PRESSURE MODELLING FOR LEAKAGE REDUCTION IN ADDIS ABABA WATER SUPPLY SYSTEM MAINS (SAINT PAUL AND RUFAEL SUB SYSTEMS)

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfilment of the Requirements for the Degree of Environmental Engineering in Masters Program

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Abstract

The city of Addis Ababa is suffering from shortage of water - the city is receiving only 48% of its demand in 2007; at the same time losing 35.73% of its supply in its distribution systems. Financial, management and technical constraints are the main bottlenecks that hamper the city from satisfying the highly growing needs of its residents while loosing such a large amount of treated water within its system.

This Masters Thesis focuses on modelling water supply pressure for leakage reduction in the mains of Addis Ababa Water Supply System, especially in Rufael and Saint Paul sub systems. EPANET modelling software prepared by the Environmental Protection Authority of America is used for the purpose.

In so doing, first of all water balance at both city and the stated sub-city levels was conducted to visualize the loss of water and for the purpose of indexing the leakage level, at their respective levels. Water supply for the entire city and water consumption as aggregated from individual customer bills was used to evaluate the total water loss at city level and at the sub distribution system levels, respectively. Finally, potential Environmental effects of leakage in the city water supply system are assessed.

Quantifying, characterizing and modelling the factors that lead to water leakage in a city water supply system is by its nature a complex task requiring well managed network and reliable data which is scarce in Addis Ababa city water supply system. As a result, cross checking and verification of data was the most significant problem faced during the study.

The paper comprises two major parts; namely Leakage Analysis part and Pressure Modelling part. It was found out from leakage analysis part that water leakage in the city was 35.67%. The ILI value was obtained to be 4.74 which is far from an ideal value of 1. Thus, there is a room for AAWSA to target for leakage reduction without affecting the current pressure system. Besides, there is similar room for leakage reduction due to excessive pressure at some nodes with pressure above the recommended maximum for leakage of 80m almost all the day as pointed out from the modelling part.

AAWSSA shall devote itself in automation of its network system using GPS coordinates for network assets and data loggers for hydraulic parameters in order to establishing reliable database conducive for both leakage management and study.

Key Words: EPANET, Leakage, Pressure Modelling, Water Supply, Water Consumption, Calibration, Environmental Impact.

Acknowledgments

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Finally, I want to thank my friend Natnael Tesfaye for his moral encouragement during my study.

List of Acronyms

AAWSA - Addis Ababa Water supply Authority

ADLI - Agricultural Development led Industrialization

a.s.l. - above sea level

CARL - Current Annual Real Losses

DCI - Ductile Iron

DMAs - District Meter Areas

ILI - Infrastructure Leakage Index

IWA - International Water Association

NRW - Non-Revenue Water

PE - Poly Ethylene

PI - Performance Indicator

PVC - Poly Vinyl Chloride

UARL - Unavoidable Annual Real Losses

UTM - Universal Traverse Mercator

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Introduction

1.1 Background

Experience suggests that technical remedies are necessary but not sufficient to solve the problems of non-revenue water which are usually also linked to broader managerial, organizational, environmental and social issues that need to be appropriately dealt with as well. High economic returns will be generated by investments in the reduction of non-revenue water as it will simultaneously increase sales, cut supply costs and save waste. In actuality, sustainable development and management of water resources need to focus on reducing water loss and wastage rather than concentrating on expansion based on new systems. This is because large volume of water is lost through transmission and distribution lines. According to International Water Supply Association (IWA), the unaccounted for water is estimated to be 20 to 30% of treated water supplies for most cities. However, this figure rises to 50% in some cities.

Financial implication of water loss is immense. For instance, since 1989, the Kansas Rural Water Authority has conducted water loss surveys identifying an annual loss of 2.387 billion gallons. The annual costs to purchase or produce this loss would have been $3.586 million (Walsky, Betzs, Posluzany, Weir and Whiteman, 2006).

The main objective of the paper is to assist reduce water leakage in Addis Ababa Water Supply System Mains based on outputs of pressure model developed for the purpose. In order to attain the objective set, the paper analyzed the current leakage status in the city both at city level and at selected sub system levels.

The Thesis paper concentrates mainly on Pressure Modeling for Leakage Reduction in Addis Ababa Water Supply Mains. Since, the mains in Addis Ababa Water Supply System are about 500kms, the paper is limited to two sub systems mains that have relatively reliable and isolated data in addition to being adjacent to one another.

Addis Ababa is located between 972000N to 1000500N and 462000E and 488000E UTM coordinates. It is situated in central highland of Ethiopia surrounded by the Blue Nile catchments in the north and the Wachacha Mountain which forms separation belt of the city from the Awash River catchments in the west (Adane Bekele, 1999). Fig. 1-1 shows the location of Addis Ababa with respect to the map of Ethiopia. The city descends from Entoto ridge which is at an altitude of 2,975m a.s.l. at Entoto to Akaki area to an elevation of about 2,050m a.s.l around Kality.

Based on Central Statistics Authority abstract for 2006, the population of Addis Ababa was estimated to be 2,973,000 in July 2006. The population of Addis Ababa is also estimated to be growing at 3.7% rate (CSA, 1994).

Addis Ababa is the largest political and commercial capital of Ethiopia with a population of about three million residents in 2006. The city covers over 540km^2 area in the same year; whereas only 200km^2 of the total area is served accounting for only 37% of the city area.

Addis Ababa is the seat of many international organizations and embassies including African Union. The city administration has, thus, a big task of supplying the city with adequate and quality water to fulfil the different demands so that the status of the city is kept.

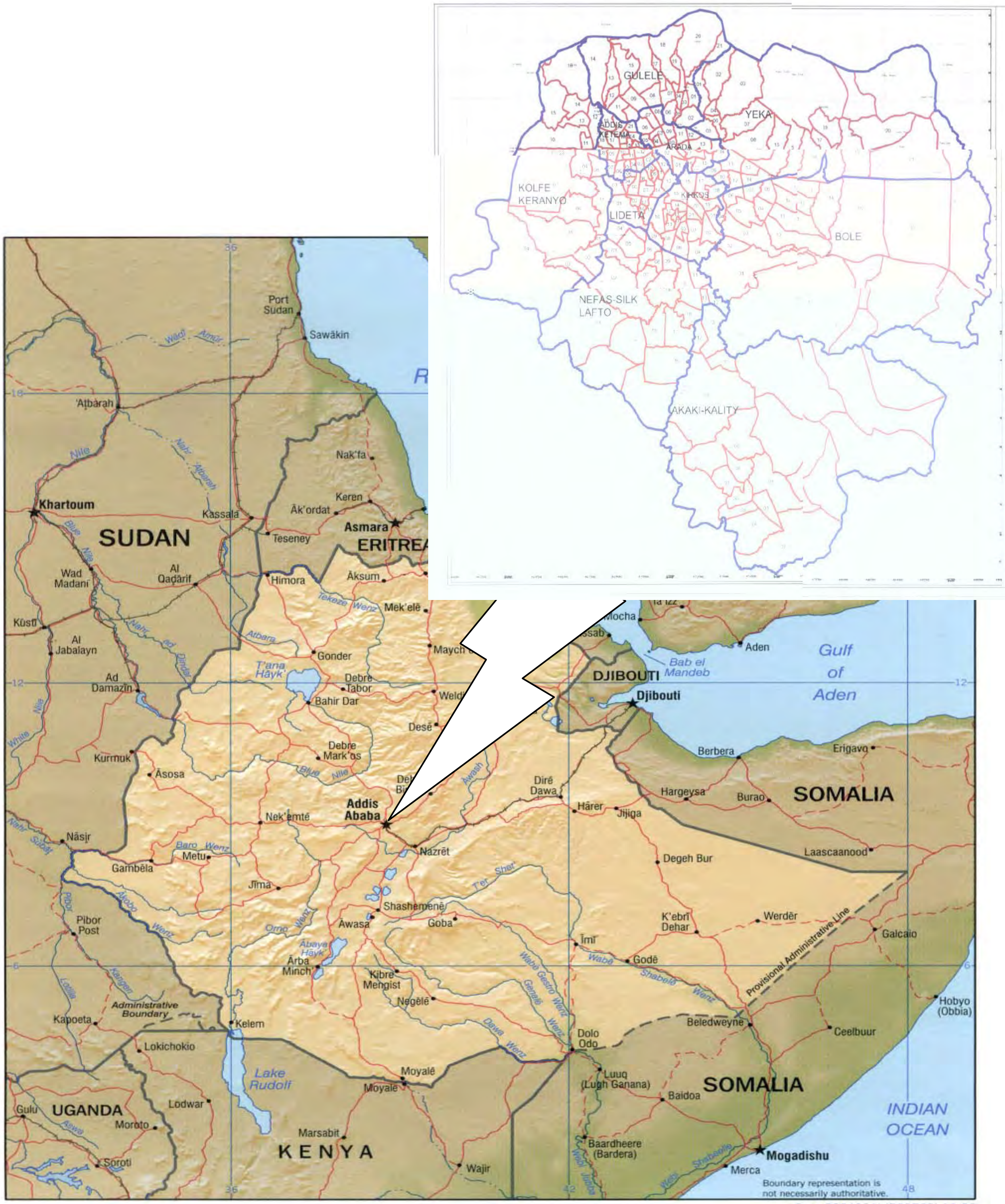

Fig. 1-1 Location Map of Addis Ababa in Ethiopia

1.1.1 Development of Water Supply System of Addis Ababa

The earliest water supply to Addis Ababa was from spring water located at the foot of Entoto hill. The water supply source of the city for the first 58 years, since its establishment in 1886, was from ground water in the form of either spring or well water. The increasing demand due to population increase could not be met with ground sources only. This necessitated the first surface water development and water treatment plant at Gafarsa, some 14 kilometres to the west of the city established in 1942. As the demand further escalated, the second water treatment plant of the city, Lagadadi water treatment plant, was commissioned in 1970 with treatment capacity of 50,000 m3/d. Due to further increase in demand for water, under its phase II project, the capacity of Lagadadi water treatment plant capacity was increased to 150,000m3/d in 1985. In 1990, the distribution systems of the city were improved, emergency water supply projects of Akaki ground water development, and Dire dam construction to supplement Lagadadi water treatment plant were the major water supply activities carried out to date (AAWSA Design Study Services, 2001).

Meanwhile, AAWSA was established in 1963, under the provisions of proclamation No. 68/1963 holding title of *Addis Ababa Water Supply services*. It was re established under proclamation No. 10/1987 with more rights and authority by the name of *Addis Ababa Water Supply Authority, AAWSA* (AAWSA broacher, 2006).

Water Consumption Categories in the City: According to AAWSA broacher of 2006, the city has about a quarter a million customers in July 2006.There are three consumption categories in AAWSA namely: Domestic non-domestic and Public Fountain. Domestic water consumption is the highest ranking consumption category in the city; followed by non-domestic consumption. Based on consumption data collected from AAWSA for 2006, the domestic, non-domestic and public fountain consumptions were 51.89%, 46.18% and 1.93%, respectively.

Water Consumption Characteristics in the City: People having in-house services that are estimated about 4% of the total population use water on average between 80 and 100 litters per capita per day, while the remaining population with access to safe drinking water

(94%) are served by communal or tap connection and use between 15 and 30 litre/capita/day.

Non domestic uses including industrial and commercial water use are about 25 litre/capita/day and 7 litre/capita/day, respectively. From the water used by industries about 40% is provided by the water authority while the remaining amount is produced by the industries themselves from deep wells.

Current Water supply Sources: According to a broacher published by AAWSA, in July 2006 the water supply sources of the city with their supply capacities are:

a) *From Gefersa Reservoir 23,000m³/d.*
b) *From Lagadadi reservoir 150,000m³/d*
c) *From Akaki well fields 30,000m³/d*
d) *From Various springs and wells 14,000m³/d.*

This amounts to a total of 217,000m³/d supply to fulfil the requirements of city water demand.

Proposed water supply projects: Besides these existing water supply sources, the envisaged water supply projects of the city to meet the ever increasing demand include:

a) *AAWSA III: through harnessing Gerbi and Sibilu rivers with a capacity of 685,000m3/d that required 550,000,000 dollars for its implementation, based on 2006 price estimate.*
b) *AAWSA ground water phase II: upgrading the current extraction of ground water from 30,000m3 to 42,000m3 for Bole bulbula, Lafto and Makanisa areas.*
c) *Upgrading water treatments of Gefersa from 23,000m3/d to 30,000m3/d. In conjunction with the deep wells to be dug around Keranyo, it is intended to serve Anfo and Keranyo areas with the expanded source from Gefersa.*

Water Tariff in the city: Based on World Bank group report, water tariffs are less than 20% of operations and maintenance costs in Addis Ababa water supply system. The current rates are:

a) *Public fountains at 1.75/m3.*

b) *Residential up to 7m3 at 1.75/m3*

c) *Residential up to 7m3 to 20m3 at 3.15/m3*

d) *Residential above 20m3 at 3.80/m3*

e) *Non Residential at 3.8/m3 altogether.*

Water supply variations: The water supply variations in the city range from intermittent supply to 24 hrs supply. Most parts of the city are receiving water on ration basis that the current supply coverage accounts only 48% of the population of the city (Capital News paper, April 15, 2007).

Water supply infrastructure: Based on AAWSA classification transmission Mains are those pipes with diameter of more than 50mm up to 1400mm. Primary pipes are pipes above 125 mm and secondary pipes are those pipes between 50mm and 125mm. The total primary and secondary lines based on 2006 data are 496 km and 1,026 km, respectively. Gafarsa transmission line consists of two 400 mm steel pipes, installed in two phases in 1955 and 1960. Lagadadi water transmission line was built in 1970 and consists of 900 mm main steel pipes.

There are 56 reservoirs at 40 different locations with capacities ranging from 50m3 to 20,000m^3. Their total capacities are 90,000m^3. There are totally 24 electrical pumps to boost pressure in the distribution system (Welday Berhe, 2005)

1.2 Description of the Study Area and its Water Supply System

There are 13 sub distribution systems in Addis Ababa city water supply system. Among these, the selected study area includes both St. Paul and Rufael sub systems taken together located at North western part of the city.

Figure 1-3 shows the location of the selected study area in Addis Ababa city administration while Figure 1-4 depicts some features of the distribution system.

Figure 1-2: Location of the selected study area in Addis Ababa

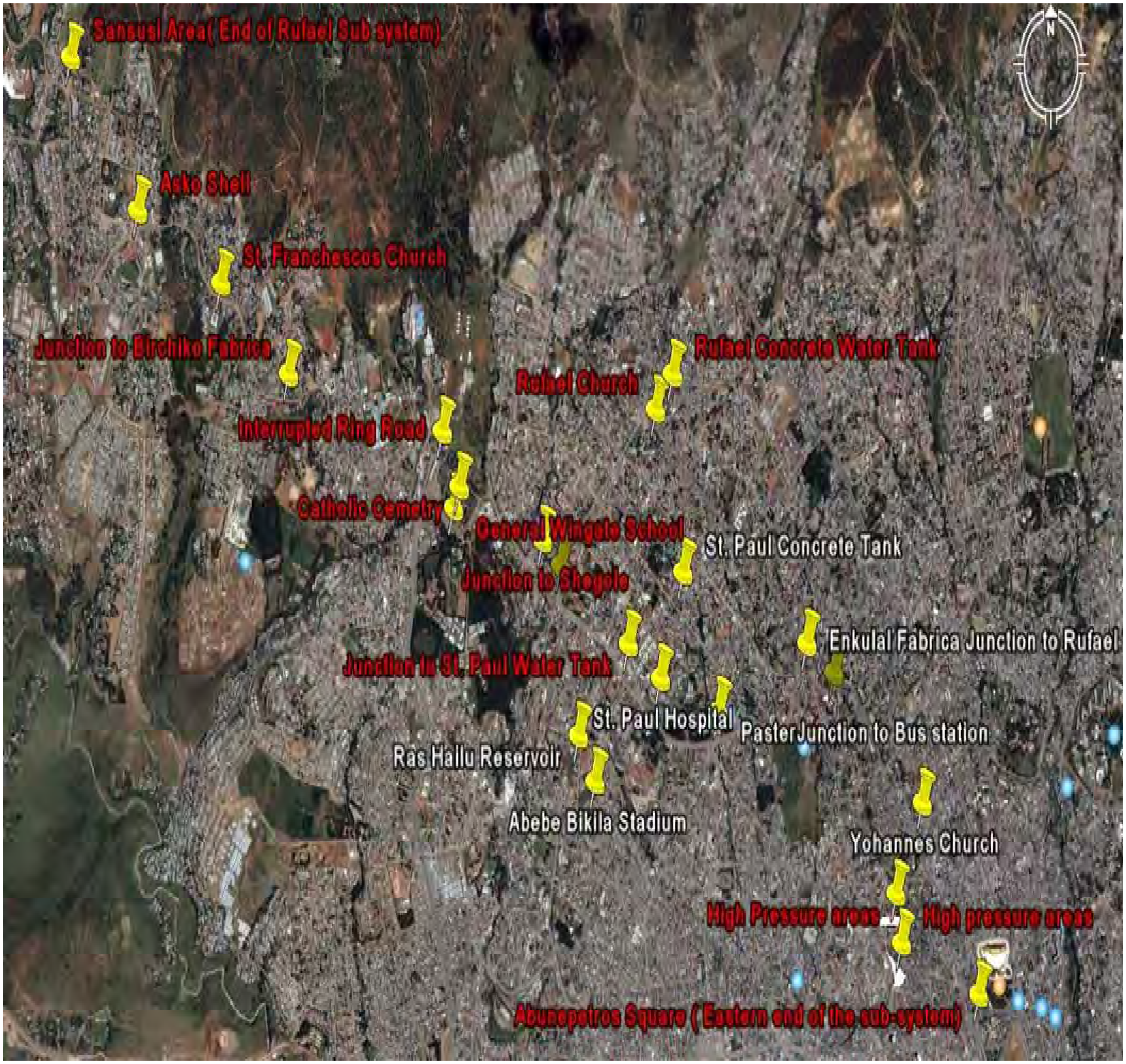

Figure 1-3: Some features of the study area

The existing water distribution network mains of the study area consists of mainly DCI pipes ranging in size from diameter 150mm to 350mm. The main pipe characteristics of the study area are summarized on table 1-1.

Table 1-1 Summary of Main Pipe Lengths in the Study Area

Pipe of:	Length (m)	Percentage
1975	11,045.83	57.00
1989	8,328.98	43.00
Diameter 150mm	4,222.09	21.79
Diameter 200mm	7,373.88	38.06
Diameter 250mm	1,831.08	9.45
Diameter 300mm	1,213.42	6.26
Diameter 350mm	4,734.34	24.44
Total	**19,374.81**	**100.00**

The distribution system has main storage at Ras Hailu Reservoir which consists of two circular reservoirs of each 2,500m3 capacities that supply to both Rufael and Saint Paul storage reservoirs under action of four centrifugal pumps. Figures 1-5a and 1-5b show areal view of Ras Hailu Reservoir and Saint paul Concrete tank.

Additionally, a pump from Ras Hailu Reservoir pumps water directly through the distribution pipes to the east along Paster up to Abune petros area; and the other pump delivers water from the same reservoir back to the west through General Winget School to Traffic Sefer area.

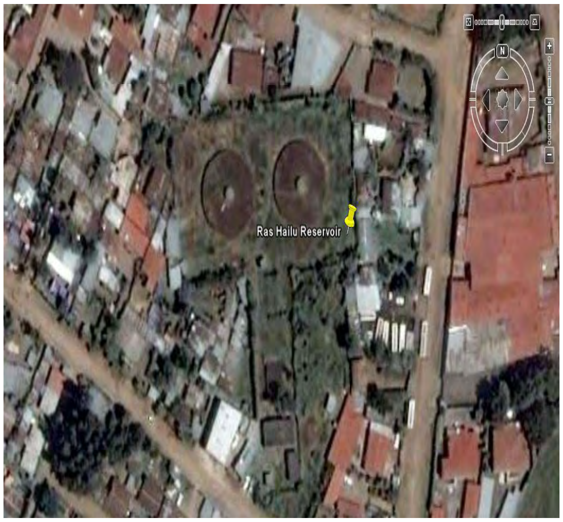

Figure 1-4 Areal view of Ras Hailu Reservoir **Figure 1-5** Areal view of Saint Paul Concrete Tank

(Modified from Google Earth)

Once pumped to Rufael and Saint Paul storage reservoirs the system operates through gravity to their respective sub systems.

Rufael water tanks consist of three circular tanks; two of them have capacity of 500 m^3 each, and an other tank has a capacity of 2,500 m^3. Saint Paul water tanks, on the other hand, consists of two circular tanks of 500m3 capacity each as can be seen on figure 1-5b depicting areal views of St. Paul Concrete Water Tank.

There is also a service tank around Asko area with a capacity of 17.8m^3 that is used for balancing the water distribution to Sansusi area and to Kale area at different times.

1.3 The Rationale of the Study

1.3.1 National Water policy

As Ethiopia is federal state, under proclamations No.33 of 1992, 41 of 1993 and 4 of 1995 duties and responsibilities of regional state include: planning, directing, and development of social and economic programs, as well as the administration, development and protection of natural resources of their respective regions. Accordingly under provisions of these proclamations Regional Governments are granted regulatory powers over their respective water resources.

It is to be noted that the existing basins around Addis Ababa that are providing and that could potentially provide water to the city are tributaries of the Blue Nile basin or that of Awash River basin located in Oromia region. Thus, according to the above proclamations, Oromia region has similar mandate as that of Addis Ababa region over management of resources originating from its region.

Moreover, Ethiopia is a country frequently hit by draught and famine. The government has prepared programmes with the objectives to enabling the economy to develop rapidly, the country to extricate itself from its dependence on food aid and make poor people to be the main beneficiaries of growth. One of the identifier pillars of the programme is Agricultural Development led Industrialization (ADLI). The country is launching a major program for the intensification of agriculture, including the large and small scale development irrigation schemes through the ADLI Policy. Under ADLI one of the special emphases given area is small scale irrigation to ensure food security. Therefore, there will be a conflict of interest over those water sources that provide water to the city in the future.

Accordingly, the Addis Ababa Water and Sewerage Authority (AAWSA) is established as a public institution that is responsible for the supply of potable water and collection, treatment and disposal of waste water and sludge of the city. Similarly, proclamation number 10 of 1995 defines the powers and responsibilities of AAWSA within the Addis Ababa city administration level. The authority is supervised by a board and it is directly responsible to the city manager.

The allocation and apportioning of resources is, anyway, mandated to the Federal Ministry offices. Although the allocation and apportioning of water resources to different regions is regulated by the Federal Ministry of Water Resources as proclamation number 197/2000 declares that "All water resources of the country are the common property of Ethiopian people and the State." Anyway, imminence of water resources extraction limit is implied.

1.3.2 Water Demand Characteristic and Leakage in the City

Attributed to water supply irregularities and shortage, the actual water demand of the city is depressed; the city is receiving only 48% of its demand in 2007. The average domestic consumption is reported to be 22 l/c/d. For developed countries, this figure varies from 100 to 530 l/c/d (EPA, 2004).

AAWSA conducted water loss study during the years 1982 and 1995; the figure of water loss in 1982 was reported to be 22% while this figure rose to 35.6% in 1995. Although the authority's target was to reduce these figures to 10% gradually, the total water loss further rose to 38.01% in 2006. This shows that the situation is worsening and the infrastructure is deteriorating. Figure 1-6 illustrates the water loss trend in the city from 1993 to 2006, the period for which continuous water loss data was available. As can be observed from the figure, the water loss trend reduced post 1995 up to 2005 following water loss study of 1995.

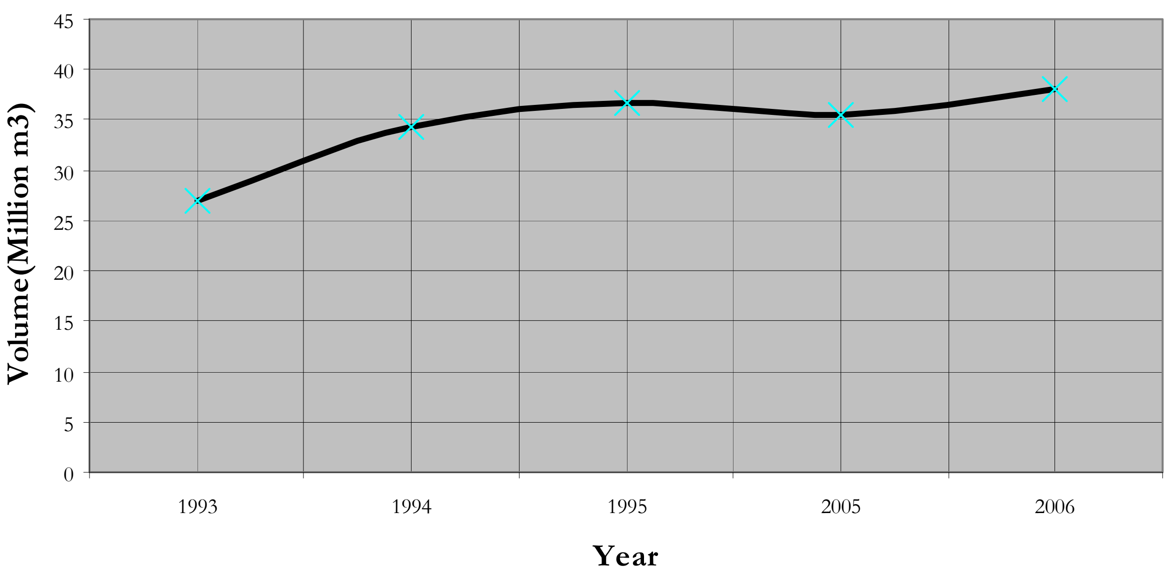

Fig.1-6 - Water loss trend in AAWSA supply system from 1993 to 2006
(Source: AAWSA data)

1.3.3 Scarcity of Raw Water

As stated earlier, the same water source will be used to ensure the economic development of Oromia region in the vicinity of the supply sources.

Besides, the distance from which potential raw surface water sources, with significant yield capacity, are to be developed for the city is getting farther than ever; obviously, with its associated additional costs of delivery to the city.

At this junction, it is important to rethink projection of the optimum number of population all the water resources within the vicinity of Addis Ababa could sustainably be used to serve the city population; without hampering the economic development of the source areas in Oromia region.

1.3.4 AAWSA IIIA Project and Challenges for the City Water Supply

The next immediate water supply plan for Addis Ababa city is termed AAWSAP IIIA; the water supply project for which the design has been completed. It is the largest of all projects undertaken to date for the supply of water for the city.

It involves massive construction works which includes:

a) *Gerbi dam and reservoir with a storage volume of 48.5Million m^3 and safe yield of 77,000m^3/d*
b) *Sibilu dam and reservoir with a gross storage volume of 347.7Million m^3 and safe yield of 611,500m^3/d.*
c) *A raw water pumping and a transmission of 1.8m diameter, 11.3km for sibilu and, 1.2m and 8.2 km long for Gebri to the treatment plant.*
d) *A two stage treatment plant construction in common for both dams.*
e) *Lined tunnel, approximately 3m diameter and 4.8 km long through Entoto Mountain.*
f) *Transfer and primary distribution system expansion, including pumping stations, service reservoirs and pipelines to convey to the distribution sub systems.*
g) *Development of the Akaki ground water and expansion of the ground water supply to deliver 75,000 m^3/d to the city*

AAWSAP IIIA project had been expected to be operational by 2005; and to be fully operational by 2010. However, it is not started yet. Once AAWSAP IIIA becomes fully completed, the total supply capacity to the city is expected to be 990,000 m^3/d (AAWSA Design Study Services, 2001)

The technical capability of AAWSA is also among the current and future challenges. A factor lacking particular concern is the resulting needs to properly manage after implementation of the project. It can be deduced that AAWSA will lose 376,299m^3/d of its intended supply after AAWSA IIIA is completed provided its current operational practices remain constant.

On the other hand, huge water supply projects also have social, economic and environmental consequences. These large projects often are associated with large land-use. The environmental impacts of such projects can be analyzed from three perspectives. Foremost, the water resources deplete from high rate of infiltration and surface water evaporation. Secondly, these direct Environmental impacts result in induced Environmental impacts like impact on flora and fauna, basin morphology and local climatic changes. The last impact of impounded water is on human health associated with changes in natural biodiversity and possibility of malaria spread.

1.4 The Research Objective

1.4.1 General Objectives

Under fore mentioned serious water stress conditions, financial and raw water source constraints, large amount of treated water escapes through main pipes and distribution systems taking large sum of the resources spent on it to collect, convey, treat and distribute it.

Apart from its financial importance, leakage prevention has both social and environmental importance in many ways. Above all the resources spent on it to collect, convey, treat and distribute comprises Environmental aspects. On the other hand, the value assigned to raw water itself is highly undermined and water shall be considered as precious commodity.

Leakage also results in deterioration of the appearance of a city along with health related social problems that contaminated water through leakage brings about. Additionally, resources required for expansion of the water network as a result of leakage related demand deficient are Environmental assets. The associated problems related to additional water harnessing, changes in river morphology, diseases related to impoundment of large volume of water is also worth mentioning.

Leakage is directly proportional to pressure in supply systems. Thus, pressure reduction results in corresponding reduction of leakage. For instance, according to a pilot study conducted in South Africa, as a result of pressure management a significant reduction in night flow rate is achieved. An average reduction of 44.12% pressure resulted in 42.57% reduction in night flow rate (leakage) (Lambert and McKenzie, 2005)

Pressure modelling enables to understand the operational mechanisms of the distribution system. It also assists in optimization of these operational mechanisms and to analyze different scenarios and their impacts. For the purpose of the thesis, modelling is used to observe the pressures at different locations of the transmission system so that the effect of excessive pressure could be analyzed for its impact on leakage.

Thus, the primary objective of the thesis paper is to generate a water supply mains model representing the existing states of the city water supply mains pressure in selected sub distribution systems and to evaluate the effect of the obtained pressures on leakage.

In doing so, the thesis assesses the water leakage in both the supply mains and distribution systems of Addis Ababa city water supply.

Finally, environmental, social and economic consequences associated with leakage in Addis Ababa water supply system, especially in the selected sub systems, will be analyzed. Since Environmental Impact Assessment is a proactive process whereby preventive measures are proposed before implementation of a project, Environmental Impact Assessment is beyond the objectives of the study.

1.4.2 Specific Objectives

Under those main objectives, the paper addresses the following specific objectives:

a) *To estimate water leakage of water in the distribution system of Addis Ababa water supply at city level.*
b) *To evaluate and compare water leakage in selected sub distribution systems of Addis Ababa water supply and index it.*
c) *To model the selected sub city water distribution main pressure for the purpose of leakage reduction.*
d) *Since leakage assessment is an analysis based on already functional project, the study will only evaluate the social and environmental consequences of leakage for the quantified leakage at selected sub city level.*
e) *Proposing AAWSA leakage reduction targets based on obtained leakage index.*

1.5 Overview of the Thesis

In addition to the introductory part, the following is a berief description of discussions held in the study under each section:

Chapter two contains literature reviews related to water supply system, water loss, leakage and pressure modelling for leakage reduction.

Chapter three discusses theoretical background of water balance, relationship between leakage and pressure, performance indexes, hydraulic modelling and other associated theoretical backgrounds of the study.

Chapter four is devoted to the methodology used in data collection and preparation, water leakage analysis, pressure modelling for leakage reduction and the methods used to reach at findings and conclusion.

Chapter five contains the first part of the study that presents the analysis of leakage and indexing using different performance indicators at both city level and at selected sub system levels.

As the core of the paper, **chapter six** is devoted to the pressure modelling part in the two selected sub distribution systems with emphasis on the findings of the model out put.

Chapter seven on the other hand discusses and interprets the model out puts, analyzes pressure distribution at critical points, recommends as to reduce leakage, simulates the model after Proposed solutions, assesses the out put of the model on leakage and assesses economical feasibility of the Proposed solutions.

Chapter eight assesses Social and Environmental Consequences of Leakage in the sub systems based on the findings of chapters four, five, six and seven.

Chapter nine deals with conclusion and Proposed solutions forwarding general approach AAWSA has to follow to identify and reduce water loss based on results and findings of the study in the two sub distribution systems.

Under Appendix, collection of major tabular data, graphs and charts on data input parameters, data out put parameters and other relevant parameters attributed to the study are included.

2 Literature Review

2.1 General

2.1.1 Design and Operation Requirements of Water Supply System

The purpose of a water supply system is to supply water ensuring that there is sufficient pressure and adequate flow to the consumer. The major issues to be addressed in ensuring the required flow and pressure from perspectives of different sources are summarized as follows:

Appropriate pressures: Pressure at any point in the system should be maintained within a range whereby the maximum pressure avoids pipe bursts and the minimum ensures that water is supplied at adequate flow rates for all expected demands. Pressure in excess of 80m increases leakage rate and occurrence of new leaks. An average pressure in mains of 50m is considered to be appropriate in many literatures (Hammer, 2003).

Situations that may give rise to negative pressures should always be avoided. Besides supply failure to customer, faecal organisms in groundwater adjacent to a pipeline and may be drawn into a pipe during transient low or negative pressures (LeChevallier, 2003).

Flow velocity: Generally, flow velocity in water supply system shall be in the range of 60cm/s to 3m/s depending on pipe sizes to avoid stagnation and to limit head loss which varies in squared proportion to velocity. The limits also avoid both high residence time and scouring on the other hand, respectively (Hammer, 2003). In order to reduce excessive head loss in pipe mains of diameter 150mm, 250mm and 350mm the velocities shall not exceed 1.2 m/s, 1.5 m/s and 1.8m/s, respectively (Hammer, 2003).

Surge events: The common causes of surge are the operation of pumps, valves and hydrants. Other events such as bursts and sudden increases in demand can cause surges. Besides possibility of burst, it can result in a deterioration of water quality because the surge can disturb deposits in the pipe or on the pipe wall. These operations may also cause low pressures that could allow ingress of contaminants. Surges create disturbance waves in the pipes back and forth causing greater head losses. The thickness of pipes shall be designed to sustain such waves; otherwise it ends in leakage and bursts in the supply lines.

The risk of significant surge, and hence water leakage and quality problems, is greater in long un branched pipes than in branched pipes, because branched pipes reduce surge. There are several techniques to avoid surge effects, among which are the following.

a) *Placing air vessels close to pumps and major valves. Air vessels are devices that have air trapped above the water. The water level changes as the pressure vary, dampening the surge event. The advantage of this system is that no power supply is needed, but the volume of air must be maintained*
b) *Controlling the rate of switching pumps to make the change in flow gradual, so that the network can absorb the effect of the change in flow*
c) *Operating valves and hydrants slowly (Metcalf and eddy, 2003)*

Integrated operations: The operation of a network should not be just a collection of uncoordinated activities such as valve and pump operation but should take account of the interactions between these activities. This requires an overall strategy adapted to local circumstances and applicable to all water quality issues. This includes:

a) *Risk assessment of each activity (e.g. valve operation) before it is undertaken and identification of actions to minimize risk*
b) *Procedures for mains cleaning, mains laying, repairs and renovations*
c) *Coordination with fire fighting services on hydrant use and awareness of which areas may be at risk of loss of pressure*
d) *Procedures for operating valves, hydrant and other fixtures on access for maintenance and prevention of surge*
e) *Service reservoir design, operation and maintenance requirements*
f) *Procedures for changing or mixing supplies in distribution*
g) *Optimization of water treatment (including a full cost–benefit analysis) so that water entering the network is of good quality and the potential for re growth in the network is minimized*
h) *Awareness of, and collaboration with, leakage reduction teams to identify where pressure reduction may result in low pressures*

i) *Good record keeping so that problems can be traced and lessons learnt.*
j) *Collaboration with consumer services to keep consumers fully informed of activities on the network and any emergency advice in the case of a water quality problem (Chambers, Creasey and Leith, 2004)*

2.2 Water Balance in Distribution Systems

Total water loss describes the difference between the amount of water produced and the amount which is billed or consumed. Leakage is one of the components of the total water loss in a network excluding apparent losses as described on IWA water balance provded on table 2-1. Leakage comprises the physical losses from pipes, joints and fittings, and overflowing service reservoirs. These losses can be severe, and may be undetected for months or even years. The larger losses are usually from burst pipes, or from the sudden rupture of a joint, whereas smaller losses are from leaking or "weeping" joints, fittings, service pipes, and connections. The volume lost will depend largely on the pressure in the system, and on the "awareness" time, i.e. how quickly the loss is noticed and dealt with. This in turn depends on whether the soil type allows water to be visible at the surface. It also depends on the leak detection and repair policy of the water supply company.

The other components of total water loss are non-physical losses, such as meter under-registration, illegal connections, and illegal or unknown use.

Quantifying the total amount of lost water is achieved by conducting a system-wide water audit, known as a water balance. Audits provide a valuable overall picture about various components of consumption and loss, which is necessary for assessing a utility's efficiency regarding water delivery, finances, and maintenance operations.

The standard IWA terminology for water balance is provided on Table 2-1 below that simplifies the definition. As can be seen from the table, water loss is classified in to two; apparent loss and real loss. These loss categories are also further classified into their sub categories. Our major concern is on the real losses in distribution systems.

Table 2-1: IWA best practice water balance

<table>
<tr><td rowspan="9">System Input Volume</td><td rowspan="4">Authorized Consumption</td><td rowspan="2">Billed Authorized Consumption</td><td>Billed Metered Consumption</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Un metered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized consumption</td><td>Un Billed Metered Consumption</td><td rowspan="7">Non-Revenue Water</td></tr>
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<tr><td rowspan="5">Water Loss</td><td rowspan="2">Apparent Losses</td><td>Un Authorized Consumption</td></tr>
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<tr><td rowspan="3">Real Losses</td><td>Leakage on Transmission and Distribution Pipes</td></tr>
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<tr><td>Leakage on Service Connections Up to Customer Meter</td></tr>
</table>

(Source: Lambert and McKenzi, 2005)

According to IWA cited in (*Lambert McKenzi, 2005)* the above terminologies are defined below:

System Input Volume: is the annual volume input to that part of the water supply system Authorized Consumption is the annual volume of metered and/or non-metered water taken by registered customers, the water supplier and others who are implicitly or explicitly authorized to do so. It includes water exported, and leaks and overflows after the point of customer metering.

Non-Revenue Water (NRW): is the difference between system input volumes and billed authorized consumption.

Water losses: are the difference between system input volume and authorized consumption, and consists of apparent losses and real losses.

Apparent losses: consist of unauthorized consumption and all types of metering inaccuracies.

Real losses: are the annual volumes lost through all types of leaks, bursts and overflows on mains, service reservoirs and service connections, up to the point of customer metering

2.3 Factors Governing Water Leakage

Deterioration of water distribution pipes is the main factor for leakage and its frequency. On the other hand, there are many factors that influence deterioration of pipe lines. These factors are categorized under either structural, internal, external or maintenance factors as described below on.

Ages of Pipes and Leakage: Many of the factors just described are age-dependent - their effect will be greater with time. Consequently, the age of a pipeline can appear to be the most significant factor affecting the likelihood of leakage, but on its own, age is not necessarily a factor.

Different installation periods or eras, show different failure characteristics. In some cases, older pipes are more resistant to failure than younger pipes. For grey cast iron pipes this can be explained by the thinner walls produced by newer casting methods. The thinner walls lead to a greater effect of corrosion and higher stress level for the same external loads Hikki (1981).

Load over Pipes: Rangawala suggest that the effects of vibration and high loading caused by heavy Lorries is thought to be a major factor affecting buried pipelines and leading to pipe failure Rangwala (2000).

Previous Failures: Goulter and Kanzemi (1988) observed the temporal and spatial clustering of water-main breaks, indicating that a previous break increased the likelihood of future breaks in its immediate vicinity. They suggested that the subsequent breaks are caused by damage during repair operations, such as pressure surge while refilling the pipe after repair or ground movements caused by excavation, backfilling and the movement of heavy vehicles Hikki (1981).

Soil Conditions and Movement: The type of soil and its permeability is an important factor, as it affects the length of time a leak is allowed to continue. In soils like clay, water from underground leaks may show on the surface fairly quickly, whereas similar leaks in chalk or sandy soil can continue indefinitely without showing. Soil conditions also affect external corrosion rates, and play an important role in pipe degradation.

Soil movement is caused by changes in moisture content, particularly in clays, causing shrinkage changes. Transverse failure of cast iron mains has been recorded to be caused due to soil movement. Soil movement factors can cause a pipeline to break, joints to move, or result in localized stress concentrations within the pipe leading to failure Erdogan and Wei(1984).

Pipe Material: The most serious problem in this category is the corrosion of metallic pipes. Internal corrosion is usually more severe in acidic waters. In the case of iron pipes, tubercles develop on the wall of the pipe, and these are associated with pitting and localized areas of metal attack. The pipe wall thickness is reduced so that the pipe loses its ability to withstand pressure, leading to eventual penetration and failure of the pipe wall, and obviously leakage.

External corrosion can arise from a number of causes - aggressive soils may cause damage because of differing levels of dissolved salts, oxygen, moisture, pH, and bacterial activity, leading to corrosion currents in the metal.

The manufacturing techniques for the different pipe material have changed considerably over the years. The evolution of casting methods for grey iron pipes is a good example of this. The first pipes were horizontally cast in sand moulds resulting in uneven wall thickness.

Later, vertical casting was introduced, resulting in more even wall thickness and allowing the supply of pipes with thinner walls Erdogan and Wei(1984)..

Pipe Diameter: There seems to be total agreement in the literature that highest number of failures is found in pipes with small diameters (Andreou, 1986; Eisenbeis, 1994). Pipes with diameters less than or equal to 200 mm have particularly large number of failures. The high frequency of failures for small pipe dimensions is explained by reduced pipe strength, reduced wall thickness and less reliable joints for smaller pipes (Wengström, 1993b).

2.4 Water Leakage Detection Techniques

There are different techniques that are used for leakage reduction globally. Among the many, some are described below. The modern techniques are very efficient but economic constraint is the bottleneck for adopting in the case of Addis Ababa.

Acoustic Equipment: Listening devices include listening rods and ground microphones; may be either mechanical or electronic. They use sensitive mechanisms or materials such as piezoelectric elements to sense leak-induced sound or vibration. Modern electronic listening devices incorporate signal amplifiers and noise filters to make the leak signal stand out. Leak inspectors conducting leak surveys work their way around the pipe network systematically and use listening rods at appropriate pipe fittings to detect the characteristic hissing sound created by leaking water. The leak detection effectiveness of listening surveys depends on the size of leaks, ambient noise from road traffic and water draw, and the degree of detail of the survey. General surveys, performed by listening at only convenient fittings such as fire hydrants and/or valves, mainly detect large leaks. On the other hand, detailed surveys conducted by listening at all pipes fittings, including curb-stops, can detect small leaks.

Ground microphones are used to pinpoint leaks by listening for leak noise at the ground surface directly above pipes at small intervals. This process is time consuming and its success depends on the experience of the user.

Noise loggers are compact units composed of a vibration sensor (or hydrophone) and a programmable data logger. They are used to leak survey large areas but they are not suitable for pinpointing leaks.

Recent models of acoustic noise loggers can be deployed permanently – leak noise is processed using onboard electronics and the stored result is transmitted wirelessly to a roaming receiver.

The effectiveness of acoustic leak-detection equipment depends on several factors including pipe size, type, and depth; soil type and water table level; leak type and size; pipe pressure; interfering noise; and sensitivity and frequency response of the equipment.

Soil type and the water table level influence the strength of leak signals at the ground surface significantly. Leak sounds are more audible on sandy soils than on clayey ones and on an asphalt or concrete surface than on grass. Leak signals are muffled if the pipe is below the water table level (Røstum, 2000).

Leak noise correlators: These are portable microprocessor-based devices that can be used in either leak survey or pinpointing modes. They are based on the cross-correlation method, which involves the measurement of leak noise (either sound or vibration) at two locations on a pipe section. Measured noise is transmitted wirelessly to the correlator, which then determines the position of the leak based on the time shift of the maximum correlation of the two leak signals, propagation velocity of leak noise, and the distance between sensing points. The distance between sensors can be read from distribution system maps when the correlator is used in survey mode but it should be measured onsite accurately when it is used in pinpointing mode. Propagation velocities for various pipe types and sizes are programmed in most correlators, but they should be measured onsite to improve pinpointing accuracy, especially for non-metallic pipes. Leak noise correlators are more efficient, yield more accurate results and are less dependent on user experience than listening devices.

However, existing equipment requires extensive training and can be unreliable for quiet leaks in cast and ductile iron pipes and for most leaks in plastic and large diameter pipes. Correlators are also expensive and remain beyond the means of many water utilities and leak detection service companies. (Røstum, 2000).

Tracer gas technique: In this technique, a non-toxic, water-insoluble and lighter-than-air gas, such as helium or hydrogen, is injected into an isolated section of a water pipe. The gas escapes at a leak opening and then, being lighter than air, permeates to the surface through the soil and pavement. The leak is located by scanning the ground surface directly above the pipe with a highly sensitive gas detector. (Røstum, 2000).

Thermography: The principle behind the use of thermography for leak detection is that water leaking from an underground pipe changes the thermal characteristics of the adjacent soil, for example, making it a more effective heat sink than the surrounding dry soil. Thermal anomalies above pipes are detected with a ground or air-deployed infrared camera. (Røstum, 2000).

Ground-penetrating radar: Radar can be used to locate leaks in buried water pipes either by detecting voids in the soil created by leaking water as it circulates near the pipe, or by detecting sections of pipe which appear deeper than they truly are because of the increase in the dielectric constant of water-saturated adjacent soil. Ground-penetrating radar waves are partially reflected back to the ground surface when they encounter an anomaly in dielectric properties, for example, a void or pipe. An image of the size and shape of the object is formed by radar time-traces obtained by scanning the ground surface. The time lag between transmitted and reflected radar waves determines the depth of the reflecting object. (Røstum, 2000).

The LeakfinderRT System: LeakfinderRT is a new system for locating leaks in municipal water distribution and transmission pipes using the cross-correlation method. This leak noise correlation system is fully realized in software for personal computers. The system utilizes the PC's soundcard and other multimedia components to record and playback leak signals. It also uses the PC's central processing unit to perform the cross-correlation operation and associated signal conditioning. Modern PCs incorporate fast CPUs and high-resolution soundcards and hence offer several advantages over existing commercial hardware implementation of the cross-correlation method.

Hardware components of the LeakfinderRT system are composed of leak sensors, wireless signal transmission system, and a PC. The use of PCs eliminates the need for a major component of the usual hardware of leak noise correlators.

Locating leaks in water distribution and transmission pipes is a classical application of the cross-correlation method. Two things make this possible. First, the propagation velocity of leak sounds in water pipes is nearly constant over the dominant frequency range of leak sounds. Second, water-filled pipes transmit leak signals for long distances. Therefore, the shape of leak signals does not change significantly as they travel away from the leak, which is a pre-requisite for a successful correlation.

The correlation function of leak noise signals measured at the two points that bracket the location of a suspected leak provides information about the time delay between the two signals. The time delay between the two leak signals is the result of one measurement point being closer to the leak location than the other. LeakfinderRT determines the time delay corresponding to the peak of the cross correlation automatically (Hunaidi, Wang, Bracken, Gambino, Fricke, 2004)

2.4.1 Earlier Hydraulic Analysis Methods and EPANET

Using mathematical models to analyze flow in water-distribution system networks has been in existence for more than 60 years since it was proposed by Cross in the 1930s called the Hardy Cross method. Basically, all hydraulic modelling of water-distribution systems are conducted by solving mathematical equations that describe the pipe network of the distribution system. For each point in time, at network nodes (junctions), an equation for conservation of mass is solved. At each network pipe (link), an equation relating head loss and flow is solved.

Modern computers greatly simplified the then highly time consuming analysis. Among the modern computer models is EPANET, a computer program prepared by Environmental Protection Authority of America that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps.

In addition to chemical species, water age and source tracing can also be simulated. EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution systems analysis.

Sampling program design, hydraulic modelling, chlorine residual analysis, and consumer exposure assessment are some examples. EPANET can help assess alternative management strategies for improving water quality throughout a system (EPANET user's Guide, 2004). The details of EPANET Algorithm are provided on appendix B.

3 Theoretical Background

3.1 Relationships between Pressure and Leakage

Pressure can affect system losses in a number of ways. The rate of leakage from leaking pipes or faulty joints will increase with a rise in pressure. In a similar way, especially in older systems, an increase in pressure even by a few meters can result in large leakage frequencies. Conversely, pressure reduction can reduce the rate at which bursts occur.

Pressure surge can happen when a pump is switched on too quickly, or a valve is opened or closed too quickly. The sudden surge in pressure can cause the pipe to fracture, or can move thrust blocks, or damage the socket. There is also some evidence that surge can cause pipes to flex and move against rocks, resulting in local stress concentrations, and sometimes pipe failure especially in PVC pipes.

The relationship between leakage and pressure can be given by orifice equation for individual leaks:

$$Q2/Q1 = (P2/P1)^{N} \qquad \text{Eqn. (3-1)}$$

Where: Q1 is the flow from the leak at pressure P1; Q2 is similarly flow through leak at pressure P2.

The value of N in equation (3-1) has considerable effect on leak reduction program.

a. If N=0.5, reducing pressure by half reduces leakage by 29 %;
b. If N=1.5, reducing pressure by half reduces leakage by 65 %;
c. If N=2.5, reducing pressure by half reduces leakage by 82 %.

Literatures agree that based on analysis conducted in different countries and cities, the value of N ranges from 1 to 1.5 for municipal distribution systems, which indicates existence of about linear relationship between leakage and pressure (Lambert and McKenzie, 2005)

Thus, reduction of pressure in distribution systems by half results in reduction of leakage by half to even 65% depending on the network condition. In addition, Lambert noted that pressure reduction and stabilization not only reduces rate of leakage but also decreases new leakage frequency.

3.2 Leakage Measurement Techniques

3.2.1 Leakage from Reservoirs

Leakage from reservoirs is measured by conducting a drop test. The aim is to measure the rate of fall of the water level over the duration of the test, with both the reservoir inlet and outlet valves closed. The reservoir should be full before the start of the test. The rate of drop can be measured in several ways and calculated as follows:

Leakage from reservoir = $\frac{(d1-d2)}{T} \times A$ ……………………………………………Eqn. (3-2)

Where: d1 = initial depth (m)

d2 = final depth (m)

A = surface area of reservoir (m²)

T = test duration (hours)

3.2.2 Leakage in Transmission Lines

By-pass method can be used for measuring trunk main leakage. To use this method the following conditions must be met:

a) *The main can be taken out of supply;*
b) *There are valves on the main;*
c) *The valves can be closed and are drop-tight;*
d) *There are usually no branch connections from the main, or they can be turned off during the test.*

The procedure of leakage measurement in transmission line is as follows:

a) *Selection of a length of main to be tested.*
b) *Isolation of the section of main under test by closing a valve at each end of the test section.*
c) *Installing a positive displacement meter on a bypass around the upstream valve.*
d) *Record the meter readings before and after the test period.*

Any leakage from the section under test will be recorded by the meter. Leakage is calculated by the following formula:

Transmission main leakage = $\frac{\Delta Q}{LT}$ ………………………………………….. Eqn. (3-3)

Where: ΔQ is the difference in water meter readings (volume) before and after the test

L is mains length considered and

T is the test duration

3.3 Performance Indicators and Leakage Indexing

The objectives of Leakage Indexing are to have Practical approaches to Leakage. There are various performance indicators of Leakage used for different purposes. The customary ones are:

a) *leakage expressed as percentage of water supply;*
b) *leakage can also be expressed in terms of water supply pipe length per day and;*
c) *leakage expressed in terms of service connections per day*

Percentage of system input volume is easily calculated and frequently quoted. It is Level One Financial PI for Non-Revenue Water (NRW). It is, however, unsuitable for assessing the efficiency of management of distribution systems. This is because calculated values of percentage NRW do not distinguish between Apparent and Real Losses; it is strongly influenced by consumption and it is difficult to interpret for intermittent supply situations.
IWA recommended 'volume/service connection/day as Level one customary PI for systems with more than 20 service connections/km of mains is when the system is pressurised. Principal reason for this suggestion is that the greatest proportion of annual real losses volume occurs on service connections. Although 'per service connection/day' is the most robust of the traditional PIs for operational management of Real Losses, it does not take into account of mains length, customer meter location (relative to the property line) or average operating pressure.

A more recent PI is the ILI, Infrastructure Leakage Index, method. The method is used in this thesis and is elaborated for it delivers meaningful practical management aspect over the customary ones.

ILI provides greater insight into the systems being analyzed, without much additional effort. ILI also incorporates pressure as a parameter in leakage indexing - which none of the customary PIs dof2 not do. For these reasons, the ILI is a more meaningful basis for performance comparisons, benchmarking, target setting and analyses (Lambert and McKenzie, 2000).

3.3.1 Definition of Terms under ILI Method

The volume of Unavoidable Annual Real Losses (UARL) is the lowest technically achievable annual Real Losses for a well-maintained and well-managed system. The volume of Current Annual Real Losses (CARL) is the existing annual Real Losses calculated from water balance. The Infrastructure Leakage Index (ILI) is the dimensionless ratio of CARL to UARL.

$$\text{ILI} = \frac{CARL}{UARL} \qquad \text{Eqn. (3-4)}$$

3.3.2 Assumptions and Equations for UARL Calculation

The equations used for calculating UARL are based on auditable assumptions for frequencies and durations of the different types of leaks, and their typical flow rates related to pressure. Above all, it is assumed that a linear relationship exists between leakage and pressure for most large systems. The required data on four key system-specific factors to calculate ILI are: length of pipes, number of service connections, location of customer meter on service connection, average operating pressure There are certain assumptions used in calculating real losses. Assumptions for new leak frequency on mains of 13/100 km/year and on service connections of 5/1000 service connections/year were based on published studies of repair statistics (Hirner and Sattler, 2001).

The simplest practical metric form, Equation (3-5), is as follows:

$$\text{UARL (lit/d)} = (18 \times L_m + 0.8 \times N_c + 25 \times L_p) \times P \qquad \text{Eqn. (3-5)}$$

Where: L_m = pipe length (km); N_c = number of service connections; L_p = total length of private pipe property line to customer meter (km); P = average pressure (metres);

3.3.3 Practical Limitations of ILI Method

UARL formula cannot be recommended with confidence for systems with fewer than 5,000 service connections; because the numbers of certain classes of new leaks from year to year may be so small that the assumption of typical average flow rates for each class in component analysis may be invalid.

Additionally, it is not recommended for Systems with less than 25 metres average pressure. The main reason is the assumption of a linear relationship is not good approximation at such low pressures for all metal, or all non-metal, systems.

Moreover, it is not commonly applied for density of connections less than 20 per km; and thus it is difficult to find well-managed fully metered systems to adequately test the UARL predictions and assumptions at such low connection densities. That may result in the UARL assumptions for frequencies of unreported leaks to be too high for long rural systems, where any significant new leak may cause a low pressure complaint or supply failure. (Lambert and McKenzie, 2000)

Addis Ababa water supply system has about 250,000 connections, and the length of mains is 496km (AAWSA, 2006). Accordingly, the supply system is out of the limitation with connection density of 504 connections per km. Based on discussion with experts, its average pressure is estimated to be 68m over 24 hrs period.

The study area comprises of about 250,000 connections, and the length of mains is 496km (AAWSA, 2006). As there is no specific water pressure data for the sub systems, the overall figure for the city is used for the study area.

3.4 Interpretation of ILI Values

In practical terms, ILI values close to 1.0 mean that 'world-class' leakage management ensuring that annual Real Losses are close to the 'Technical Minimum' value at current operating pressures. However, such low ILI values are only likely to be economically justified when marginal costs of water supply are relatively high, or water is scarce, or both. Some Water Utilities with well-deserved reputations in advanced management of Real Losses, and ILI values of less than 1.5 are recorded.

ILI value depicts that Proposed solutions may exist for lowering Annual Real Losses to around an inverse of the Current Annual Real Losses, if there are no changes in the current pressure management regime. Besides, it indicates that additional changes in Real Losses will result from changes in the pressure management.

3.5 Hydraulic Modeling in Water Supply Systems

3.5.1 Hydraulic Modelling

Water supply models are used to support decision-making in different ways. In its broader sense water supply network modelling in general is a key input:

a) to understand how the water supply system operates under various demand/flow scenarios,

b) to assess the performance of the water supply system in the event of various failure events

c) to assess the impacts of proposed operational modifications

d) to review the impacts of proposed developments

e) to provide the supporting information for a planning study.

In this study, however, it is focused on the first most important purpose of network modelling with special emphasis on the effect of pressure on leakage. Fig. 3-1 illustrates Water leakage and Pressure Relationship with time in distribution Systems based on European experience.

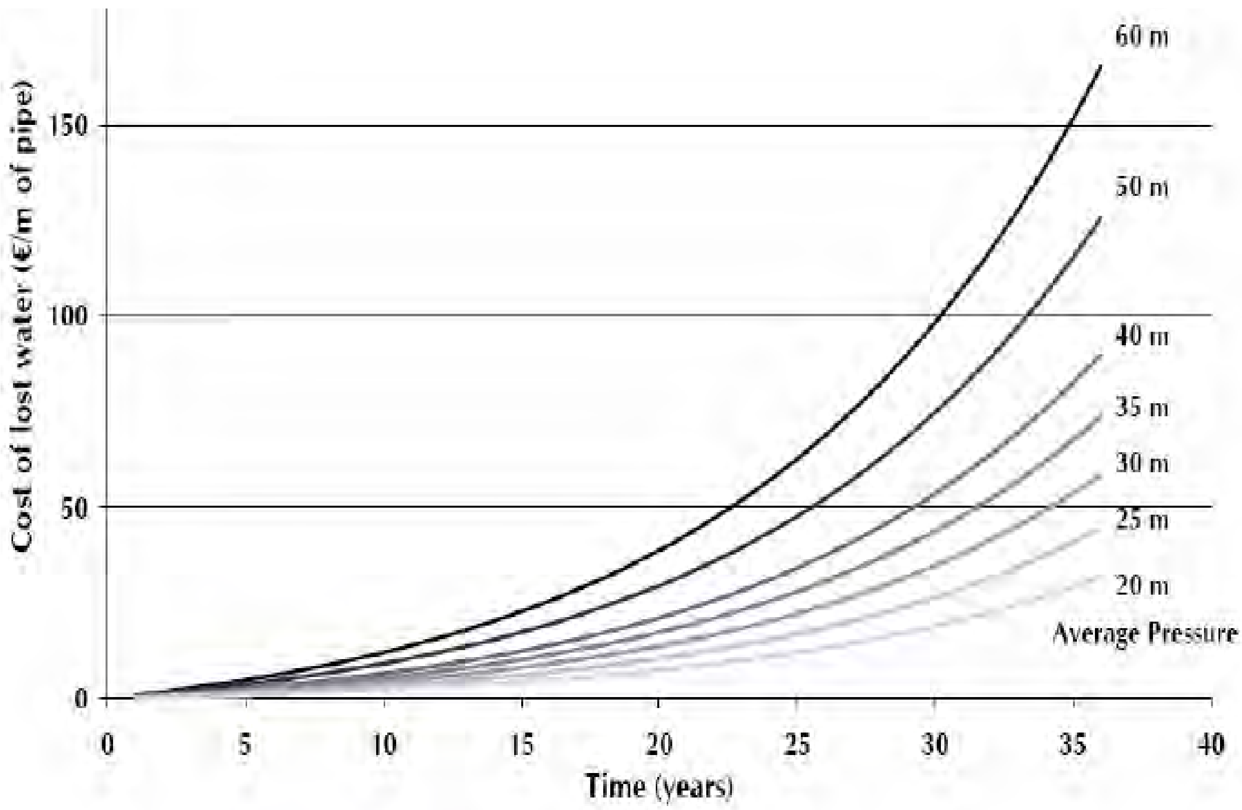


Fig. 3-1 Water Leakage and Pressure Relationship with Time in Distribution Systems
Courtesey Cobacho, Enrique Cabrera and Pardo (2007)

3.5.2 Calibration and Sensitivity Analysis

Calibration consists of determining the physical and operational characteristics of an existing system and the data, which after being input to the computer model, will yield realistic results. Calibration of a water distribution model is a two-step process consisting of (Walski, 1983):

a) *Comparison of pressures and flows predicted with observed pressures and flows for known operating conditions (i.e., pump operation, tank levels, pressure-reducing valve settings), and*
b) *Adjustment of the input data for the model to improve agreement between observed and predicted values.*

It involves the process of fine-tuning a model until it simulates field conditions for a specific time horizon (such as maximum hour conditions) to an established degree of accuracy.

According to American Water Works Association Engineering Computer Applications Committee "true model calibration is achieved by adjusting whatever parameter values need adjusting until a reasonable agreement is achieved between model-predicted behavior and actual field behavior (AWWA Engineering Computer Applications Committee, 1999)

Once a model is considered to be calibrated, it can then be used to, among other purposes, estimate hydraulic characteristics of the real system at locations where measured data are unavailable or unknown, spatially and temporally.

According to the AWWA Engineering Computer Applications Committee (1999) there are ten sources for possible error that could cause poor agreement between simulated model values and measured field values. These sources of error provide a potential list of parameter values that can be adjusted during the model calibration process and are:

a) *errors in input data (measurement and typographical),*
b) *unknown pipe roughness values (i.e., Hazen-Williams "C-Factors"),*
c) *effects of system demands (distributing consumption along a pipe to a single node),*
d) *errors in data derived from network maps,*
e) *node elevation errors,*
f) *errors introduced by time variance of parameter values such as tank water levels and pressures,*
g) *errors introduced by a skeletal representation of the network as opposed to modeling all small-diameter pipes,*
h) *errors introduced by geometric anomalies or partially closed valves,*
i) *outdated or unknown pump-characteristic curves, and*
j) *poorly calibrated measuring equipment including data measurement and tank water-level monitors.*

3.5.3 EPANET

Basically, all hydraulic modeling of water-distribution systems are conducted by solving mathematical equations that describe the pipe network of the distribution system. For each point in time, at network nodes (junctions), an equation for conservation of mass is solved. At each network pipe (link), an equation relating head loss and flow is solved.

Among the modern computer models is EPANET, a computer program, prepared by Environmental Protection Authority of America that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps.

In addition to chemical species, water age and source tracing can also be simulated. EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution systems analysis.

Sampling program design, hydraulic modelling, chlorine residual analysis, and consumer exposure assessment are some examples. EPANET can help assess alternative management strategies for improving water quality throughout a system.

These can include:

a) *altering source utilization within multiple source systems,*
b) *altering pumping and tank filling/emptying schedules,*
c) *use of satellite treatment, such as re-chlorination at storage tanks,*
d) *·targeted pipe cleaning and replacement.*

Running under Windows, EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded network maps, data tables, time series graphs, and contour plots.

The main objective of using EPANET in this paper is for hydraulic modelling emphasizing on pressure. EPANET contains a state-of-the-art hydraulic analysis engine

Frictional Head loss Formula: The hydraulic head lost by water flowing in a pipe due to friction with the pipe walls can be computed using one of three different formulas:

a) *Hazen-Williams formula*
b) *Darcy-Weisbach formula or*

c) Chezy-Manning formula

Each formula computes head loss between the start and end node of the pipe based on formula 3-6:

$h_L = A \times q^B$ …………………………………………………………... Eqn. 3-6

where h_L = head loss (Length), q = flow rate (Volume/Time), A = resistance coefficient, and B = flow exponent.

Although Chezy formula is the most precise of all, the Hazen-Williams formula is the one used in this study because of its readily available c-values. Besides, it is the most commonly used head loss formula in many countries. However, it cannot be used for liquids other than water.

3.5.4 Allocation of Demands

Basically, billing meter aggregation method is used in demand allocation to nodes where kebele boundaries belong to only one sub system. Otherwise, the flow distribution technique is used as illustrated below.

Flow distribution strategy involves distributing lump-sum area water-use data among a number of service polygons and, by extension, their associated demand nodes. The lump-sum area is a polygon for which the lump-sum water use of all of the service areas and their demand nodes within it is known but the distribution of the total water use among the individual nodes is not known. The known flow within the lump-sum area generally is divided among the service polygons within the area using one of two techniques; either equal distribution or proportional distribution.

The proportional distribution Proposed solution divides the lump-sum flow among the service polygons based upon one of two attributes of the service polygons; the area or the population. In this method, the greater the percentage of the lump-sum area or population that a service polygon contains, the greater the percentage of total flow assigned to that service polygon will be.

3.5.5 Calibration Criteria and Recommended Pressure Values

The goodness of fit criteria used was correlation value of 0.98 sufficient for the data set. The following criteria are also adopted from the USA for poor data set to verify the calibration results.

a. An average pressure difference of ±4.3 psi(2.96m) with a maximum difference of ±14.2(9.78m) psi for a "poor" data set (Walski, 1983); and
b. The difference between measured and simulated values should be ±5 psi(3.445m) to ±10 psi(6.89m)

The maximum pressure in mains is considered not to exceed 80m to limit leakage and stresses on pipes. The absolute maximum is, however, 103m (Mark J.Hammer, 2003).

From manufacturer's catalogues excess pressure that all the pipes can sustain as obtained from the specifications is found to be 172m for DCI pipes and 137m for PVC pipes. EPANET proposes pressure to be limited to 80m to 100m, similar to that recommended by Mark Hammer.

4 The Research Methodology

4.1 Selection of the Study Area

This study is conducted on Addis Ababa Water Supply Mains particularly in Rufael and Saint Paul sub systems representing the current operational situations. Both sub systems are chosen due to their relative isolation from other sub systems, their proximity to one another and at the same time that they are interconnected to one another. Additionally, their data is relatively available. The selected sub systems are situated in wide elevation range from 2,440 m a.s.l. to high grounds of up to 2,815m a.s.l. Thus, the selected sub systems, under gravity flow condition, cover elevation range of about 375m which could depict the effects of pressure on leakage easily.

4.2 Selection of Modeling Software

The manual approach to hydraulic analysis like hardy cross method is full of iterations and is highly time consuming, liable to introduction of errors and is not user friendly. Thus, nowadays, manual procedures are impractical for hydraulic analysis of large water networks.

Among the many commercially available hydraulic soft wares, EPANET is chosen for modeling pressure in the sub systems for many reasons. Among all, flexibility, simplicity of operation, user friendly outputs and availability of the software at no cost are the major ones. Last but not least, the software is highly versatile and applicable to a variety of networks that it is chosen for the study. The parameters used in modelling using EPANET are provided on Table 4-1.

Table 4-1 parameters used in modelling using EPANET

Units	Head loss	Trials	Accuracy	Unbalanced	Demand Multiplier	Tolerance
LPM	Hazen Wiliams	40	0.001	Continue 10	1	0.01

4.3 Collection of Data for Leakage Analysis and Modeling

Physical communication was made to Addis Ababa water and sewerage authority Research Department prior to commencing the study in order to preliminarily ensure the availability of the basic data for the study.

From its inception, it was planned to collect secondary data from AAWSA and other respective offices of the city and supportive qualitative information through discussion with local experts of AAWSA.

Accordingly, the required data are collected from different departments of AAWSA and other institutions as described below.

The total annual water produced and distributed to the distribution systems and the water billed that was aggregated from the individual customer meter readings were used to quantify the total water loss for the entire city and for the sub systems, respectively, on monthly basis. With the aim of determining apparent losses, meter inaccuracies and theft of water are studied. Meter inaccuracies are estimated based on AAWSA recommendations; whereas water theft is neglected based on previous study in the city. As per AAWSA report the annual water consumed for fire fighting and that consumed by the water authority itself was estimated to be an average of 66,656 cubic meters for both years, which accounts only 0.08 % of the total annual water supply. The real losses are calculated by deducting estimated meter inaccuracies from total losses.

To visualize the trend of water loss in the city, available water supply and consumption data is used for periods from 1990 to 1995; and 2005 to 2006.Accordingly, the percentage water loss is increasing for almost all of the years except for the year 2005, a year after leak detection project had been carried out and implemented as provided on Fig. 1-6 under chapter one.

4.3.1 The City Water Network Data

A digital water network for the entire city including their attribute like the size, age and material of the pipes has been collected in AutoCAD format. The collected pipe network mainly comprises of main pipes that cover the major part of the city; it was collected from water services section of AAWSA.

Volume of water supplied to the city and volume consumed in the city was collected for the duration of Sept. 2004 to August, 2005.

According to AAWSA classification, pipes above and equal to 125mm are termed main lines and those below 125mm are termed secondary pipes. Based on 2006 inventory of AAWSA, the city supply mains are about 496 kms long while the secondary ones are 1,060km long. The supply system has about 250,000 connections in the same year.

The cadastral information of the city was collected from a colleague who has been working on the area. The data includes information on buildings, parcels and blocks for each 303 Kebeles which are located in 28 former Weredas.

4.3.2 The Sub- System Water Network Data

Basically the entire city is sub-divided in to thirteen sub-systems, but in practice the pipe networks at the local level were interconnected to each other.

All the sub systems have water consumption data; whereas most of them lack water supply data due to malfunctioning of bulk meters. As explained earlier, two sub systems, Rufael and Saint Paul, have relatively reliable supply data but they are interconnected while relatively isolated from other sub systems that both systems are considered for the study altogether. Volume of water supplied to both sub-systems and volume consumed was collected for the duration of Sept. 2004 to August, 2006. In practice the data for 2006 has been used throughout the study, while the 2005 data has been used to evaluate data quality.

4.3.3 The City Reservoirs

The information on location of most of the water reservoirs were collected in conjunction with the main water network of the city. The reservoir data including their capacity, years of construction and material of construction were also collected. The locations of some of the reservoirs were not exactly indicated in the network, and the document found from the planning department indicates only the surrounding where they are located. Some of the reservoirs serve as transfer point to other reservoirs located elsewhere in addition to serving as a distribution to the surrounding areas. There are 56 reservoirs at 40 different locations with capacities ranging from 50m3 to 20,000m3. Their total capacities are 90,000m3. There are totally 24 electrical pumps to boost pressure in the distribution system (Welday Berhe, 2005).

Figures 4-1, 4-2 and Table 4-1 below show Photograph of Rufael reservoir, aerial view of the concrete tank at Rufael and reservoirs coordinates, respectively.

Figure 4-1 One of the Reservoirs at Rufael

(NB. patterned overflow leakage)

Figure 4-2 Areal view of Concrete Tanks at Rufael

(Modified from Google earth)

Table 4-2 Coordinates of Reservoirs and Water Tanks

X-Coordinates	Y-Coordinates	Node
469271.64	999602.49	"Reservoir RasHailu"
469989.11	1001383.54	"Tank Rufael"
470147.74	1000575.93	"Tank Saint Paul"

4.3.4 Water Supply

The main sources of water supply for the entire city are the "Gafarsa" and "Lagadadi" dams located 25 km East and 12km North West from the centre of the city, respectively.

Based on broacher of AAWSA, during the study, around 217,000m3 per day water is distributed from Gafarsa, Lagadadi, Dire reservoirs, Akaki ground water scheme and some wells and springs in the city.

Attributed to failures in most of the bulk meters, there was no organized water supply data, the annual water supply at city level has been collected from water services section. However, monthly supply data was not available at sub systems level that evaluations at sub systems were not possible.

4.3.5 Water Consumption

In order to evaluate the water supply loss, consumption data of each Kebele were collected from the computer Information Technology Service (ITS) of AAWSA. There were about 250, 000 customers in the entire city distributed over former Wereda and Kebele during the study year. The same data is also used in aggregation of nodal demands that is an input for the modelling.

There was discrepancy between the aggregated consumption data obtained from the water services section and the detailed data obtained from the Information Technology Services. The detailed data is used in reconciliation after cross checking with respective data of 2005.

4.3.6 Population Data

Census data of the country conducted in 1994 has been collected from Addis Ababa University, Technology faculty Post Graduate Library. The abstracted projected form by each Kebele for the year 2006 is used. Population growth rate of 3.78% is also taken from the same document.

4.3.7 Discussion with Local Experts

Although the study predominantly was planned to be conducted using secondary data, in order to support it qualitatively, discussion with local experts of AAWSA has also been carried out. The main motives behind were to check the pipe layout on the ground, to check places of high leakage frequencies and to clear out and assure data accuracy.

4.4 Data Verification

4.4.1 Water Network Data

The data on city water network that has been collected from Water Services Division was in AutoCAD format, updated in 2006.

While the network was reviewed in its original format, the pipe diameter, year of construction and its material type were written as an annotation. These attributes were analyzed thoroughly in conjunction with contour map, cadastral map, road network map and map of former kebele boundaries of the city for consistency.

4.4.2 Water Consumption Data

The data collected for more than 250,000 customers with in the entire city has been collected in Excel format after exporting from Oracle; aggregated at kebele level. The consumption data was collected for each month by six branches and compiled by the ITS.

As the data is very large, verifying its quality before aggregating to different levels was necessary.

The data quality checking criteria used were:

a) *Comparison with other months data*

b) *Comparison between respective months of years 2005 and 2006* *c) Evaluation based on expected consumption levels with respect to the season*

While the consumption data was reviewed, significant differences between consecutive months were observed that might be caused due to non-regular reading of meters. This has been also explained by the local experts.

The aggregated values were used for leakage analysis as these aggregated values were not affected much as the lower and higher individual values can be balanced to each other after minor adjustments for exceptional differences. For modelling purpose, water consumption of the year 2006 was used.

4.5 The Study Procedure

The processes involved in the study work are presented and described schematically on Fig. 4-3.

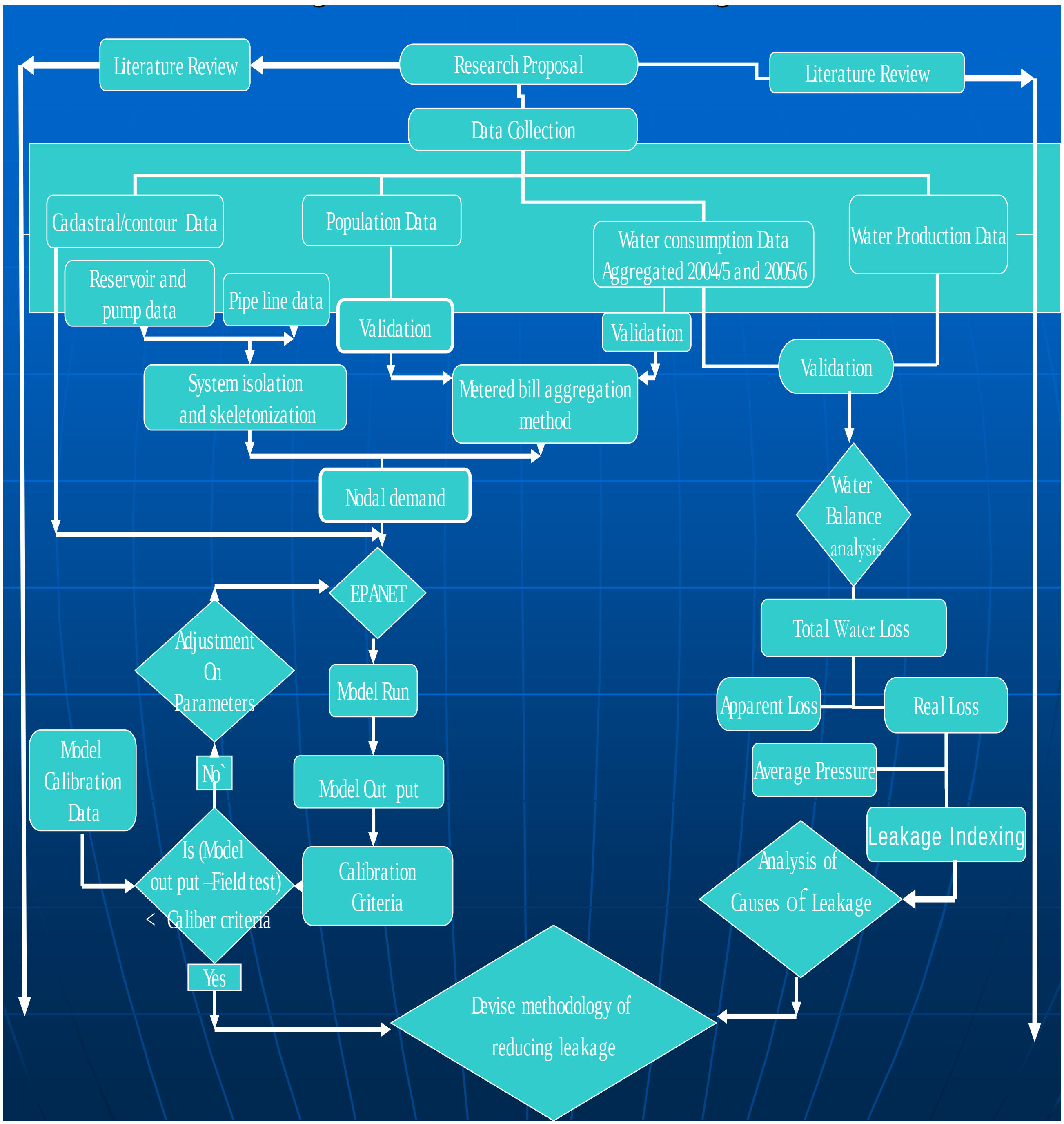


Fig. 4-3 Schematic diagram of the research process

4.6 Setting Performance Indicators

The total water loss analysis at both city and sub-system levels have been evaluated and compared using three traditional approaches of measuring loss, percentage of supply, loss per length of mains and loss per connection methods. However, special emphasis is given to a modern leakage indexing methodology called Infrastructure Leakage Index (ILI) method.

4.7 Limitations to the Study

Availability of reliable data is among the factors that influence the process and out puts of a study. Water supply to a city and to its sub systems is relied on bulk meter reading. In the city of Addis Ababa most bulk meters were not functional during the study. This can not be counter checked using any other method except for cross checking with supplies of other years. Although this method is employed in the study, the issue poses data reliability question.

On the other hand, the city receives water supply intermittently in shifts due to shortage of supply. Besides, the rationing time is not strictly observed at pumping stations that consistent results could not be obtained at fixed location and fixed time.

Last but not least, the water supply network is interconnected from sub system to sub system that there is no practical boundary between flows of sub systems.

5 Part I - Analysis of Leakage and Indexing

5.1 City Level Water Balance

As per AAWSA report the annual water consumed for fire fighting and that consumed by the water authority itself was estimated to be an average of 66,656 cubic meters for both years, which accounts only 0.08 % of the total annual water supply. Out of this, the authority estimates that its water use for flushing water supply pipe is 30,000m^3/yr.

Based on AAWSA water leakage detection project, flow meter inaccuracies are estimated to be 6% of the annual supply. Accordingly, meter inaccuracy for the year 2006 is 1,896.873.96 m3. Apparent loss from theft is significant in many African countries; however, this is negligible in AAWSA system (BCEOM in JV with G2C, 1997).

Apparent losses = Meter inaccuracies + Unauthorized consumption

= 1,896.873.96 m^3

As can be seen on Table 5-1 water balance for the year 2006 are provided. Accordingly, total water supply to the city was 83,170,607m^3 while the corresponding consumption was 51,556,041m^3; resulting in total losss of **31,614,566.00m^3**. The city level real loss is calculated to be the total loss minus apparent losses which equals **35.73%** of total water distribution or 29,717,692.04 m^3 for the year 2006.

Real Loss = Total Loss - Apparent Losses

= 31,614,566 m^3 – 1,896.873.96 m^3

=29,717,692.04m3

5.2 Sub system Level Water Balance

Only, aggregated annual water supply data was obtained from the water services unit for sub systems that comparison of leakage based on monthly basis was not possible. Supply of 7,437,196.73m^3 was reported to Rufael sub system while an average consumption of 4,544,425.31 m^3 was extracted from consumers' data. Accordingly, average leakage in Rufael sub system was found to be 2,892,771.42 m^3 accounting for 36.56 % of the water supplied to the sub system.

Similarly, an average supply of 6,269,259.35 m^3 was reported while an average consumption of 3,341,895.32 m3 was extracted from consumers' data for Saint Paul sub system. Accordingly, average loss in Saint Paul sub system was found to be 2,751,722.18 m^3 accounting for 43.89 % of the water supplied to the sub system.

The leakage in both sub systems is, however, 5,820,135.45 m^3 accounting for 39.91% of the water supply in the two sub systems. Table 5-1 summarizes these water balances at sub distribution level and at city levels using different indexing methods.

Table 5-1 City Level and Sub System Level Leakage for the year 2006

Sub system	Supply (m³)	Consumption (m³)	Loss (m³)	**Leakage (m³)	Leakage (%)	No of Connection	*Pipe Length (km)	Leakage m³ /km/d	Leakage m³ /conn/d
Rufael	7,437,196.73	4,544,425.31	2,892,771.42	2,719,205.14	36.56	11,153.00	90.75	82.09	0.67
Saint Paul	6,269,259.35	3,341,895.32	2,927,364.02	2,751,722.18	43.89	8,806.00	74.37	101.37	0.86
Both Sub Systems	13,706,456.07	7,886,320.63	5,820,135.45	5,470,927.32	39.91	19,959.00	165.12	90.78	0.75
City level	83,170,607.00	51,556,041.00	31,614,566.00	29,717,692.04	35.73	250,000.00	1,522.00	53.49	0.33

NB *Pipes above diameter 50 are considered as it shall include distribution pipes too.

**Based on AAWSA study, meter inaccuracies are estimated to be 6% of the total losses

5.3 Calculation of ILI Values

As stated in the theoretical background, UARL is an empirical formula based on specified assumptions and depends on total pipe lengths, number of service connection and existing pressure in the system. Although the ILI approach is a recent development that will probably be refined in the future incorporating other parameters like pipe diameter and friction factors, the latest metric empirical formula is employed here in the context of the study area.

The simplest practical metric form of ILI is described under Equation (5-1):

UARL (lit/d) = $(18 \times L_m + 0.8 \times N_c + 25 \times L_p) \times P$ ....(5-1)

Where: L_m = pipe length (km); N_c = number of service connections; L_p = total length of private pipe property line to customer meter (km); P = average pressure (metres)

a) City level

For Addis Ababa water supply system the following data are estimated in 2006:

Lm=1,522 km

Nc=250,000

Lp=4mx250, 000 (average) =1,000 km

$P_{avg.}$ *=68m*

UARL= (18x1,522+0.8x250,000+25x1,000) x68 lit/d = **17,162,928.00 lit/d**

From table 5-1, the Current Annual Real Loss is obtained to be **29,717,692.04 m3/yr** which is equivalent to **81,418,334.36 lt/d** for the year 2006.

Hence,

ILI for 2006 = $\frac{CARL}{UARL} = \frac{81{,}418{,}334.36}{17{,}162{,}928} = \mathbf{4.74}$

b) Sub-system level

At sub-systems level, for the specific study area, the following data are estimated in 2006:

$Lm = 165.12\ km$

$Nc = 19{,}959.00$

$Lp = 4m \times 19{,}959.00\ (average) = 79.84\ km$

$P_{avg.} = 68m$

UARL= (18x165.12+0.8x19,959+25x79.84) x68 lit/d = 1,423,604.48 lit/d

From table 5-1, the Current Annual Real Loss is obtained to be **5,470,927.32 m3/yr** which is equivalent to **14,988,841.97 lt/d** for the year 2006.

Hence,

ILI at sub-systems level for 2006 = $\frac{CARL}{UARL} = \frac{14{,}988{,}841.97}{1{,}423{,}604.48} = 10.53$

5.4 Interpretation of ILI Values

In practical terms, ILI values close to 1.0 mean that 'world-class' leakage management ensuring that annual Real Losses are close to the ***'Technical Minimum'*** value at current operating pressures.

a) City level ILI value

The result of this study shows that AAWSSA could possibly reduce its real loss to UARL values which equals 6,269,555.28 m^3/yr, provided it is economically justified with respect to the marginal cost of supply of the same volume of water.

It means that overall 23,448,136.76 m^3 of the leaking water could have been saved if appropriate leakage control measures were implemented; which is equivalent to 78.90% of the current leakage in the existing system; or 28.19% of the total annual water supply can be saved even without changing the existing pressure regime. This figure is an indication of the existing operational and leakage control mechanism in AAWSA.

Thus, AAWSA can strive for this 28.19% reduction target in its strategy in the long run, even, without changing the current pressure regime. However, it shall be understood that there is also open room for pressure reduction that will also decrease the leakage further.

b) Sub-systems level ILI value

Similarly, at sub –systems level AAWSSA could possibly reduce real loss to an inverse of the ILI values of the annual water leakage which equals 519,556.25 m^3/yr, provided it is economically justified with respect to the marginal cost of supply of the same volume of water.

It means that overall 4,951,371.07 m^3 of the leaking water could have been saved if appropriate leakage control measures were implemented; which is equivalent to 90.50% of the current leakage in the existing sub-systems; or 36.12% of the total annual water supply to the sub-systems can be saved even without changing the existing pressure regime. This figure is an indication of worse existing operational and leakage control mechanism in the sub-systems as compared to city level.

Thus, AAWSA can strive for this 36.12% reduction target in its strategy the sub systems in the long run, even, without changing the current pressure regime. However, it shall be reminded that there is also open room for pressure reduction that will also decrease the leakage further.

5.5 Comparison of City level and Sub-systems level leakage

At city level, leakage expressed in terms of volume (m^3) per km per day is 52.19 while that expressed in terms of volume (m^3) per connection per day is 0.33; the corresponding ILI value is 4.74.

At sub systems level, the corresponding gross values are: 90.78, 0.75 and 10.53, respectively. All these indicators confirm that the situation at the selected sub-systems level is worse as compared to city level figures.

5.6 Assessment of Causes of Leakage in the sub-Systems

Ages of Pipes: Many of the factors for water leakage are age-dependent - their effect will be greater with time. Consequently, the age of a pipeline can appear to be the most significant factor affecting the likelihood of leakage, but on its own, age is not necessarily a factor. 57% of the pipes in the sub systems are above 30 years while the remaining 43% are only 20 years. Accordingly, the significance of this factor on leakage is limited for DCI pipes. The average age of main pipes in the sub-systems are summarized on table 1-1.

Load over Pipes: Vibration and high loading caused by heavy Lorries is thought to be a major factor affecting buried pipelines and leading to pipe failure. The pipelines in the study area are layed away from major loadings; accordingly, load over pipes is not among governing factors in the study area.

Previous Failures: Temporal and spatial clustering of water-main breaks, indicates that a previous break increased the likelihood of future breaks in its immediate vicinity. In the conditions of the study area, there are no proper compactions carried out during maintenance operations. Based on discussion with local experts, frequent breaks are caused within vicinity of repair operations which indicates that this factor is among requiring careful considerations.

Soil Conditions and Movement: The type of soil and its permeability is an important factor, as it affects the length of time a leak is allowed to continue. As the soil type in the study area is silty clay, leaks in this soil can continue indefinitely without showing. This significantly affects leakage by delaying reporting time.

Soil movement is caused by changes in moisture content, particularly in clays, causing shrinkage changes. In combination to the prevailing cast iron pipes in the system, soil movement moves joints, or result in localized stress concentrations within the pipe leading to failure. However, this is beyond the scope of the study.

Pipe Material: In the case of iron pipes, tubercles develop on the wall of the pipe, and these are associated with pitting and localized areas of metal attack. The pipe wall thickness is reduced so that the pipe loses its ability to withstand pressure, leading to eventual penetration and failure of the pipe wall, and obviously leakage.

External corrosion can arise from a number of causes - aggressive soils may cause damage because of differing levels of dissolved salts, oxygen, moisture, pH, and bacterial activity, leading to corrosion currents in the metal. However, this area requires further study beyond the scope of this study to conclude its importance level.

Pressure: The prevailing pressure at some nodes exhibits above the recommended 80m value; which is among parameters requiring attention. Pressure contributes to about 18% of leakage in the sub-systems.

Operational practices: As can be seen from leakage indexing part, most of the leakage is attributed to this factor in both city level and sub system levels. The contribution to leakage as a result of poor operational practices is about 78%.

6 Part II - Hydraulic Modeling in the Study area Supply Mains

6.1 Model Scope

From its inception, the paper has been focusing on modeling pipeline mains in Addis Ababa water supply system for leakage reduction. The main lines in the supply system are more than 496 kms that forced limiting the scope further.

Accordingly, in the aim of selecting an appropriate sub system, skeleton of the city water supply network mains was analyzed. Finally, the paper concentrated on sub systems of Rufael and Saint Paul as a unit. The reasons for choosing the two sub systems altogether was considering the relative availability of data, relative confinement of supplies within the systems and too much cross flows within these sub systems.

6.2 Key Assumptions and Limitations

The assumptions and limitations provided below are partly simplifying and are based on actual conditions. However, the assumptions will not alter the goal of obtaining a model close to real situation as long as both temporal and spatial representative testing is conducted during calibration and simulation, that will be verified by comparison of actual versus simulated values.

6.2.1 Reliability of data

Most bulk meters in the systems are not functional during the study. Thus, the reliability of data on the water supply from reservoirs and tanks to sub systems as obtained from the water services section is questionable. Rather it seems approximation of past records by the time the meters were operational.

On the other hand, water consumption data aggregated from customers' bills has some discrepancy from that provided by water services section in summarized form.

6.2.2 Intermittent supply

The city is being supplied water currently on intermittent basis. It is estimated that the city is receiving only 48% of the actual demand in 2007. This highly undermines the real demand that the model out put may vary from the actual situations by the time of sufficient supply.

6.2.3 Friction factors

Pipe friction factors are normally considered solely based on age and pipe material. It is well known that as metal pipes age their roughness tends to increase due to encrustation and tuburculation of corrosion products on the pipe walls. This increase in roughness produces a lower Hazen-Williams C-factor.

Although, bedding slope, water treatment standard, existence of leakage points and availability of dead ends highly influence friction factors, C-factor in the paper is limited to pipe material and age due to shortage of reliable data.

6.2.4 Bends and minor losses

As part of skeletonization, some meandering bends are approximated with straight lines in the system. This will, obviously, have its own conservative effect on the out puts of pressure in the model. However, this assumption will not alter the real situation as long as both temporal and spatial representative testing is conducted during calibration and simulation.

6.2.5 Demand category

AAWSA considers three consumption categories in its distribution system; namely residential, non residential and fountain. The selected study area is highly residential that the effect of other consumption categories is ignored in the model. Besides, the share of fountain in the city supply system is only 1.93% that it is also neglected.

6.2.6 Reservoir versus tanks

Based on definition of EPANET it is assumed that the only reservoir in the sub systems is Ras Hailu Reservoir. Accordingly, it is considered as the only source for both Saint Paul and Rufael sub systems with unlimited supply. Saint Paul and Rufael reservoirs are considered as tanks since they are simply meant for flow balancing in the sub systems. The limitation, here, is that Ras Hailu reservoir might be in deficit of supply contrary to the definition of reservoir.

6.2.7 Cross connections

It is also assumed that there are no cross-connections to other sub systems or vice versa. However, practical situations are different from this fact. There are actual connections out side their sub system boundaries in each distribution sub systems.

6.3 The Procedure of Modeling

Two broad sets of data used for modeling the network are Pipe network data and Water Demand data. Under these broad categories are further sub divisions provided hereunder.

The Pipe Network Data: The water supply layout data consisted of spatial coordinates along with elevations for water nodal elements in the two sub systems including 32 km water pipe mains, 45 connection nodes, 6 centrifugal pumps and 8 circular storage tanks at three locations. Figure 6-1 shows one of the pumps (Left) at Ras Hailu reservoir that pumps to Rufael with its control panel (Right).

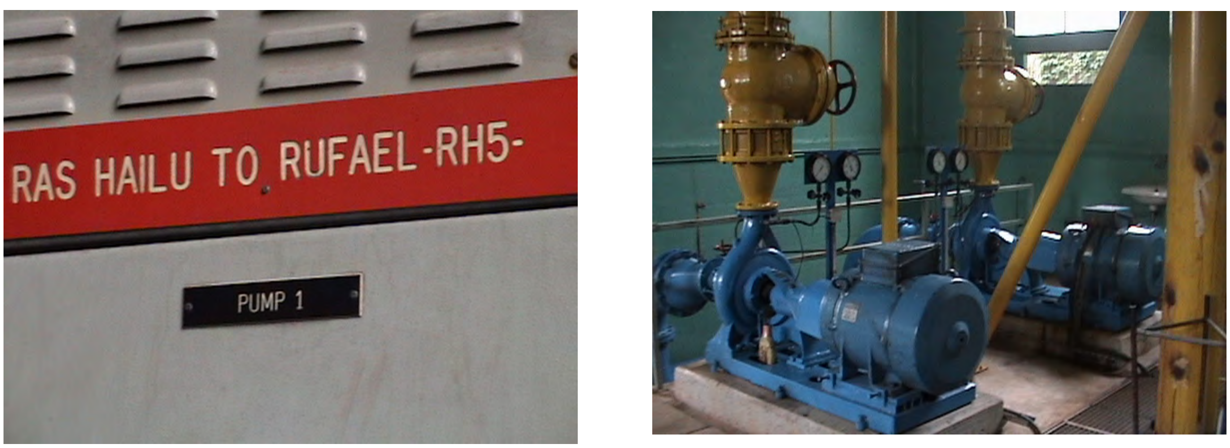


Figure 6-1 One of pumps and its control at Ras Hailu Reservoir control room

The Pipe data comprises pipe diameter, pipe length, pipe material, pipe age, pipe location in UTM coordinates and nodal elevations. Pipe friction factors are taken from standards for the respective pipe characteristic and age.

The pipe mains layout map obtained from Heavy line section of AAWSA was in AutoCAD format. Cadastral Map of the city of Addis Ababa with scale of 1:2,000 having contour interval of 2m was also in AutoCAD format. Besides, the old and new kebeles and kifleketema boundaries are obtained in similar AutoCAD format. Additionally, an AutoCAD format of Addis Ababa city road network is also utilized. These spatial data are obtained from both AAWSA and sources other than AAWSA.

According to a source from AAWSA, 37.72 % of the pipes in the network are above 20 years as provided on table 6-1.

Table 6-1 Pipe age in years in the city network with pipe sizes greater than 100mm in 2006

AGE CATEGORY	PIPE LENGTH (m)	PERCENTAGE of total
Less than 10 Years	52,047.00	7.67
10-20 years	370,675.00	54.61
20-30 years	189,911.00	27.98
30-40 years	31,980.00	4.71
40-50 years	34,096.00	5.02
Total	**678,709.00**	**100.00**

Water Demand Data: Water consumption data by category and by kebele is obtained from AAWSA computer services section. Population data of each kebele, projected based on 1994 census data, are used in determination of nodal demands. The area coverage of each kebele and area of kebeles in each sub system is carefully taken from the AutoCAD map of old and new kebele boundaries. These data are used to generate water demand at each node. Summarized demands at each node are provided under Appendix A.

Demand Pattern: Initial water demand pattern is generated in consultation with AAWSA experts. It was then modified during calibration to meet actual field data.

6.3.1 Model Reduction and System Isolation

It was important to develop a reduced model for convenient analysis of the water network. Skeletonization was used to reduce the model in size, while still preserving the network integrity, thus allowing similar analysis on a reduced model. System isolation was also implemented as a part of the skeletonization process. Accordingly, main pipes in Rufael and Saint Paul sub systems were sorted and isolated that simplified the complex pipe layout of the city. The skeleton of the pipe layout is provided on fig. 6-2.

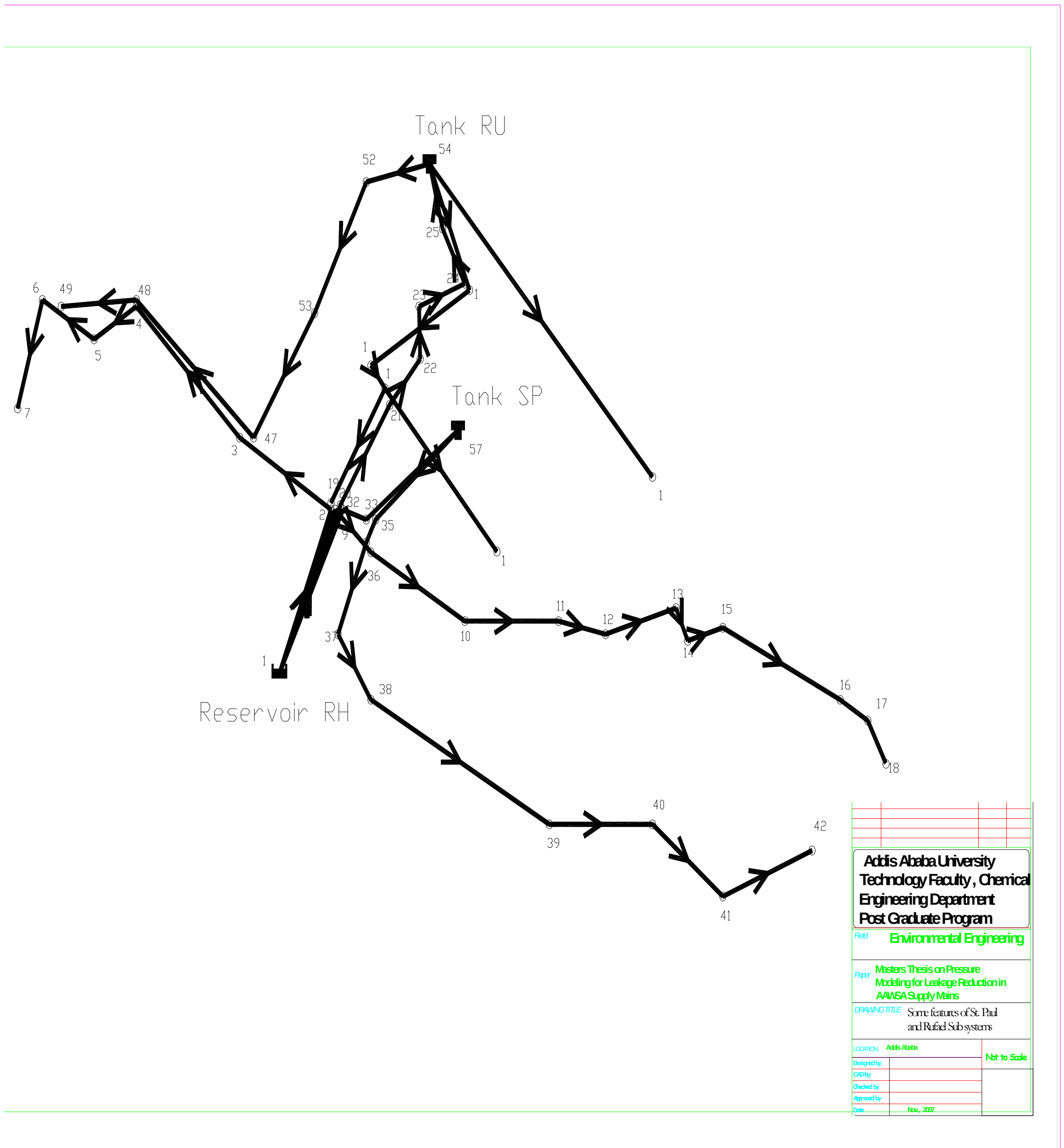


Fig 6-2 The skeleton of the pipe layout in AAWSA supply system

6.3.2 Allocation of Demands

Flow distribution strategy involves distributing lump-sum area water-use data among a number of service polygons and, by extension, their associated demand nodes. The lump-sum area is a polygon for which the lump-sum water use of all of the service areas and their demand nodes within it is known but the distribution of the total water use among the individual nodes is not known. The known flow within the lump-sum area is divided among the service polygons within the area using proportional distribution technique.

The proportional distribution Proposed solution divides the lump-sum flow among the service polygons based upon one of two attributes of the service polygons; the area or the population. In this method, the greater the percentage of the lump-sum area or population that a service polygon contains, the greater the percentage of total flow assigned to that service polygon will be.

In this study, the proportional distribution method is used. With this context, each service polygon has an associated demand node.

6.3.3 Model Verification

Calibration of a water supply network model requires the need for real time pressure monitoring in critical sections of the distribution system to verify the actual value with respect to modeled parameter values.

Most water supply schemes have some form of data logging systems which allow real-time measurement of system performance. However, in AAWSA this is not the case. Thus, observed reservoir levels and measured pressures at specific time and location were used to verify the model. These calibration runs were conducted on six selected points each measured four times followed by adjustments on hydraulic parameters each time calibration tests are conducted.

6.3.4 Setting Calibration Criteria

The goodness of fit criteria used was correlation value of 0.98 sufficient for the data set as discussed under chapter 3.

6.3.5 Hydraulic Calibration

Test Protocol: A test protocol was developed to ensure that successful data-gathering would occur during the field-test activities. The following procedure was used during the field tests:

a) *The water tap was closed*
b) *A pressure gauge was installed on the water tap*
c) *The water tap was reopened, and a pressure reading was obtained from the pressure gauge and recorded*
d) *The pressure gauge was removed*

The collection of field data provides an opportunity to understand the operation of the real system at a specified number of locations and times. Accordingly, six data sets have been created to keep the consistency between the measured pressures and operation scenarios. To better reflect the system condition for each calibration dataset, demand patterns are adjusted along with pipe roughness and pump operation conditions.

Four calibration runs have been conducted for all data sets. Each run has improved gradually the goodness-of-fit between the observed pressures and the simulated pressures after time consuming trials.

Boundary Conditions: The initial comparison of the simulated and the observed tank levels for the tank at Saint Paul and at Rufael reservoirs indicated significant discrepancy between the actual and simulated result of 2.5 m and 2.8m, respectively.

6.3.6 Sensitivity Analysis

The collection of field data provides an opportunity to understand the operation of the real system at a specified number of locations and times.

After careful review of the data and the possible sources of error, parameters are narrowed to three categories that are believed to provide the greatest source for error in the study and therefore, subjected to possible adjustment during the calibration process. The parameters are:

a) *Pipe roughness values (Hazen-Williams "C-Factors"),*
b) *System demand patterns, and*
c) *Pump operation pattern and pump-characteristic curves (head versus flow values).*

A sensitivity analysis confirmed that, variation in "C-Factor" has insignificant influence on system pressures.

The system demand factors are used to distribute the average daily nodal consumption values over hourly time steps of an Extended Period Simulation. Initial estimates of system pattern were obtained from discussions held with AAWSA experts. During the calibration process, the individual hourly factors were modified by a trial and error method using the generalized practical consumption pattern (e.g. peak demand hours and low demand hours). It is important to note, however, that although individual hourly factors were modified, the total system-wide demand was not modified during the calibration process.

The spatial (nodal) distribution of consumption in terms of an average daily value derived from records provided by AAWSA, was not modified during the calibration process. Rather, as discussed above, the system demand factor pattern in EPANET was used to distribute the average daily consumption value to an hourly time step value.

A third model parameter that was adjusted during the calibration process was the pump operation period and pump-characteristic curves. Initial pump-characteristic curve data were provided by AAWSA. This was a key parameter that was justifiably modified during the calibration process.

Modifications were made to the original data so that system operations observed values during the tests are as close as possible to the simulated values. Trial and error method was used for four test rounds at six testing locations.

6.3.7 Correlation of Observed and Simulated Data

In order to accommodate the hourly-time step pattern of the Extended Period Simulation required by EPANET, the one-minute sampling data that were collected during the field tests were averaged to one-hour time periods.

Pressure measurements were taken four times at the selected six points and the model is run to assess actual system performance against model outputs. At initial level of calibration, correlation between the measured and computed data was only 0.854 which gradually fulfilled the correlation criteria with correlation between observed and simulated values finally being 0.985. Table 6-1 summarizes the final pressure readings at the selected test points.

Table 6-2 Final calibration readings at selected nodes

NO.	NODE	LOCATION	PARAMETER	DATE	TIME	READING (M)
1	9	Ras Hailu Reservoir	Pressure	July 21, 2007	3:20 PM	71
2	13	Paster to Enkulal Fabrika area	Pressure	July 21, 2007	3:40 PM	80
3	54	Rufael Reservoir	Water level	July 21, 2007	2:50 PM	1.5
4	57	St. Paul Reservoir	Water level	July 21, 2007	3:00 PM	0.0
5	18	Abune Petros area	Pressure	July 21, 2007	4:00 PM	98
6	49	G. Wingate	Pressure	July 21, 2007	3:40 PM	66

NB- Tests were conducted on nearby individual connections and additions were made to compensate for losses up to mains.

The criteria for each data and for the mean are also fulfilled except for node 18 with mean error of 15.78. The final correlation result is provided on Table 7-1.

6.4 Findings during the Study

6.4.1 Customer Service Operational Criteria

AAWSA specifies 10m head pressure at property line location for residential buildings; whereas, other countries specify correspondingly 20m head. The maximum pressure, however, shall not exceed 80m to limit leakage and stresses on the reticulation system (Hammer, 2003). In the study area, even negative pressures prevail for limited period of time.

6.4.2 Water Demand and Supply

Different countries have different consumption rates. In the USA municipal water use in general is 2,270 l/d/metered services (Hammer, 2003).

The corresponding value for Addis Ababa is 565 l/d/metered service, which exhibits that it is under acute shortage of water supply. It is not surprising with the fact that the city is also receiving only 48% of its demand in 2007(Capital newspaper, April 15, 2007). As a result, rationing of water in the city during the study period was a common phenomenon.

6.4.3 Disproportionate Mains

There are some specific technical issues concerning the existing water supply system, although beyond the scope of the paper, in the studied sub distribution systems such as inadequate pipe sizes and disproportional mains spatial distribution is observed.

These issues are critical to the western extremes of the sub systems. During the field study, it was learnt that Asko area up to Sansusi on one hand and Kale or Bottle Factory area on the other hand are provided water intermittently at intervals of four days. The areas are devoid of main line for more than 5 kms on the main road.

6.5 Out Puts of the Model

Nodal pressures for selected time periods are provided under Appendix A. From the out put of the model, critical nodes are identified that have significant consequences on leakage and system performance in general.

6.5.1 Junctions with excessive pressure

Results of the Model: The following junctions are identified as having excess pressure that accelerates leakage.

a) *On gravity mains from Saint Paul to Johannes church lower area; junctions 41 and 42 exhibit pressure in excess of 80m for almost all the twenty four hours.*

b) *On pressurised main from pump 57 to Abune Petros area, junctions 10, 11, 12, 13 and 15 exhibit excess pressures from 12:00 AM to 4:00 AM, from 1:00 PM to 5:00 PM and from 9:00 PM to 12:00 AM. Junctions 17 and 18 are almost all under excess pressure over twenty four hours except* *for 7:00 AM and PM. The highest pressure in the two sub systems is also recorded on junction 18 to be 113.78m from 10:00 PM to 4:00 AM.*

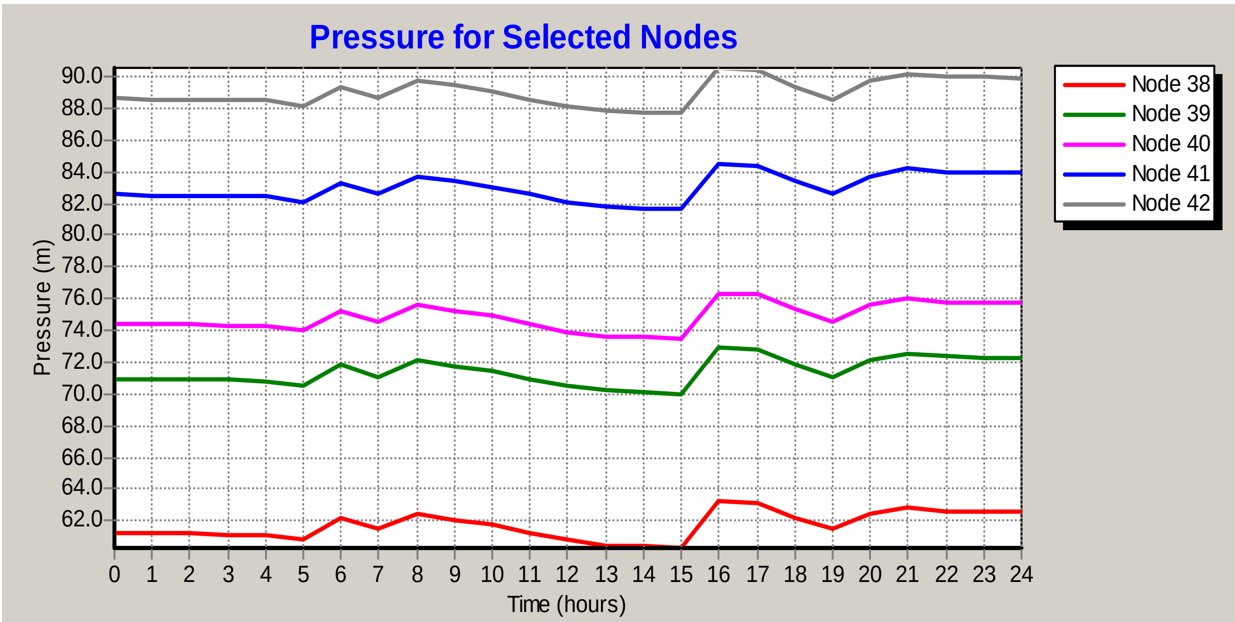


Fig. 6-3 Time Series Plot for selected Nodes - along gravity line from Ras Hailu Reservoir to node 42

6.5.2 Junctions with negative pressure

As mentioned under literature review, negative pressure has significant impact on water quality and the resulting social and Environmental effects.

The following junctions are identified as having negative pressure on the main line itself:

a) *On pressurised main from pump 57; junctions 14, 15 and 16 exhibit negative pressures at 6:00 AM and 7:00 AM.*

b) *On pressurised main from pump 58; junction 7 exhibits negative pressure at 7:00 AM.*

Technical specifications of AAWSA specify a minimum pressure at the property boundary to be 10m. This criterion is considered at the remotest elevated connection with its head loss from the main by extending the model to Sansusi area even though the mains are about 5kms far away from it. Accordingly, a remotest connection around Sansusi area shows that negative pressure prevails for most hours of the day. As a result, the area receives at an intermittent rationing schedule of four days interval from a 17m3 balancing reservoir pumped to most of the areas located west of General Wingate School.

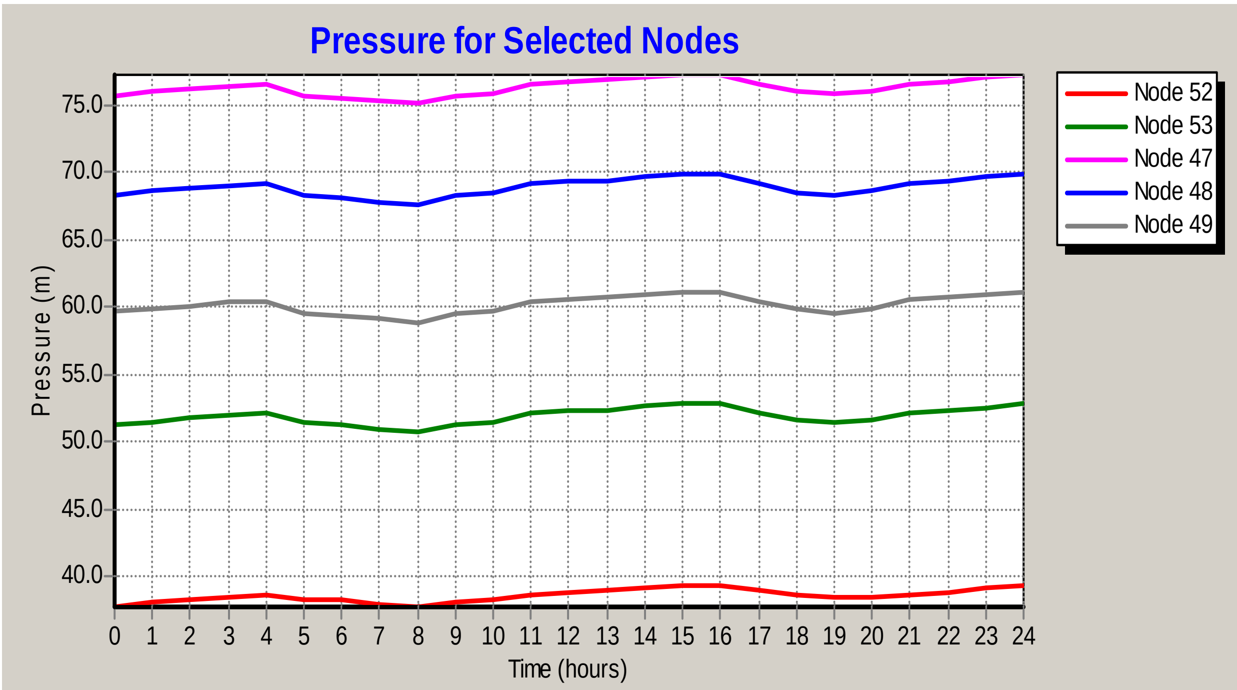


Fig. 6-4 Time Series Plot for selected Nodes -along Rufael to General Wingate area

6.5.3 Junctions with high fluctuations in pressure

The following junctions are identified as having high pressure fluctuations on the main line.

a) On pressurised main from pump 57; junctions 14, 15 and 16 exhibit fluctuations in pressures ranging from 71.19, 80.54, and 79.09 to -11.06, -7.95, and -11.48, respectively.

b) On pressurised main from pump 58; junction 7 exhibits pressure fluctuations from 10.27m to 56m.

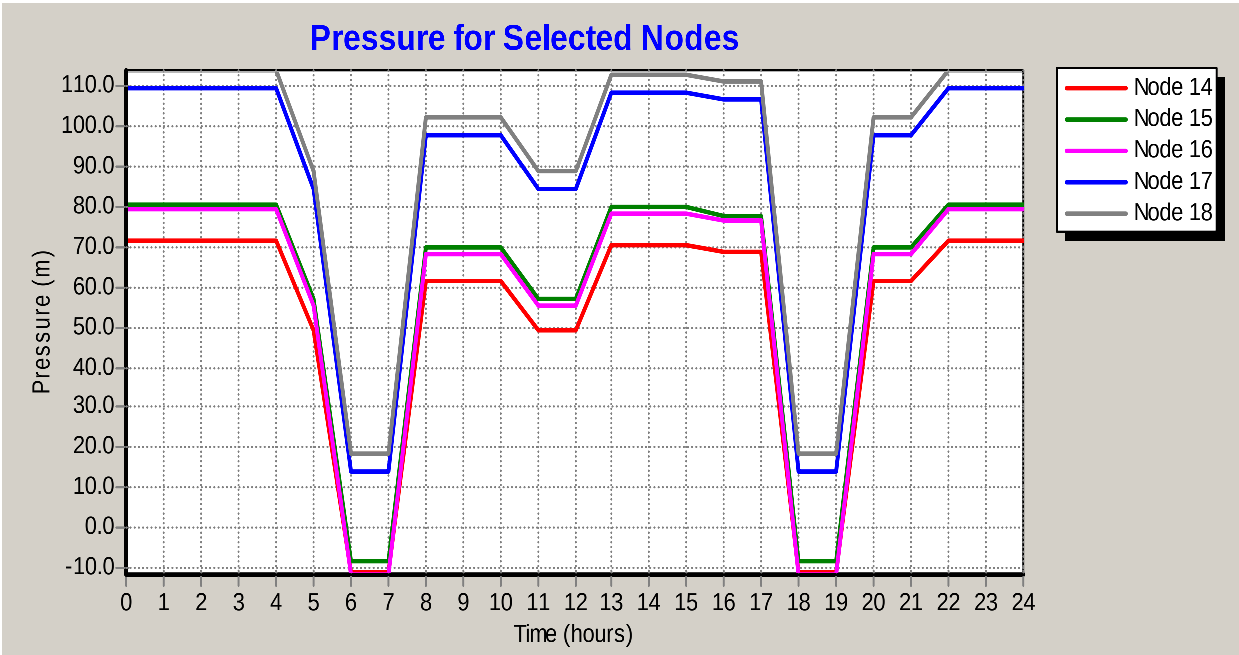


Fig. 6-5 Time Series Plot for selected Nodes-along Pressurized line from Ras Hailu to Abune petros area node 18

7 Interpretation of the Model Out put

7.1 Pressure Distribution

Critical pressure distribution in the network is shown on Fig. 7-1. Pressure distribution in the study area at 12:00 AM shows that 28 % of the network is under pressure in excess of 80m and 50% is in excess of 70 m at this time. On the other hand at 7:00 AM, pressure in excess of 80m is obtained on only 10% of the nodes.

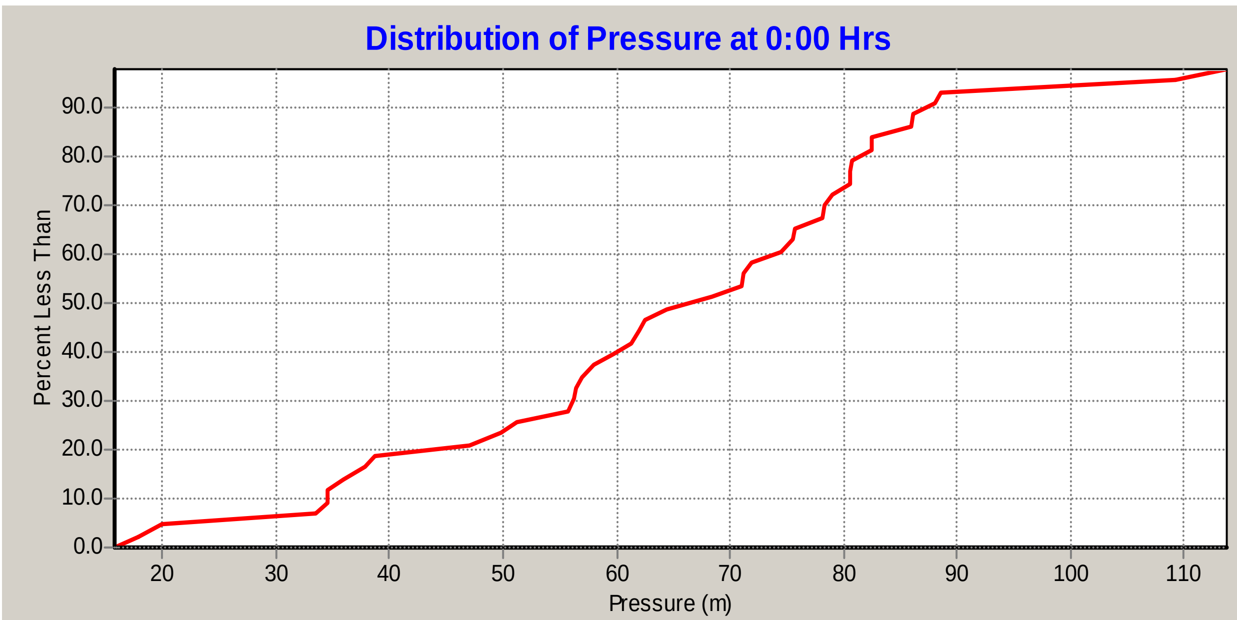


Fig. 7-1 Pressure distribution plot in the two sub systems Pre Proposed solutions

On the other hand, from the Pressure distribution plots, at 12:00AM, 18% exhibit less than 30m pressure, while at 1:00 AM, 26 % are below 30 m.

From literature, the recommended average pressure in mains is 50m, thus the system has to be upgraded to approach the recommended value at the stated time.

Thus, nocturnal pressure control is the most appropriate measure to reduce the effect of pressure on leakage.

Finally, calibration readings are conducted on selected critical nodes in order to check whether the out puts of the model comply to the outputs of the model at the critical points. Accordingly, table 7-1 elaborates calibration readings at theses critical nodes versus the model outputs.

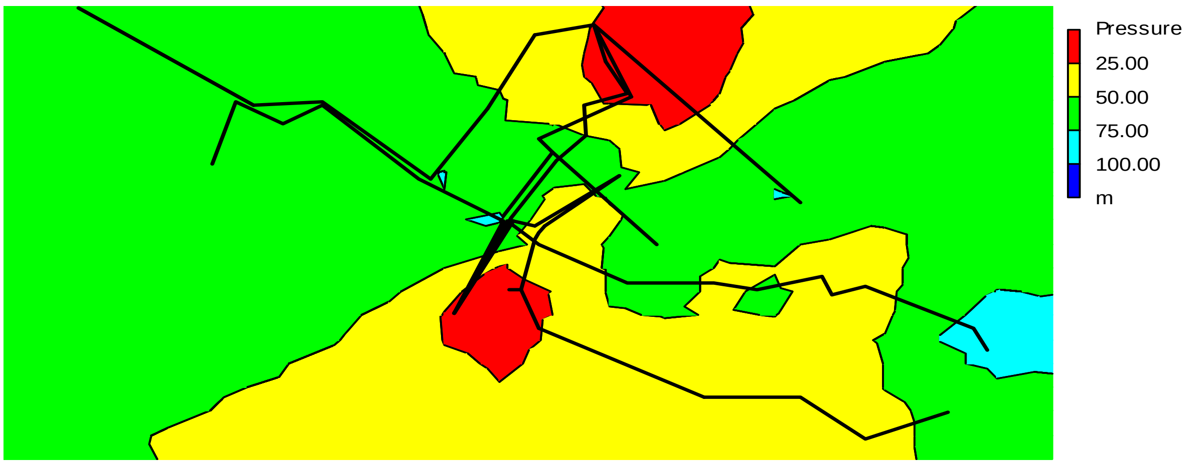
Table 7-1 Comparison of calibration readings with model outputs at critical nodes

NO.	NODE	LOCATION	PARAMETER	DATE	TIME	EPANET OUTPUT	READING (M)
1	41	St. Paul tank to *chew berenda* area	Pressure	Dec.22, 2007	3:20 PM	81.71m	80
2	42	St. Paul tank to *chew berenda* area	Pressure	Dec.22, 2007	3:40 PM	87.75m	88
3	11	RH reservoir to *Abune Petros* area	Water level	Dec.22, 2007	2:00 PM	85.57m	83
4	13	RH reservoir to *Abune Petros* area	Pressure	Dec.22, 2007	2:20 PM	85.34m	83
5	17	RH reservoir to *Abune Petros* area	Pressure	Dec.22, 2007	4:00	106.34m	100
6	18	RH reservoir to *Abune Petros* area	Pressure	Dec.22, 2007	4:20	110.79	108

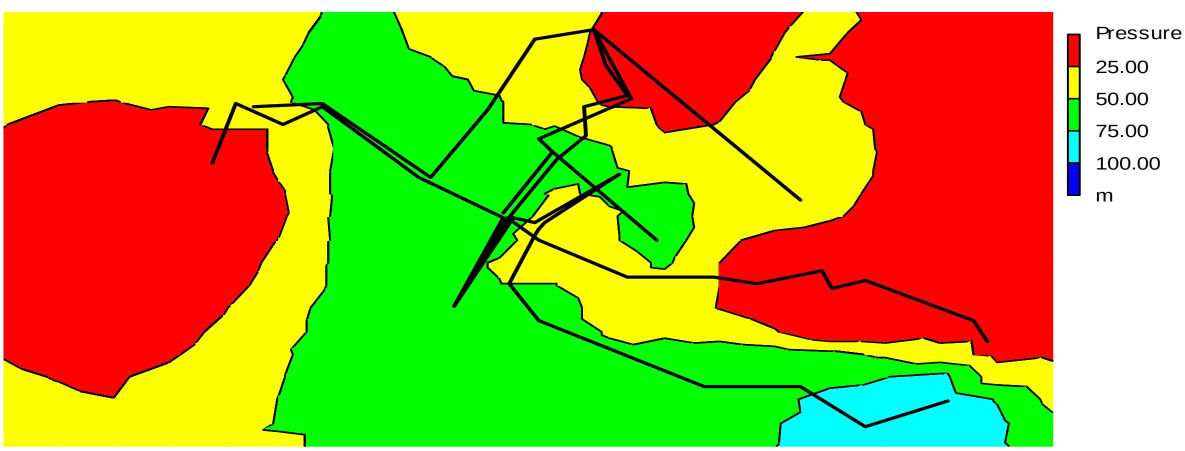
Upon comparison for final check at these critical points, it was observed that the simulated pressure shows somewhat differences from the actually measured values.

Accordingly, the differences on the nodes 41, 42, 11, 13, 17 and 18 were 1.71, 0.25, 1.43, 1.56, 1.64 and 3.21m, respectively. The highest difference being 3.21m at node 18; the reasons for the differences could be among the limitations explained earlier.

Although there is rarely documented data on locations of leakage in the sub systems, based on discussion with AAWSA experts, the most frequent leakage area conforms to the high pressure areas indicated on fig. 7-2.



i **Fig. 7-2** Pressure Contour plot at 12:00 AM Pre Proposed solutions



ii **Fig. 7-3** Pressure Contour plot at 7:00 AM Pre Proposed solutions

7.2 Proposed Solution to Reduce Leakage

From the analysis and modeling parts, it can be concluded that pressures are higher only during low demand period in night times except along gravity line from Saint Paul to node 42. As can be observed, pressure on junctions 41 and 42 are above 80m all the day.

Accordingly, the following solutions are proposed based on the findings of the previous chapters.

Proposed solution I: Time of day control approach is best suited for conditions like that of Addis Ababa where pressures are higher only during low demand period in night times. Its effect on customer pressure should however be considered before hand; exception here is along gravity line from Saint Paul to node 42.

It is recommended that pump 56 be changed to variable speed pump to be operated at 40m head from 7:00 PM up to 11:00 AM along the pressurized main from Ras Hailu reservoir to Abune Petros along junctions 9, 10, 11, 12 through junction 18 through adjustment on its pattern.

Proposed solution II: Since pressure on junctions 41 and 42 are above 80m all the day, it is further recommended that either an all time operating Pressure Reducing Valve (PRV) be installed along the gravity main line from Saint Paul tank to lower areas of *Johannes church* area on junction 41; or flow balancing tank be constructed next to junction 38.

Basically pressure reducing valve (PRV) is used to limit the pressure at a point in the pipe network that the valve will be closed if the *gauge pressure*, that is measured using pressure gauge, on the downstream side exceeds that on the upstream side. However, the side effect of PRV is that excessive pressure may develop downstream in case the PRV fails.

As a result, introducing flow balancing tank of 500m3 capacity at junction 37 may solve the problem. The simulation result of the model after incorporation of the recommendation shows gauge pressure downstream the tank is between 25m and 55m for almost all the 24 hours, which indicates that the pressure is well below the maximum of 80m.

Proposed Solution III: Diameter 150mm main pipe line from General Wingate school to Asko area terminates at Awelia, some 300m distance from General Wingate, followed by diameter 110mm pipe up to Asko area. The diameter 110 pipe used is inadequate that the supply to Sansusi on one hand and Kale or Bottle Factory area on the other hand are provided intermittently at intervals of four days. The areas are devoid of main lines for more than 5 kms on the main road. Thus, increasing the pipe diameter to 200mm up to Asko area is the proposed solution.

7.3 Simulation of the Model Post Proposed Solutions

The above proposed solutions are finally simulated to observe whether the desired reduction in pressure is achieved or not.

Accordingly, figures 7-3 through 7-6 illustrate pressure situations at selected time and nodes. The pressure distribution at the critical 12:00 AM hour is provided on figure7-7. From the plot it can be seen that the excessive pressure above 80m in the network is reduced to less than 5% of all the pressure in the system. Pressure less than 15m in the system is also limited to only 2%. The pressure in the system is within the best pressure range with 78% of the pressure distribution between 30m and 80m.

As the final simulation based on the proposed solutions reveals, individual values have also significantly improved.

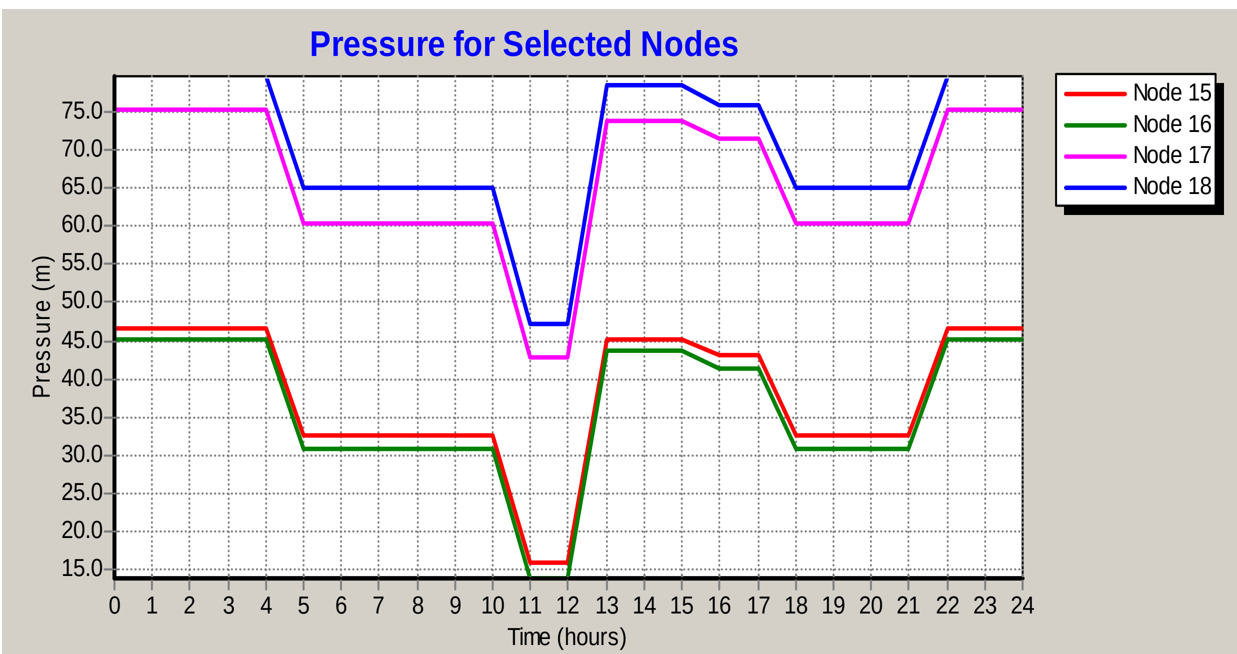


Fig. 7-4 Time series plot in the sub systems Post Proposed solutions along Pressurized line from Ras Hailu to Abune Petros area

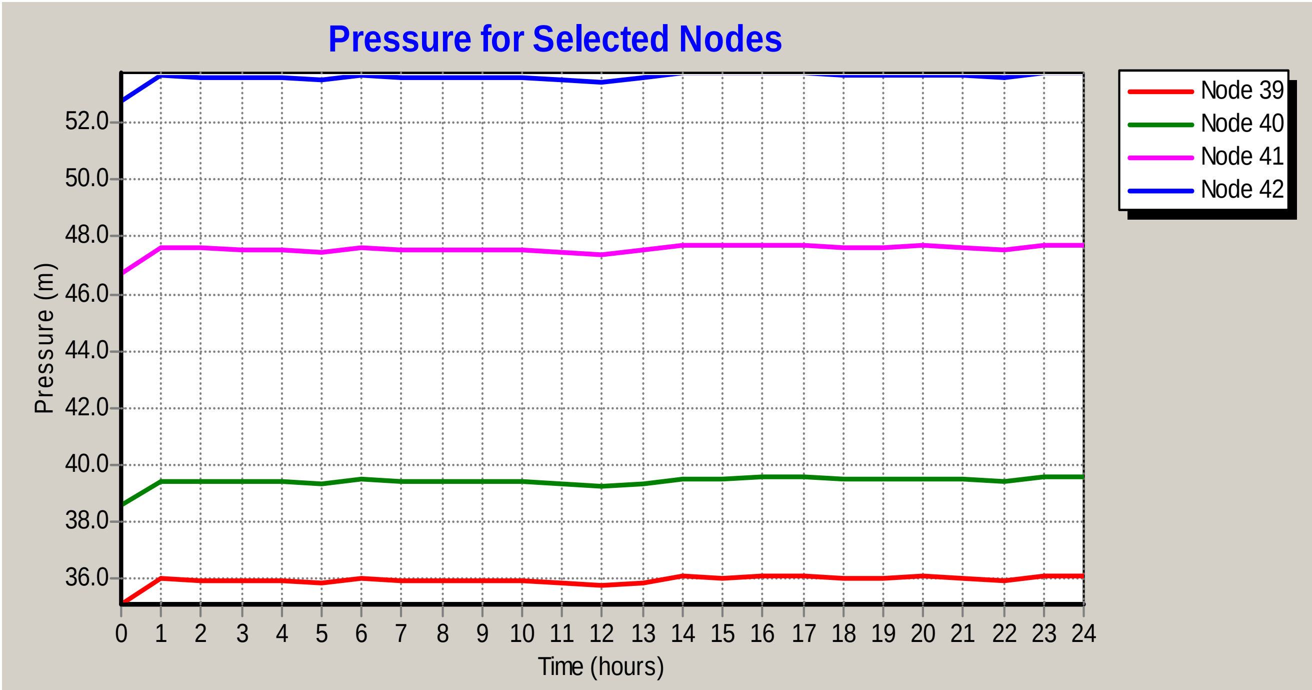


Fig. 7-5 Time series plot in the two sub systems Post Proposed solutions along Gravity line from St. Paul to Chew Berenda area node 42

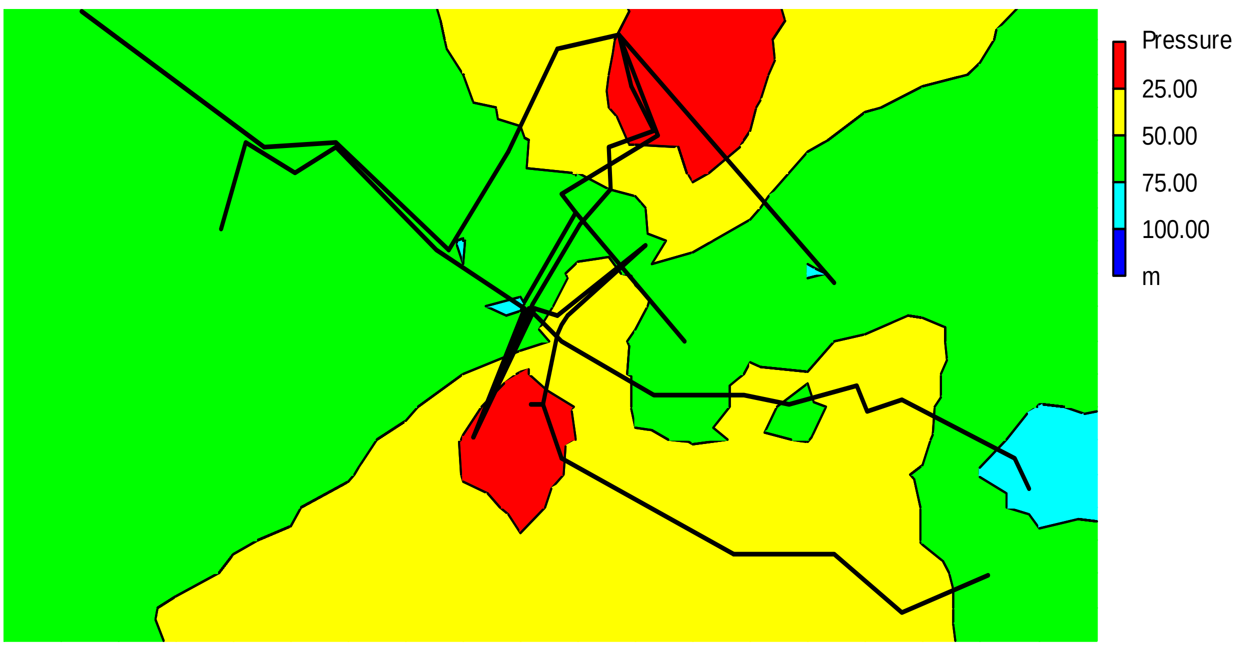
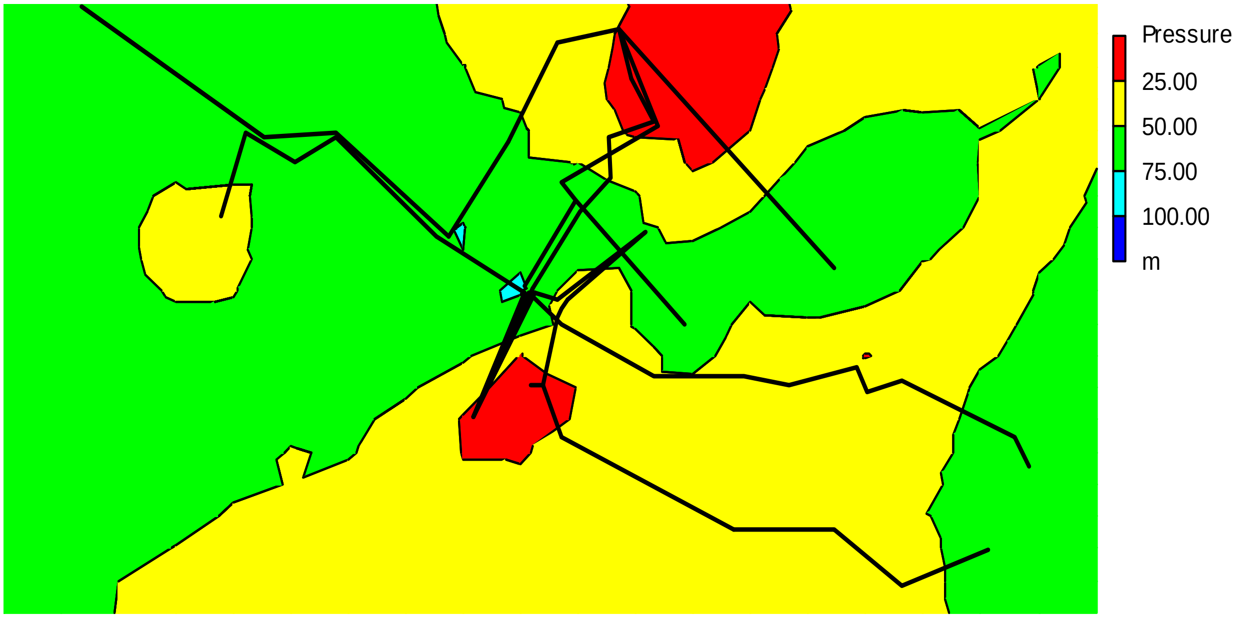


Fig. 7-6 Contour plot at 12:00 AM Post proposed solutions in the two sub systems



i **Fig. 7-7** Contour plot at 7:00 AM Post Proposed solutions in the two sub systems

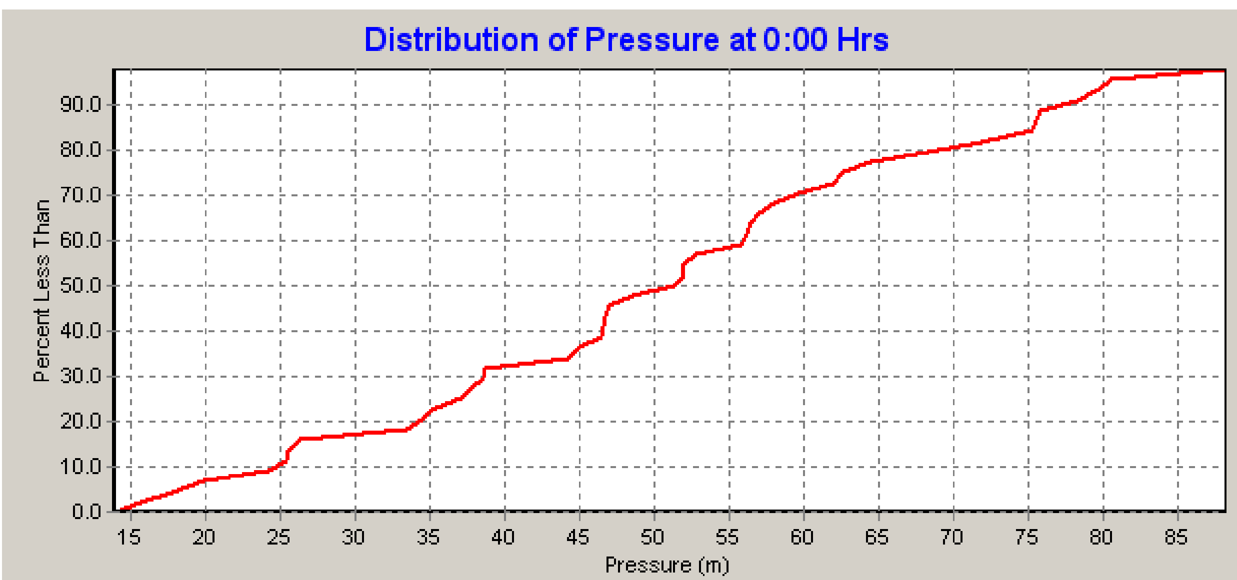


Fig. 7-8 Pressure Distribution plot in the two sub systems Post Proposed solutions

7.4 Assessment of Out-Put of the Model on Leakage

According to a pilot study conducted in South Africa, as a result of pressure management a significant reduction in night flow rate is achieved. An average reduction of 44.12% pressure resulted in 42.57% reduction in night flow rate. Since no particular research is conducted in Addis Ababa concerning the relation between pressure and leakage in the supply system, and from the fact that literatures support this linear assumption, one can adopt similar relationship for the case of Addis Ababa, too.

An average pressure of 52m would have been achieved after appropriate measure is implemented. It was observed that average pressure was 68m before the proposed solutions. Thus, the 20% reduction in pressure will result in about 20% reduction in leakage considering linear relationship between leakage and pressure. Annual net water leakage in the two sub systems being 5,470,927.32 m^3, the volume of water to be saved in the two sub systems alone as a result of pressure management only is estimated to be 1,094,185,46 m^3 annually.

Similarly, assuming the same pressure regime prevails in the entire city, and annual net water leakage at city level being 29,717,692.04 m^3, the volume of water to be saved at city level as a result of pressure management only is estimated to be 5,943,538.41 m^3 annually. That is about 20% of the total leakage.

7.5 Feasibility of the proposed solutions

7.5.1 Economic Analysis of the Pressure Control Proposal

In order to estimate the financial burden on the authority, cost estimates of the proposed solution are summarized on Table 7-2 as follows:

Table 7-2 Cost Estimate of the proposed solutions

Item	Unit	Quantity	Unit Price	Total	Remark
Bulk meters(diameter 250mm)	No.	1	50,000	50,000	
Gate valves(diameter 250mm)	No.	2	70,000	140,000	
500 m3 RC Tank	No.	1	1,500,000	1,500,000	
Total				**1,690,000**	
Contingency				**169,000**	
Grand total				**1,859,000**	

NB- Proposal for pipe change is not included as it is proposed solution which does not involve pressure

Table 7-3 provides power consumption of different pumps in the two sub systems.

Table 7-3 Pumping Energy Consumption

	Percent	Average	Kw-hr	Average
Pump	Utilization	Efficiency	/m3	Kw
55	37.50	75.00	0.24	10.44
56	62.50	75.00	0.31	91.33
57	100.00	75.00	0.24	11.96
58	100.00	75.00	0.26	13.36

From table 7-3, the average power consumption of the pumps is about 0.2625 Kw-hr/m3. On the other hand, the total leakage from *leakage analysis* part indicates that in the two sub systems 5,470,927.32 m3 water is being lost annually. Accordingly, the excess power consumption cost due to leakage alone is:

Power cost = 0.2625x5,470,927.32m3x0.273birr/kw-hr=392,060.33 birr annually.

On the other hand, Table 7-4 summarizes the financial benefits of the proposed solution.

Table 7-4 Financial Benefit of the Proposed Solution

Item	**Unit**	**Volume**	**Unit Price (birr)**	**Total (birr annually)**
Estimated water saved	m3	1,164,027.09	2.45	2,851,866.37
Pumping energy saving	M3	5,470,927.32	0.2625x0.273	392,060.33
				2,935,283.46

Besides, unquantifiable environmental and social benefits are there which are not included in this financial analysis quantitatively; rather these are provided qualitatively under Chapter Six. Economical analysis is conducted using the net Present value method of discount as given below.

$$NPV=\sum\frac{\left[(C_t-C_o)\right]_t}{(1+r)} \qquad \text{……(7-1)}$$

Where:

t - the time of the cash flow

n - the total time of the project

r - the discount rate

C_t - the net cash flow (the amount of cash) at time t.

C_0 - the capital outlay at the beginning of the investment time (t = 0)

7.5.2 Assumptions

The estimated rate of return is assumed to be 10 % with investment period of 10 years. Accordingly, Cash out flow at t=0, = 1,859,000; and administrative cost of 500,000 birr is assumed annually.

Annual cash inflow **= 2,935,283.46**; Discount rate = 10%

Table 7-5 Economic analysis using net present value of the proposed solutions

TIME (YR)	**PRESENT VALUE FORMULA**	**VALUE**
t=0	-1,859,000.00	-1,859,000.00
t=1	(**2,935,283.46**-500,000)/1.1^1	2,213,894.06
t=2	(**2,935,283.46**-500,000)/1.1^2	2,012,630.96
t=3	(**2,935,283.46**-500,000)/1.1^3	1,829,664.51
t=4	(**2,935,283.46**-500,000)/1.1^4	1,663,331.37
t=5	(**2,935,283.46**-500,000)/1.1^5	1,512,119.43
t=6	(**2,935,283.46**-500,000)/1.1^6	1,327,567.25
t=7	(**2,935,283.46**-500,000)/1.1^7	1,374,654.03
t=8	(**2,935,283.46**-500,000)/1.1^8	1,249,685.45
t=9	(**2,935,283.46**-500,000)/1.1^9	1,136,077.69
t=10	(**2,935,283.46**-500,000)/1.1^10	1,032,797.90
Total		**14,024,855.40**

As can be seen from the table, the proposed solutions are profitable for AAWSA to implement.

7.6 Summary

a) Sub-system Level Leakage Control

At sub-systems level an average pressure of 52m would have been achieved after appropriate measure is implemented from the prevailing average of 68m. The water leakage in the two sub systems being 5,470,927.32 m^3, the corresponding annual net volume of water to be saved in the two sub systems as a result of pressure management alone is estimated to be 1,094,185,46m^3 annually.

From the point of view of operational, leakage control and management, at sub –systems level AAWSSA could possibly reduce real loss to 519,556.25m^3/yr, provided it is economically justified with respect to the marginal cost of supply of the same volume of water.

It means that overall 4,951,371.07 m^3 of the leaking water could have been saved if appropriate leakage control measures were implemented; which is equivalent to 90.50% of the current leakage in the existing sub-systems; or 36.12% of the total annual water supply to the sub-systems can be saved even without changing the existing pressure regime.

Thus, AAWSA can strive for 36.12% reduction target in its strategy in the sub systems in the future, even, without changing the current pressure regime.

b) City Level Leakage Control

Similarly, assuming the same pressure regime prevails in the entire city, and annual net water leakage at city level being 29,717,692.04 m3, the volume of water to be saved at city level as a result of pressure management alone is estimated to be 5,943,538.41 m^3 annually.

From point of view of operational leakage control and management, at city level, AAWSSA could possibly reduce real loss to 6,269,555.28 m^3/yr, provided it is economically justified with respect to the marginal cost of supply of the same volume of water.

It means that overall 23,448,136.76 m^3 of the leaking water could have been saved if appropriate leakage control measures were implemented; which is equivalent to 78.90% of the current leakage in the existing system; or 28.19% of the total annual water supply.

Thus, AAWSA can strive for 28.19% reduction target in its strategy in the long run, even, without changing the current pressure regime.

c) Conclusion

As can be comparatively seen, the major and primarily attention seeking area is not pressure related control; rather it is implementation of operational leakage control measures. Thus, the primary focus of AAWSA with respect to leakage reduction shall be on operational and management efficiency than pressure control. Besides, it can be learnt that leakage control in the sub systems is worse than at city level.

8 Assessment of Potential Social and Environmental Effects of Leakage

8.1 General

Environmental Impact Assessment (EIA) is a tool used to identify the environmental, social and economic impacts of a project prior to decision-making. It aims to predict environmental impacts at an early stage in project planning and design, find ways and means to reduce adverse impacts, shape projects to suit the local environment and present the predictions and Proposed solutions to decision-makers (EPA, 2004).

Since EIA is a proactive measure in ensuring sustainability of a project whereas the water supply project of Addis Ababa is already in place and leakage is an existing situation, the potential Environmental impacts of leakage are addressed in the paper. Thus, formal environmental impact assessment is not expected from the paper. The thesis paper rather makes analysis on the direct and indirect environmental impacts of leakage through additional infrastructure required to maintain the volume of lost water.

8.2 Direct Potential Impacts

Effect on Resource Depletion: Above all, the direct Environmental Impact of Leakage is depletion of scarce resource, water. This issue is highly undermined due to the low value assigned to water as an economic good. The city is loosing 29,717,692.04 m3 annually with all the resources spent on it in its abstraction, treatment and distribution.

Effect on Human Health: For water quantity to act as an absolute constraint on hygiene, it must be available only in very small quantities. A minimum for basic health protection corresponds to 'basic access' and experience shows that this is equivalent to a water collection of less than 20 l/c/d, of which about 7.5 liters is required for consumption.

Dividing the lost water of 29,717,692.04 m^3 per year in AAWSA supply system by the basic minimum of 20 l/c/d, one can estimate the number of people whose health is being taken away by leakage in the city to be **4,070.9** people daily. This much of the population of Addis Ababa could have received the basic minimum water simply by forbidding the water loss.

Diseases primarily transmitted through consumption include infectious diarrhea, typhoid, cholera and infectious hepatitis. Study conducted on diarrhea suggests that improvement of water availability has significant impact on reductions in diarrhea accounting for a median 25% reduction attributed to water availability.

Inversely, it can be inferred that for the city is under high water stress condition - being served only 48% of the requirement. Accordingly, it can be deduced that **12 %** of the population of the city is currently susceptible to diarrhea case as a result of the mentioned loss of water.

8.3 Indirect Effects of Leakage

8.3.1 Potential Negative Indirect Effects

The construction of new reservoirs and infrastructure brings with them a number of problems. The timescales are significant, suitable sites can be hard to find often requiring compulsory purchase and in many locations it is associated with substantial local opposition.

Some of the major potential negative impacts of water supply projects arising from leakage at their different stages (Planning, Construction & Operation) are:

a) Psychological impact (disturbance of people along the project areas);

b) Loss of arable land: In Ethiopia arable and grazing land is scarce commodity; the largest part of the population depends on agriculture and cattle rearing. This situation becomes more critical in the vicinity of Addis Ababa where potential harnessing of water sources will be planned.

c) Damage on natural vegetation along the project areas;

d) Soil pollution due to residues of water treatment chemicals, spill of fuel & oil;

e) Water pollution;

f) Damage on houses and Infrastructures (including roads, telephone lines, power lines and water lines);

g) Accidents/Safety hazards (from motors, machineries, constructed structures, chlorine gas, explosions, fire, etc.);

h) Causing more water to be lost through infiltration and evaporation;

i) Change in biodiversity due to habitat & food web changes;

j) Flooding of upstream & downstream areas due to filling of raw water reservoirs; &

k) Potential underground disturbance

l) Disrupting flows and ecology down stream

m) Potential vector borne diseases

n) Alteration of the micro climate

Indirect Effect on Health and productivity: As a factor contributing to water scarcity, leakage further reduces the capacity of industrial and commercial sectors to grow, thus inhibiting economic growth. On the other hand, with out a minimal level of economic standing, a person's health and well-being are compromised.

Indirect Effect on Household income generation: Productive water use may be critical among the urban poor in sustaining livelihoods and avoiding poverty and therefore has considerable indirect influence on human health (Fass, 1993; Thompson et al., 2001). Traditionally productive uses are not considered as important. However, it is increasingly recognized that productive uses of water have particular value for low-income households and communities and have health and well-being benefits (Thompson et al., 2001). Direct health benefits are derived for example from improved nutrition and food security from garden crops that have been watered. In urban areas, this often is essential for low-income communities to meet nutritional requirements and may offer additional income from small-scale sales.

Indirect Effect on Supply reliability: It is likely that the nature of supply discontinuity will affect the hygiene condition. Whilst regular discontinuity may cause more hardship, this may be mitigated to some extent if the interruption in supply is predictable as this will allow the household to develop coping strategies for water collection. The greatest problems is felt when discontinuity is frequent, but very unpredictable. The current AAWSA water rationing method is common example and leads to collection of water from piped networks at odd hours, including late at night.

8.3.2 Potential Positive Indirect Effects

Some of potential positive impacts of water supply additional project at their different stages (Planning, Construction & Operation) are:

a) Job opportunities for skilled & unskilled labor;

b) Training opportunities, knowledge transfer & promotion for AAWSA employees;

c) Domestic & foreign business opportunities;

d) Fish resource development in raw water reservoirs.

Table 8-1 summarizes the above points, here under.

Table 8-1 Parameters for Analysis of Potential Environmental effects of Additional Infrastructure in Compensation for Leakage

		Positive impact very likely	***Positive impact possible***	***No impact***	***Negative impact possible***	***Negative impact very likely***
		A	***B***	***C***	***D***	***E***
Hydrology	*1-1 Low flow regime*					X
	1-2 Flood regime				X	
	1-3 Operation of dams				X	
	1-4 Fall of water table				X	
	1-5 Rise of water table				X	
Pollution	*2-1 Solute dispersion*				X	
	2-2 Toxic substances				X	
	2-3 Organic pollution				X	
	2-4 Anaerobic effects				X	
	2-5 Gas emissions				X	
Soils	*3-1 Soil salinity*				X	
	3-2 Soil properties				X	
	3-3 Saline groundwater			X		
	3-4 Saline drainage			X		
	3-5 Saline intrusion			X		
Sediments	*4-1 Local erosion*		X			
	4-2River morphology					X
	4-3 Channel regime					X
	4-4 Sedimentation					X

Ecology	*5-1 Project lands*					X
	5-2 Water bodies					X
	5-3 Surrounding area				X	
	5-4 Valleys and shores			X		
	5-5 Wetlands and plains			X		
	5-6 Rare species			X		
	5-7 Animal migration				X	
Socio-economic	*6-1 Population change*					X
	6-2 Income amenity					X
	6-3 Human migration					X
	6-4 Resettlement					X
	6-5 Sites of value			X		
	6-6 Regional effects				X	
	6-7 User involvement		X			
	6-8 Recreation		X			
Health	*7-1 Water and sanitation*			X		
	7-2 Habitation			X		
	7-3 Health services			X		
	7-4 Nutrition			X		
	7-5 Relocation effect				X	
	7-6 Disease ecology					X

	7-7 Disease hosts					X
	7-8 Disease control					X
	7-9 Other hazards					X
Imbalances	*8-1 Pests and weeds*				X	
	8-2 Animal diseases			X		
	8-3 Aquatic weeds				X	
	8-4 Animal imbalances				X	

8.4 Mitigation Measures of the Water Supply Upgrading Project

There are various mitigative measures to minimize and/or to alleviate potential negative impacts of the water supply upgrading projects resulting emanated from loss of water through leakage. Accordingly, table 8-2 Summarizes mitigation measures.

Table 8-2 Summary of mitigation measures

		Mitigation measure
Hydrology	1-1 Low flow regime	*Allow a reasonable amount of water for downstream*
	1-2 Flood regime	*Warn downstream flood situations*
	1-5 Rise of water table	*Take precautionary measures not to spill chemicals and contaminants*
Pollution	2-1 Solute dispersion	*Keep EPA standards*
	2-2 Toxic substances	*Apply preference of construction materials*
	2-3 Organic pollution	*Take precaution not to spill fuel, oil & chemicals on soil*
	2-4 Anaerobic effects	*Avoid transport of fertilizers to the reservoir*
	2-5 Gas emissions	*Apply preference of construction materials*
Soils	3-1 Soil salinity	*Avoid drainage of salts to the reservoir*
	3-2 Soil properties	*Apply reforestation/plantation programs*
Sediments	4-1 Local erosion	*Apply reforestation/plantation program*
	4-2 River morphology	*Backfilling the excavated soil*
	4-3 Channel regime	*Construct retaining walls ,Backfilling the excavated soil*
	4-4 Sedimentation	*Apply reforestation/plantation program, Backfilling the excavated soil*
Ecology	5-1 Project lands	*Construct retaining walls ,Apply preference of construction materials*
	5-2 Water bodies	*Carefully handle wastewater, Take precaution not to spill fuel, oil & chemicals on soil*
	5-3 Surrounding area	*Apply reforestation/plantation programs*
	5-4 Valleys and shores	*Construct retaining walls, Apply preference of construction*

		materials
	5-5 Wetlands and plains	*Carefully handle wastewater*
	5-6 Rare species	*Construct buffer zone*
	5-7 Animal migration	*Apply reforestation/plantation programs*
Socio-economic	6-1 Population change	*Carry-out consultation process*
	6-2 Income amenity	*Give adequate/reasonable financial compensation*
	6-3 Human migration	*Create awareness about the importance of the projects*
	6-4 Resettlement	*To find appropriate land for resettlement*
	6-5 Sites of value	*Construct retaining walls , Carefully handle wastewater*
	6-6 Regional effects	*Apply reforestation/plantation programs*
	6-7 User involvement	*Carry-out consultation process*
	6-8 Recreation	*Apply preference of construction materials, Carefully handle wastewater*
Health	7-1 Water and sanitation	*Apply preference of construction materials, Carefully handle wastewater*
	7-3 Health services	*First aid and health facility shall be established*
	7-5 Relocation effect	*Promote modern agricultural inputs*
	7-6 Disease ecology	*Health facility shall be established*
	7-7 Disease hosts	*Protective measures shall be taken*
	7-8 Disease control	*Training on possible diseases that may occur*
	7-9 Other hazards	*Carefully handle waste water*
Imbalances	8-1 Pests & weeds	*Apply preference of construction materials*

9 Conclusions

9.1 General

Sustainability is undermined by the excessive loss of water in transmission and distribution systems. Experience suggests that technical remedies are necessary but not sufficient to solve the problems of non-revenue water which are usually also linked to broader managerial, organizational, environmental and social issues that need to be appropriately dealt with as well. High economic returns will be generated by investments in the reduction of non-revenue water as it will simultaneously increase sales, cut supply costs and save waste. In actuality, sustainable development and management of water resources need to focus on reducing water loss and wastage rather than investing in supply capacity expansion and new distribution systems – the route AAWSA shall follow.

9.2 Methodology to Manage Leakage

9.2.1 Leakage Management Policy

Current leakage management practice of AAWSA is based on *maintain as leak occurs* approach. In 1999 there was good strategically guided beginning of leakage management although discontinued years back.. The authority focuses on harnessing new supply sources leaving about 30million m^3 of treated water escaping through its systems annually.

It is from the policy and the strategy that leakage management has to start in order to attain sustainability. Thus, AAWSA shall establish clear leakage control strategy besides harnessing new supply sources to fill the demand supply gap.

9.2.2 System Audit

Once the management is convinced of the need for policy change and make structural adjustments with the required resources, it has to evaluate its status as to the level of leakage. This is done through system audit discussed in detail under literature review.

9.2.3 Leakage Indexing

Once the authority has come up with the level of leakage in its distribution system, as part of its strategy it has to set target for further reduction. There will always be a level of leakage which is technically un avoidable depending on the economic level of the leakage reduction activities specific to that area. Economic Level of Leakage (ELL) is achieved when marginal cost of leakage reduction is above the corresponding cost of supply of the same volume of water. Accordingly, AAWSA shall strive for 29.4 % target as obtained from the study.

9.2.4 Capacity Building

For an effective management of water supply service in general and water loss and leakage in particular, water supply providing institutions must have an appropriate organizational management.

Shortage of qualified and experienced personnel is the major problem of AAWSA. Thus, intensive capacity building work shall be conducted for proper leakage management.

Besides, for water supply utilities to become financially sustainable, it is necessary that water charges be increased to financially viable levels; management information and accounting systems including billing and collection be improved; degree of real autonomy and accountability be established; and operation and maintenance be improved.

9.2.5 Establishing District Meter Areas (DMA)

The principle of zoning and DMA is a hierarchical way of evaluating and managing losses that covers a number of levels beginning with measurement at the supply and ends at the customers meter for an estimate of consumption. The network system of the entire city has to be sub-divided into different zones and sub-zones. Implementation of a zoning scheme whereby the complete water distribution network is broken down into manageable segments enables easy metering, monitoring and analyzing; it also creates better ground for further operations related to loss analysis and control.

In establishing zoning and district meter areas, the areas have to be investigated based on the ground elevation difference and distance from the respective service reservoirs. Literatures recommend metering at each hierarchy and permanent disconnection of zones and districts from one another. Besides the following are recommended:

a) Zones to include from 10,000 to 50,000 properties
b) District meter areas (DMA) to incorporate around 3,000 properties
c) Sub-district meter areas shall include about 1,000 properties

9.2.6 Updating Network Data

Proper record keeping enables availability of correct and up-to-date data. It is an important step for evaluating water loss. Supply zone and district meter areas (DMAs) records should also relate to both physical records and records for leakage analysis and modeling.

In AAWSA, most of the network records like the pipe sizes, material types and years of installation are collected and stored in AutoCAD format. This is an important step for managing the network as well as monitoring the water loss, but other records like maintenance and pressure records need also integrated with the network.

9.2.7 Night Flow Measurement

The city water supply system is characterized by un functioning meters and valves. It is essential to repair or replace all meters and valves so that night flow measurement at different levels would be possible. Night Flow Measurement is the first and simplest method that can assist in estimating locating and evaluating losses within a zone or district once the system is divided into DMAs. As most of the valves and meters are not functional in the systems nowadays, night flow measurement is not being conducted in the city.

9.2.8 Regular Inspection of the Water Network

There are various methods of leak detection and location in use globally. Ones the locations of highly suspected leaking networks are identified due attention should be given to inspect these areas and any leaks should be well recorded as it will be a good base for further maintenance or replacement.

9.2.9 Proper Maintenance

One of the major causes of water loss is the usage of poor quality materials and poor workmanship. Due care should be taken while maintaining existing networks and installation of new ones in this respect. While rehabilitation of any mains is planned, due attention should be given to maintain as well the service connections fed from the mains.

Additionally, as some of the main lines in the city are older than 30 years, they need be replaced.

9.3 Recommendations to Reduce Leakage

9.3.1 General

Based on the findings of the leakage indexing part of this study, the major and primarily attention seeking area in the existing AAWSA system, to reduce leakage, is implementation of operational leakage control measures than pressure related control. Thus, the primary focus of AAWSA with respect to its leakage reduction strategy shall be enhancing its operational and management efficiency than prioritizing pressure control measures. Besides, it can be learnt that leakage control in the selected sub systems is worse than at city level that the sub systems are among the priority sub systems along leakage reduction strategy.

9.3.2 Operational Control

a) Sub-system Level

The annual water leakage in the two sub systems is **5,470,927.32 m**^{3}. From the point of view of operational leakage control at sub –systems level, AAWSSA could possibly reduce real loss to **519,556.25 m**^{3}**/yr,** provided it is economically justified with respect to the marginal cost of supply of the same volume of water.

It means that overall **4,951,371.07 m**^{3} of the leaking water could have been saved if appropriate leakage control measures were implemented; which is equivalent to **90.50%** of the current leakage in the existing sub-systems; or **36.12%** of the total annual water supply to the sub-systems.

Thus, AAWSA shall strive for **36.12%** reduction target in its strategy in the sub systems in the future without changing the current pressure regime.

b) City Level

Similarly, assuming the same pressure regime prevails in the entire city, and annual net water leakage at city level being **29,717,692.04 m3**. From point of view of operational leakage control at city level, AAWSSA could possibly reduce real loss to **6,269,555.28 m^3/yr,** provided it is economically justified with respect to the marginal cost of supply of the same volume of water.

It means that overall **23,448,136.76 m^3** of the leaking water could have been saved annually if appropriate leakage control measures were implemented; which is equivalent to **78.90%** of the current leakage in the existing system; or **28.19%** of the total annual water supply to the city.

Thus, AAWSA shall strive for **28.19%** reduction target in its strategy in the long run, even, without changing the current pressure regime.

9.3.3 Pressure Related Control

From hydraulic modelling part, an average pressure of 52m would have been achieved in the sub-systems after appropriate measure is proposed. It was observed that average pressure was 68m in the two sub-systems before the proposed solutions.

Thus, about 20% reduction in leakage is expected considering linear relationship between leakage and pressure. Annual net water leakage in the two sub systems being **5,470,927.32 m^3**, the volume of water to be saved in the two sub systems as a result of pressure management alone is estimated to be **1,094,185,46 m^3** annually.

From the point of view of financial, technical; and social and environmental considerations, the recommendation is feasible. However, it is further recommended to revise water supply data before implementing this proposal using primary data logged for the purpose than using secondary data used in this study. Extensive calibration data including nocturnal records and geographical distribution is a pre requisite before implementation of the proposed solutions of this thesis as access to resources and personnel for flow calibration tests limited this study.

Time of day control approach is best suited for conditions like that of Addis Ababa where pressures are higher only during low demand period in night times. Its effect on customer pressure should however be considered before hand. This can be achieved through maintaining about 50m pressure in the mains.

It is recommended that pump 56 be changed to variable speed pump to be operated at 40m head from 7:00 PM up to 11:00 AM along the pressurized main from Ras Hailu reservoir to Abune Petros along junctions 9, 10, 11, 12 through junction 18 through adjustment on its pattern.

Since pressure on junctions 41 and 42 are above 80m all the day, it is further recommended that either an all time operating PRV be installed along the gravity main line from Saint Paul tank to lower areas of *Johannes church* area on junction 41; or flow balancing tank be constructed next to junction 38 . Due to the negative side effects of using PRV, flow balancing tank of 500m^3 capacity is proposed at junction 37. The simulation result shows pressure downstream the tank is between 25m and 55m for almost all the 24 hrs.

10 Appendices

10.1 Appendix A – Model Input Data

10.2 Appendix B – EPANET Algorithm

10.3 Appendix C –Model Outputs

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Declaration of originality

This thesis is my original work and has not been presented for a degree in any other university, and that all sources of material used for the thesis have been duly acknowledged.

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