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# Operational Tools for Decision Support in Leakage Control

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## Abstract

Water utilities particularly in the developing countries are still grappling with challenges of high water losses due to leakage. District Meter Areas, pressure management and network hydraulic modeling have proven to be powerful engineering tools for reducing leakage in many developed countries notably in the UK. Despite their apparent success, these tools have not had wide application in the developing countries partly due to inadequate information on cost-benefit analyses to support management decision making in implementation of pressure management policies. To address this constraint, this paper develops a decision support tool for predicting the associated benefits to make a sound financial case for investment in pressure management strategies. The predicted benefits by the tool are compared with those derived using network hydraulic modeling to give users confidence in the tool results. The predicted benefits are illustrated on a real-developing world case study in Kampala city, Uganda. Predictions by the tool and the network hydraulic model indicate that reducing average pressure in the DMA by 7.5 m could result into annual net benefits of Euro 56,190 and Euro 66,910 respectively without compromising the customer level of service. The results obtained indicate that the predicted net financial benefits compare fairly well.

**Key words:** Decision Support; Developing Countries; Leakage Control; Pressure Management; Water Distribution Network.

## INTRODUCTION

One of the main challenges facing water utilities worldwide is the high levels of water losses in the distribution networks. A recent World Bank study estimated that more than 32 billion cubic meters of treated water is lost annually as leakage from urban water supply systems around the world and half of these losses occur in developing countries (Kingdom *et al.*, 2006). The same report estimates the full cost of water losses from urban water utilities in developing countries to be as much as US$5 billion per year. In light of global pressures of growing demand and increasing water scarcity, water utilities particularly in the developing countries need to operate more efficiently for sustainable service delivery.

Non-revenue Water (NRW) is reaching alarming levels in many cities of the developing countries notably in Sub-Saharan Africa. A recent performance assessment study on African water utilities reports NRW figures of 40% in Lagos, 55% in Dar-es-salaam, 51% in Nairobi, 58% in Maputo, 52% in Lusaka and 51% in Blantyre among others (WSP, 2009).

Water leakage accounts for a significant amount of NRW in many cities of the world. It varies from 3% of the water put into the distribution systems in well managed systems to over 50% in poorly managed systems (Puust *et al.*, 2010). In Kampala, the capital city of Uganda, more than 8 million $m^3$ of treated water physically leak from the water supply system per year (Mutikanga *et al.*, 2009). Clearly, it is unacceptable, that where public utilities are starving for additional revenues to finance expansion of services and where most connected customers receive water irregularly, that water is also heavily wasted.

It is now widely acknowledged that Pressure Management (PM) in conjunction with District Meter Areas (DMAs) is as a powerful proactive leakage management tool (Farley and Trow, 2003; Puust *et al.*, 2010; Thornton *et al.*, 2008). Many water utilities have reported network pressure reduction, and, inter alia, leakage (Babel *et al.*, 2009; McKenzie *et al.*, 2004; Pilipovic and Taylor, 2003). Although these case studies report significant leakage reduction, they did not provide optimal solutions. Research studies have indicated that further leakage reduction could be obtained by applying optimization techniques such as genetic algorithms (GAs) (Savic and Walters, 1995), mathematical programming (Vairavamoorthy and Lumbers, 1998), and multi-objective optimization (Nicolini *et al.*, 2011). PM by optimal storage tank levels using a hybrid optimization model (GAs and artificial neural networks) for predicting leakage reduction has also been reported (Nazif *et al.*, 2010). PM does not only reduce leakage but extends useful life of infrastructure, reduces operation and maintenance costs through reduced frequency of main breaks and energy consumption, improves customer service as a result of reduced water supply interruptions and is a demand management tool (Girard and Stewart, 2007; Lambert and Fantozzi, 2010).

Despite numerous benefits of PM, it is hardly applied in Water Distribution Networks (WDNs) of the developing countries. One of the major barriers is inadequate economical information to support decision-making for adoption of pressure management strategies. It is therefore prudent to develop appropriate planning tools and methodologies for predicting potential savings of PM projects (Ulanicki *et al.*, 2000). This paper develops an appropriate management decision support tool (herein referred to as PM-COBT) for evaluating the associated cost-benefits of using Pressure Reducing Valves (PRVs) to reduce leakage and promote use of PM strategies in the water utilities of the developing countries. Water distribution modeling that explicitly account for pressure-dependent leakage requires precise data and skilled human resources that are often lacking in water utilities of the developing countries. This tool will be useful for network engineers and utility managers for quickly gauging the potential of pressure management in the water distribution systems, without the need for rigorous network hydraulic analysis.

The following section briefly describes the case study, followed by the methodology used for developing the tool. Subsequent sections present the tool, the network hydraulic model, application, results and conclusions.

## Case Study Background

The Uganda National Water and Sewerage Corporation (NWSC) is responsible for the delivery of water supply and sewerage services in 22 large towns in Uganda including Kampala city which is herein referred to as our case study.

The service area encompasses an area of about 350 km$^2$ with population estimated to be 1.5 million inhabitants. Water supply has not kept pace with population growth and has resulted in water shortages and low pressures in most parts of the distribution system. The current water supply averages 147,955 m$^3$/day. The condition of the network has deteriorated over the years, due to poor operating practices and inadequate strategic asset management (Mutikanga *et al.*, 2009). The average number of failures reported is 1,175 breaks/100 km/year. These are 50 to 75 times higher compared to figures reported in England and Wales (Thornton *et al.*, 2008). An average of less than 40 breaks/100 km/year is considered acceptable (Pelletier *et al.*, 2003). Non-revenue water averages about 43% of system input volume or 22 million m$^3$ per year (NWSC, 2009). The case study DMAs are presented in Table 1. The work presented in this study focuses only on DMA1 as DMA2 did not yield significant benefits to justify investing in PM strategies.

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**Table 1 |** Water Supply Profile of DMA1 and DMA2

| Description | Unit | Kitintale-DMA1 | Kawuku-DMA2 |
|---|---|---|---|
| Supply Regime | | Intermittent | 24-Hour |
| Service Connections | No. | 5,443 | 354 |
| Average Length of Private Connection | m | 25 | 25 |
| Total Pipe Length | km | 42.4 | 2.5 |
| Pipe Sizes (DN) | mm | 40-400 | 40-100 |
| Supply Zone Elevation | m | 1136-1222 | 1143-1174 |
| Average Water Demand | $m^3$/day | 6,167 | 288 |
| Average Billed Consumption | $m^3$/day | 2,752 | 225 |
| Non-revenue Water | $m^3$/day | 3,415 | 63 |
| Average Zonal Pressure | m | 61.5 | 48 |

## METHOD

The methodology used to develop the Pressure Management Cost-Benefit Tool (PM-COBT) is based on:

1. Bursts and Background Estimates (BABE - concepts)
2. Pressure-leakage relationships (FAVAD principles)
3. Pipe flow-Nodal Head (Q-H) equations

### BABE Concepts

In BABE analyses, components of leakage are considered in three categories (Lambert and Morrison, 1996):

- Background (undetectable) leakage – small flow rate, runs continuously
- Reported leaks and bursts – typically high flow rates but short duration
- Unreported leaks and bursts – moderate flow rates, duration depends on intervention policy

Although the water balance quantifies the total volume of leakage for the audit year, it does not provide a breakdown of leakage components (background, reported and unreported). The BABE methodology enables assessment of volumes of leakage components and allows identification of suitable reduction strategies.

### Fixed and Variable Area Discharges (FAVAD) Principles

Modelling leakage depends on understanding the hydraulics of leaks and how to incorporate the hydraulics into existing models of the water distribution system. The hydraulic equation for fully turbulent flow rate (L) through a hole of area (A) subject to static pressure (P) follows the square root principle according to Equation 1 (Thornton *et al.*, 2008).

$$L = C_d\ A \times (2gP)^{0.5} \qquad (1)$$

Where L is the flow rate ($m^3$/s), $C_d$ is the discharge coefficient: a dimensionless factor of less than 1; g is the gravitational constant in m/$s^2$ ; P is the pressure in metres head.

In practice some types of leaks, $C_d$ and A (and the effective area $C_d$ x A) can be pressure-dependent. This is the premise of the FAVAD paths concept (May, 1994). The effect of operating at different pressures is modelled by FAVAD principles. The basic FAVAD equation for analysing and predicting changes in leak flow rate ($L_0$ to $L_1$) as average pressure changes from $P_0$ to $P_1$ is (Lambert and Fantozzi, 2010):

$$L_1/L_o = (P_1/P_0)^{N1} \quad (2)$$

Numerous field and laboratory tests from various countries have shown that *N1* could vary from 0.5 to 2.3 depending on the type of leak, pipe material and failure type (Greyvenstein and van Zyl, 2007; Lambert and Fantozzi, 2010).

## Flow-Head Loss (Q-H) Equations

When water flows in a pipe network, it loses energy due to internal friction and turbulence. The loss of energy is commonly referred to as head loss. The head loss in a pipe is classified into: (i) frictional head loss and (ii) minor head loss due to minor appurtenances (Bhave and Gupta, 2006). The flow-head loss relationship can be expressed as follows:

$$H_L = K^*Q^2 \quad (3)$$

Where $H_L$ is the headloss (m), K is the head loss coefficient ($m^{-5}.h^2$) and Q is the flow rate ($m^3/h$).

## Estimation of Network Head Loss Coefficient (K)

The frictional factor (K) for the network can be estimated as follows:

1. Based on the 24-hour field measurements (pressure and flow) at the inlet and pressure at the critical point (CP) and Average Zonal Point (AZP) in combination with MNF analysis (Equation 4) and FAVAD principles (Equation 2), estimate hourly pressure-dependent leakage beginning with the hour of minimum night flow.
2. Estimate hourly nodal demands (assumed to be pressure-independent) as the difference between hourly total DMA inflow and nodal outflows (pressure-dependent leakage) computed in step 1.
3. Calculate the head loss ($H_L$) as the difference in pressure between the inlet point and the CP as well between the inlet and the AZP.
4. Calculate the K-factors for each hour of the day for CP and AZP using Equation 3. These hourly K-factors are assumed to be representative for the entire network for a specific hour of the day.

## Analysis of Different PRV Settings

With known K-factors, nodal demands and nodal outflows (pressure-dependent leakage), it is now possible to assess the impact of different pressure reducing scenarios based on different PRV settings. The objective is to reduce excessive pressure at the inlet point of the DMA while ensuring that the minimum required pressure at the various nodal points especially the CP is not violated. This problem could be straight forward if the relationship between pressure-dependent leakage and nodal heads and pipe flows were linear. Unfortunately, they are not and the problem is a nonlinear programming (NLP) problem which is rather difficult to solve.

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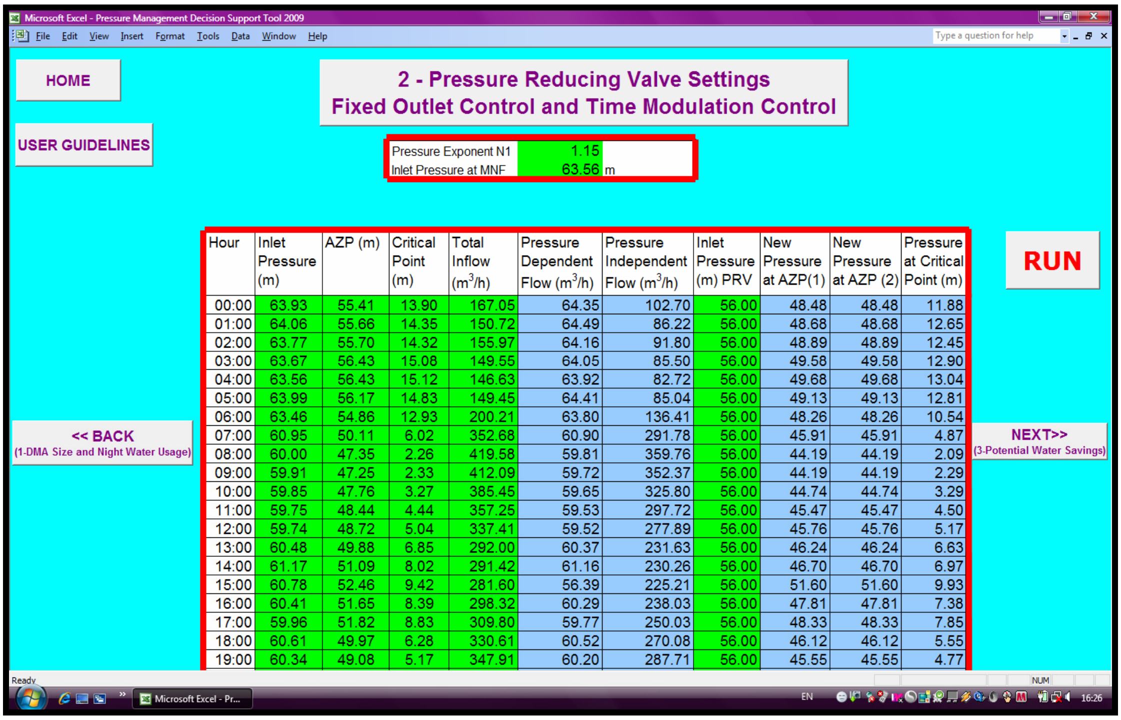
The different PRV setting options are analyzed by solving a NLP problem using sequential linear programming (SLP) techniques. SLP is an iterative procedure that involves linearization of the objective function and constraints until a termination criterion is met. The optimal valve control non-linear problem for leakage minimization in WDNs using SLP has long been solved by previous researchers (Hindi and Hamam, 1991; Jowitt and Xu, 1990) and is outside the scope of this paper.

The optimal PRV setting is selected to ensure the availability of flow at the CP during the maximum consumption period without violating the required minimum pressure of 10 m (DWD, 2000). In practice however, this minimum pressure requirement is not adhered to and some areas receive water at very low pressures of about 4 m. For the final PRV selected settings, the nodal outflows for each hour are computed (Equation 2) and together with the already established nodal demands, the new DMA inflows are estimated. The difference between the initial (before PM) and predicted DMA inflows (after PM) is taken to be the potential water savings. Further details on the methodology can be found in the user guideline module of the tool or in McKenzie (2001). The tool can be accessed from NWSC-Uganda or UNESCO-IHE Institute for Water Education, Delft on request.

## DECISION SUPPORT TOOL

A spreadsheet decision support tool (PM-COBT) using MS Excel® as a platform and coded using visual basic was developed to predict the potential benefits of pressure management in a given DMA. The screen shot of the computational worksheet of the tool where pressure can be lowered using PRV settings, is shown in Figure 1.

The tool has options of selecting the pressure management regimes using either the fixed outlet control or the time modulation control valve. However, for maximum leakage reduction, advanced flow modulated PRVs could be considered. Optimal time schedules can be converted to flow modulation curves by plotting scatter plots of flows against heads more efficiently and timely (Ulanicki *et al.*, 2008).




| Hour | Inlet Pressure (m) | AZP (m) | Critical Point (m) | Total Inflow (m³/h) | Pressure Dependent Flow (m³/h) | Pressure Independent Flow (m³/h) | Inlet Pressure (m) PRV | New Pressure at AZP(1) | New Pressure at AZP (2) | Pressure at Critical Point (m) |
|---|---|---|---|---|---|---|---|---|---|---|
| 00:00 | 63.93 | 55.41 | 13.90 | 167.05 | 64.35 | 102.70 | 56.00 | 48.48 | 48.48 | 11.88 |
| 01:00 | 64.06 | 55.66 | 14.35 | 150.72 | 64.49 | 86.22 | 56.00 | 48.68 | 48.68 | 12.65 |
| 02:00 | 63.77 | 55.70 | 14.32 | 155.97 | 64.16 | 91.80 | 56.00 | 48.89 | 48.89 | 12.45 |
| 03:00 | 63.67 | 56.43 | 15.08 | 149.55 | 64.05 | 85.50 | 56.00 | 49.58 | 49.58 | 12.90 |
| 04:00 | 63.56 | 56.43 | 15.12 | 146.63 | 63.92 | 82.72 | 56.00 | 49.68 | 49.68 | 13.04 |
| 05:00 | 63.99 | 56.17 | 14.83 | 149.45 | 64.41 | 85.04 | 56.00 | 49.13 | 49.13 | 12.81 |
| 06:00 | 63.46 | 54.86 | 12.93 | 200.21 | 63.80 | 136.41 | 56.00 | 48.26 | 48.26 | 10.54 |
| 07:00 | 60.95 | 50.11 | 6.02 | 352.68 | 60.90 | 291.78 | 56.00 | 45.91 | 45.91 | 4.87 |
| 08:00 | 60.00 | 47.35 | 2.26 | 419.58 | 59.81 | 359.76 | 56.00 | 44.19 | 44.19 | 2.09 |
| 09:00 | 59.91 | 47.25 | 2.33 | 412.09 | 59.72 | 352.37 | 56.00 | 44.19 | 44.19 | 2.29 |
| 10:00 | 59.85 | 47.76 | 3.27 | 385.45 | 59.65 | 325.80 | 56.00 | 44.74 | 44.74 | 3.29 |
| 11:00 | 59.75 | 48.44 | 4.44 | 357.25 | 59.53 | 297.72 | 56.00 | 45.47 | 45.47 | 4.50 |
| 12:00 | 59.74 | 48.72 | 5.04 | 337.41 | 59.52 | 277.89 | 56.00 | 45.76 | 45.76 | 5.17 |
| 13:00 | 60.48 | 49.88 | 6.85 | 292.00 | 60.37 | 231.63 | 56.00 | 46.24 | 46.24 | 6.63 |
| 14:00 | 61.17 | 51.09 | 8.02 | 291.42 | 61.16 | 230.26 | 56.00 | 46.70 | 46.70 | 6.97 |
| 15:00 | 60.78 | 52.46 | 9.42 | 281.60 | 56.39 | 225.21 | 56.00 | 51.60 | 51.60 | 9.93 |
| 16:00 | 60.41 | 51.65 | 8.39 | 298.32 | 60.29 | 238.03 | 56.00 | 47.81 | 47.81 | 7.38 |
| 17:00 | 59.96 | 51.82 | 8.83 | 309.80 | 59.77 | 250.03 | 56.00 | 48.33 | 48.33 | 7.85 |
| 18:00 | 60.61 | 49.97 | 6.28 | 330.61 | 60.52 | 270.08 | 56.00 | 46.12 | 46.12 | 5.55 |
| 19:00 | 60.34 | 49.08 | 5.17 | 347.91 | 60.20 | 287.71 | 56.00 | 45.55 | 45.55 | 4.77 |

**Figure 1 |** Screenshot of the Tool (PM-COBT).

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To be able to use the tool, basic infrastructure and system data (e.g. length, material and diameter of pipes, number of service connections, average zone pressure) and coefficients and default values (e.g. typical flow rates of leaks and bursts at some standard pressure, number of active population at hour of MNF, toilet flush volume, FAVAD *N1* values, costs of repairing leaks, cost of PRVs) are required. In addition to basic infrastructure data, the user must provide 24-hour pressure measurements at the inlet point, AZP and the CP. Also 24-hour flow measurements for the inlet are required. This information is nowadays easy to collect with aid of pressure and flow data loggers. More information on tool data requirements is provided in the user guideline module of the tool.

## DMA DESIGN, SET-UP AND OPERATION

Two DMAs were planned, designed and set-up for the pilot study to facilitate data collection required now and in the future to validate predictions made by the tool and the hydraulic model. The DMAs were selected to fairly represent the entire network characteristics. The DMA system data has been summarized in Table 1. For details on principles of DMA design and operation, the reader can refer to Farley and Trow (2003).

## WATER DISTRIBUTION NETWORK MODELING

The network hydraulic model was built using EPANET 2.0 (Rossman, 2000) and calibrated using internationally accepted procedures of water distribution model calibration (Savic *et al.*, 2010; Speight *et al.*, 2010). The case study network model (DMA1) is shown in Figure 2. The network model for DMA1 contained a reservoir, a tank, 112 pipes and 94 junctions. The model is applied for Extended Period Simulation (EPS).The demand allocation, model calibration and validation are briefly explained.

In order to determine the DMA leakage profile, night flow measurements were carried out and MNF assessed. To calculate the leakage at MNF time ($Q_L$), Equation 4 was applied to the DMA (Farley and Trow, 2003).

$$Q_L\ (t_{MNF}) = Q_{DMA}\ (t_{MNF}) - \text{Legitimate Night-Time Uses} \qquad (4)$$

For estimation of legitimate night uses, detailed field investigations are required. In the absence of such detailed studies, McKenzie (2001) proposed use of 6% of total population and average use of 10 litres/person/hour at time of MNF. However, these are default values for South African conditions and may not be valid for other countries. The use of active population percentage at time of MNF depends on socio-economic life styles of the population and probably on level of urbanization. The default value of 10 l/person/hour depends on toilet flush capacity. In some countries where water use efficiency is being promoted, toilets have been retrofitted to more efficient sizes of say 6l/flush. In this study, we used 10% as the active population coefficient and average use of 3 litres/person/hour was measured at time of MNF. Although there is a high percentage of active population in the DMA, water use is low as most households are in urban poor settlements where houses lack internal plumbing. Use of pit latrines instead of flush toilets is the norm for most households. Assessment of legitimate night-time use is crucial for accurate leakage reduction predictions. Over-estimation will lead to low leakage levels while under-estimation will lead to high leakage levels, thus exaggerating potential water savings.

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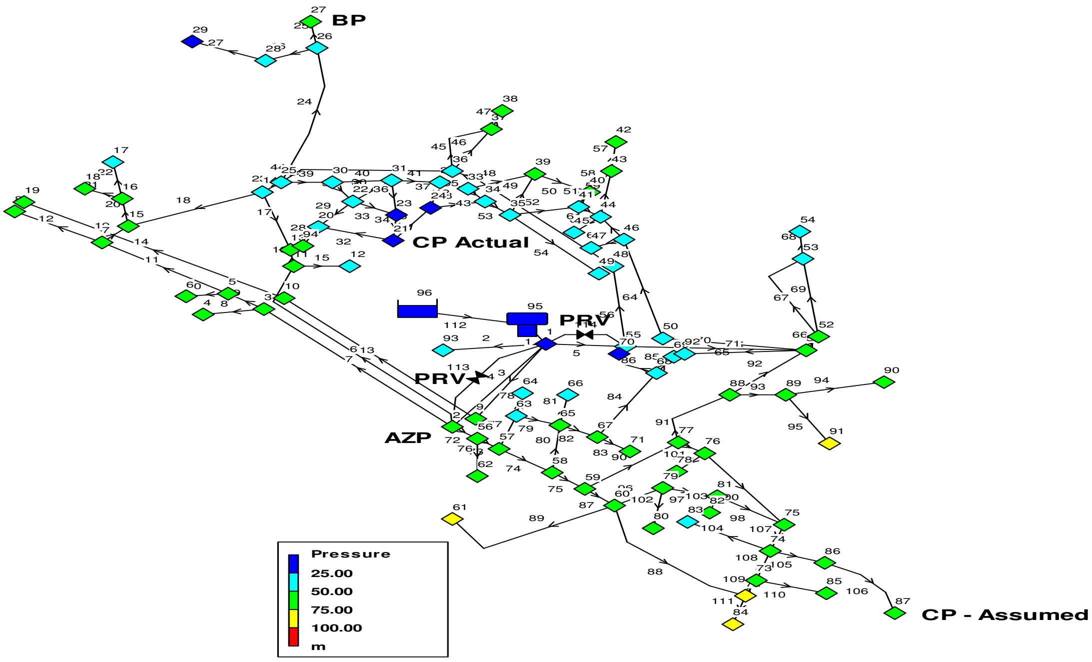


**Figure 2 |** Network Model Layout of DMA1.

The hourly leakage rate ($Q_L$,$t$) throughout the day is calculated by multiplying the Night-Day-Factor (NDF) with the leakage rate at MNF based on pressure-dependent leakage (Fanner *et al.*, 2007).

$$Q_L\,(t) = Q_L,\,(t_{MNF})\ \mathrm{x}\ [P(t)/P(t_{MNF})]^{N1} \qquad (4)$$

Where, $Q_L\,(t)$ is the leakage rate at the hour t (t ≠ $\mathrm{t_{MNF}}$), $t_{MNF}$ is the MNF hour, $Q_L$, ($t_{MNF}$) is the leakage rate at the MNF hour, $P(t)$ is the average hourly nodal pressure at the hour t ($t \neq t_{MNF}$), $P(t_{MNF})$ is the average hourly nodal pressure at the MNF hour, *N1* is the pressure exponent. The drawback of the MNF method is that it does not exactly reveal how this leakage is distributed in the network.

## Pressure-Dependent and Pressure-Independent Flows

In order to assess the impact of pressure reduction in the DMA, total flow into the DMA was split into two components: nodal outflows (pressure-dependent leakage) and nodal demands (pressure-independent). This split is rather subjective and based on simplified assumptions. In order to perform hydraulic simulations, leakage was incorporated in the models using the emitter devices of the EPANET 2 hydraulic network solver as outlined by various researchers (Tabesh *et al.*, 2009; Wu *et al.*, 2010). The emitter nodes allow leakage to be modeled using appropriate pressure-dependent outflow relationships as shown in Equation 5.

$$Q_i = K_i(P_i)^{N1} \qquad (5)$$

where $Q_i$ is the leakage flow at node $i$, $P_i$ is the pressure at node $i$ and $K_i$ is the emitter coefficient for the node $i$, estimated as a function of pipe and soil characteristics. *N1* is the pressure exponent. By trial and error, the emitter coefficient ($K_i$=0.00208) was fixed for all nodes since the pipes from which leakage occurred were not known.

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## Model Calibration and Validation

In order to accurately measure night flows and apply MNF concepts for leakage assessment, special arrangement was made to ensure 24-hour supply to DMA1 during the study period. The model was manually calibrated, tested and validated by comparing measured pressure and flow values with model simulated results with the main objective of minimizing the discrepancies between the two. During the calibration process, pipe roughness coefficients and base demands were adjusted as calibration parameters.

The calibration methodology resulted in a maximum error of 3.8 m for pressure and 11 l/s for flows. The criteria used to accept the errors were: (i) less than 5 m for pressure and (ii) less than 5% of the MNF measured at the inlet point (29 l/s). The mismatch between the integral areas under the observed and simulated inflows particularly at the hour of MNF in Figure 3 could be due to variations in the actual night use by users, errors in the measurement equipment, erroneous estimation of pipe roughness coefficients and localized calibration to a small part of the network. Although there was a mismatch between observed and simulated flows, the correlation coefficients were very close to one indicating acceptable performance and good model representation of the system behavior. However, the calibration process could be improved by optimization techniques with the objective function of minimizing the differences between measured and computed flows (Savic *et al.*, 2010). The model is calibrated and verified for EPS and Figure 3 shows the observed and simulated inflow and leakage.

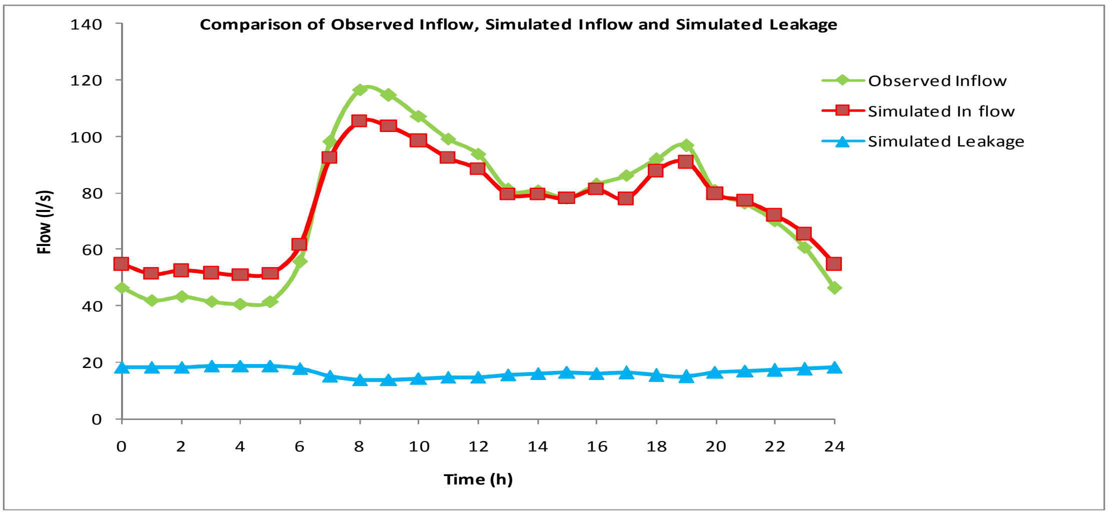


**Figure 3 |** Comparison of computed and observed flow into the system.

## APPLICATION TO CASE STUDY

The tool and model have been applied to DMA1 of the KWDN using field data and information from the NWSC procurement department to compare the estimated costs of the PM project against cost-savings of the projected benefits. Where data was not available, gaps were filled using data from literature.

The net benefits derived from a pressure reduction scheme proposed for implementation in DMA1, were estimated from the difference between related costs before and after introduction of the scheme, using the following cost model (Awad *et al.*, 2008):

$$NPM = CLR + CBR + CCR + CDER - CPRV - CMM - CED \tag{3}$$

Where, $NPM$ = net benefit from pressure reduction (€/year), $CLR$ = benefit from leakage reduction (€/year), $CBR$ = benefit from reduced pipe burst frequency (€/year), $CCR$ = benefit from customer complaints reduction (€/year), *CDER* = benefit from direct energy reduction (€/year), *CPRV* = annual cost of installation, construction and commissioning pressure reducing valves (€/year), *CMM* = annual cost of maintenance and monitoring (€/year), and *CED* = initial cost of engineering and design (€/year).

## RESULTS AND DISCUSSION

The Decision Support Tool (DST) and Network Hydraulic Modeling (NHM) have been applied to DMA1 in Kampala. The results presented herein are based on predictions of pressure reduction using Fixed Outlet PRVs which are considered more appropriate for water utilities in the developing countries that are just starting to work with pressure management systems. They are relatively cheap in terms of investment cost and easy to operate and maintain.

The analysis results presented in Table 3 indicate that lowering pressure by 7.5 m results into annual net financial benefits of over 56,000 Euros. The DST predictions are more conservative as it predicts about Euro10,000 (or 16%) less benefits compared to the NHM. Although the model may not be very precise in predicting financial benefits it is still useful as the predicted savings are generally within 10 to 20% of those actually achieved in practice (McKenzie, 2001). For the same pressure reduction, a saving of water of at least 4% can be achieved in the DMA.

**Table 3 |**

| | Average Pressure (m) | Total Inflow ($m^3$/d) | Savings ($m^3$/d) | Savings (%) | Net Benefits (€/yr) |
|---|---|---|---|---|---|
| No PM | 63.5 | 6,167 | | | |
| DST | 56 | 5,913 | 254 | 4 | 56,190 |
| NHM | 56 | 4,885 | 1,282 | 21 | 66,910 |

The analyses of different pressure profiles at the critical point are shown in Figure 4. It is evident from Figure 4 that the critical point pressure predicted by the DST is higher than that predicted by the network hydraulic analysis confirming the conservativeness of the tool in its predictions. In the absence of measured data, the NHM results could be used as input data for the tool, thus complimenting each other.

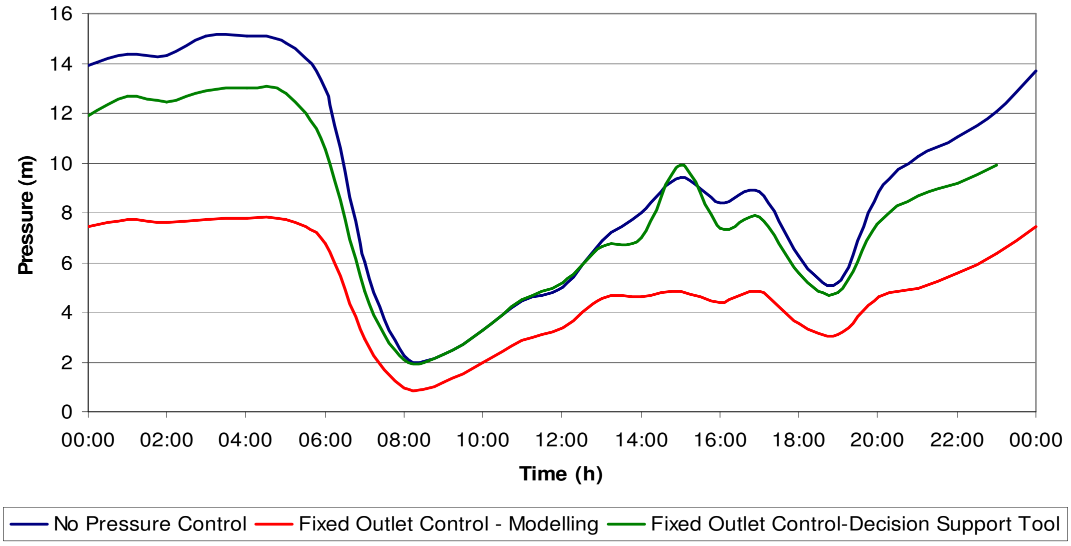


**Figure 4 |** Comparison of CP pressure predicted from the NHM and DST.

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It is important to note that the cost-benefit results obtained should not be generalized because of the uncertainty and subjectivity of data used and assumptions made. For instance, variation in repair and maintenance costs will have a significant influence on cost savings of the pressure management scheme. Lastly, socio costs like traffic disruptions and others were not quantified.

## LIMITATIONS OF THE TOOL

The decision support tool has the following limitations:

- It may under estimate the potential net water savings and divert attention and expectations away from pressure management strategies. This shortcoming is an inherent property in the tool's methodology.
- The head loss–flow relationship used by the tool to determine network friction factors, considers all influences on head loss to be lumped in one parameter. This may lead to under estimation of head loss and subsequent prediction of high values of critical point pressure.
- The tool uses coefficients and default values developed from a series of field testing in the developed countries (e.g. FAVAD *N1* values, BABE typical flow rates of leaks and bursts at some standard pressure) – all of which may cumulatively lead to erroneous estimates of net benefits. It is important to make necessary modifications to suit local conditions.
- The tool does not provide information such as flow and pressure in the rest of the DMA. Calibration is localized at only three points (inlet, AZP, and CP) and this could lead to erroneous predictions.

## CONCLUSIONS

Leakage reduction in urban water distribution systems is a major challenge for water utilities especially in the developing countries. Pressure management is one of the proactive options for leakage reduction. There are various methods and tools for leakage reduction based on pressure management. However, most of the tools and methods require a lot of resources and are hardly applied in the developing countries. This paper presents an appropriate decision support tool for evaluating potential benefits of implementing pressure management strategies in developing countries. The planning tool will be very valuable in providing insight into the financial benefits of implementing PM projects and justify the investment decisions. For operational synergies however, it is advisable to compliment the DST with NHM as reliable data and resources become available with time for more accurate predictions of financial benefits and water savings. It is the authors hope that the developed DST will act as a stimulus to promote use of PM strategies as part of the broader leakage management policies in the water utilities of the developing countries to recuperate water losses.

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