicted values. So one generation is completed. The fitness measure is taken into account when evaluating the next generation of the GA operation. To find the optimal calibration solutions, fitter solutions will be selected by mimicking the Darwin's natural selection principal of "survival the fittest" (Wu et al., 2002).

Walski (2004) evaluated the performance of automated calibration for water distribution systems. A real network computer model generated correct values for roughness, demands, flows and head to be used on the calibration of the system rather than using observed data. Walski concluded that when given true values of field measurements, the automated calibrator can correctly solve for the calibration parameters however, the user must keep in mind following issues (Walski, 2004):

1. There must be a reasonably large number of observations.
2. Observations must have head loss significantly greater than error in measurement in head loss.
3. The calibrator works best when a reasonable range of values for the unknowns is given.
4. When calibrator does not give good agreement, the results can be useful in identifying the source of the problem initially.

Wu and Walski (2005) proposed a progressive calibration procedure including generating sensible roughness adjustment grouping for the optimal model calibration by automated tools.

Walski et al. (2006) developed a small scaled network model in a laboratory to perform a calibration over that model by Darwin calibrator and resulted that automated calibration methods works well in estimating pipe roughness, demands and locating closed valves.

2.7. Current Practices in Calibration

Speight and Khanal (2009) conducted a survey among the ten US water utilities to get information about model and calibration usage currently in the industry. Population served by the participating utilities ranging from 50,000 people to more than 1 million. All the participating utilities have computer models for master planning and most of them use their models on daily basis. Since almost all utilities uses Hazen-Williams equation for modeling, most of them perform calibration for C-factors. For the parameters during calibration, utilities responded on many of the asked parameters including C-factors, valve settings, demand patterns, tank level etc. Most of them reported that they have developed in-house criteria for calibration. But, despite the advances in technology, water utilities in USA fall behind the current developments.

2.7.1. Automated Calibration for Large Systems

In 2001, Darwin Calibrator was used to calibrate water distribution model of the city Guayaquil. Guayaquil has a population of 2.3 million people located in Ecuador. The undertaker company that is responsible for operating and managing Guayaquil's water systems has adopted WaterGems and Darwin Calibrator for hydraulic network simulation. Applying Darwin calibrator enabled engineers to identify and quantify unaccounted-for-water and to save many trial-and-error hours (Wu et al., 2004).

In 2006, City of Sydney developed a water model and undertaken flow balance and hydraulic grade calibration. Tank levels and system demands are calibrated (Clark and Wu, 2006).

CHAPTER 3

AUTOMATED CALIBRATION

In this study, automated calibration software named as Darwin Calibrator is used to perform calibration of water distribution networks. Darwin Calibrator is an additional module for the hydraulic modelling software WaterCAD (Haestad Methods, 2003).

3.1. Calibration Objective

Calibration problem is simply adjusting roughness value of pipes and nodal demands in the order that the difference between field measurements (pressures and flows) and the simulated model results are minimized. In this study, GA produces the adjusted model parameters (roughness values and nodal demands) to achieve the minimum discrepancy between model results and field measurements by adjusting the mentioned parameters. This discrepancy is formulated in order to calculate the fitness of the solutions produced by GA. Darwin Calibrator can solve for three different fitness functions namely: (1) minimizing square differences, (2) minimizing absolute differences and (3) minimizing maximum absolute differences. Minimizing the sum of difference squares is used as objective function (F) in this study. The calibration objective can be formulated as below.

$Minimize$: $F(X)$

$Subject\ to$: $\overline{f_i} \leq fi \leq \underline{f_i}$

$\overline{m_i} \leq fi \leq \underline{m_i}$

where;

$$F\ (X) = \frac{\sum_{np=1}^{NH} w_{nh}\left(\frac{Hsim_{nh}-Hobs_{nh}}{Hpnt}\right)^2 + \sum_{np=1}^{NF} w_{nf}\left(\frac{Fsim_{nf}-Fobs_{nf}}{Fpnt}\right)^2}{NH+NF} \tag{3.1}$$

$Search\ for$: $X = (f_i, m_j)$ *i= 1,..., NI; j=1,..., NJ*

Here, X denotes for the set of model parameters (roughness and demand values) ; $\overline{f_i}$ and $\underline{f_i}$ is the upper and lower bounds for roughness factor in pipe group; $\overline{m_i}$ and $\underline{m_i}$ are the limits for the demand adjustment factor in pipe group *j; NI* is the number of roughness groups; *NJ* is the number of demand groups and $F\ (X)$ is the objective function.

Also here;

$Hsim_{nh}$ = nh-th simulated hydraulic grade,

$Hobs_{nh}$= nh-th observed hydraulic grade,

$Fsim_{nf}$ = nf-th simulated pipe flow,

$Fobs_{nf}$ = nf-th observed pipe flow,

$Hpnt$ = Hydraulic head per fitness point,

$Fpnt$ = Flow per fitness point,

w_{nh} = Weighting factor for observed hydraulic grades,

w_{nf} = Weighting factor for observed pipe flows,

NH = Number of observed hydraulic grades,

NF = Number of observed pipe flows.

w_{nh} and w_{nf} represent a normalized weighting factor for observed hydraulic grades and flows respectively (Wu et al., 2002); they are given as:

$$w_{nh} = Hobs_{nh} / \sum Hobs_{nh} \quad (3.2)$$

$$w_{nf} = Fobs_{nf} / \sum Fobs_{nf} \quad (3.3)$$

The weighting factors may also take many other forms, such as no weight (equal to 1), linear, square, square root and log functions (WaterCAD User Manual); it is taken as 1 in this study.

Hydraulic head/flow per fitness point ($Hpnt$ / $Fpnt$) enables multi objective optimization by providing an approach to weigh the relative importance or impact of both type of differences (head and flow) between model results and field tests; it is also introduced as dimensionless into the formulation (Equation 3.1). In default, they are set as 0.3 m and 0.63 l/s in Darwin Calibrator. In terms of calibration, a pressure within 0.3 m of a measured pressure is as good as a flow within 0.63 L/s of a measured flow (Bentley Systems, 2012). In order to define these values, the precision of the measuring instruments should be considered. Head/flow per fitness point should not be lower or higher than the accuracy of the measuring instruments. Generally, digital output from related instruments provides data at this accuracy.

Darwin Calibrator uses genetic algorithm method to achieve the minimization of the objective function.

3.2. Genetic Algorithm Optimization

Genetic Algorithm (GA) technique is a computational tool developed to help mathematical programming problems (Lingireddy and Ormsbee, 1999). GA reaches to the most favourable answer by imitating the mechanism of natural selection.

GA optimization starts with coding the decision variables called as “genes”. Each increment in the possible solution set can be coded as binary numbers (genes) in the upper and lower bound limits of the solution set (Goldberg, 1987). Assume that there is a calibration problem to adjust the roughness values of the system shown in Figure 3.1.

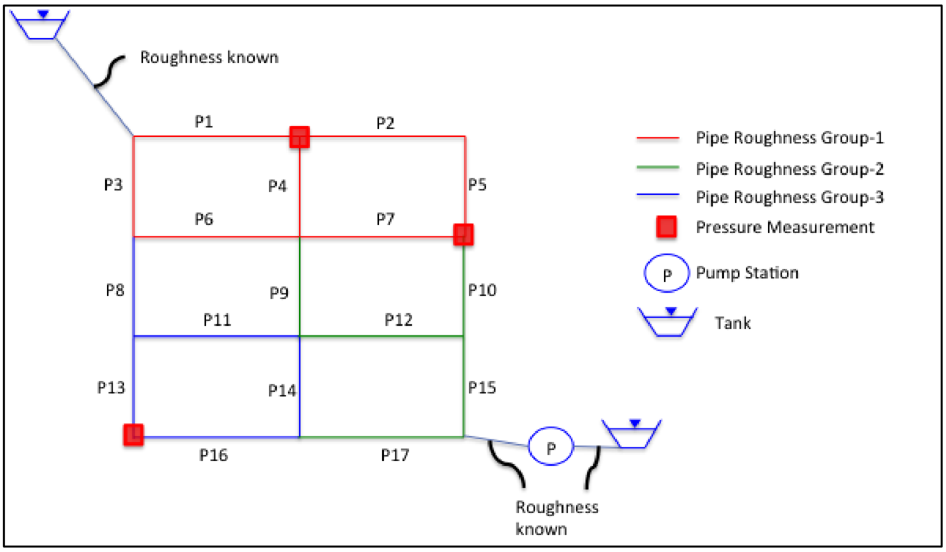


Figure 3.1. Network Layout for GA Example

For different pipe roughness values, unique binary numbers are assigned randomly as shown on Table 3.1.

Table 3.1. Representative Binary Codes for Pipe Roughness Values

Pipe Roughness	Binary Code
60	000
70	001
80	010
90	011
100	111
110	100
120	110
130	101

Then, GA generates an initial population of solutions (chromosomes) using a random number generator. Random number generator assigns either 1 or 0 to each bit position for nine character strings for three pipe roughness groups. These strings are called as chromosomes and shown on Table 3.2.

Table 3.2. Initial Population Solutions (Chromosomes) Generated by Random Generator

	Roughness Group-1			Roughness Group-2			Roughness Group-3			
1	1	1	1	0	0	1	1	0	0	1st chromosome
2	0	0	1	0	0	0	1	0	1	2nd chromosome
3	1	1	0	1	0	0	0	1	1	3rd chromosome
.										
.										
.										
.										
.										

Next step is computing the objective function. Each gene (binary code) is converted to the corresponding pipe roughness value and the hydraulic solver computes the variable (hydraulic grades) in the objective function for each solution (chromosome) in the population and passes it to GA processors. Then GA calculates the objective function, in other words the fitness of the each trial solution accordingly.

Now, GA operators are used to reproduce offspring solutions. There are mainly three operators: selection, crossover and mutation.

Selection

The probability that a string is selected to reproduce offspring solution is based on its level of fitness. GA selects fittest solutions by using a method known as roulette-wheel selection (Goldberg, 1989). The theory replicates the natural selection process that fitter individuals will have a higher probability to survive and be used for future generations. Thus the roulette wheel slots are sized according to the computed fitness of each solution (Figure 3.2). The number of times the roulette is spun equals to the size of the population. The solutions that are selected by the roulette will be used for breeding the next generation solutions.

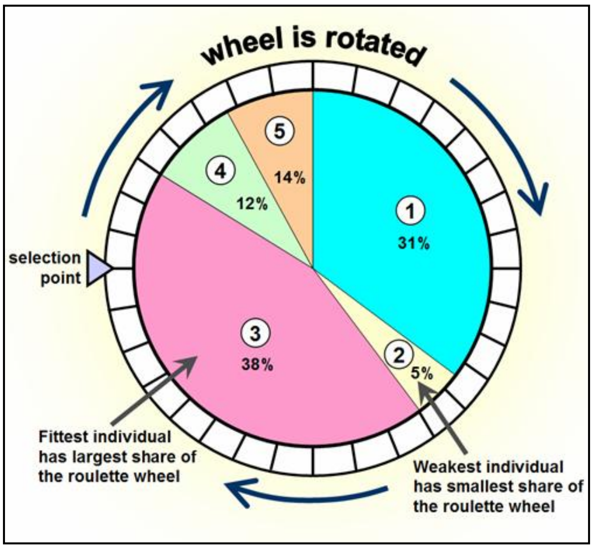


Figure 3.2. Roulette-Wheel Selection (Newcastle University Engineering Design Centre, 2012)

Crossover

Next, the crossover operator is applied to exchange bits between two parent strings in order to form two child strings (Figure 3.3). There is no fixed method to execute crossover. However, the only general procedure is to transfer the genetic material from parents to forward, introducing enough variation, to enable them to become fitter.

	Roughness Group-1			Roughness Group-2			Roughness Group-3			Fitness
Parent-1	1	1	1	0	0	1	1	0	0	6.82
Parent-2	0	0	1	0	0	0	1	0	1	8.48
Child-1	1	1	1	0	0	0	1	0	0	5.06
Child-2	0	0	1	0	0	1	1	0	1	8.95

Figure 3.3. Crossover

Mutation

Mutation creates a new chromosome simply changing some part of it. If the change is beneficial, the new chromosome is carried forward on the other hand if the change causes a weakness then it is likely that the individual will die out. GA operators do mutation by changing 0 to 1 or vice versa bit by bit considering the user defined mutation probability as shown in Figure 3.4.

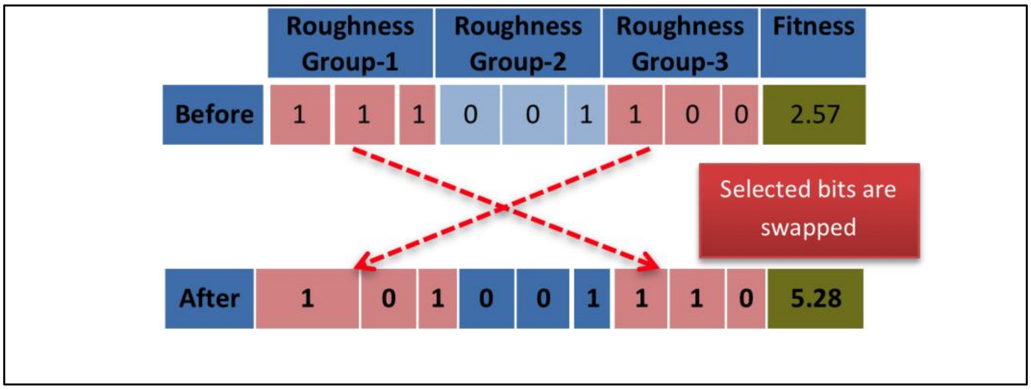


Figure 3.4. Mutation

So the first generation step is completed. The GA then repeats the same steps till the fittest solution or the termination criteria are achieved (Newcastle University Engineering Design Center, 2012).

There are a few options for the termination criterion. If one of the following criteria is satisfied, Darwin Calibrator stops to run:

1. User specified fitness tolerance: Solver stops, if the desired fitness tolerance is achieved.

2. Maximum number of iterations: if the maximum number is exceeded, solver displays the last solution as result.

3. Maximum number of non-improvement generations: if solver cannot improve fitness anymore, it stops.

A usual GA optimization can be summarized as follows (Walski et al., 2003) (Figure 3.5):

1. Initial population of solutions (chromosomes) is generated randomly.

2. Fitness values of the solutions in the initial population are computed.

3. New populations are generated using operators that are inspired by genetic transformation (selection, crossover and mutation).

4. Fitness values of new solutions are calculated.

5. Step 3 and 4 are repeated till the termination criterion is reached.

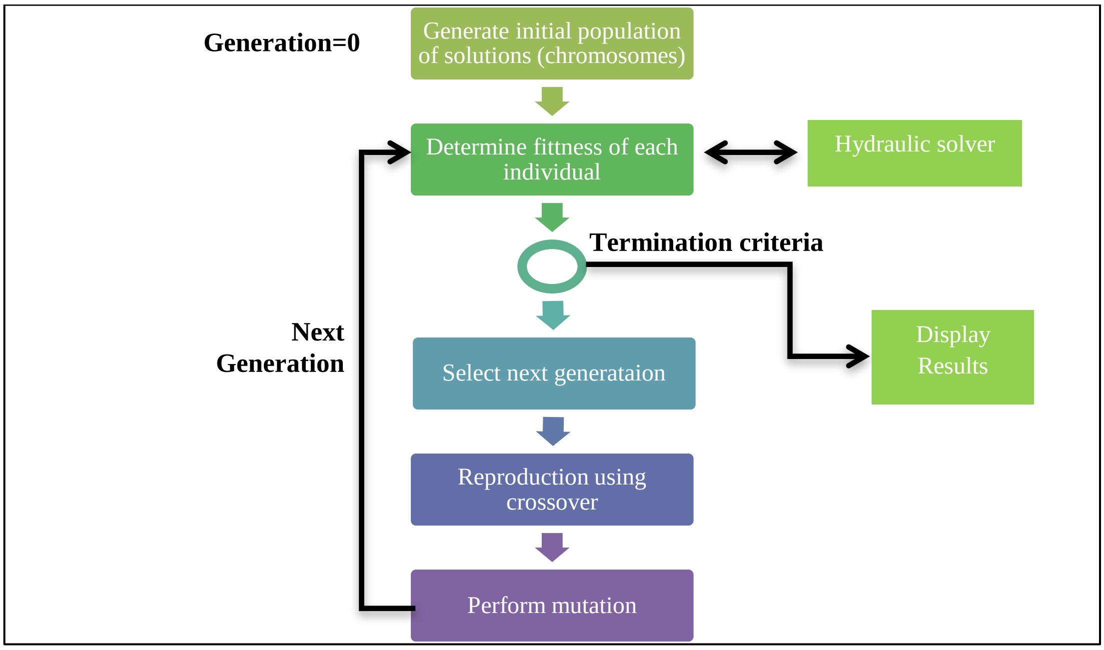


Figure 3.5. Basic Genetic Algorithm for Calibration (Newcastle University Engineering Design Center, 2012)

Advantages of genetic algorithms over traditional optimization techniques can be summarized as (Goldberg, 1987):

1. It simultaneously evaluates optimal solution vectors. In many search methods, solutions are explored from point to point in the decision space by using decision rules. However, GAs start from an initial sets of solution (chromosomes) shifting to many sets in parallel thus reducing the probability of a false solution.

2. It does not require gradient information. Regardless of the starting population, genetic algorithm is applied generation by generation using objective value information and randomized operators guide the creation of new offspring populations. It just requires objection function value.

3. It employs probabilistic rules; therefore being not deterministic, it can assure a robust solution.

3.3. Automated Calibration Software

Constructing the hydraulic model is the first step of automated calibration. Hydraulic models can be easily created using WaterCAD. WaterCAD can be employed to analyze and design water distribution networks and can be used for operational purposes. To construct a hydraulic model with WaterCAD, firstly, system components such as reservoirs, pipes, junctions, valves, and pumps are created; afterwards, properties of these components (roughness values, elevations, demands etc.) are assigned. WaterCAD can solve the network hydraulics and display pressures, hydraulic grades, pipe velocities etc.

Next step is inserting the recorded field data to the Darwin Calibrator. Darwin Calibrator allows the user to enter the following field data; observed measurements, boundary conditions and nodal demand adjustments. Observed measurements are hydraulic grades, pressure values and pipe flows that are spotted during field tests. Boundary conditions can be described as tank levels, status of pumps and valves. Nodal demand adjustments allow the user to define additional flows such as fire flow flushed during field experiments.

Next stage, deciding on adjustment groups, is one of the most important issues for automated calibration. It is not likely that all pipes and nodes in a system will have totally different roughness values and nodal demands. So every single pipe and node should not be calibrated separately. Instead they should be grouped to have a successful automated calibration. Grouping reduces the size of the problem, makes it possible to find the optimal solution and avoids issues where several identical pipes would end up with very different roughness values because of small inaccuracies in field measurement (Walski et al., 2006). In conclusion, it is better to group similar elements (pipes and nodal demands) and to calibrate those groups rather than calibrating each pipe or each nodal demand separately. Wu et al. (2002) has observed that when the number of unknowns greatly exceeds the number of useful observations, there is little confidence in the calibration results. It is because that there are too many solutions that can match the observed flows and heads.

In most of the studies, pipes are grouped according to their diameters. Grouping with respect to pipe diameters may be reasonable if their velocities are at the same ranks. As shear stress at pipe wall is directly related with velocity, pipes with same velocities can be assumed to have similar physical characteristics at their inner pipe walls. Because of that grouping according to the pipe diameters may lead to incorrect solutions. Furthermore, velocities of pipes decrease significantly at the far end regions of the networks. As the velocity decreases, self-cleaning of pipes occurs at much more lower levels which means that more materials will accumulate at the pipe wall. In this case, pipes with the same size can transmit totally different flows and may have different physical conditions. Because of the reasons mentioned above, in this study, pipes are grouped according to their velocities at the normal flow condition.

Furthermore, Darwin Calibrator allows you to set up calibration criteria. Calibration criteria specify how the calibration is evaluated. The Calibration Criteria contains the following controls (Bentley Systems, 2011):

- Fitness Type
- Head/Flow per Fitness Point
- Flow Weight Type

It is possible to increase fitness by adjusting these controls. But increasing fitness in this way does not mean that, your calibration results get more accurate. There may be millions of solutions that can match the observed data. But the virtue for having accurate calibration lies behind the trustable field measurements and most accurate hydraulic model.

3.4. Performance of Automated Calibration

Prior to the case study (Chapter 4), several automated calibration runs are performed to learn and understand the calibration software better. In this manner, calibration capabilities of the software are also questioned. Accuracy of the calculations of Darwin Calibrator holds vital importance for the results of this study.

Key question is that in what levels of accuracy can the Darwin Calibrator do calibration calculations? Regardless of the calibration method, to be able to trust calibration results of a study, we must be sure about the observed field data and hydraulic network model. Because calibration is a process that field data and hydraulic models results are matched, any error in the field data or hydraulic model will yield to incorrect results.

Assume a case in which field data are hundred percent correct. All the measurements are accurate and precise. Hydraulic network model is totally compatible with the constructed network that is laid under the streets. Also we are sure about the nodal demands. Since the observed field data sets and the constructed hydraulic model are perfectly accurate, calibration results of the software should be precise. This specific case can be created as follows: pipe roughness is assigned to a reasonable value; in addition, a hydrant flow is introduced. The model is run and results of this application of the model (hydraulic grades, flows) with the assigned pipe roughness value and hydrant flow are taken as the field data set. This kind of data is also called as synthetic data. When automated calibration is performed with the synthetic field data set, it is expected that Darwin Calibrator should yield results that matches with the assigned roughness values and flows used to generate the synthetic data.

Above procedure is applied to Yayla Subzone of Ankara N8.3 distribution network (Figure 3.6). This network will also be used for the case study. Two different cases are created to study the performance of the calibration software. In both of cases, an additional flow of 50 l/s is flushed from the hydrant and pipe roughness value is assumed as 75 for all pipes while generating synthetic data. First case consists of pressure readings at the fire hydrant and at the inlet of the system (measurement chamber) and flow reading at the inlet (Figure 3.6). On the other hand, in the second case, number of readings is increased as shown in Figure 3.7. In addition to readings in the first case, four additional pressure and flow readings are provided in the second one. It is expected that by increasing number of readings better calibration results will be achieved.

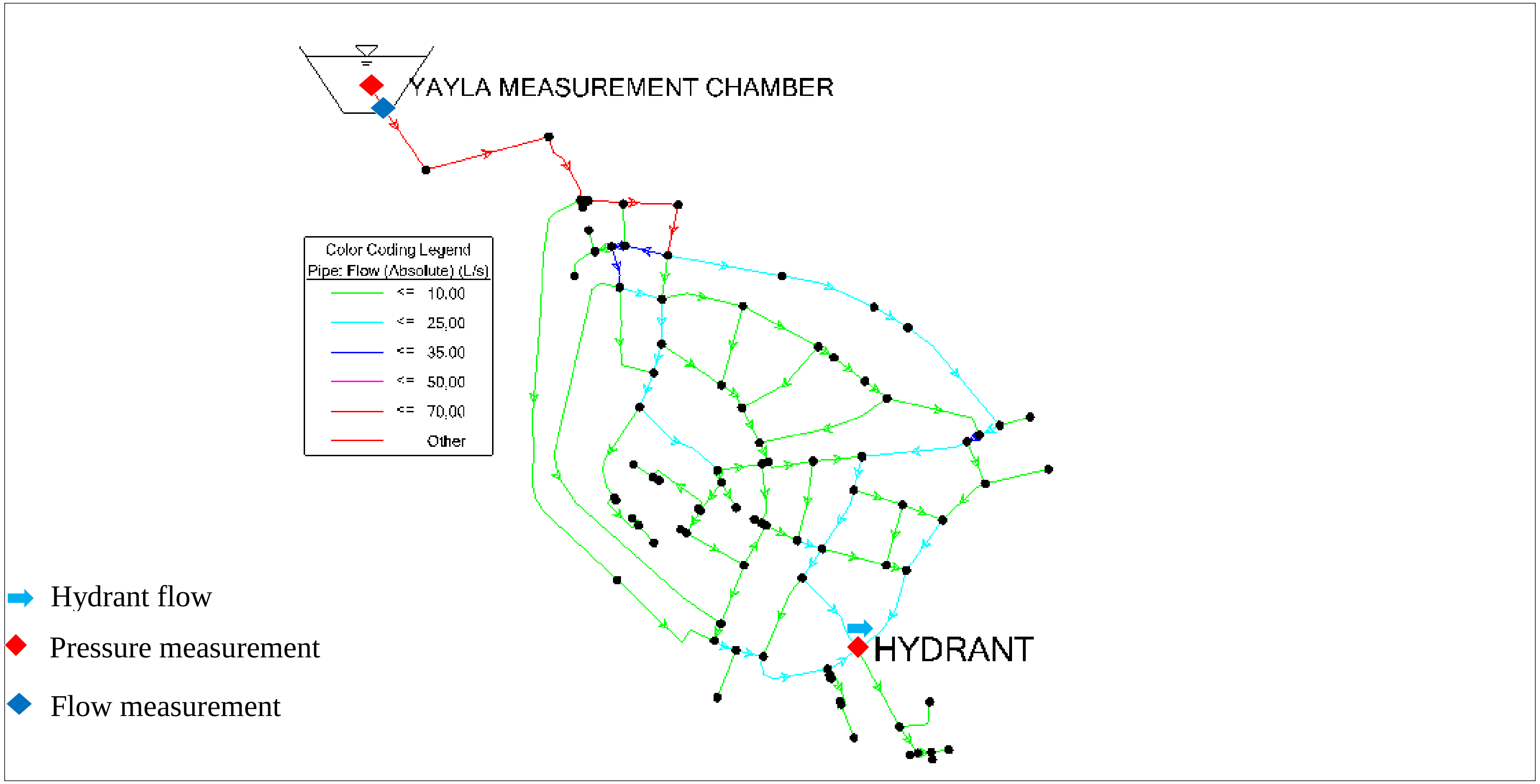


Figure 3.6. Synthetic Data Generation-Case 1

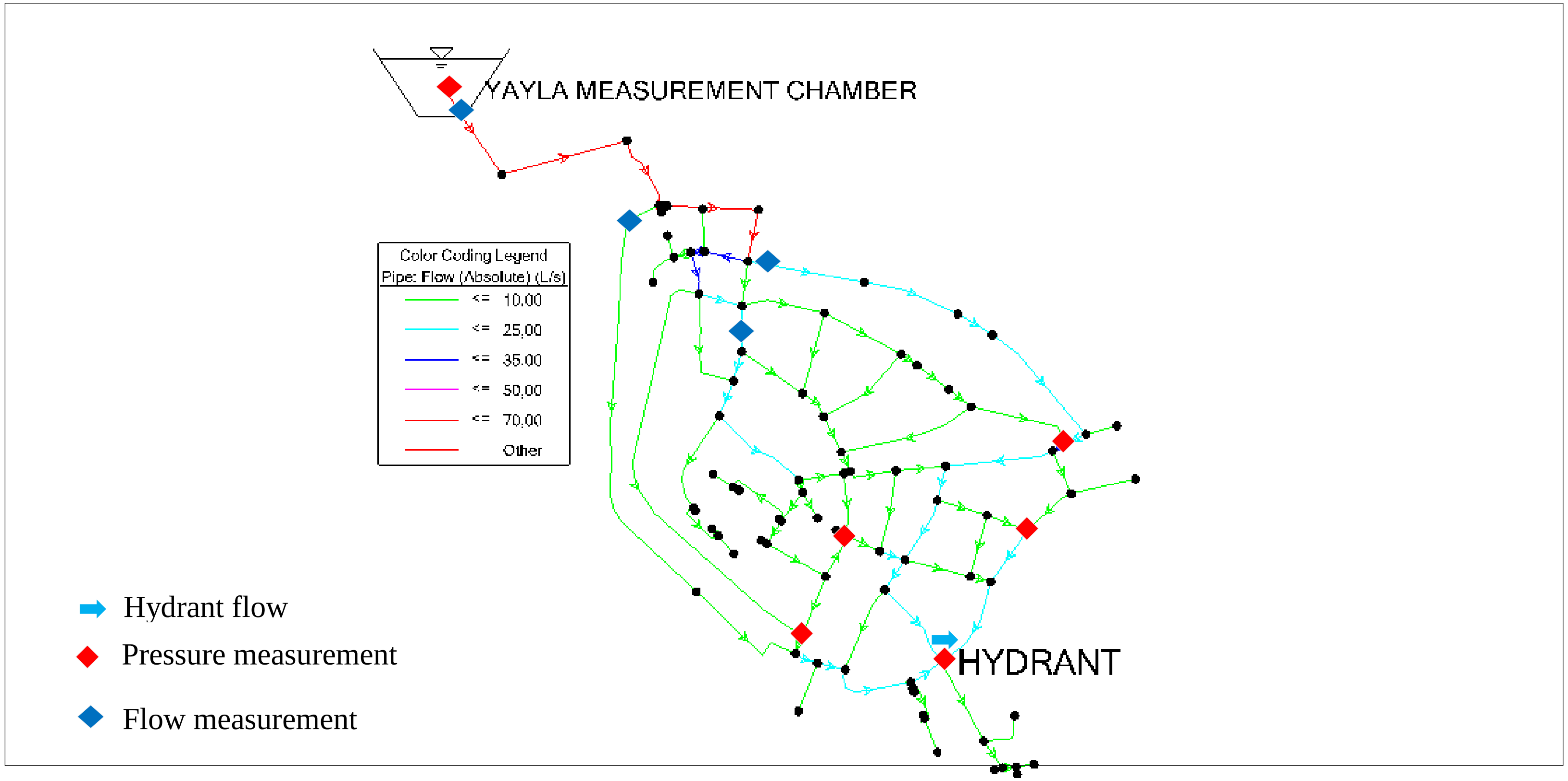


Figure 3.7. Synthetic Data Generation- Case 2

In order to simulate observed data; synthetic data are created by using a roughness value of 75. But the model is constructed with a pipe roughness value of 130.00 as an initial estimate. It is expected that Darwin Calibrator should calculate the adjusted roughness as 75.00. Results are presented below (Table 3.3).

Table 3.3. Results of Calibration Calculations with Synthetic Data Sets

Case	Model Roughness	Adjusted Roughness	True Roughness	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
1	130.00	74.09	75.00	1,135.78	1,135,78	0.00
2	130.00	73.76	75.00	1,148.47	1,148.58	0.11
				1,146.54	1,146.66	0.12
				1,148.53	1,148.65	0.12
				1,145.85	1,145.93	0.08
				1,135.78	1,135.61	-0.17

It is understood that Darwin Calibrator can solve for roughness values easily and get very close to the true value if the correct field data is provided; however, Darwin Calibrator cannot reach the exact true value. It may be because of the nature of the GA. This issue is also discussed with one of the developer of the software (Walski, 2012). It is mentioned that yielded result is in the range of the capabilities of the software.

Moreover, alternative scenarios are produced on Case-1. Some errors are introduced to the synthetic data to gain information on the sensitivity of the Calibrator to data quality. Solutions are presented on Table 3.4.

Table 3.4. Summary of Darwin Calibrator Results with Data Errors

True Roughness	Error At Pressure Reading (m)	Adjusted Roughness	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Diff. (m)	Fitness Ratio
75.00	0	74.09	1,135.78	1,135.78	0.00	3.90
	-0.10	73.90	1,135.68	1,135.67	-0.01	3.90
	-0.50	73.15	1,135.28	1,135.29	0.01	3.89
	-1.00	72.23	1,134.78	1,134.79	0.01	3.90
	-5.00	65.86	1,130.78	1,130.78	0.00	3.90

As the quality of the data decreases, adjusted roughness is diverged at distant numbers from the true roughness value as expected. But up to a certain range of error in the data, solutions are acceptable.

CHAPTER 4

CASE STUDY:
CALIBRATION OF N8.3 PRESSURE ZONE OF WATER DISTRIBUTION NETWORK, ANKARA

4.1. General Information about N8.3 Water Distribution Network and the Case Study

N8.3 pressure zone is located at the northern part of Ankara, within the boundaries of Keçiören and Yenimahalle counties. Treated water is supplied from pump station P23 to six DMA's (district metered areas) and stored at the tank T53. General layout of N8.3 pressure zone network is referenced on a satellite view (Google Inc., 2009) in Figure 4.1. This zone serves for approximately 50,000 people. Total population is distributed to the subzones in proportion with their service areas (Table 4.1).

Table 4.1. Populations of DMA's in N8.3 Pressure Zone

DMA	Population
Yayla	10,228
Northern Sancaktepe	7,756
Southern Sancaktepe	5,248
Şehit Kubilay	11,161
Upper Çiğdemtepe	7,791
Lower Çiğdemtepe	7,816

Topographically, this zone is located on an undulating territory, having elevations between 1075 m and 1120 m above sea level. It is located at the northern part of the city, deemed to be quite away from the city centre. Thus, the region is still under development: (i) mostly, residential and small commercial areas constitute the zone; there are also schools and green areas; but there are no industrial regions, (ii) economical level of the society living in the zone is low. Therefore, water demand amounts occur usually at low levels. But in the last years, multi-storey buildings started to replace single storey buildings as a result of revised zoning plans and population growth. Hence, the water consumption rates are increasing. Total demand of the system in the noontime is recorded as 65 l/s approximately. In fact, the daily demand curve (DDC) should be constructed in the calibration studies. In this study DDCS could not be measured because of the limitations of the water authority Ankara Water and Waste Water Administration (ASKI). Instead, instantaneous pressure and discharge measurements have been carried out. Discharges recorded during field tests of this study are distributed to the junctions (demand nodes) by using methodology explained in Chapter 2. DDCs of N8.3 Pressure Zone DMAs (Şendil, 2013) are given in Chapter 2.

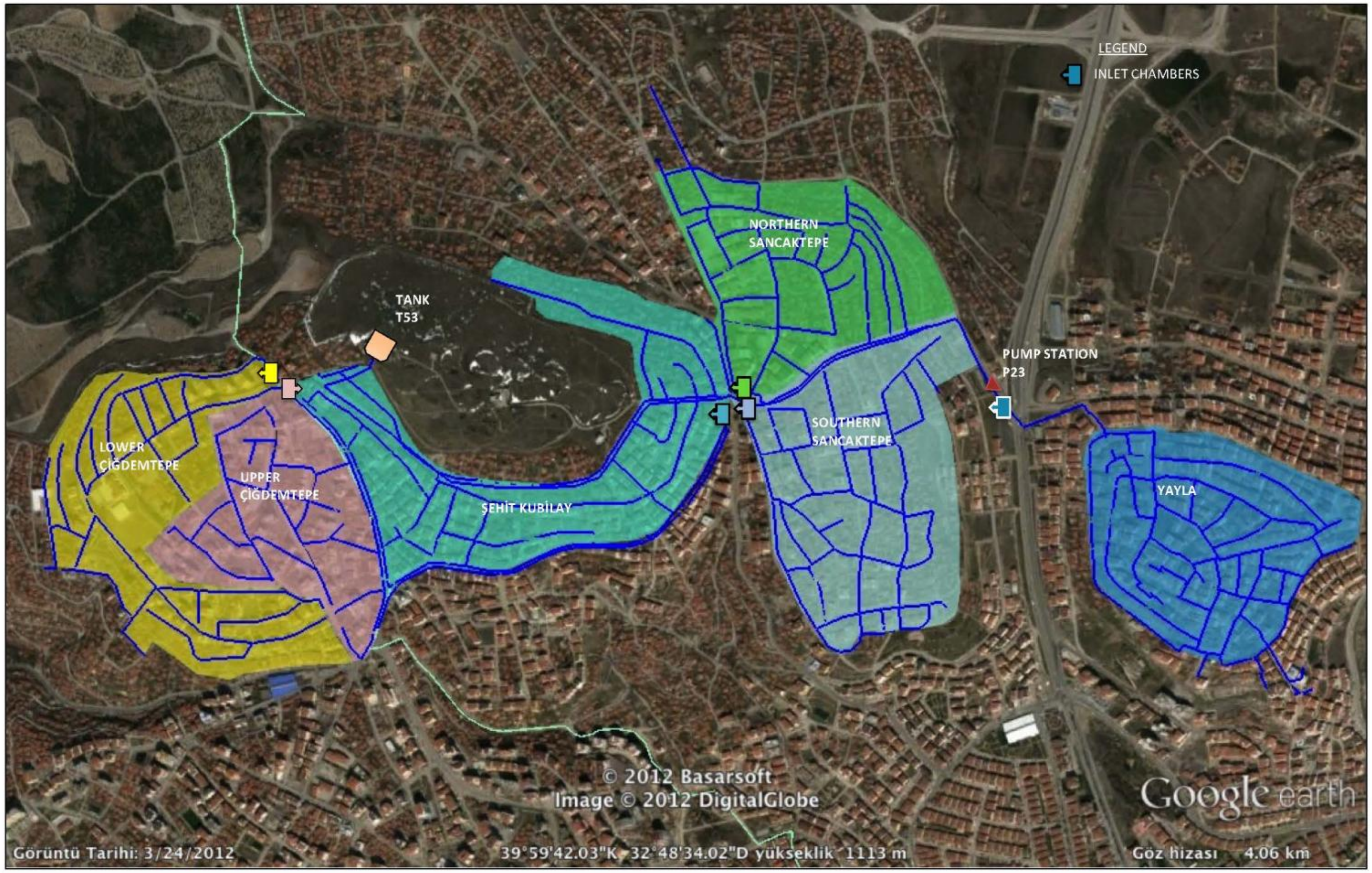


Figure 4.1. N8.3 Pressure Zone-General Layout

ASKI is a governmental institution and it is responsible for the water supply and distribution system of the city. Recently, ASKI carried out field exploration studies especially at N8.3 and N7 networks. These exploration studies aimed to locate exactly pipes and valves; then these studies were followed by digital network mapping works. All the system components are digitized using appropriate GIS tools. Furthermore, during the explorations studies unknown underground valves were also discovered. Consequently, all these improvements led to a better network model. With a better network model, engineers had chance to carry out studies and modifications to operate network more efficiently. Earlier, the whole pressure zone was interconnected with other zones. With the contribution of researchers and technical staff of ASKI, system has been divided into six DMA (subzones), basically for leakage control purposes. After having applied necessary valve operations, water is directed into the DMAs through a single path and isolated subzones are created (Figure 4.1). At the inlet of each subzone, inlet chambers are constructed for controlling and measuring purposes. Mobile flow meters and pressure gauges can be easily installed at these chambers for pressure and flow monitoring.

Moreover, infrastructural exploration studies and digital mapping works mentioned above enabled to carry out different hydraulic studies such as leakage detection (Özkan, 2001), calibration (Ar, 2011), and pump scheduling (Şendil, 2013). Also, Bektas (2010) completed a leakage management study, which was started in Ankara but then carried out for Antalya. This calibration study is realized as advancement to Ar (2011) calibration work. In this study genetic algorithm (GA) is used. For this calibration study, field tests are conducted at three different locations of each subzone. At each inlet location, both pressure and flow are measured. Additional pressure measurement locations are fire hydrants and a characteristic location in the surrounding. This third point is chosen at the far end of the system and on one of the main flow paths so that maximum pressure drops are experienced during the hydrants were flushing.

Due to problems raised during field studies, not all the subzones could be calibrated. Only Yayla, Upper Çiğdemtepe and Northern Sancaktepe subzones are calibrated since the field tests results of the other subzones have to be ignored as they hold significant errors. Field data collection requires serious organizational arrangements. Data collection is planned several days before going to the field. Staff from the water utility and vehicles to reach the study area is to be prearranged. Also, hydrants are flushed in advance to check the collection setup. Therefore, it is very difficult to repeat the field tests, in case that a problem is faced. Limitations of ASKI also prevent repeating the tests.

4.2. Calibration of Water Distribution Network of Yayla Subzone

Yayla Subzone is located at the east side of the N8.3 pressure zone. At the time of the tests, the average flow rate in the system was 13.06 l/s. Field measurements are conducted at the locations shown in Figure 4.2; these readings (recorded at 10:00 am on 03.05.2012) are given on Table 4.2. Rate of flow that is flushed from the hydrant is not measured by a flow meter. In fact, it is assumed that the hydrant flow is equal to the difference of measured flows during so called high and low flow conditions at the inlet location. P1 and P2 test nodes are placed on the main flow carrying paths as seen in Figure 4.3.

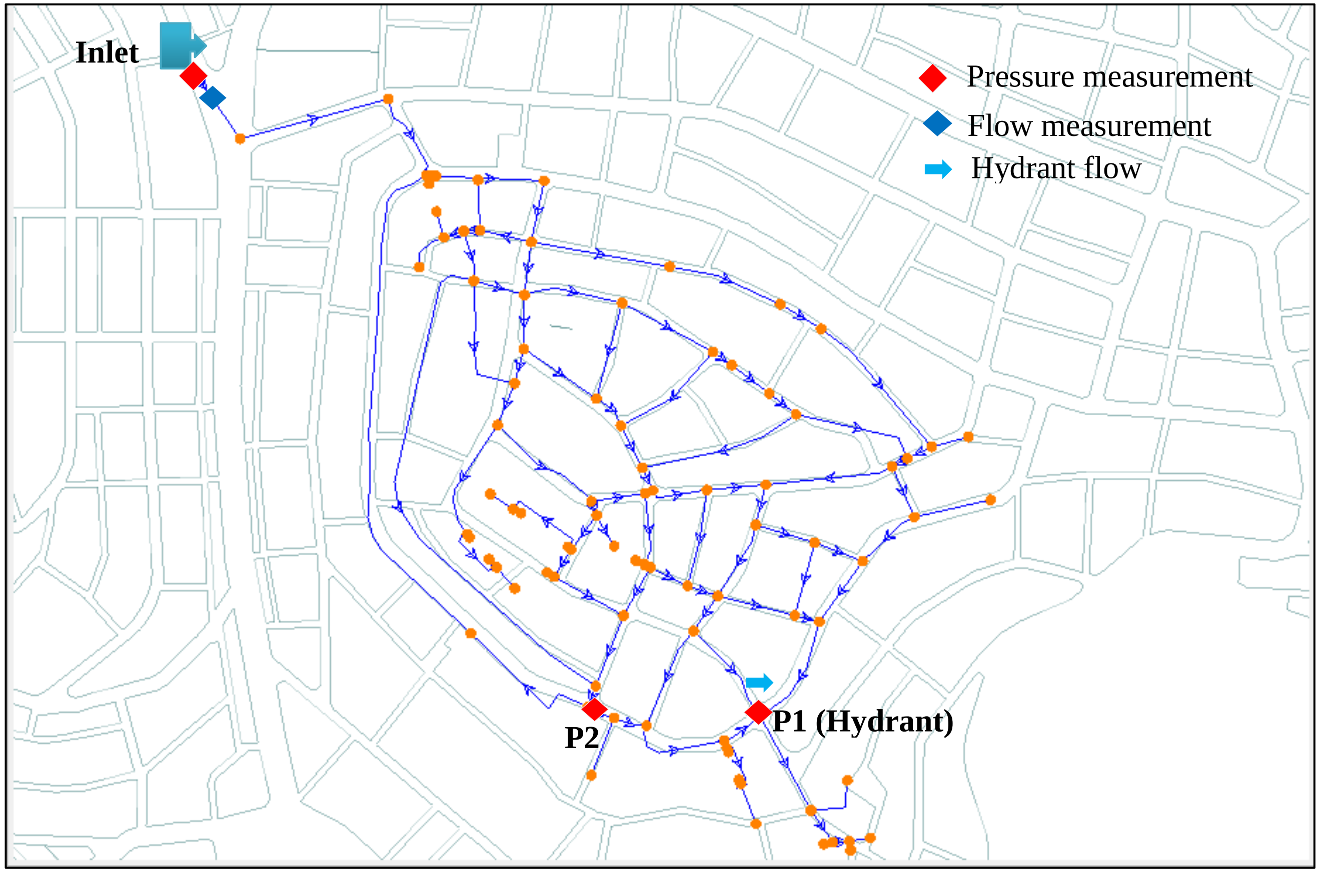


Figure 4.2. Hydraulic Model and Measurement Locations for Yayla Subzone

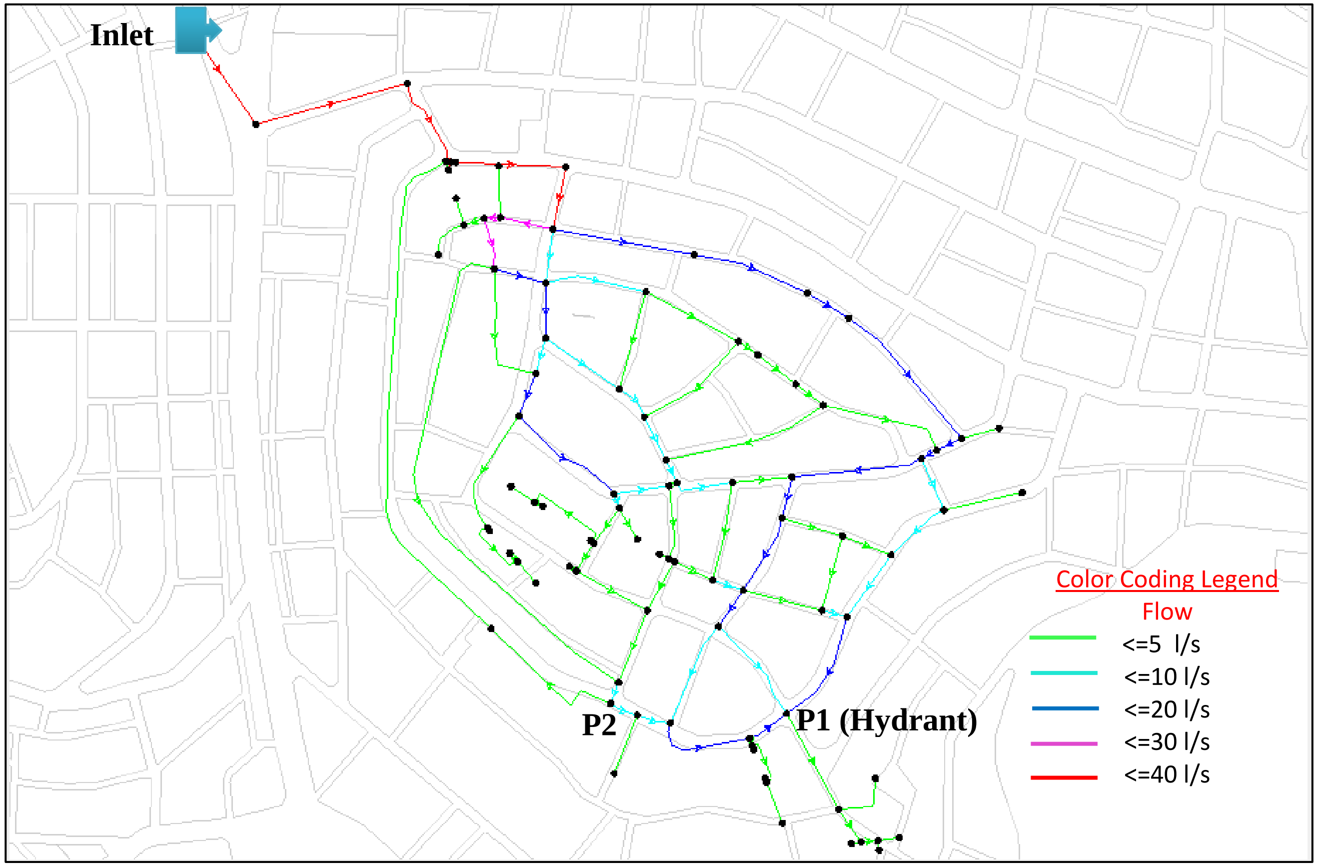


Figure 4.3. Pipe Flows of Yayla Subzone at Fire Flow Condition

At P1 and P2, 14.28 m and of 8.03 m pressure drops are experienced during high flow condition respectively.

Table 4.2. Pressure and Flow Readings for Yayla Subzone

TEST NODE	Elevation (m)	LOW FLOW			HIGH FLOW		
		Pressure (m)	Hydraulic Grade (m)	Flow (l/s)	Pressure (m)	Hydraulic Grade (m)	Flow (l/s)
INLET	1,056.41	96.89	1,153.30	13.06	96.89	1,153.30	45.00
P1 (Hydrant)	1,088.22	67.31	1,155.53	0.00	53.03	1,141.25	31.94
P2	1,092.31	66.00	1,158.31	-	57.97	1,150.28	-

Case 1: Single Roughness Group

Calibration results are presented on Table 4.3.

Table 4.3. Calibration Results for Yayla Subzone – Case: 1 Single Pipe Roughness Group

Adjustment Group	Calibrated Hazen-Williams C	Test Data	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
All Pipes	70.10	P1 (Hydrant)	1,141.25	1,141.24	-0.01
	116.90	P2	1,150.28	1,150.28	0.00
	79.60	P1	1,141.76	1,143.77	2.01
		P2	1,150.28	1,147.14	-3.14

Calibrated roughness value using the P1 data is approximately same as the result of the study carried by Ar (2011). Ar (2011) found roughness value 68.80 for Yayla Subzone. On the other hand, calculated C-factor value by using the P2 data is found to be higher than the one calculated by using the P1 data. This is due to the fact that observed hydraulic grade at P2 is considerably higher than P1. In fact, two test locations are not distant from each other. This variation may be because of various reasons (i) one of the pressure-measuring tools may be non-calibrated or malfunctioning (ii) Also, there may exist not completely open valves that are currently unknown. (iii) Furthermore there may occur excessive leakage at the portions of the network, which have significant influence on P2 readings. Calibration performed by using P1 and P2 data together presented the C-factor as 79.60.

Case 2: Pipe Roughness Grouping With Respect to Pipe Velocities

In Figure 4.4, it is seen that average velocity at the Yayla network is around 0.08 m/s. Therefore pipes are grouped into two; pipes having velocity greater than 0.08 m/s and pipes having velocity lower and equal to 0.08 m/s. This also maintains a continuous line between the inlet and the test location. Grouping pipes above and below velocity of 0.08m/s have yielded unreasonable results. The same criterion is also applied for the other two subzones. Calibration results are given on Table 4.4.

Table 4.4. Calibration Results for Yayla Subzone- Case 2: Pipe Roughness Grouping With Respect to Pipe Velocities

Adjustment Group	Hazen-Williams C	Test Data	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
V > 0.08 m/s	76.80	P1 (Hydrant)	1,141.25	1,141.26	0.01
V ≤ 0.08 m/s	65.30				
V > 0.08 m/s	100.40	P2	1,150.28	1,150.20	-0.08
V ≤ 0.08 m/s	212.10				
V > 0.08 m/s	356.90	P1&P2	1,150.28	1,150.23	-0.05
V ≤ 0.08 m/s	47.60		1,141.25	1,141.29	0.04
V > 0.12 m/s	50.70	P1 (Hydrant)	1,141.25	1,141.16	-0.09
V ≤ 0.12m/s	100.00				
V > 0.04 m/s	57.70	P1 (Hydrant)	1,141.25	1,141.25	0.00
V ≤ 0.04m/s	400.00				

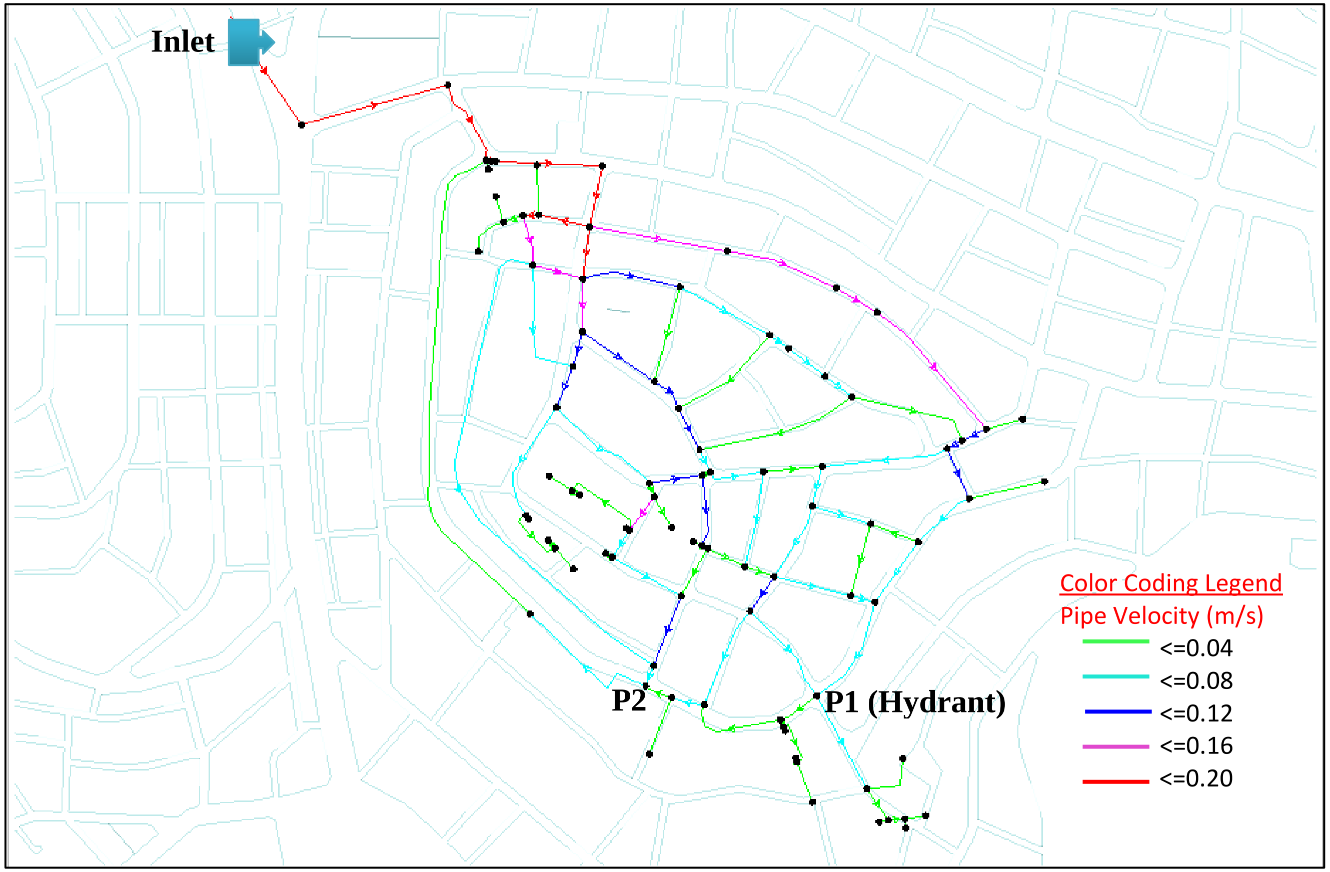


Figure 4.4. Pipe Velocities of Yayla Subzone at Normal Flow Condition

The results that are obtained by using P1 data are reasonable as roughness value for the pipes having greater velocity is expected to be higher. Also outcomes of P1 data are compatible with Case 1 results. However, results using the P2 data are problematic; Pipes having velocities greater than 0.08 m/s have lower C-factor value than the ones with lower velocities. Besides the C-factor value for the pipes having velocity lower than 0.08 m/s is 212.10, which is irrelevant. It can be said that the uncertainties and the disconformities along the portions of the network affecting the P2 node readings may be considerably higher. And there may be some problems with the measurements. It can also be seen that calibration by using P1 and P2 data together has resulted in irrelevant figures because of the reasons mentioned above.

4.3. Calibration of Upper Çiğdemtepe Subzone Water Distribution Network

Upper Çiğdemtepe is a relatively small network compared to the other ones with respect to pipe length and service area. Network model is presented in Figure 4.5. At the initial studies, it is discovered that at the far ends of the network, desired pressure drop during high flow condition cannot be achieved. It is because of the low demand condition of the subzone. So P2 node is shifted to the inner side of the network, close to the hydrant by necessity. Results of field tests are presented on Table 4.5 (recorded at 10:00 am on 10.05.2012).

Table 4.5. Pressure and Flow Readings for Upper Çiğdemtepe Subzone

TEST NODE	Elevation (m)	LOW FLOW			HIGH FLOW		
		Pressure (m)	Hydraulic Grade (m)	Flow (l/s)	Pressure (m)	Hydraulic Grade (m)	Flow (l/s)
INLET	1,115.28	40.80	1,156.08	6.58	40.80	1,156.08	24.11
P1 (Hydrant)	1,115.14	41.82	1,156.96	0.00	28.05	1,143.19	17.53
P2	1,104.18	49.18	1,153.36	-	45.54	1,149.72	-

Case 1: Single Pipe Roughness Group

For Upper Çiğdemtepe Subzone, roughness value using P1 data is found to be 72.00, which is compatible with result of Yayla subzone. Ar (2011) found 31 for the same subzone. Reasons for the difference between the results of P1 and P2 data may be due to consistency degree of network model (valve conditions, pipe connections etc.) or may be due to the measuring instruments. Results are presented on Table 4.6.

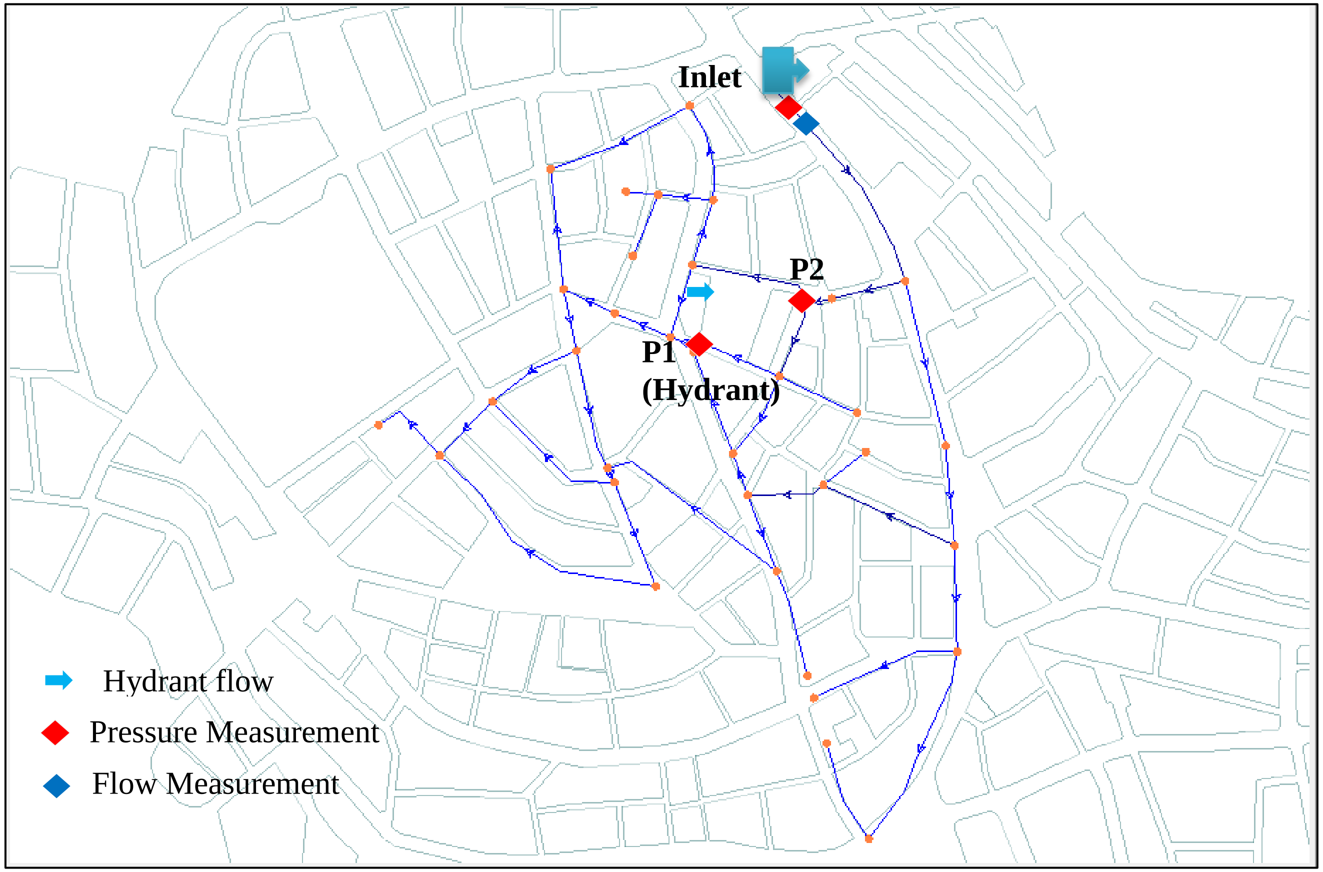


Figure 4.5. Hydraulic Model and Measurement Locations for Upper Çiğdemtepe Subzone

Table 4.6. Calibration Results for Upper Çiğdemtepe Subzone – Case 1: Single Pipe Roughness Group

Adjustment Group	Calibrated Hazen-Williams C	Test Data	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
All Pipes	72.00	P1 (Hydrant)	1,143.19	1,143.21	0.02
	94.80	P2	1,149.72	1,149.73	0.01
	79.10	P1&P2	1,143.19	1,145.26	2.08
			1,149.72	1.147.20	-2.52

Case 2: Pipe Roughness Grouping With Respect to Pipe Velocities

It can be seen in Figure 4.6 that most of the pipes, which are located along the main flow line (between the inlet and the hydrant) have velocities between 0.08 m/s and 0.12 m/s. So the pipes are grouped in accordance with 0.08m/s velocity (Table 4.7).

Table 4.7. Upper Çiğdemtepe Calibration Results – Case 2: Pipe Roughness Grouping With Respect to Pipe Velocities

Adjustment Group	Calibrated Hazen-Williams C	Test Node	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
V > 0.08 m/s	72.70	P1 (Hydrant)	1,143.19	1,143.19	0.00
V ≤ 0.08 m/s	52.90				
V > 0.08 m/s	94.70	P2	1,149.72	1,149.72	0.00
V ≤ 0.08 m/s	78.60				
V > 0.08 m/s	93.70	P1	1,143.19	1,143.22	0.03
V ≤ 0.08 m/s	15.00	P2	1,149.72	1,149.70	-0.02

Results for data sets P1 and P2 are reasonable since pipes with higher velocities have greater roughness values as expected. But there is an inconsistency between results of P1 and P2 data sets. This inconsistency can also be seen at the result for the P1&P2 data set.

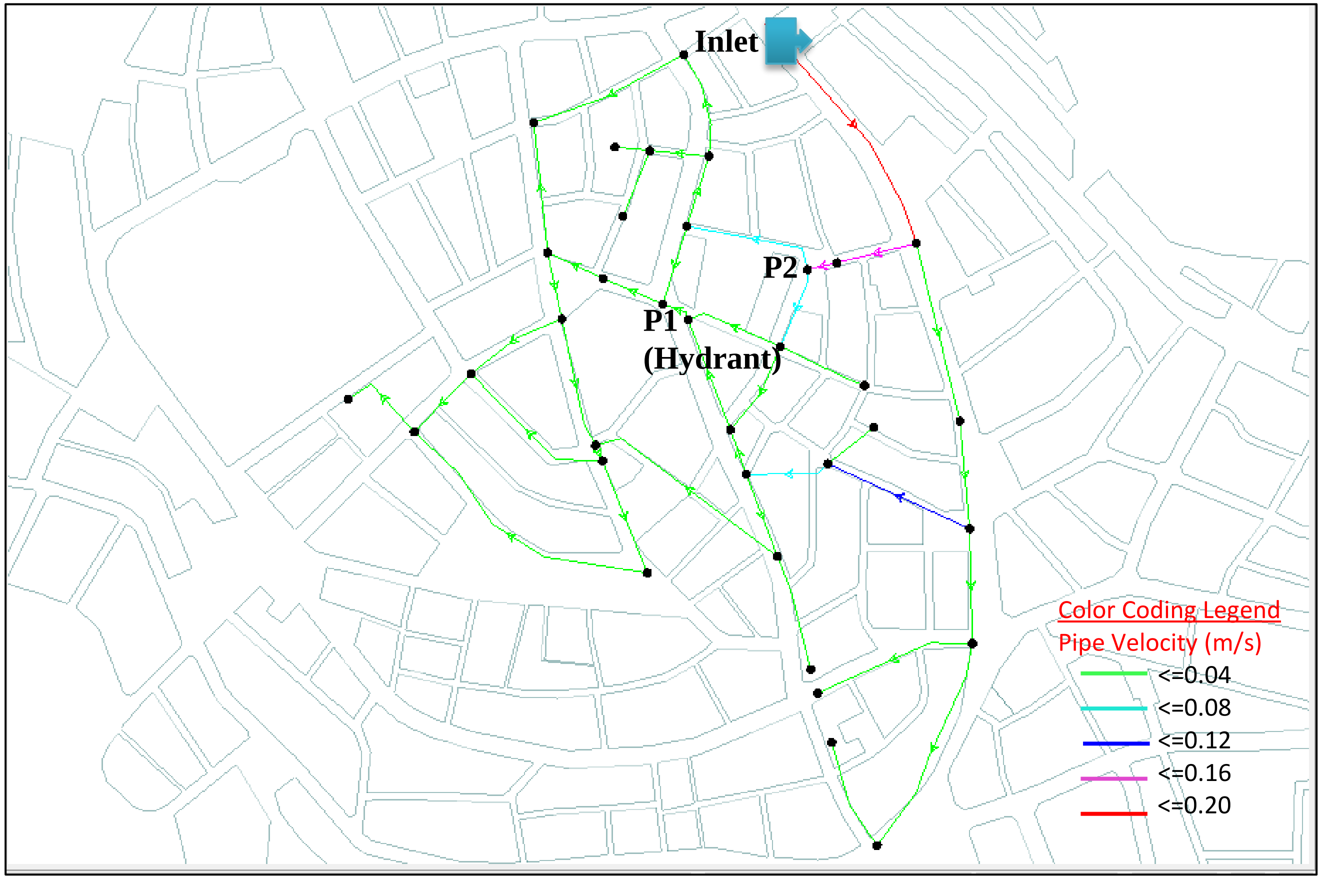


Figure 4.6. Pipe Velocities of Upper Çiğdemtepe Subzone at Normal Flow Condition

4.4. Calibration of Northern Sancaktepe Subzone Water Distribution Network

Hydraulic model constructed for Northern Sancaktepe network and the test points to collect calibration data are shown in Figure 4.7. Water consumption at this subzone is lower than the other ones. Test results are presented on Table 4.8 (recorded at 15:00 pm on 17.05.2012).

Table 4.8. Pressure and Flow Readings for Northern Sancaktepe Subzone

TEST NODE	Elevation (m)	LOW FLOW			HIGH FLOW		
		Pressure (m)	Hydraulic Grade (m)	Flow (l/s)	Pressure (m)	Hydraulic Grade (m)	Flow (l/s)
INLET	1,115.28	62.00	1,157.37	5.56	61.50	1,156.87	25.00
P1 (Hydrant)	1,088.16	69.35	1,157.51	0.00	65.78	1,153.94	19.44
P2	1,094.82	62.77	1,157.59	-	60.06	1,154.88	-

Case 1: Single Pipe Roughness Group

Results are presented below, on Table 4.9. In this subzone, C-factors are found a bit higher than the previous ones. Ar (2011) found out C-factor as 89 for this subzone.

Table 4.9. Calibration Results for North Sancaktepe Subzone - Single Pipe Roughness Group

Adjustment Group	Calibrated Hazen-Williams C	Test Data	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
All Pipes	109.90	P1 (Hydrant)	1,153.94	1,153.94	0.00
	89.50	P2	1,154.88	1,154.88	0.00
	102.70	P1&P2	1,153.94	1,153.55	-0.39
			1,154.88	1,155.33	0.45

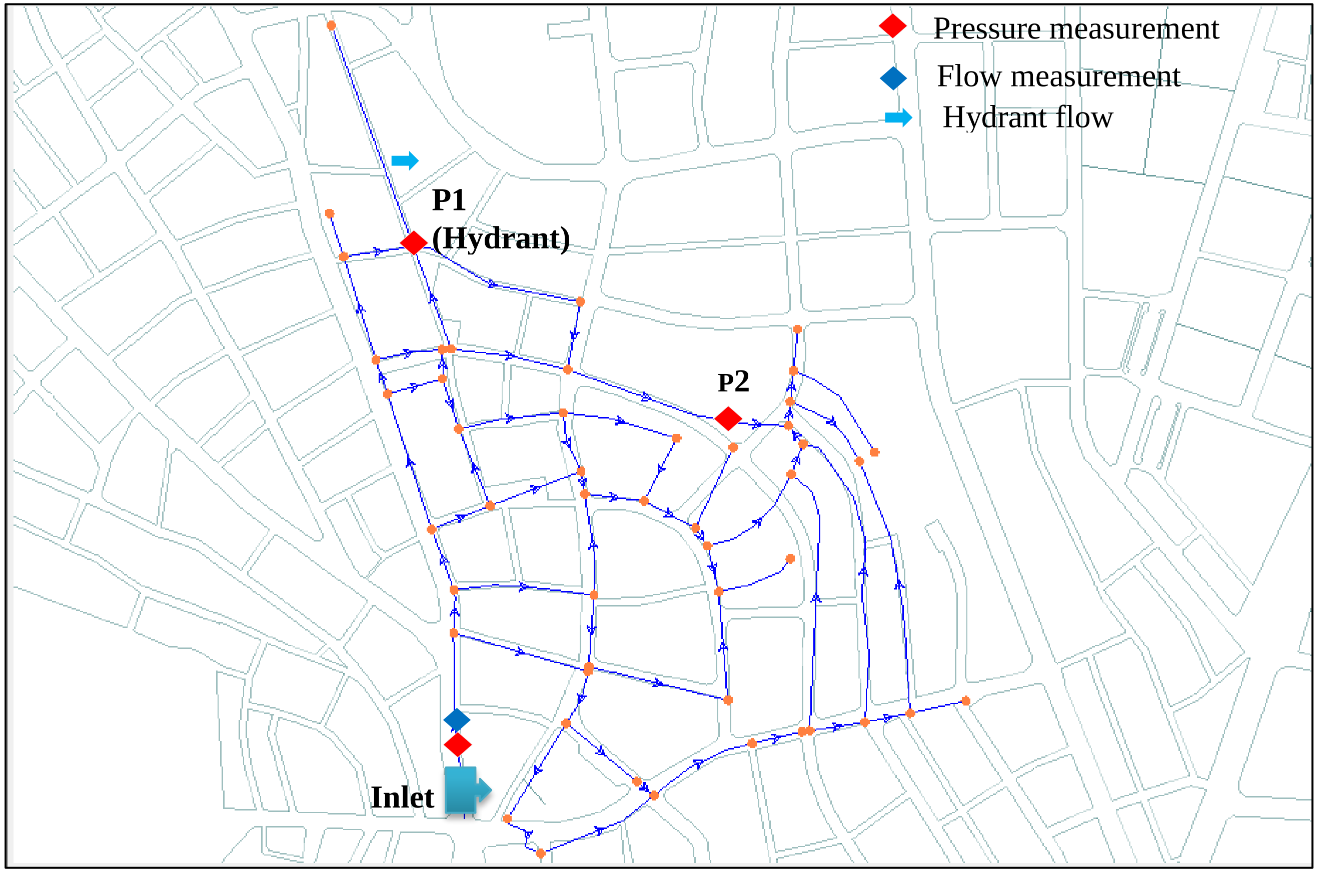


Figure 4.7. Hydraulic Model and Measurement Locations of Northern Sancaktepe Subzone

Case 2: Pipe Roughness Grouping With Respect to Pipe Velocities

It can be seen from the Figure 4.8 that most of the pipes at the Northern Sancaktepe subzone have velocities around 0.08 m/s. So the pipes are with respect to 0.08m/s. Calibration results are presented on Table 4.10.

Table 4.10. Northern Sancaktepe Calibration Results - Pipe Roughness Grouping With Respect to Pipe Velocities

Adjustment Group	**Hazen-Williams C**	**Test Node**	**Observed Hydraulic Grade (m)**	**Simulated Hydraulic Grade (m)**	**Difference (m)**
V > 0.08 m/s	137.20	P1 (Hydrant)	1,153.56	1,153.56	0.00
V ≤ 0.08 m/s	91.80				
V > 0.08 m/s	74.10	P2	1,154.53	1,154.53	0.00
V ≤ 0.08 m/s	179.30				
V > 0.08 m/s	81.00	P1	1,153.56	1,153.53	-0.03
V ≤ 0.08 m/s	125.00	P2	1,154.53	1,154.75	0.22

Results for data sets P1 and P2 to be reasonable with regard to the pipe velocities since pipes with higher velocities has greater roughness values as expected. The inconsistency between the two data sets has yielded the above result for the P1&P2 data set.

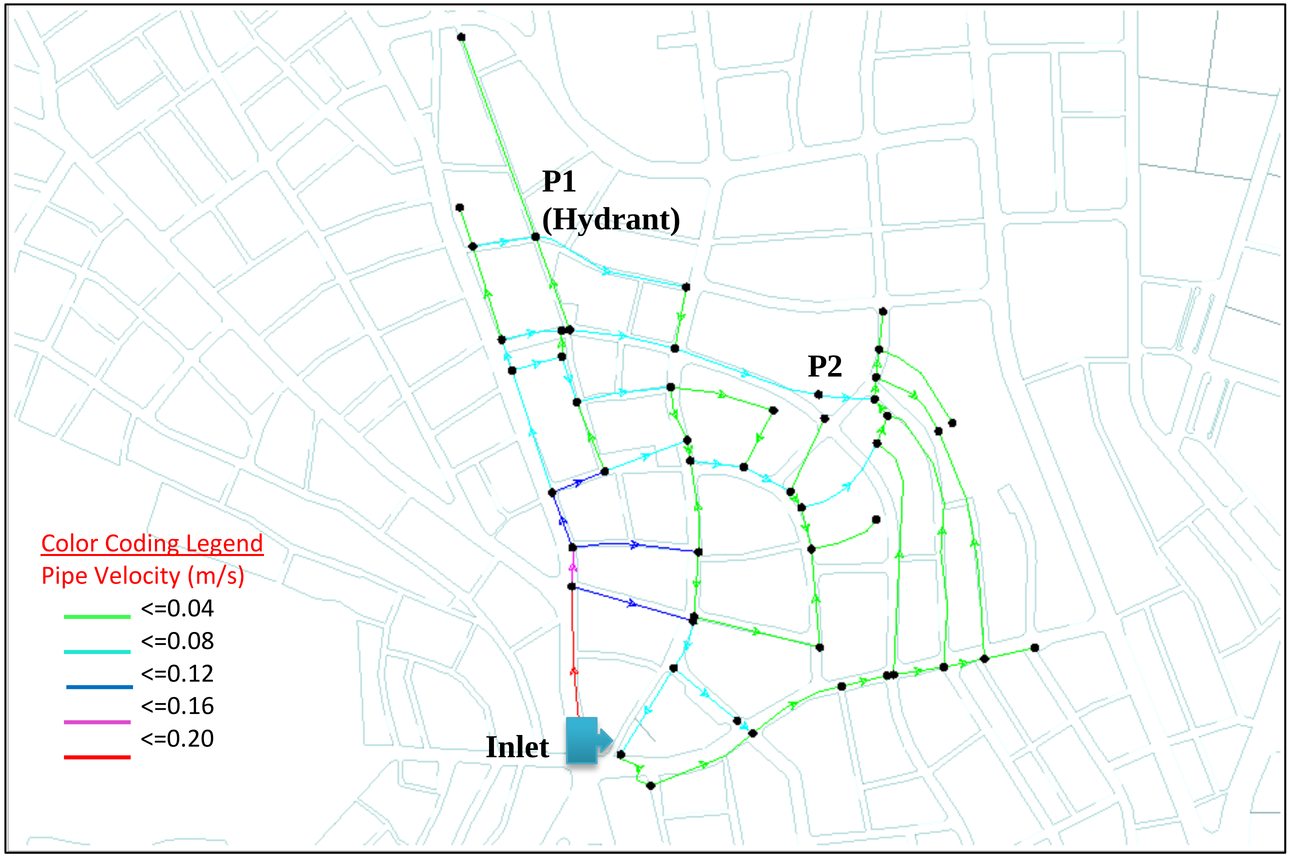


Figure 4.8. Pipe Velocities of Northern Sancaktepe Subzone at Normal Flow Condition

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

There exist many methods to calibrate water distribution networks. They can be grouped as iterative methods (trial and error methods), explicit methods and implicit methods. Being relatively slow, iterative methods are the basis of calibration studies. Today, on the other hand, implicit methods, by using global optimization tools, provide rapid and more consistent results even for large water distribution networks. Darwin Calibrator software used in this study uses genetic algorithm (GA) optimization which simultaneously produces hundreds of possible solutions, evaluates them and selects the fittest ones and reproduces offspring solutions among fittest solutions by imitating the mechanism of natural selection. Thus, the accuracy of the search process increases leading to better calibration results.

In Chapter 3, calibration trials using the so-called true data (generated data) showed that Darwin Calibrator is a fast and reliable tool to calibrate water distribution networks. It enables to; (i) manage the field data flexibly (ii) select calibration criteria, (iii) weigh the importance and accuracy field data results, (iv) create roughness and demand groups, (v) realize multi-objective optimization, (iv) view results spontaneously. Thus, Darwin Calibrator can solve calibration problems rapidly and yield reliable results with some minor inaccuracy that can be ignored, if accurate data (field data, network configuration) is provided.

In the case study, calibration results with single pipe roughness grouping case are consistent with the previous studies (Ar, 2011). Furthermore, adjustment groups are created for pipes having similar properties. Roughness values of pipes having higher velocities are found greater than the pipe having lower velocities as expected in some DMA's. But the yielded figures are not indicative since calibration data sensitive to the pipe groups could not be provided. Also problems with the measurements lead to uncertainty. Therefore to be able to make more meaningful interpretations, data should be repeatable. In order to obtain repeatable results, more "good" data should be obtained:

(i) using officially calibrated instruments,

(ii) elevation data should be checked with care,

(iii) a well educated team should be incorporated into the field works,

(iv) the water authority should be conscious about the uses of a well calibrated hydraulic model.

After getting the calibration results, discrepancies between simulated and the calibrated results should be examined. Inconsistencies should be identified. Otherwise, obtained calibration result is just a figure that represents the sum of the errors in the field data. In fact, in this study, causes of inconsistencies could not be inspected in details by means of field investigations because of limitations.

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