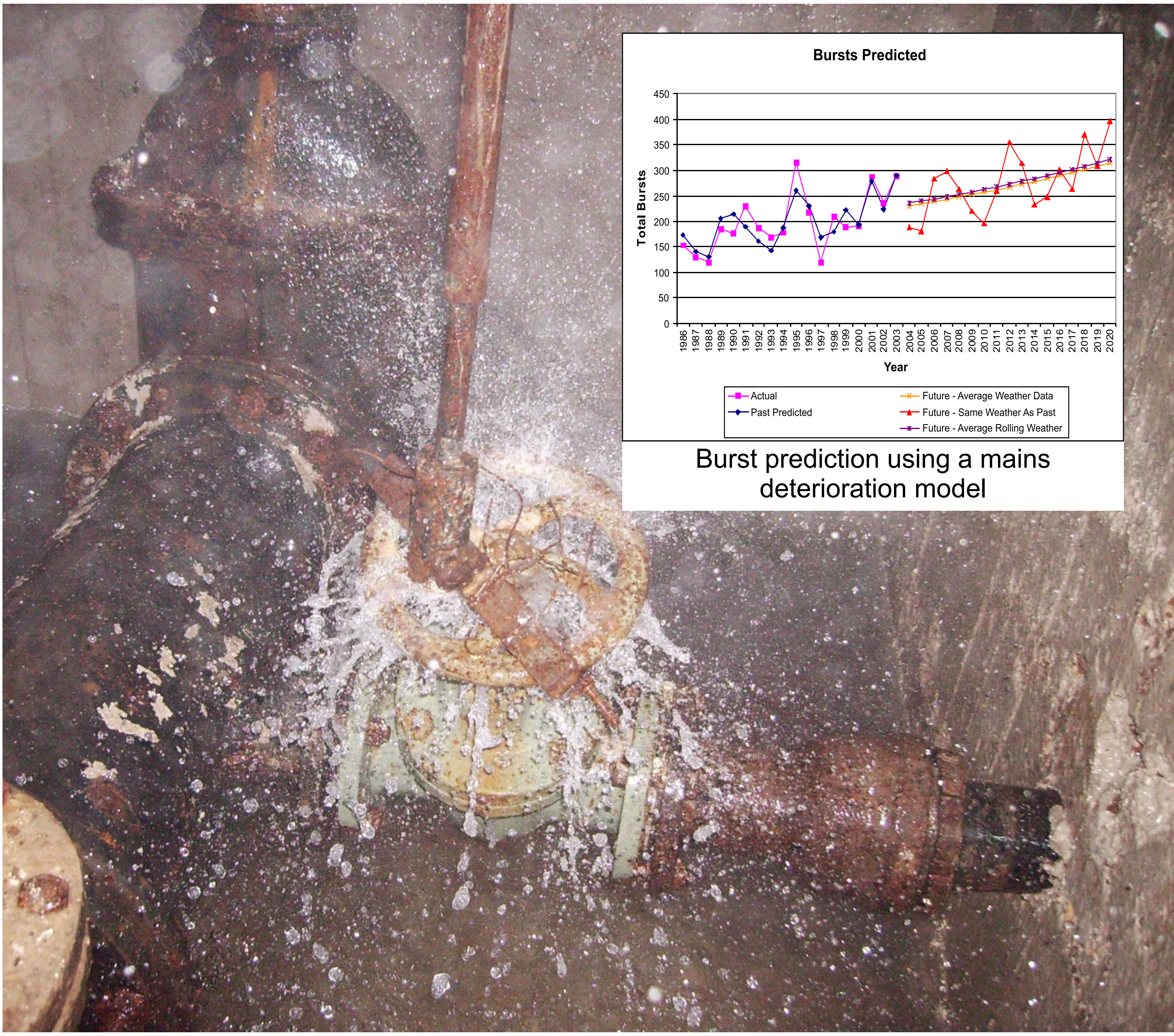
Water Loss 2007

23 - 26 September

Conference Proceedings
Volume III

Bucharest - Romania



Burst prediction using a mains deterioration model

IWA
International Water Association

Specialist Group
Efficient Operation and Management
Water Loss Task Force

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ASOCIAȚIA ROMÂNĂ A APEI
ROMANIAN WATER ASSOCIATION

IWA International Specialised Conference
23 – 26 September 2007
Bucharest, Romania

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ASOCIAȚIA ROMÂNĂ A APEI
ROMANIAN WATER ASSOCIATION

Foreword

One of the major challenges facing many water utilities around the world is a high level of water losses either through real losses (leakage) or apparent losses (meter under-registration, theft of water). This difference between the amount of water put into the distribution system and the amount of water billed to consumers is known as “Non-Revenue Water” (NRW). According to a recent World Bank discussion paper[1] the total cost to water utilities caused by NRW worldwide can be conservatively estimated at $15 billion/year.

Not understanding the magnitude, sources, and cost of NRW is one of the main reasons for insufficient NRW reduction efforts around the world. Only by quantifying NRW and its components, calculating water loss performance indicators, and turning volumes of lost water into monetary values can the NRW situation be properly understood and the required action taken.

For the last ten years the Water Loss Task Force (WLTF) of the IWA's Specialist Group on "Efficient Operation and Management of Urban Water Systems" is developing and advocating new concepts and methodologies that can help water utilities to reduce water losses more efficiently.

A part of the WLTF's efforts is the organisation of specialised conferences and the biggest so far was "Leakage 2005", an event that took place in Halifax, Canada in September 2005. More than 50 high quality papers were presented during this three day event.

Source: Water and Sanitation Program of the World Bank

Two years have passed since and the global water industry is showing even more interest in the work of the WLTF – and especially in the WLTF's 2007 conference: “Water Loss 2007” in Bucharest, Romania where some 90 papers from around the world will be presented, the majority of them included in these proceedings.

I like to take the opportunity to thank the members of the Scientific Committee (Francisco Cubillo, Prof. Anton Anton, Bambos Charalambous, Tim Waldron, Mary Ann Dickinson, Malcolm Farley, Marco Fantozzi and Dewi Rogers) for reviewing close to 120 abstracts and helping me to put the program for “Water Loss 2007” together.

However, it would not have been possible to organise “Water Loss 2007” and publish these proceedings without the enormous efforts of ARA, the Romanian Water Association. I would like to thank the Management and the Staff of ARA for all the hard work, in particular Cristina Popescu, Eugenia Demetrescu, Silviu Lacatusu, Daniel Zaharia and Vasile Ciomos.

In June 2007 I had the opportunity to visit SABESP, the water utility of São Paulo, Brazil. One of their leak detection specialists showed me the Leakage 2005 proceedings – downloaded from the Internet and nicely printed and bound. He referred to it as the "best water loss management publication". I sincerely hope that the “Water Loss 2007” proceedings will be considered an equally useful reference document for water loss management professionals around the world.

Roland Liemberger
Chair, Scientific Committee

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[1] The Challenge of Reducing Non-Revenue Water (NRW) in Developing Countries - How the Private Sector Can Help: A Look at Performance-Based Service Contracting, WSS Sector Board Discussion Paper #8, World Bank, 2006, by William D. Kingdom, Roland Liemberger, and Philippe Marin

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VOLUME 2

VOLUME 3

Session C9

POSTER

Appropriate materials to use for potable water service connections

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Keywords: Potable water service connections; Leakage control; Polyethylene pipes; Standards and specifications; Sleeving and reinstatement trial.

Introduction

The problem of leaks from water service connections has always been one of the major causes of concern for the Water Services Corporation in its quest to manage leakage. An aggressive leakage control program was consequently implemented. In the mid-nineties, research was conducted and the Corporation opted to stop using galvanised pipes and fittings and turn to polyethylene materials. It was soon obvious that although the new material did not corrode like its predecessor, leakage became more prominent in the service pipe.

History

Polyethylene pipes and fittings are bought by tender in the following sizes: 20mm, 25mm, 32mm, 50mm and 63mm. The Water Services Corporation has been purchasing polyethylene pipes and fittings since 1997. From 1997 to date, 30 contracts have been issued to purchase polyethylene pipes of the above mentioned sizes.

In these ten (10) years, the Water Services Corporation has bought polyethylene pipes from seven (7) different suppliers. The value of these contracts amounted to Lm 342,717 (€ 800,000) and a total of 1918 km of polyethylene pipe was bought for the above mentioned sizes.

During the past three (3) years, checking of pipe dimensions and pressure testing started to determine which supplier was awarded the contract.

Assessment of the present situation with regards to failures

An assessment was carried out to determine the present situation with regards to failures. Failure data was compiled for both Malta and Gozo for the months of September, October and November 2006 as shown in table 1. Pictorial comparison is shown in figure 1. Details of all damages found for both 'active' and 'passive' leakages were categorized as follows:

- Plastic pipe
- Plastic fitting
- Metal (including mains, galvanised pipes and fittings, tappings, banjos, stopcocks, stfs)

Table 1: Failure data

Defects during Sept-Oct-Nov 2006	Central	Gozo	North	South	Totals
Plastic pipes	679	123	510	1211	2523
Plastic fittings	594	80	136	493	1303
Metal	304	51	309	227	891
Totals	1577	254	955	1931	4717

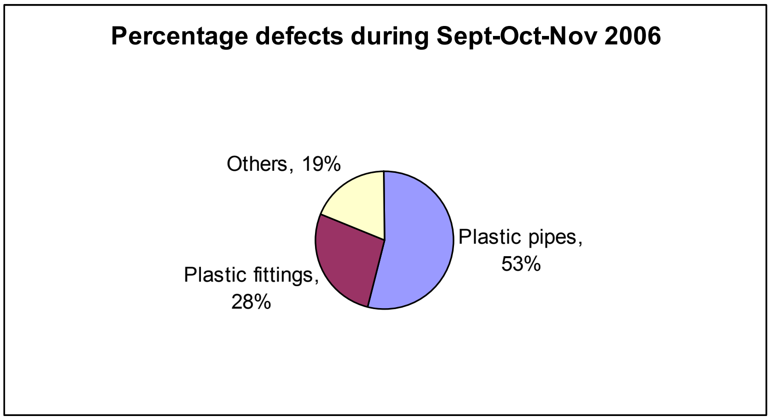

Figure 1: Failure data

This data clearly showed that pipes and fittings used for potable water, service connections are the main contributors to causing leaks. The actual pipes and fittings found were also collected to form a representative sample of the defects found. The defects found were as follows:

- Plastic pipe defects were due to exterior damage in the way they were reinstated as part of the road build up, pipe splitting along its pipe length and pinhole defects. This indicates that the pipe quality ordered was not always up to the required quality standards.
- Plastic fitting defects were due to both material quality and exterior damage in the way they were reinstated as part of the road build up.
- Metal defects were due to corrosion of material, both externally and internally, due to the time they were in the ground. This corrosion was also due to the quality of the water and the surrounding substrate material.

Since figure 1 indicated that pipes are the major concern for service defects resulting in leaks (53%), the water services corporation concentrated on pipe failures.

Comparison of Polyethylene pipes ordered

The total length of km of polyethylene pipe ordered per size is shown in figure 2. Since most potable water service connections are installed using the 20mm pipe, it is the most used size. Figure 2 also gives a pictorial comparison of the different sizes of polyethylene pipe ordered.

Size	Pipe ordered km
20mm	1269.7
25mm	248.2
32mm	174
50mm	65.4
63mm	160.5
Total	1917.8

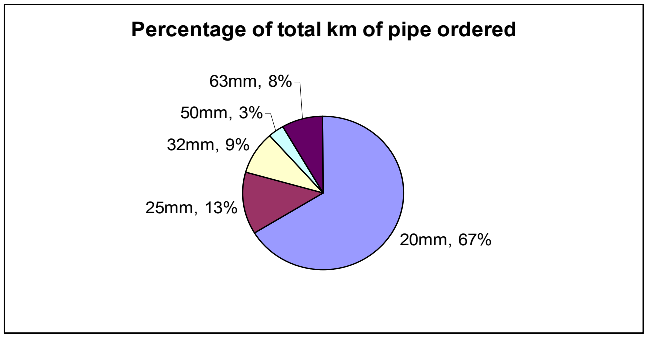

Figure 2: Polyethylene pipe ordered per size

The correlation of pipe to pipe supplier to quantity supplied and time

A period was set to collect the defective pipes which resulted in water leaks. Defective pipes were collected from both Malta and Gozo. Between 15th January and 15th February 2007, these defective pipes were collected and analysed. A total of 189 defective pipes were collected and these originated from six (6) different suppliers. To date, seven (7) different suppliers were used by the Water Services Corporation in the purchasing of its pipes. In this sample, 8.5% of defective pipes had illegible or incomplete print and could not be assigned to any supplier.

Table 2 shows the defects and defects per km ordered from this pipe sample. Defective pipes were sorted according to the sizes found. The pipes without information printed on the pipes were not considered. Pictorial comparison is shown in figure 3. Figure 3 shows that the most pipe defects were from the 20mm size. However, when considering the km ordered, the 25mm and 32mm sizes contribute to more defects than the 20mm size.

Table 2: Defects from pipe sample

Size	Pipe ordered Km	Defects from pipe sample	Defects per km ordered
20mm	1269.7	108	0.09
25mm	248.2	37	0.15
32mm	174	22	0.13
Totals	1691.9	167	

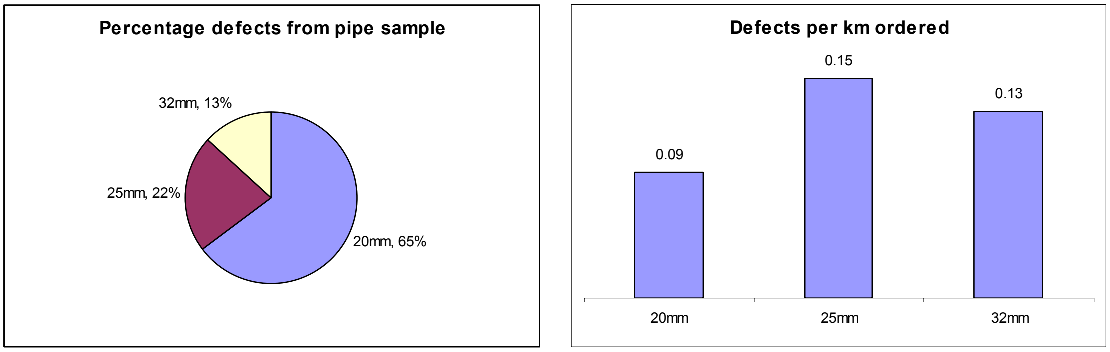

Figure 3: Percentage defects from pipe sample (left) and defects per km ordered (right)

Defective pipes were analysed by size according to:

- km of pipe ordered
- sample defects
- defects per km ordered per supplier and
- defects per km ordered per supplier, per year from contract date

These results are tabulated in table 3 as follows:

Table 3: Defect analysis by supplier

Size	Km of pipe ordered	Sample defects	Defects per km ordered	Defects per km ordered per year from contract date
20mm	Supplier 1	Supplier 2	Supplier 2	Supplier 2
25mm	Supplier 2	Supplier 2	Supplier 2	Supplier 2
32mm	Supplier 1	Supplier 2	Supplier 2	Supplier 2

Table 3 above showed that supplier 2 had the most defects. This clearly indicates that the quality of this pipe ordered was not always up to the required quality standards. The pipe leaked not only due to exterior damage in the way they were reinstated as part of the road build up. It also leaked due to the quality of the pipe since reinstatement was constant for all suppliers.

The tenders used to buy these pipes have been analysed and the literature supplied by these suppliers studied. In this literature, each supplier has given us the way their pipe should be reinstated as part of the road build up.

The polyethylene service pipe was not installed and reinstated according to the supplier's recommendations. This means that an explanation from our suppliers cannot be demanded for the pipe failures being encountered due to pipe quality. Had the supplier recommendations been adhered to, the Water Services Corporation would today be in a position to demand explanations and take legal action as required.

Standards and Specifications

The European standard for plastic piping systems for water supply – Polyethylene is EN 12201:2003 and is divided as follows:

- EN 12201 – Part 1: General
- EN 12201 – Part 2: Pipes
- EN 12201 – Part 3: Fittings
- EN 12201 – Part 4: Valves
- EN 12201 – Part 5: Fitness for purpose of the system
- EN 12201 – Part 7: Guidance for the assessment of conformity

EN 12201 – Part 6: Recommended practice for installation was not published and existing national practices would be applicable. Locally, this is legal notice LN 364 of 2003 – Road Works (Design and Construction Standards) Regulations 2003.

This standard is also a local, national standard. The Malta Standards Authority (MSA) has approved and endorsed this standard under the auspices of the European Committee for standardization.

These standards are detailed and comprehensive. To order polyethylene pipes, it is not possible to confirm these standards by quoting them as part of the tender procedure. Firstly, it is impossible to compare suppliers by asking for parts of the standard. Secondly, leaving out any part of the standard would be asking for an inferior product. Therefore, all suppliers approached for the purchase of pe pipes and fittings must:

- Conform to EN 12201
- Manufacturing facilities must conform to ISO 9001 for quality systems
- Produce third party certification that products actually conform to EN 12201.

- Be nominated by independent organizations, groups, associations, regulating bodies and/or federations.

Present technical specifications to purchase polyethylene pipes

The Water Services Corporation is presently ordering polyethylene pipes by tender procedure.

What is the Water Services Corporation ordering today?

- A black, HDPE pipe for potable water installations
- Nominal working pressure of 20bar (PN20)
- Nominal diameter of 20, 25, 32, 50 and 63mm
- Material class PE100
- We are specifying standards EN12201

What is the Water Services Corporation not asking for today?

- A widely-produced PE100 pipe instead of a PN20 one
- The suppliers recommendation method of installation for this pipe to last over 50 years without leaking
- Third party certification ensuring that manufacture is complying with EN12201
- Affiliation of the manufacturer with independent organizations, groups, associations, regulating bodies and/or federations.

This study has led to these being included in the new specifications.

Theory

Polyethylene has a long, proven track record for water distribution. The introduction of PE100 has broadened the range of pipe applications even further. Material designation PE100 gives the long term strength of the material. This is the minimum required strength of 10MPa (hoop stress) and is obtained from regression analysis on test data from long term pressure testing.

The standard dimensional ratio (SDR) refers to the geometry of the pipe. It is defined as the ratio of the nominal outside diameter to the nominal wall thickness. Therefore, a higher SDR indicates a thinner walled pipe at any given diameter. SDR11 and SDR9 are commonly used for potable water service connections.

High quality plastic materials are ensured by continuously monitoring three fundamental properties:

- Creep rupture strength (ISO1167 & EN921) – Internal pressure test
- Stress crack resistance (ISO13479) – Pipe notch test
- Resistance to rapid crack propagation (ISO13477) – S4 test

Table 4: Testing of pipe samples

Property	Test Method	CEN/ISO Standard Requirements
Creep rupture strength	Pressure test at 20°C and 12.4MPa	>100h
Stress crack resistance	Pipe notch test at 80°C and 9.2bar	>165h
Resistance to rapid crack propagation	S4 test at 0°C	$PC \geq \frac{MOP}{2.4} - \frac{13}{18}$

To ensure that the highest quality is adhered to, independent, internationally respected laboratories carry out the tests described in table 4 above. The tests are repeated periodically to ensure that manufacturers meet these stringent requirements.

The lifetime of a service connection depends on five factors:

- The pipe operating conditions of temperature and pressure
- The pipe material used
- External pipe loading due to traffic loading, type of road build up and reinstatement
- The environment surrounding the pipe (chemical loading in contaminated soil)
- Installation conditions and methods

A reputable supplier will supply the required pipes and fittings depending on the application required by the Water Services Corporation and guide the Water Services Corporation on how to install it. This, together with local standards and on site monitoring should start the process of having service connections lasting for a long time.

Sleeving trial

Two other projects related to this one were the sleeving and reinstatement trials. The sleeving trial consisted of passing the service pipe through a pvc, 50mm sleeve buried in lean mix. This was tried as part of different pipe laying projects being carried out.

The advantage of this sleeve was that in the event of any failure, the part of the pipe that was sleeved could be threaded through again. However, the area around the tapping ferrule and stopcock was not sleeved. The tapping and stopcock area would still need to be dug up. The existing fittings also necessitated the installation of a 90° bend, just after the banjo, in the tapping area.

Figure 4: Tapping area with bend and start of sleeve (left) and sleeve from tapping area towards pavement

The long term effect of sleeving on detection sounding practices is not yet known but needs further investigations. Also, the sleeve may be installed just as a precautionary measure and must not serve as a cushion to camouflage bad workmanship.

Reinstatement trial

The reinstatement trial was carried out to experience the concept of a one stop visit policy to carry out a job, with the necessary equipment and set up required. Included in the scope of this trial was a study of the time required to carry out a standard job together with the effect on quality with time. A standard job is a 7m new service installation or equivalent.

Figure 5: Truck loaded with equipment and materials for reinstatement (left) and a first attempt, new service at Marsascala (right)

Three vans comprising a tail lift were rented out since this type of vehicle was not available in house. All the necessary equipment was loaded. This included H&S equipment and materials required to carry out the job. Cutting was done with the tarmac saw and reinstatement was done with the vibro tamper. Cold mix was used for resurfacing.

This trial exhibited the difference when comparing the normal work practice to this trial. It showed that investment in equipment will pay back as time spent on the job is directly proportional to quality. This trial showed that quality is strongly a function of time, and a job well done is cheaper in the long run.

Adhering to standards and sustaining to adding personnel is a key issue in keeping up with the number of daily jobs that need to be done. Any subcontracting will require strict and expensive control.

The way forward in potable water, service connections

Polyethylene was introduced in the late 1950's and has undergone tremendous development to reach the world-wide position it enjoys today. Compared to traditional materials, PE pipe installations are the most competitive by combining the following key advantages:

- Ease of handling due to flexibility and lightweight
- Leak-tight installation due to excellent fusion welding possibilities
- Long life with low operational costs

- Capability for relining existing pipelines
- Possibility for on site extrusion, alternative installations
- Chemical resistance

The outstanding quality of PE pipe is documented by international standardization bodies. PE has a long, proven track record for water distribution purposes. The introduction of PE100 material has broadened the range of applications even further. This report recommends that the Water Services Corporation is to retain the polyethylene plastic material for pipes as long as quality pipes are purchased and installed as recommended by the supplier.

Alternative solutions

Three alternative solutions were identified, relating to the way the pipe is installed as part of the road build up. These are:

Solution 1: It is recommended that the mains distribution pipes are moved towards the pavement, away from the tarmac resurfaced roads. The verge underneath the pavement curb may be used. Even if each installation is as perfect as possible, the roads are flexible and the network underneath the tarmac surface is bound to be damaged one day.

This implies the installation of dual mains and this means double the investment required. However, using good quality materials and reinstatement practices, added costs are offset with leak and maintenance free systems. It is therefore cheaper in the long run.

Solution 2: It is recommended that the water mains are installed in the verge on one side of the road and the service connections crossed across the road. Pipes across the road may be sleeved as an added safeguard.

Solution 3: Instead of sleeves, composite pipes may be used. This is a skin coated polyethylene pipe that permits below ground installation without the need for specially selected backfill surround. Further studies need to be carried out with this composite pipe to analyse its suitability in our local scenario. A sample of this type of pipe has just been obtained for installation.

Although the trials carried out and alternative solutions lead to improvements, the Water Services Corporation must not reinvent the wheel as regards to potable water service connections. Abroad, this work is all carried out by licensed contractors and plumbers which are subcontracted by the respective water utilities. The utility supervises the work and accepts it depending on the quality standards required. The need to visit this work being carried out abroad and the need to contact our counterparts in water utilities in Europe and benchmark with them is imperative.

In this manner, the Water Services Corporation will be in a position to start a new era with regards to potable water service connections. The effect of the last ten years of installing polyethylene pipes and fittings has been demanding excessive leakage control input. The success attained in leakage control is at a price. The implementation of the above recommendations should result in a complete turn around to using quality products in our roads, installed in a quality fashion. This will be cheaper in the long run, will improve the image of the Water Services Corporation and earn respect from customers.

Conclusions and recommendations

- Retain the polyethylene plastic material for pipes.
- Purchase only high quality pipes (and fittings) according to international standards.
- Create working relationships with suppliers of pipes and fittings.
- Use quality suppliers as indicated by recognized associations.
- Upgrade tender specifications as suggested above.
- To standardize pipes to 20mm, 32mm and 63mm.
- Correct normal, service connection installation. Install service connections as per supplier recommendations and local road works, reinstatement standards (LN 364 of 2003).
- Decide on reinstatement method and procure corresponding pipe, or vice versa.
- Improve upon supplier recommendations by installing main in the verge on one side and sleeve the required services across the road.
- Create a pipe traceability installation system.
- Visit and benchmark with water utilities abroad.
- Control the installation procedures being carried out by third parties when installing recommended pipes and fittings.
- Carry out further studies regarding pipes and fittings: push fit fittings, compression fittings, fusion and butt welding, composite pipes.

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WaterPipe project: an innovative high resolution Ground Penetration Imaging Radar (GPIR) for detecting water pipes and for detecting leaks and a Decision-Support-System (DSS) for the rehabilitation management of the water pipelines

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Keywords: GPIR, DSS, inspection, leak, rehabilitation, water

Project full title: Integrated High Resolution Imaging Ground Penetrating Radar and Decision Support System for WATER PIPEline Rehabilitation

***Acronym:** WATERPIPE*

***Proposal/Contract No**.: 036887*

***Project type**: Specific Targeted Research Project (STREP) – F P 6*

***Project website:** http://www.waterpipe-eu.org/*

***Project start date:** 1/11/2006*

***Project Budget:** 3.3 million EURO - out of which 2.2 million EURO – EC contribution*

***Duration:** 36 months*

Contractors:

The coordinator:		
Microwave and Fiber Optics Laboratory – Institute of Communications and Computer Systems	ICCS	Greece
Other contractors:		
AMGA Group	AMGA	Italy
Compania Aquaserv – Tirgu Mures	AQUASERV	Romania
PipeHawk	PIPEHAWK	UK
Huberg	HUBERG	Italy
Hydrosave UK	HYDROSAVE	UK
TECNIC	TECNIC	Italy
RISA Sicherheitsanalysen	RISA	Germany
Advanced Microwave Systems	AMS	Greece
Laboratory of Applied Electrodynamics Physics Department Tbilisi State University	LAE - TSU	Georgia
Istanbul Technical University Civil Engineering Faculty Environmental Engineering Department	ITU - EED	Turkey

1. Project Description:

Many EU cities are experiencing increasing problems with their water pipeline infrastructure. The cost of replacing these old, worn-out systems, if left to deteriorate beyond repair, is astronomical and clearly beyond the resources of many communities. Replacement, however, is not the only choice as many of these systems can be rehabilitated at 30 to 70 percent of the cost of replacement. Accordingly, resources are now increasingly being allocated to address pipeline rehabilitation management issues. Due to the emphasis on sustainable management, risk-based approaches for the rehabilitation management of the water supply network need to be developed. Rehabilitation decisions should be based, inter alia, on inspection and evaluation of the pipeline conditions. Yet, utilities cannot locate a number of their old pipes and current inspection technologies typically do not provide the needed detailed information on pipeline damage.

1.1. Objectives

The objectives of this work are:

- To develop a novel, high resolution imaging ground penetrating radar for the detection of pipes, leaks and damages and the imaging of the damaged region and evaluate it at a test site.
- To produce an integrated system that will contain the equipment in ‘1’ and a Decision-Support-System (DSS) for the rehabilitation management of the underground water pipelines that will use input from the inspections to assess, probabilistically, the time-dependent leakage and structural reliability of the pipelines and a risk-based methodology for rehabilitation decisions that considers the overall risk, including financial, social and environmental criteria.
- To field test the equipment and the DSS.

1.2. Issues Addressed:

Waterpipe addresses the issues under the topic II.3.3.’Advanced Technologies for Locating, Maintaining and Rehabilitating Buried Infrastructures by developing new, reliable technologies for water distribution- incorporating also performance and risk based approaches- for locating assets, identifying defects, identifying leaks, monitoring and rehabilitating the buried infrastructures. The effectiveness of the developed technologies and tools will be evaluated. The main aim is to improve operation, rehabilitation, serviceability, pollution prevention and safety and thus minimizing direct and indirect costs, including the environmental and socioeconomic ones. Included in the participants are industries, end-users and SMEs.

2. Benefits

The utilities will obtain significant benefits:

- The most conservative method for water utilities to estimate the economic benefits derived from the improved leak and damage detection and rehabilitation programme offered in this work is to consider the value of the water saved plus electricity used to pump and chemicals used to treat that volume. Another method, additionally, contemplates the reduced cost of inspection and rehabilitation that will be achieved through the proposed package1. In a third method, which is perhaps the most appropriate, the utility's total variable costs plus the sizable costs of the fixed capital investment for a given period are divided by the volume of the water sold. Even with the most conservative of the above estimates, though, the water utilities will obtain significant savings.

- Additional benefits to the utilities offered by the proposed package include the extension of life of their network, the lessening of emergency overtime for employees, the reduction of infiltration in the sewer system and thus, the reduction in the amount of waste water that needs treatment, the reduction of the likelihood of lawsuits, the reduction of the probability of water contamination, the reduction of the probability of streets and neighbouring utilities collapsing and the improvement in public relations (more reliable service, fewer rate increases).
- Furthermore, utilities will acquire a tool that will permit them to take into account, in an integrated way, costs, benefits, efficiency, social and environmental implications of decisions on rehabilitation and will characterise the sustainability dimension of their rehabilitation decisions so that they can manage their water supply system in the context of implementing the EU Water Framework Directive. Moreover, this tool will offer invaluable help for presenting their cases to their regulators.

3. Expected results:

3.1. Ground Penetrating Imaging Radar (GPIR)

A novel, high resolution, GPIR will be designed and built for detecting buried water pipelines, for detecting leakage and damage in water pipelines and for imaging the damaged region.

Software will be developed that will convert inspection results into quantities needed for a quantitative assessment of the structural condition of the pipeline. These quantities include the location of damage along the pipeline, the location of damage in the damaged cross-section, type of damage (deformation, cracks, open joints) and size of damage. Moreover, the error in describing the above will also be provided.

3.2. Decision Support System for water pipeline rehabilitation (DSS)

The DSS will include:

3.2.1. A software implementation that will determine leakage rates and the probability of having these leakage rates over time.

A methodology will be developed based on inputs from the GPIR on the size of openings and based on the pipe material, diameter and pressure will determine the flow rate of leakage at the time of the inspection.

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[1] The utilities using the DSS will reduce the leak inspection cost through optimal re-inspection timing. Additionally, they will obtain savings in the cost of the engineer who assesses the pipeline condition and selects remedial measures. Furthermore, they will obtain savings due to optimisation of rehabilitation.

A methodology will also be developed to assess, probabilistically, the evolution of the flow rate of leakage over time. For this assessment, the influence of the amount of leaked water (which increases with time) on soil erosion and the Modulus of Soil Reaction and through them on the size of openings will be considered (soil erosion causes bending of the pipe and opening of the defects at the invert while a reduced Modulus of Soil Reaction promotes crushing of the pipe and the opening of horizontal cracks at the crown, invert and springlines). The result will be estimates of flow rates as well as the probability of having these flow rates over time.

A Bayesian procedure will be developed to update the above flow rates when new inspection results become available

3.2.2. A software implementation that will determine, probabilistically, the level of contaminants as a function of time

A methodology will be developed for the probabilistic assessment of concentrations of pathogens and other contaminants in the pipeline over time because of loss of physical integrity. These concentrations will be a function of the location of the pipeline (e.g. proximity to sewers) and are positively related to the size of the openings.

A Bayesian procedure will be developed to update the estimates of concentrations when new inspection results on the size of defects become available.

3.2.3. A software implementation that will determine the probability of crushing, bending and joint failure over time.

A methodology will be developed based on inputs from the GPIR on the location, type and size of damage and based on the material of the pipeline, type of soil surrounding the pipeline, loading, water table and age of the pipeline, that will determine the structural adequacy of the pipeline at the time of the inspection.

A methodology will also be developed to assess, probabilistically, the evolution of the structural adequacy of the pipe. This will be based on (a) the evolution of the loading system, (b) the deterioration of pipeline materials' properties and (c) the evolution of the soil-structure interaction.

A Bayesian procedure will be developed to update the above when new inspection results become available.

3.2.4. A software implementation that will permit water utility managers to decide which pipelines to rehabilitate, when and how and when to re-inspect individual pipelines.

A risk-based methodology will be developed for the rehabilitation management of the trunk and distribution water mains. It will attempt to answer the questions: Which pipeline should be rehabilitated? How? When? When should there be the next inspection for specific pipelines?

The first step will be to select serviceability based measures of performance (or failure modes) that might include bursts, interruptions to supply, water quality, leakage and pressure.

To estimate the risk for each point in time for each pipeline, all the consequences of burst failure, leakage failure, water quality failure, etc. will be identified and assessed quantitatively or, if this is not possible, qualitatively. The probabilities of occurrence of each of these consequences will also be assessed.

A range of rehabilitation schemes will be identified and evaluated. As their benefits are similar (reduce leakage and/or increase structural reliability), for specific pipelines they will be selected primarily based on costs. Included will be all associated direct and indirect costs while net present values of costs will be used when possible. On the other hand, the level of risk as a function of time for the various pipelines will guide project prioritisation. Additionally, a methodology will be developed for re-inspection times for specific pipelines based on their level of risk.

3.2.5. A GIS module, a database and a data manager (see Fig. 1).

The data base will serve as the repository of all project related data, such as, data on the pipe material, data on the location of leaks, data on the orientation of cracks (horizontal, vertical, etc.), length of cracks, opening of cracks and changes in the vertical and horizontal diameters and the results of the calculations and the scenario data ('what-if' analysis results).

These data will be available to the other modules via the data manager, a set of subroutines that will hide from all other modules the implementation details of the database. The data manager will be the only access point to the database and it will provide a customised view for each particular module providing access control mechanisms and assuring the data integrity in the data base.

A specific view of the data manager will be to access the spatial data of the water pipe network which will be stored in the GIS module. The incorporation of the GIS module will facilitate the user to examine a specific area of pipes or a specific pipe and evaluate the results in a visual manner. Incorporation of the GIS module will also make it possible to consider factors such as proximity to other utilities, extent of traffic, etc

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3.2.6. An Expert System with a knowledge base and a query manager

The expert system will coordinate the various modules of the DSS, provide easy access to the system via a user-friendly interface and take care of the maintenance of the domain knowledge and the relevant data base.

The integrated DSS will invoke all modules as required and display, via the user interface, all necessary data to the end user. The user will be able to estimate the present status of the water pipe network or a specific pipe or he will be able to experiment with scenarios to estimate the possible status of the water pipe network depending on the desired inputs of the scenario. The interpretation of user queries, the invocation of the appropriate module(s) to a specific sub-query as well as the composition of responses to the user will all be functions of the query manager.

Included in the questions that the user might be able to ask are questions of the following type:

- When will a specific pipeline segment, of given diameter, wall thickness, material, joint type, age, soil type, depth of cover, etc. with a specific leakage and structural damage when inspected 3 years ago, reach a point that the probability of crushing failure will be 80%?
- What will be the best method to rehabilitate the pipeline at this point?
- What will be the direct cost to rehabilitate the pipeline at this point?
- What will be the indirect cost to rehabilitate the pipeline at this point?
- What will be the impact on risk to rehabilitate the pipeline at this point?
- What will be the water quality risk at this point?
- What will be the leakage flow rate at this point?
- What will be the water quality risk at the same pipeline when the probability of crushing failure is 50%?
- What will be the impact on risk if the pipeline is rehabilitated five years later?

4. Structure of work plan

The work is broken down into 8 workpackages as follows:

- WP1 - Development of the High Resolution GPIR
- WP2 - Evaluation of the Prototype Equipment and Software Produced in WP1 on Pipes of Known Performance (in Terms of Location, Damage)
- WP3 - Development of Methodologies for the Time-Dependent, Probabilistic, Assessment of Leakage Rate, Structural Reliability and Concentration of Contaminants in Water Pipelines
- WP4 – Development of a Methodology for the Risk-Based Rehabilitation Management of the Water Supply Network
- WP5 – Development of the DSS Modules on Leakage, Structural Reliability and Concentration of Contaminants
- WP6 – Development of the DSS Module on the Risk–Based Rehabilitation Management of the Water Supply Network
- WP7 – Development of the Expert System, the Data Base, the Data Manager and the GIS Module in the DSS. System Integration and Package Integration.
- WP8 – Field Evaluation of the GPIR and the DSS

The graphical presentation of the workpackages showing their interdependencies is presented in figure 2.

5. Ground Penetrating Imaging Radar- Fundamental Specifications

The main specifications of the GPIR equipment are the followings:

- Maximum Imaging Depth: 2 meters from the earth surface.
- Spatial-imaging resolution in 3D: better than 5 cm.
- Acquisition time of underground image of 1X1 m^2 in less than 10 secs.
- The radar should be mounted on a truck for continuous measurements.
- Power consumption requirements: less than 500 Watt.
- The operation should be fully automated- no user intervention- only to be on the level of supervision to take action if something goes wrong.
- The GPIR should be capable of surveying 1km in less than 2.8 hours.

Note: the above specifıcatıons are ındıcatıve and should be consıdered as mınımum performance to be achıeved ın the waterpıpe research project. the desıgn of gpır ıs open to ımprove the above parameters.

6. General Presentation of S.C. COMPANIA AQUASERV S.A.

S.C. COMPANIA AQUASERV S.A., with its main headquarters in Tirgu Mures is at present one of the most appreciated companies in the field of the public drinking water supply and sewerage services in Romania. The company is the successor of the former Aquaserv Autonomous Regia that supplied drinking water and sewerage service in the town of Tirgu Mures.

S.C. COMPANIA AQUASERV S.A. is the result of the association of the town of Tirgu Mures with the Mures County Council and with the localities Sighişoara, Reghin, Luduş, Târnăveni, Iernut and Cristuru Secuiesc, these localities representing the shareholders of the company.

The company holds a first class operation licence for the public drinking water supply and sewerage services.

Starting on the 15th of January 2007 the company operates in the municipalities Tirgu Mures, Sighişoara, Târnăveni, Luduş, Iernut and Cristuru Secuiesc.

Service portfolio of the company includes:

- Production and distribution of the drinking water
- Collecting and treatment of urban wastewater
- Collecting and management of storm water

Key figures:

Share capital:	2.1 million Euro
Annual turnover:	7.0 million Euro
Operating profit:	1.8 million Euro
Investments: approx.	1.0 million Euro / year
Number of employees:	800

Technical-economic data:

Drinking water production capacity:	230.000 m3/day
Total length of water distribution network:	600 km
Total length of sewer network:	600 km

Envisaged market:

- drinking water supply: population of more than 250,000 inhabitants
- sewerage: population of more than 204,000 inhabitants

7. AQUASERV`s role in WATERPIPE

The contribution of Aquaserv company in development of the workpackages are the followings:

7.1. WP1 – Development of a High Resolution GPIR

- Provision of end-user requirements

7.2. WP3– Development of Methodologies for the Time-Dependent, Probabilistic, Assessment of Leakage Rate, Structural Reliability and Concentration of Contaminants in Water Pipelines

- Provision of end-user requirements

7.3. WP4 – Development of a Methodology for the Risk-Based Rehabilitation Management of the Water Supply Network

- Provision of requirements
- Identification and assessment of the direct costs to this utility other than the rehabilitation costs
- Assessment of the length of interruptions of supply and their relationships to leakage, structural damage and the rehabilitation methods
- Identification and modelling of the relationships between leakage and structural damage and low pressure

7.4. WP5 – Development of the DSS Modules on Leakage, Structural Reliability and Concentration of Contaminants

- Provision of requirements
- Provision of data so that the DSS can be customized for its network

7.5. WP6 – Development of the DSS Modules on the Risk-Based Rehabilitation Management of the Water Supply Network

- Provision of requirements
- Provision of estimates of the direct costs to this utility other than the rehabilitation costs
- Provision of data so that the DSS can be customized for its network

7.6. WP7 – Development of the Expert System, the Data Base, the Data Manager and the GIS Module in the DSS. System Integration and Package Integration

- Provision of data so that it can be customized for this utility
- Provision of user requirements

7.7. WP8 – Field Evaluation of the GPIR and the DSS

- Selection of pipes of its network to be tested (blind tests).
- Evaluation of the DSS
- Evaluation of the cost effectiveness of the package

8. Tables and Figures

Figure 1.

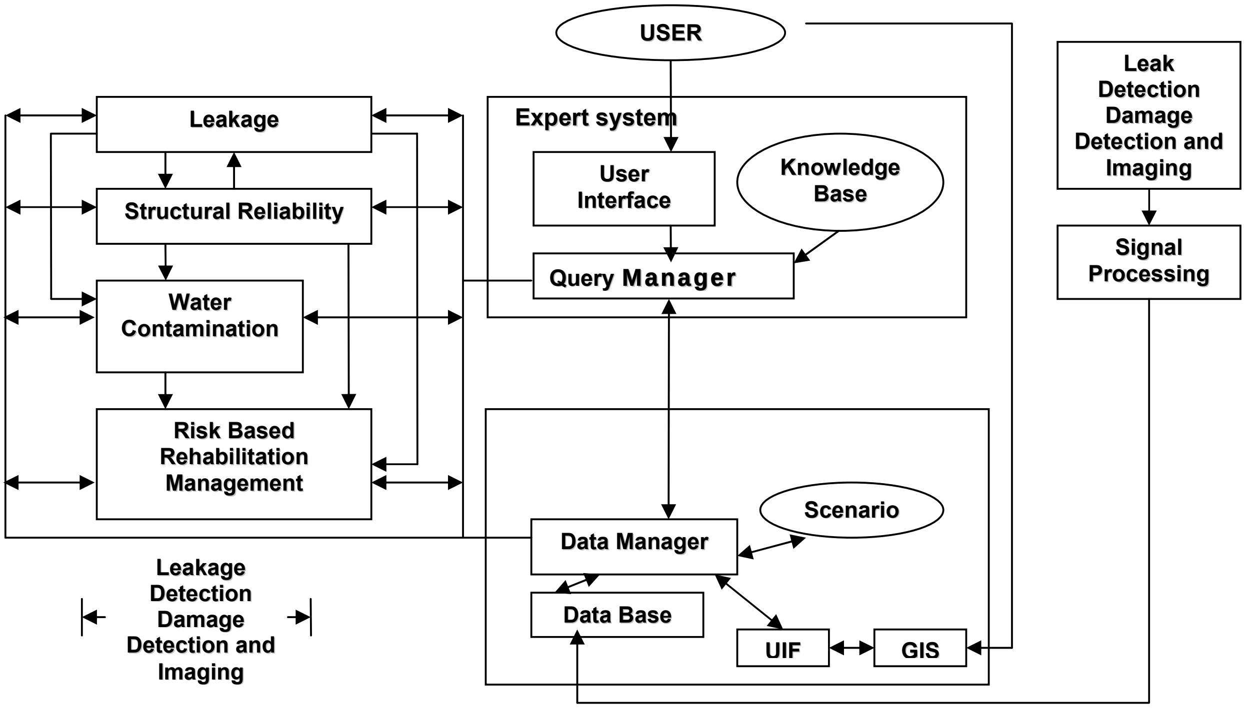
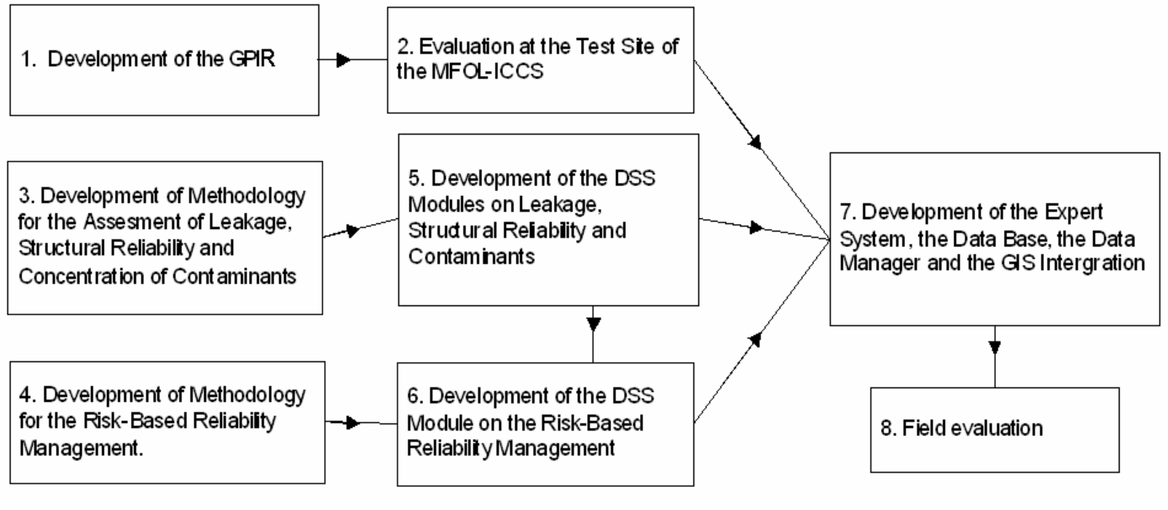


Figure 2.



9. Contacts

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Comprehensive solution of water loss reduction

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Keywords: Leakage reduction

Introduction

Leakages are only one of the fundamental attributes of municipal piping networks. With regard to the fact that increased leakages may have a number of causes, it is necessary to examine a range of influences and take these into account in the resultant long-term solution. A complex approach in resolving the proposed measures is essential for minimising both investment and operational costs. The entire problem is thus described from this wider perspective.

Main problems of municipal networks

The condition and main problems of municipal networks in CEE are given primarily by historical development, in which costs for maintenance and primarily renewal of infrastructure were maintained at a minimal level. Water mains and service pipes in central areas are of considerable age in the majority of towns, which results in considerable problems from the perspective of water leakages, high breakdown rate, incrustation of networks (“overgrowth of mains”) and the connected reduction of capacity, and in many cases also problems with the quality of the supplied water. A lower age of the water mains network usually indicates a housing estate development, where water mains were predominantly built within a very short period. Here too however there are considerable problems with breakdowns and leakages, in many cases paradoxically greater than in the much older mains in central areas. This is due primarily to the low quality of the material of water conduits and construction works, as well as high and frequently fluctuating pressures within the network.

In the past the design for the dimensions of water mains often ensued from preconditions which differed markedly from today’s reality. Primarily with regard to the decline in water consumption over the last 15 years a large part of the networks has been re-dimensioned. This has negative consequences, primarily on the quality of water, since although the secured water quality is sufficient upon discharge from the treatment plant this quality deteriorates within the water mains network. It is necessary to be aware that in our conditions there may be problems not only within public water mains networks, but also in internal distribution systems of buildings, which are frequently insufficiently maintained, ageing and undersized. This increases requirements for excessively high pressure within the water mains network.

Examples of the age distribution of the network and its relationship to the breakdown rate of mains are illustrated in the following two graphs. The first (Fig.1.1) represents the central area of the town, where water mains were constructed at the turn of the 18th and 19th centuries and in the 1940s. Relatively intensive reconstruction of the network also took place from the end of the 1970s. The average breakdown rate of mains is closely related to the age thereof. The second graph (Fig. 1.2) shows an extensive network of housing estate development, in which the predominant part was constructed in the period 1985-1990. From the graph it is evident that the breakdown rate of the mains is highly variable and indicates problems of construction works and piping material used in the housing estate construction in the years 1970-75. It is an interesting fact that for the mains constructed before 1990 the breakdown rate is up to two times higher than the breakdown rate of older mains in the central development.

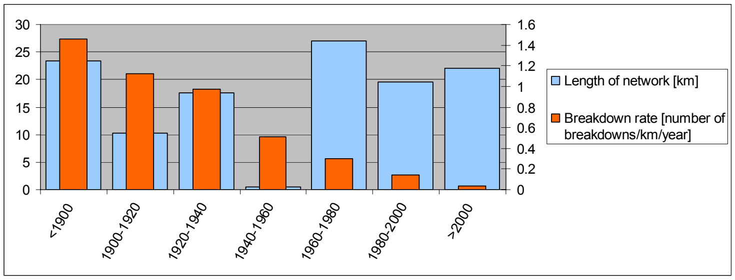

Figure 1. 1 Example of age distribution of network and dependence of breakdown rate on age – central municipal development

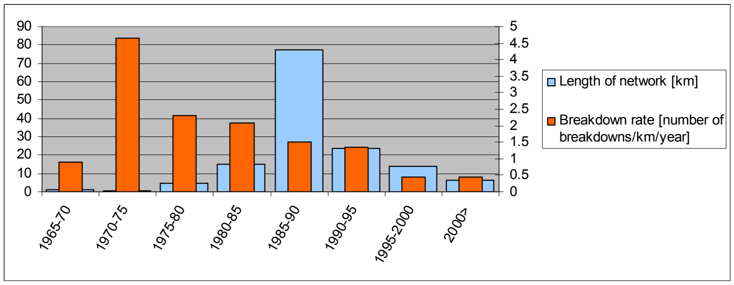

Figure 1. 2 Example of age distribution of network and dependence of breakdown rate on age – housing estate development

Almost always problems in the water mains network are not of a network-wide character but are rather local, primarily with regard to low capacity, leakages and breakdown rate. Experiences from a number of projects devoted to the study of leakages within the water mains network can serve as an example, in which the predominant part of the leakage of the entire water mains was identified within a relatively small section of the network:

- Daruvar (Croatia) - identification of network with length 21% with 80% of leakage.
- Gánovce (Slovak Republic) – identification of network with length 16% with 97% of leakage.
- Plzeň - Litice (Czech Republic) – identification of individual leakage of 4.8 l/s in 960 m of network, remainder of other examined network with length of approx. 29 km with relatively low leakage.
- Ústí nad Labem (Czech Republic) – identification of network with length 36% with 90% of leakage.

The predominantly unsuitable technical condition of the water mains network connects also to relatively limited possibilities for measurement of flows and pressures and a low volume of data which is usable for evaluation of the technical condition of the networks. From this perspective it is necessary to devote considerable attention also to the technical records in connection with evaluation of the technical condition of piping such as GIS, records of breakdowns and causes thereof and to the progressive supplementing of gauges.

Today's urban development trends differ markedly from the plans which were valid at the time of construction of the water mains. It can be said that today the development of residential construction is considerably more erratic. Every community or municipal district is attempting to develop and not all are succeeding in implementing their plans. Primarily from the perspective of requirement for water, the development of industrial zones is still more "inscrutable". Almost all municipalities are attempting to stimulate interest from investors. The offer of possible industrial zones frequently exceeds demand and the actual implementation is dependent on several factors. It is still more difficult to estimate the requirement for water which shall be implemented in the future industrial zones. An example of the large difference in average water requirement per hectare of industrial land depending on the use thereof is illustrated by the following graph.

Principles of complex approach to planning development of municipal networks

As the last experiences show, problems in municipal water mains networks combine a whole range of mutually linked factors. For a complex solution to these problems whilst minimising both investment and operational costs, it is possible to make successful use of modern technology of assessing and proposing measures in water mains networks which are methodically based on the application of computer technology. A mathematical model calibrated on the basis of measurements in terrain, application of digital technology and linkage to external data sources enables assessment of the existing system of water supply and the proposed conception of its development on the basis of simulation of the behaviour of the distribution system and evaluation of the hydraulic and qualitative parameters of water supply. At the same time this approach enables the construction of a tool for systematic solution of a whole range of conceptual and operational tasks based on the assessment of operational and investment impacts and optimisation of the proposed measures. The requirement for a conceptual tool, by which it is possible to systematically develop and specify proposed measures on the basis of specifying inputs, is relevant primarily from the perspective of the considerable uncertainty and dynamics of urban development within the territory of the municipality as given both by the gradually changing requirements of the municipality and the variable demands of the investors. The stipulated and approved development conception of the water mains network in linkage to the territorial plan shall become the basis for the territorial planning documentation and the basis for the processing of a long-term investment plan. The fundamental target of the complex approach is a harmonisation of the requirements of the municipality from the perspective of future development and conceptual plans in the system of water supply with operational requirements, plans and activities. In this manner it is possible to achieve a considerable increase in the effectiveness of the measures taken and primarily savings on operational and investment resources. The following can be indicated examples of operational activities which are closely connected with the development conception of the system:

- Optimisation of pressure ratios and borders of supply zones.
- Optimisation of operation of pumping stations.
- Optimisation of measuring system and monitoring of leakages, primarily division of network into measurement sections.
- Evaluation of technical condition and planning of reconstruction of water mains network.
- Reduction of leakages and removal of certain operational problems primarily in connection with the increased capacity of the existing water mains network.

The significance of solution to the problem of water leakages is not dependent only on their current level, but on an overall assessment of their impacts on the problem of water supply from an economic, as well as a technological and ecological perspective. In connection herewith, the “feasibility” of investments made into rectifying leakages is often mentioned, in which it is necessary to take into account negative factors of leakages such as:

- increase of operational costs for production, transport and distribution of water,
- generated investment costs for strengthening capacity of system to supply water, both on strengthening water mains pipelines and on the construction of new sources,

- exhaustion of capacity of quality and cheap sources of water and necessity of substitution thereof with poorer quality and more expensive sources,
- overburdening of technology of water treatment plants and reduction of quality of produced water,
- customer dissatisfaction due to insufficient standard of water supply (reduced pressures, suspensions of water supply) and potential financial losses on the part of customers,
- damages caused by leaking water into infrastructure and real estate,
- increased burdening of sewerage system and waste water treatment plants,
- closely connected costs for repairs.

The financial and time demands placed by detection of leakages using the methods which are today now standard, such as use of a correlator and ground microphones, are very substantial. For this reason it is essential to conduct such an operation not throughout the entire water mains system, but only in areas where there is a large leakage. Identification of such areas is possible by means of a study of the water mains network in a combination of measurement and a mathematical model. On the basis of the results of this method it is possible not only to increase the effectiveness of the activity of research groups, but in the realistic conditions of operational companies to considerably reduce leakages in water mains networks within a relatively short time. From the perspective of the long-term strategy of reducing water leakages the construction of a system of measurement and evaluation of inflows into separated measurement sections is of fundamental importance. A further important strategic measure for the long-term increase of the life span of the network, reduction of leakages and breakdown rate is optimisation of pressure ratios in the water mains network.

Project “2 supply zones of water mains network in municipality of Ústí nad Labem – mathematical model”

The procedure and results of a complex approach to the solution of municipal water mains networks are presented on the processed project “2 supply zones of water mains network in municipality of Ústí nad Labem - mathematical model”. This project was processed by the company DHI Hydroinform a.s. in 2006. The project was ordered by the companies SVS a.s. and SČVK a.s, with the aim of examining the benefits of a complex assessment of the municipal water mains network and application of mathematical modelling for two selected supply zones – Předlice and Kolonka in Ústí nad Labem. In this project the basic outcomes of the mathematical model were used for the following tasks:

- Proposal for solution of development conception of water mains network in defined territory.
- Optimisation of pressure ratios and borders of supply zones.
- Modelling of water quality – age of water.
- Detailed study of water mains network in problematic localities.
- Distribution of leakages in water mains network.
- Solution of monitoring system from perspective of resolving water losses.

The project was completed at the turn of 2006/2007 and on the basis of its very positive results works were ordered for 2007 and 2008 on the Master plan for water supply of Ústí nad Labem. Within the framework of this Master plan the resolved territory shall be extended to the entire municipality. At the same time the resolved problem is extended from the perspective of evaluation of the technical condition of the distribution system, compilation of a plan for renewal of the network and the long-term plan of investments into the water supply system.

Mathematical model

The primary activity which conditions use of the model as a fundamental tool for the resolution of defined tasks is naturally the actual construction and calibration of the model. The entire solution resides in the maximum use of data inputs and an extensive measurement campaign conducted by the processor in close co-operation with the operator of the water mains network (Fig. 1.3).

The most up-to-date data are used for the creation of the model, primarily the graphic information system (GIS), data from invoicing of water (CIS), data of the control system, digital model of terrain etc. Within the framework of the project a large number of measurements were conducted within the resolved territory within several measurement campaigns. The measurement campaign M1 and M2 served for basic information about pressure and flow ratios in the network, for calibration of the model, for detailed research into the behaviour of the network and for verification of behaviour of the network and for verifying the capacity of the piping network. All activities such as preparation of measurement campaigns, installation of instruments, necessary handling in the network and evaluation of data were conducted in close co-operation with the operator. A high degree of linkage and co-ordination of activities was achieved within the framework of the measuring teams.

Figure 1.3 Examples from performance of measurement campaigns

Water leakage in water mains network

Selection of supply zones with significant leakage

For stipulating significant supply zones from the perspective of leakage it is necessary first of all to conduct a complex evaluation of all supply zones from the perspective of the balance of water consumption and supplied water. In this it is necessary to include all the available information. The result is an overview not only of the development of leakages but an overall view of the components of inflow into the individual supply zones. This concerns primarily the following data sources:

- Records of development of water requirement over last X years (in components: water supplied, invoiced, not invoiced and leakage).
- Database from CIS (customer information system).
- Database of relevant locations.
- Consumptions of large consumers (operational measurement of course of consumption, primarily during night hours).
- Archive of SCADA system data at control centre.
- Data of flows from conducted measurement campaigns (course of flows over 24 hours, week).
- Records of status of individual water meters.

- GIS of piping network, hydrants and cocks.

On the basis of the conducted analysis and subsequent discussion with operation it is then possible to perform selection of supply zones with a significant leakage and subsequent proposal for measurement campaign (division of selected supply zones into measurement sections, placement of gauges and handling of regulating valves).

Night measurement campaign concentrating on localisation of leakage

Measurement of distribution of leakages in the water mains network is based on the temporary division of the piping network into measurement sections. Measurement sections are separated by closure of regular or zone valves in combination with measurement of night inflows (leakages) into measurement sections, supplemented by measurement of pressure ratios. The leakage is evaluated in individual measurement sections with regard to large night consumers. Performance of the measurement campaign ensues from the operational conditions, primarily the suitability of installation of portable flow gauges and the possibilities of separating parts of the network by closing with functional cocks.

Evaluation of leakage

Evaluation of distribution of leakages in the water mains network ensues from an evaluation of changes of the measured inflow into the network during the course of the measurement campaign (Fig. 1.4). The measurement campaign is conducted during night hours, in which interventions are made with the target of the progressive switching/closure of individual measurement sections. Leakage into measurement sections is then evaluated with regard to night consumption of large clients and if applicable the objectified night inflow. In addition to the actual size of the leakage [l/s] it is of very fundamental importance to evaluate also other leakage indicators such as unit leakage [l/s/km], which takes into account the length of the piping network and indicates locations of the network where subsequent detection of the leakage and repair of hidden leakages will be most effective.

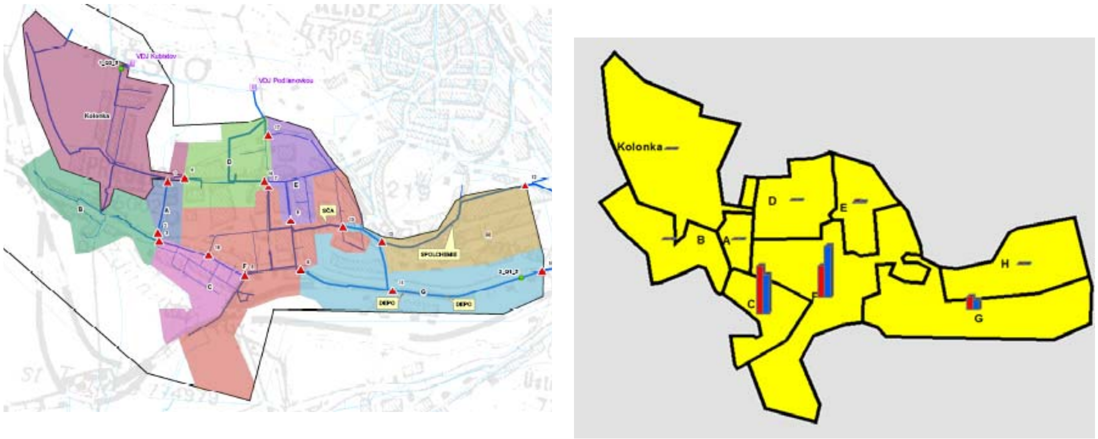

Figure 1. 4 Plan of measurement campaign for evaluation of leakages of water mains network and illustration of determined leakages in network

Determined results of existing distribution of water leakages

The Kolonka supply zone is practically without leakages. On the other hand large leakages of water were identified in the Předlice supply zone, which represent the main current operational problem of the zone and limit the capacity of the network from the perspective of optimisation of its operation, development of the network and securing a sufficient capacity for the development territory in this zone. On the basis of the first measurement of leakages in the Předlice zone 90% of the total leakage was identified in 36% of the length of the network. After finding and repairing hidden breakdowns by operation in these areas a reduction of leakage by 16.8 l/s was verified in the identified area (C+F) within the framework of the second measurement campaign. If this status was permanently sustainable this result represents an operational saving of approx. CZK 2.4 million per annum.

Calibration of model and operational study of water mains network

The aim of the actual calibration of the model is to set the model in order to ensure that it functions in the same manner as the actual water mains network. A further target is to determine parts of the network where non-standard behaviour is occurring from the perspective of pressure ratios. On the basis of the results of the conducted measurement M2, when the network was burdened by consumptions from a hydrant in a number of selected locations, points were identified in the network where limitation of its capacity occur. A number of points were determined where the actual capacity of the network is less than the theoretical capacity. The main causes of this reduction were throttling of the network in locations of flow measurement and throttled or entirely closed cocks. The most serious operational problem of the network capacity was identified in the Kolonka supply zone. Here there is a considerable pressure drop during increased consumption of water from the network. The cause was identified in the model as the closed piping DN 100 in Škroupova street. This operational problem reduces the reliability of the existing system of water supply and most importantly fundamentally limits further construction development and linking of consumptions in the zone.

Conceptual solution of water supply system, including optimisa-tion of pressure ratios and measurement system

The proposed measures which ensued from the performed analyses monitor several mutually linked tasks:

- Removal of existing limitations of operation of water mains network.
- Securing of good conditions for water supply for existing construction development and for prospective development surfaces.
- Optimisation of pressure ratios.
- Supplementing of monitoring system from perspective of monitoring water losses and proposal of measurements for improving operational conditions within resolved territory.

Evaluation of existing water supply system

The proposed measure must be backed up by a detailed evaluation of the existing water supply system. It is necessary to take an individual approach to the identified problems with complex measures, i.e. the measures must resolve as many problems as possible simultaneously. The main problems of the resolved territory were the following:

Kolonka supply zone

- Water leakages are minimal.
- The closed piping in Škroupova street limits the capacity of the water mains network from the perspective of exceptional states at present and almost completely prevents further addition of consumers within the zone.

- Even following the rectification of the above-mentioned operational problem the capacity of the water mains network is at its limit.
- The pressure ratios in the existing water mains network correspond to a heavily supplied construction development.
- It is not possible to secure sufficient pressure ratios by the existing system in the area of the planned industrial zone.

Předlice supply zone

- Large water leakages were identified in the supply zone, which are both the main current operational problem of the zone and also limit the capacity of the network from the perspective of optimising future operation, development of the network and securing a sufficient capacity for developmental territories.
- The existing possibilities for measuring flows and evaluating leakages are considerably limited.
- Operational conditions in the zone are changing markedly on the basis of sufficient water in the Telnice source. At the time of insufficient water in this source the zone is supplied only from one source. This alternation of operational states is limiting for the rational optimisation of operation.
- The existing pressure ratios in the predominant part of the water mains networks considerably exceed requirements ensuing from the height of the supplied construction development. It is necessary to propose measures to reduce and stabilise pressure ratios.

Securing network capacity for connection of new development surfaces

From the perspective of resolving insufficient pressure ratios in the highest situated areas of the new development surfaces the following are expected:

- connection of development surfaces to existing water reservoirs and completion of circuit of network,
- construction of strengthening pressure stations via which part of the development territory shall be supplied.

Completion of the circuit of the network with new gravitational pipelines via the development territories shall secure not only water supply in the newly connected development surfaces, but is also of great importance from the perspective of fundamental strengthening of the network capacity and increasing the reliability of water supply to the zone. Primarily in the Kolonka supply zone this is a fundamental precondition for securing the future reliable functioning of the water mains network. Division of the territory for the function of new additional pumping stations was conducted on the basis of a simulation of pressure ratios in the model.

Optimisation of pressure ratios

Optimisation of pressure ratios is a measure which is important primarily from the perspective of the long-term impact on reduction of water leakages and the breakdown rate and increasing the life span of the network. According to experiences from abroad a reduction of pressure by 10% causes a reduction in the breakdown rate by 25%. Optimisation of pressure ratios is thus important primarily for savings on operational and investment means. Upon evaluation of the optimum pressure ratios in the network it is necessary to start out primarily from the height of the supplied construction development (Fig. 1.5). Pressures of 40 m w.c. may be low in a high rise development, but too high for a development comprising family houses.

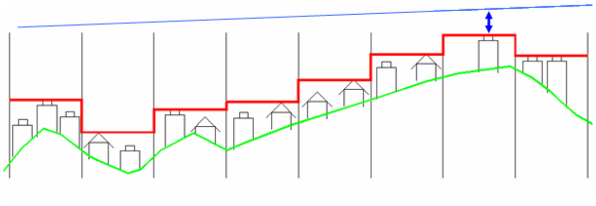

Figure 1. 5 Principle of taking into account height of construction development in assessing pressure ratios in water mains network

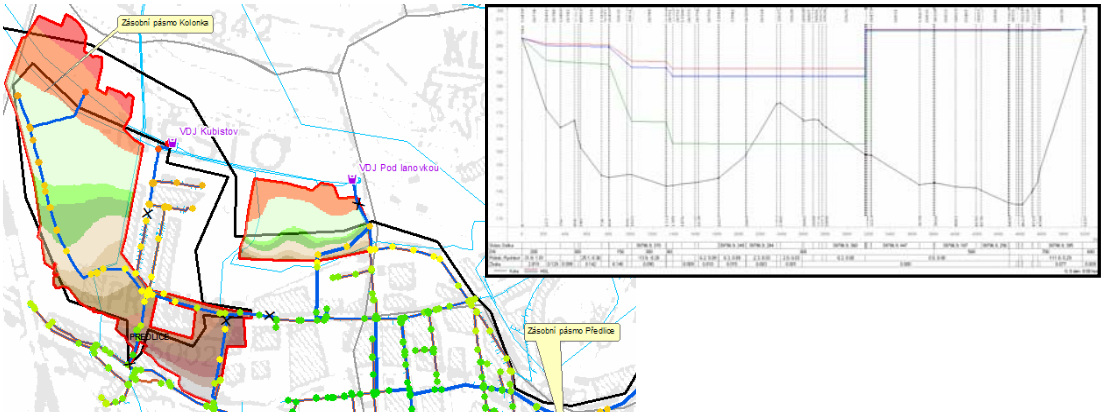

Figure 1. 6 Illustration of evaluation of pressure ratios in network and in development surfaces (insufficient pressures for development surfaces and high pressures in Předlice zone in existing state), illustration of lengthwise profile with charted dimensions of pressure line – indicates problematic points

Through simulation in the model it is possible to evaluate the pressure ratios above the height of the construction development relatively precisely (Fig. 1.6). At the same time it is possible to evaluate the causes of the main problems which prevent optimisation of pressures. The following illustration shows an evaluation of pressure ratios in the territory for the existing network and for the territory of development surfaces. In addition the lengthwise profile shows points of extreme drop in the pressure line, i.e. places where pipeline capacities are to be extended, pressures distorted in development surfaces and shows problems of the existing system. On the basis of a precise determination of problems and requirements it was possible to minimise the proposal for investments to optimise pressure ratios. This resided primarily in the following:

- Rectification of determined operational problems.
- Construction of one shaft with a reduction valve in Jateční street.
- Strengthening of the network capacity – extension of water pipeline capacities DN 100, 150 from 1952 in Jateční street in section from new reduction valve in a westerly direction after transition to profile DN 200 with length of 204 m.

The results of the simulation of pressure ratios following optimisation show a considerable reduction of pressure ratios in the water mains network to values which correspond better to the attached construction development. As against the existing status, in which the average pressure in the Předlice zone is 50.1 m w.c. there was a reduction by 14.3 m w.c. to a value of 35.8 m w.c. In the Kolonka supply zone no fundamental change of pressure ratios is required.

Supplementing of monitoring system from perspective of monitoring water losses

The proposed technical solution of the prospective status of the water mains network considerably increases possibilities for operational variants, which in connection with the optimising of pressure ratios increases possibilities for monitoring inflows into parts of the network and evaluation of leakages and non-invoiced water. From the perspective of supplementing the monitoring system the following measures are proposed:

- Measurement shall be secured for all parts of the network supplied independently from individual reservoirs, strengthening pumping stations or reduction valves. No construction of new water meter shafts as separate investments is envisaged.
- The measurement system must evaluate both for the purposes of a balance comparison of inflows into areas and for the purposes of monitoring minimum night inflow. For this reason equipment of data transmission from the control centre (CC) is proposed for all zone and district gauges in the network. A simple and cost-effective solution of monitoring water meters by an optical or other sensor and GSM transmission is envisaged. Such a technical solution does not require connection to electrical energy or a complicated system of data connection. In addition the level of operational costs can be minimised today through choice of appropriate technology.
- The system of measurement, connections and cocks in the network must enable alternative switching of parts of the network to individual gauges to a sufficient extent.
- In addition to the implementation of new gauges and supplementing of existing gauges with remote transmission, the operation of a number of key cocks in the water mains network is proposed. The proposal for the project resolves also the operational scenario, which enables switching of parts of the network of individual inflow gauges and thus enables far more detailed study of the status of leakages and non-invoiced water.

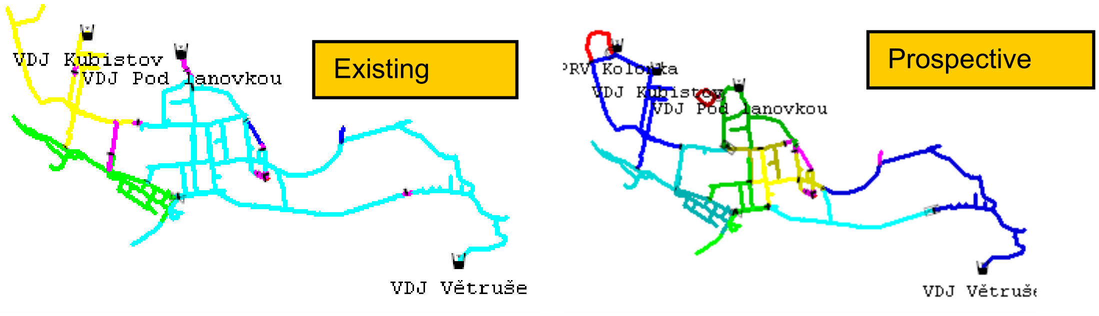

Figure 1. 7 Comparison of individually measured parts of network – existing state (3 parts) and after implementation of measures (13 parts)

On the basis of the proposed measures, the possible separation of the water mains network is increased from the perspective of measuring inflows from the present 3 areas with an average pipeline length of 8.5 km to 13 areas with an average network length of 2 km, whilst minimising the necessary investment costs. This provides preconditions for very effective monitoring of the network, primarily in areas where frequent breakdowns and water leakages occur.

Conclusion

The problems of individual water mains networks and parts reflect a whole range of factors. Of these the most significant are the increasing age of water mains networks and the connected deterioration of their technical condition, breakdown rate and water leakages. Here this is underlined primarily by a historical debt which we owe in the renewal of water mains networks. The in many places precipitous urban development places large demands on the flexibility of technical measures in the water mains network from the perspective of securing the conditions for quality water supply in new areas.

As is shown with increasing frequency, an optimum solution from a technical and investment perspective very often resides in a combination of operational and investment measures. For example securing sufficient capacity for new consumers can be resolved at least partly by reducing the enormous leakages of the network in question, whereas necessary strengthening of capacity can be connected with an replacement of piping in a worse technical condition. From this there emerges the necessary of resolving the development conception of the water mains network in harmony with solution of the problem of water leakages and other current operational problems in the network. Another very significant component of the conceptual solution is the optimisation of pressure ratios and optimisation of measurements and monitoring of the water mains network as a means for saving on operational resources and increasing the life span of water mains piping and connections. The proposal of such an optimum solution must reside in a detailed analysis of the problems of the existing water supply system and a subsequent simulation of the prospective status in a mathematical model calibrated on the basis of measurements in the terrain.

The existing precipitous development of water management technology is a fundamental precondition for the application of a complex assessment of water mains networks as a fundamental tool for resolving complex problems of water management systems and for optimising the expenditure of investment and operational resources. Here primarily the use of digital technologies, modelling of water mains systems and modern measurement procedures are applied. For these technologies to be genuinely effective it is necessary to combine deep knowledge of consultants and designers about the existing technical possibilities with a practical approach to problems and primarily effective co-operation with the operator, administrator and local government. The presented project of the complex solution of the two supply zones in Ústí nad Labem fulfilled these preconditions.

HIDDEN BENEFITS OF SMALL SCALE PERFORMANCE BASED PUBLIC PRIVATE PARTNERSHIPS

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ABSTRACT

Many water distribution systems in South Africa are deteriorating due to many years of neglect resulting in a serious maintenance backlog. Recent government legislation has introduced free basic water to all South Africans up to a limit of 6 Kl/month per property which in turn causes certain confusion regarding payment among many residents. These key issues and others have led to serious problems with service delivery specifically in the low income areas where the maintenance has been neglected for more than 30 years in some cases. The potential for support from the Private Sector has been highlighted at the highest levels within government as a possible solution to addressing the existing backlogs despite the fact that there are relatively few successful projects to support this view.

This paper presents the results after 2 years of operation of a small scale public private partnership in one of the largest low income areas in South Africa where the Sebokeng/Evaton Pressure Management Project was commissioned in July 2005. The savings both in terms of volume of water saved as well as financial savings to the municipality are impressive and exceed all initial expectations. The most interesting aspect of the project, however, is not the savings achieved from the installation, but the numerous other additional benefits that have materialised which were not originally anticipated when the project was commissioned. Such benefits, include the identification of many network problems that had been undetected for more than 9 years as well the sudden interest in helping the residents by several government and semi-government organisations. These organisations were unable or unwilling to provide any support to the area prior to the successful Public Private Partnership.

The project represents a significant advancement in Public-Private Partnerships (PPP's) and clearly demonstrates that small scale Public Private Partnerships can be viable despite the general view that this type of project is confined to large scale initiatives due to the effort and expense in developing the PPP type of contract. The paper provides details of the processes involved in setting up and implementing such a project and highlights that the model used by the Project Team to address leakage in Sebokeng and Evaton can be adapted for use in other areas and other applications to improve service delivery throughout South Africa as well as elsewhere in the world where conditions permit.

The paper presents the results from the project after 2 years of operation and summarises some of the many additional benefits that have arisen from the project.

INTRODUCTION

Emfuleni Local Municipality is shown in **Figure 1** and is located to the south of Johannesburg in the industrial heartland of South Africa. A separate water utility called **Metsi-a-Lekoa** was established several years ago to manage the supply of potable water to approximately 1.2 million residents of the Municipality of which 450 000 are located in the Sebokeng and Evaton areas. Water is supplied to Metsi-a-Lekoa from the local bulk water provider which is one of the largest providers of potable bulk water in the world.

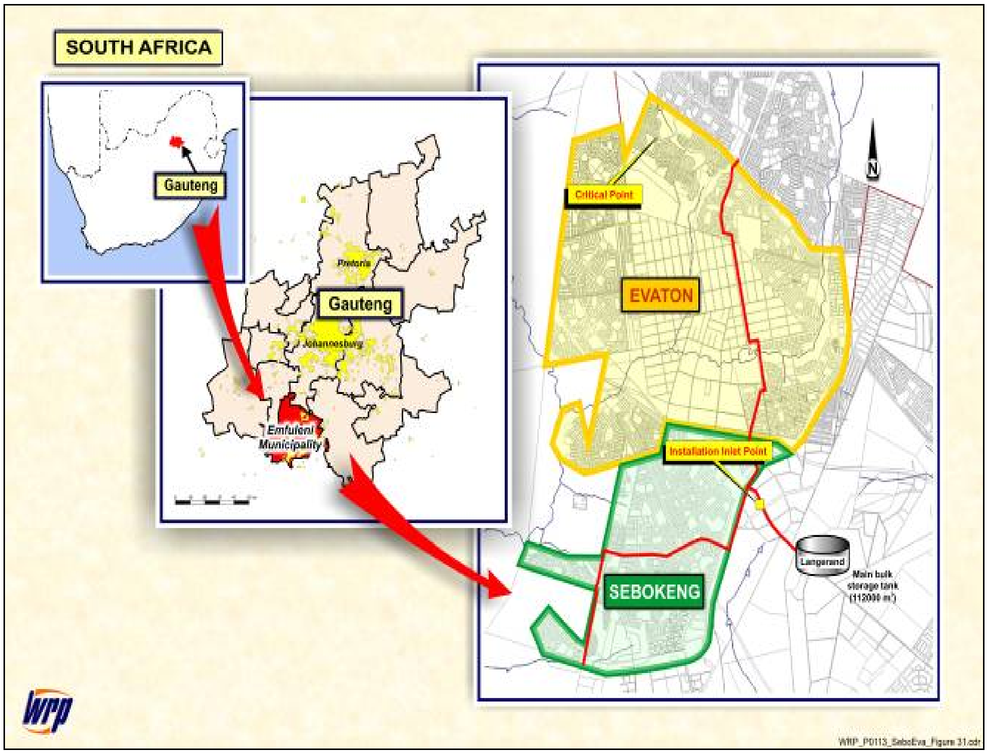

Figure 1: Location Plan

The areas are predominantly low-income residential areas with approximately 70 000 household connections, each of which is supplied with an individual water supply as well as water borne sewage. The combination of low income coupled with high unemployment has resulted in a general deterioration of the internal plumbing fittings over a period of many years causing high levels of leakage. The leakage at the start of the project was known to be extremely high as indicated by a minimum night flow (MNF) in the order of 2 800 m^3/hr as shown in **Figure 2**. This is one of the highest MNF's recorded anywhere in the world and represents almost two Olympic sized swimming pools of water every hour during a period when demand for water should be minimal. It should be noted that there is virtually no storage in the Sebokeng and Evaton areas, either at bulk reticulation level or domestic property level. The high MNF is therefore almost completely due to leakage, most of which occurs inside the properties and is therefore not evident from normal visual inspection. It should also be noted that since most of the leakage occurs inside the households, the leaking water returns to the sewage treatment plant through the sewer network which is often overloaded to such an extent that spillages of raw sewage into local river courses are a common occurrence in the area.

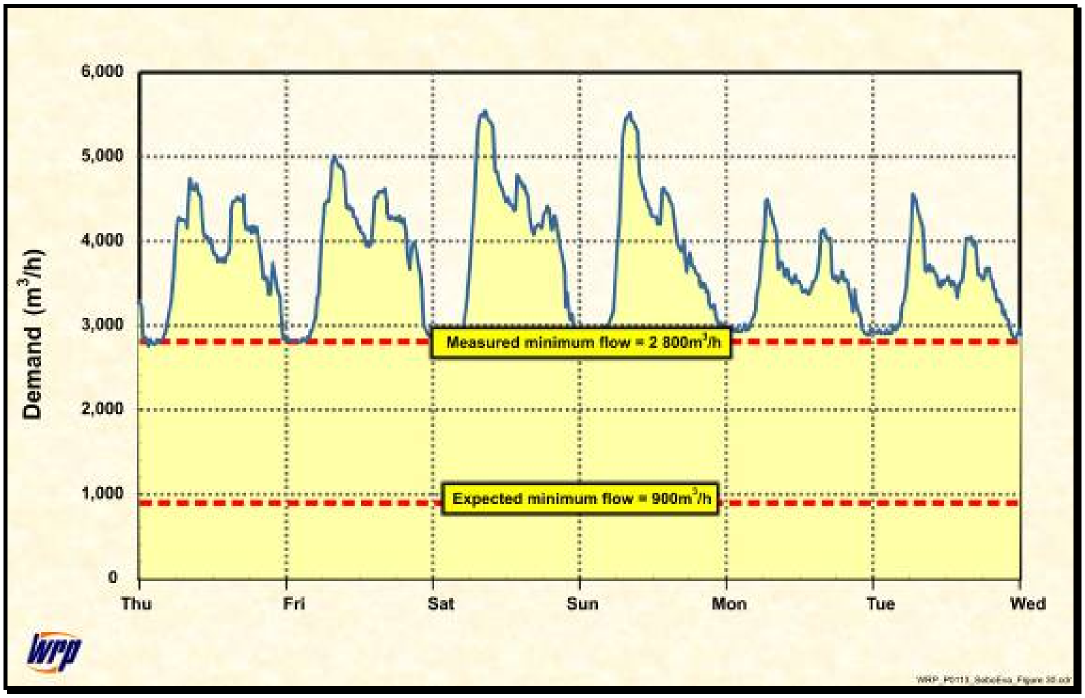

Figure 2: Initial MNF entering Sebokeng and Evaton (July 2003)

It was estimated that the wastage in the area before the project was commissioned was in the order of 80% of the water supplied to the area which in turn represented an annual water bill of approximately R120 million per year (±$20 million).

In 2004, the Municipality appointed WRP Pty Ltd to design and commission what is understood to be one of the largest advanced pressure management installations in the world as the first phase of a long term strategy to reduce wastage in the area. The project involved no financial input from the Municipality and even the initial capital costs were borne in total by the Project Team. The project was, effectively, a small scale Public Private Partnership involving a simple risk-reward model and the original concept is discussed in detail by Mckenzie and Wegelin (2005). It basically reduces water pressure in the area during of-peak periods and in this manner reduces the water lost through leakage.

BENEFITS OF THE PROJECT

The most obvious benefits from the project are clearly the savings in water purchases by the Municipality from the bulk water provider due to the reduced leakage in the Sebokeng and Evaton areas. The initial projected savings of approximately R20 million (±$3.3 million) per year (Mckenzie and Wegelin, 2005) were in fact exceeded and after the first full year of operation the actual savings achieved were closer to R27 million (±$4.5 million) as highlighted in the subsequent paper by Mckenzie and Wegelin (2006). At the time of writing this paper, the project had been operational for 2 years and the initial level of savings had been maintained as shown in **Table 1** and again graphically in **Figure 3**.

Table 1: Summary of savings for first 2 years of operation

Month	Water Use (m^3)		Savings		
	Expected	WRP Actual	(m^3)	Rands	US$
Jul-05	3,074,241	2,438,310	635,931	1,755,170	292,528
Aug-05	3,083,840	2,460,620	623,220	1,720,088	286,681
Sep-05	3,093,130	2,459,070	634,060	1,750,005	291,668
Oct-05	3,102,729	2,406,260	696,469	1,922,254	320,376
Nov-05	3,112,018	2,421,960	690,058	1,904,561	317,427
Dec-05	3,121,618	2,427,780	693,838	1,914,992	319,165
Jan-06	3,131,217	2,337,020	794,197	2,191,983	365,331
Feb-06	3,139,887	1,997,250	1,142,637	3,153,678	525,613
Mar-06	3,149,486	2,200,560	948,926	2,619,036	436,506
Apr-06	3,158,776	2,118,830	1,039,946	2,870,250	478,375
May-06	3,168,375	2,055,280	1,113,095	3,072,142	512,024
Jun-06	3,177,664	2,076,990	1,100,674	3,037,861	506,310
Jul-06	3,187,263	2,149,000	1,038,263	3,010,964	501,827
Aug-06	3,196,863	2,296,197	900,666	2,611,930	435,322
Sep-06	3,206,152	2,393,860	812,292	2,355,647	392,608
Oct-06	3,215,751	2,545,230	670,521	1,944,511	324,085
Nov-06	3,225,041	2,107,670	1,117,371	3,240,375	540,063
Dec-06	3,234,640	2,384,830	849,810	2,464,449	410,741
Jan-07	3,244,239	2,387,810	856,429	2,483,644	413,941
Feb-07	3,252,909	2,212,620	1,040,289	3,016,839	502,806
Mar-07	3,262,508	2,411,900	850,608	2,466,764	411,127
Apr-07	3,271,798	2,067,000	1,204,798	3,493,914	582,319
May-07	3,281,397	2,393,900	887,497	2,573,741	428,957
Jun-07	3,290,687	2,500,000	790,687	2,292,991	382,165
Total	**76,382,228**	**55,249,947**	**21,132,281**	**59,867,789**	**9,977,965**

The savings achieved in the first two years of operation of the installation exceeded all expectations of both the Project Team as well as the Municipality and are the most obvious benefits to accrue from the project. After operating and managing the installation for two years, several other benefits also became apparent which were not initially anticipated. In particular the following benefits have been achieved each of which will be discussed in turn:

- Defer upgrading of infrastructure
- Identification of bottlenecks in the system;
- Identification of problem infrastructure;
- Identification of bulk meter errors;
- Catalyst for funding;
- Improved Municipality Status
- Creation of National WDM fund;
- Catalyst for other WDM interventions;
- Sustainability of Savings.

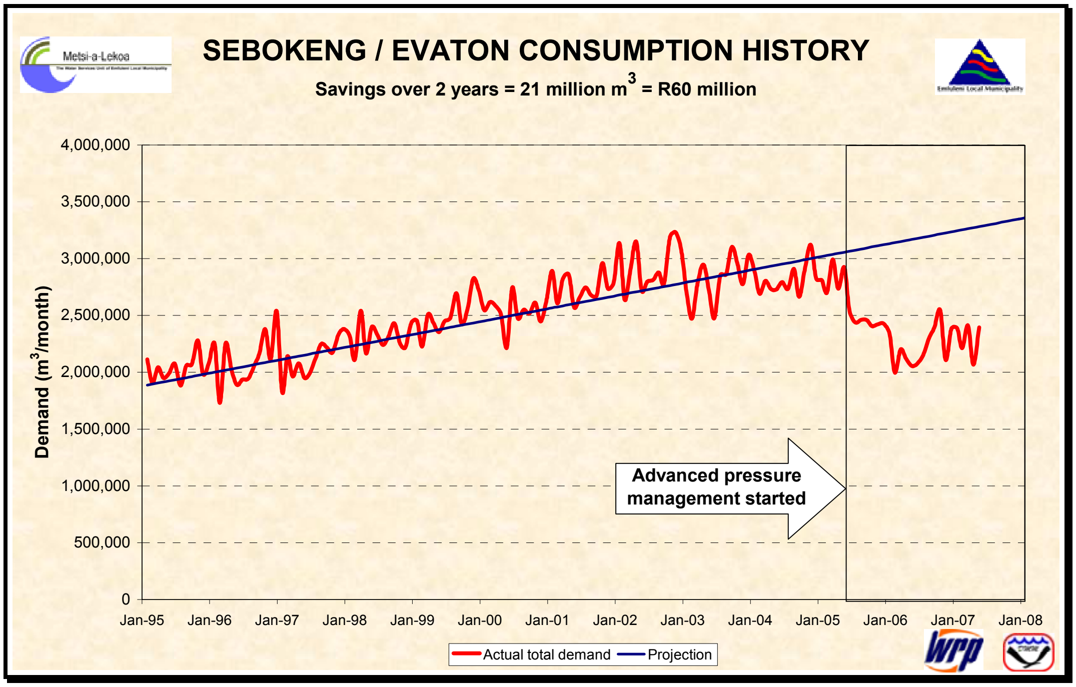

Figure 3: Historical water consumption in Sebokeng and Evaton areas

Defer upgrading of infrastructure

With the implementation of the advanced pressure management system the water demand was reduced to 1997 figures as can be seen in **Figure 3**. The reduced water demand also had a significant impact on the sewer flows entering the treatment plant which reduced from 2500m^3/h (July 2003) to 1800m^3/h (July 2005) as shown in **Figure 4**. As a result of the project the project the client has gained a reprieve of at least ten years on the upgrading of the water supply and sanitation infrastructure. The reduced pressures have also resulted in a significant reduction in the number of bursts experienced in the area which in turn will prolonging the life of the infrastructure.

Identification of bottlenecks in the system

Under normal circumstances with large scale pressure management projects, the system pressures are gradually reduced during the off-peak periods to ensure that some minimum level of service is achieved at the critical point in the system which is the point experiencing the lowest pressure at the time. The critical point can usually be identified from the reticulation layout drawings or from a hydraulic model of the system if such a model is readily available. The critical point is then monitored continuously as the pressure management activities commence.

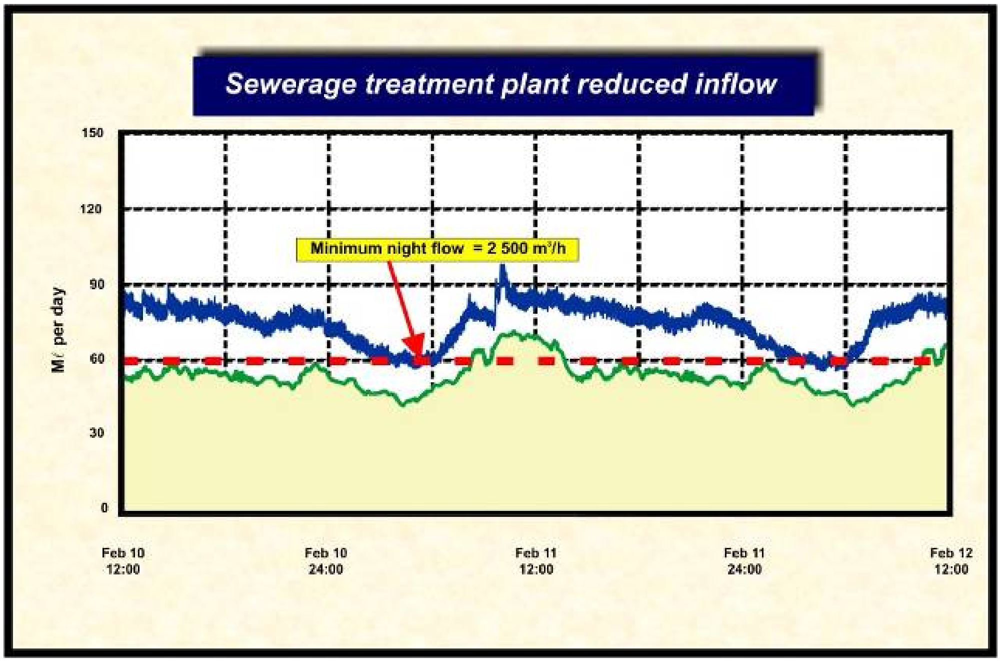

Figure 4 : Reduced sewer inflows

In the case of the Sebokeng/Evaton project, the above process did not proceed to plan and numerous communities complained of low water pressures in areas which theoretically should have had no problems. On closer examination of the unexpected problem areas, it was found that many of the problems were caused by poor maintenance or inappropriate operation of various boundary valves and/or control valves. In many cases, the boundary valves were left in a semi-open position due to the fact that the operations staff did not know if they should be open or closed. The operations staff also had the habit of closing sections of pipe in cases where a burst had occurred instead of repairing the burst and reopening the pipeline. This has caused serious bottlenecks in the system which only became apparent when the pressures were lowered. In each case the project team had to cease all further pressure reduction activities and undertake a full investigation involving significant field work to identify and correct all problem valves and/or sections of pipeline in a particular area. Following the corrective measures, it was normally found that the overall level of service to the specific community improved significantly when the system was reinstated to its original configuration.

Identification of problem infrastructure

In addition to the identification of bottlenecks as discussed previously, there were numerous cases where serious problems were found in the basic reticulation infrastructure. One of the most common problems identified was that of "missing" pipes or connections. In several cases it was found that connections from smaller pipes (200 mm or less) had not been made to the bulk mains running through a particular area. In one case, it was found that of the 4 connections shown on the "as-built" plans, only one had in fact been commissioned. The remaining three connections were sealed with a blank flange plate just before the connection point. In this instance, the community of approximately 3 000 residents had been experiencing intermittent supply (water available only during the night-time periods) for almost 9 years and had stopped complaining many years ago since nothing was ever done to alleviate the problem. On excavating the three mystery connections and adding the necessary T-pieces (see Figure 5), the area was restored to full system pressure on a 24-hour basis for the first time.

Although the additional connections actually increased the water use in the one problem area, it allowed the pressure to the whole of Sebokeng and Evaton to be lowered during the off-peak periods which more than made up for any small local increase in use during the remaining periods.

Figure 5: Installation of missing "T-Piece "in Sebokeng

The other key problem identified with the infrastructure was the identification of "missing" valves which were not shown on any reticulation drawings but were thought to exist by the project team due to the manner in which the system was responding to the water pressure.

Figure 6: Identification of missing valve and chamber after cleaning

Figure 7: Removal of valve body

As can be seen in **Figure 6** and **Figure 7** the valve chamber had been buried with rubble and rubbish for approximately 30 years and when the valve was eventually unearthed, it had seized completely. In many cases, the valves are more than 60 years old and must be removed completely and refurbished in order to restore the reticulation system to its normal operating condition. This type of problem has been a common occurrence and it is anticipated that more than 100 large valves similar to that shown in **Figure 7** will have to be refurbished. It is difficult to replace such valves with new valves due to the different flange drilling patterns and dimensions since all new valves use the metric specifications while those in the system are all based on the older imperial system. Refurbishing the valves results in very significant cost savings compared to a normal replacement policy.

Identification of Meter Errors

The bulk water supplier responsible for the water supply to Metsi-a-Lekoa is one of the few bulk water suppliers in South Africa that does not use check meters in many of its supply installations. The water supplied into Sebokeng and Evaton is measured by two 600mm diameter mechanical meters – one on each supply pipe. No problems regarding the quantity of water supplied had ever been raised until the new installation was commissioned. After the new installation had been commissioned, however, it was clear that there was a discrepancy between the bulk water meter readings and the meter readings in the installation. Unfortunately the magnitude of the discrepancy was around 4% in the favour of the bulk water provider. The acceptable accuracy range of the meters was in the order of 3% and thus it was technically possible to record a 6% difference and still be within the acceptable accuracy limits for both sets of meters.

Fortunately for the Municipality, one of the main meters operated by the bulk water provider started to malfunction in March 2006 which was immediately picked up by the project team since the difference between the two sets of readings changed from 4% in the favour of the bulk supplier to just over 2%. In the following month the difference changed significantly to over 12% in favour of the Municipality. This situation continued for a few months as shown in **Figure 8** before it was formally identified by the bulk supplier and the meters were changed. It was found that one of the two bulk meters had failed and the other was found to be approximately 5% over-recording. Following the replacement of the two bulk meters, the difference between the two sets of meters was negligible for a period of approximately 6 months after which the differences have again started to creep up in favour of the bulk supplier.

This is quite an interesting example and one that has significant financial implications to both the bulk water supplier as well as the Municipality. The lost revenue during the period in which the bulk meter was malfunctioning was in the order of R3 million (±$0.5) and obviously the bulk water provider wanted to recover this from the Municipality. The other side of the argument is that the remaining functioning meter was found to be over-recording by over 5% which is outside the acceptable range of accuracy and the meter had been in operation for several years. If it is accepted that such mechanical meters do not speed up as they get older, then it can safely be assumed that the bulk meters had been over-recording by at least 5% since they had been installed. A rough estimate of the financial implications for such over-recording put the total additional cost of water to the Municipality at approximately R15 million (±2.5 million).

The new installation is clearly a very valuable asset to the Municipality since it now provides some form of verification of the bulk water meters which had previously always accepted as accurate with no form of check. This is yet another unexpected benefit from the project.

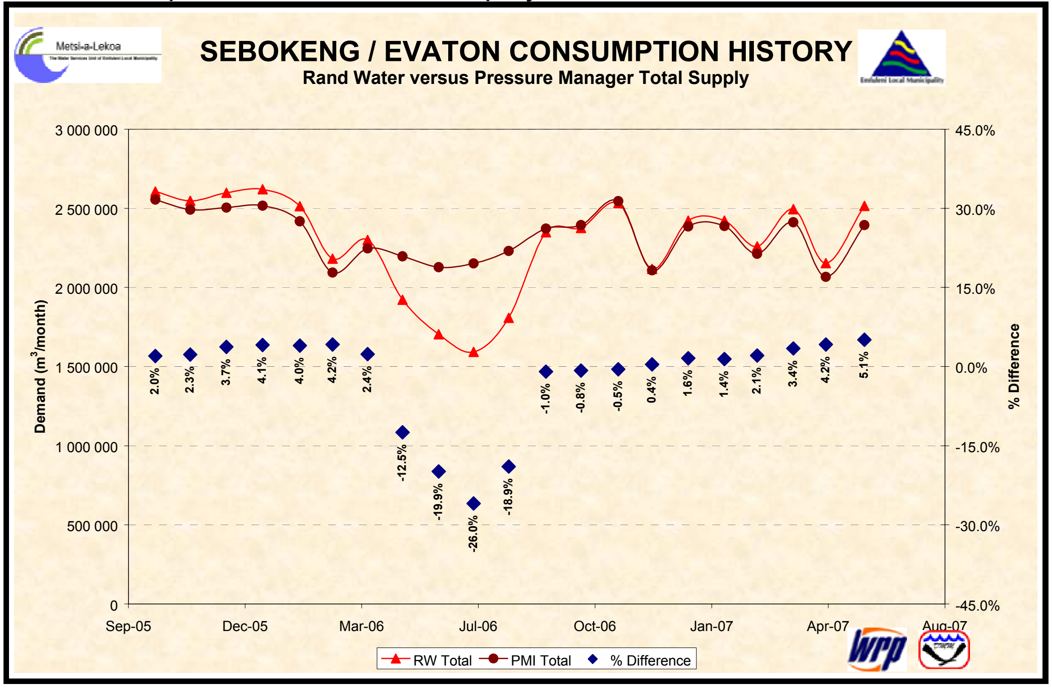

Figure 8: Meter errors between bulk supplier meters and Municipality meters

Catalyst for Funding

Prior to the implementation of the project, the Municipality was unable to access any funding for WDM activities of any nature and even the various "development" banks were unwilling to provide funding for the project. Once the project had been completed, however, and the results were published, the situation changed dramatically and suddenly there were several organisations (including the bulk water provider) wishing to invest funding into Sebokeng and Evaton. One of the main supporters of the project is now the Department of Water Affairs and Forestry (DWAF) which is the national custodian of all raw water in the country and also fulfils the role of regulator countrywide. After assessing the savings from the Sebokeng/Evaton pressure management initiatives, DWAF realised the value of such projects and created a new budget to help overcome the funding difficulties that originally threatened to halt the project. Approximately R50 million (± $8 million) has been allocated for 2007 and if successful, the budget will be increased in future to encourage WDM activities throughout South Africa. Of the R50 million (± $8 million) allocated to WDM activities by the Government, more than R10 million (± $1.7 million) has been allocated to support WDM activities in Sebokeng and the surrounding areas.

In addition to the injection of DWAF funding, the Municipality itself is now in the position that it has surplus funds for the first time as a result of the R30 million (± $5 Million) savings made during the first year of operation. Approximately R10 million (± $1.7 million) from the savings has been returned to the water utility to match the DWAF funding which brings the total funding available to upgrade the system for 2007 to more than R20 million (± $3.4 million). Prior to the project, the Municipality had virtually no budget for maintenance of the system since all funds were being used to support the water account from the bulk supplier.

Improved Municipality Status

Prior to the project, the only publicity received by the Municipality was usually with regard to spills of untreated sewage in the Vaal River. Such spills were due in part to the poor infrastructure of the multiple sewage pumping stations and in part to the huge sewage inflows which in turn were caused to a large extent by high internal household leakage. Since the project has been completed, it has created significant positive publicity for the Municipality and has picked up no fewer than 4 national awards for technical engineering excellence. The publicity surrounding the project has created awareness at the highest levels in government and the project has been acknowledged in Parliament by the Water Portfolio Committee as a model which should be repeated throughout South Africa wherever conditions permit.

The positive exposure from the project has also created a general atmosphere of success within the Municipality and the municipal managers who supported the project have also been able to promote their own personnel through various radio and television interviews on the project. In effect, the project has created a turning point within the Municipality and the general perception of the Municipality has changed from negative to positive.

Catalyst for Other WDM Interventions

Perhaps one of the most important benefits to arise from the project is the fact that it has demonstrated what can be achieved with relatively little funding and combined support from both the private and public sectors. Following the successful completion of the project, the Municipal managers have since been able to motivate for and gain approval for several additional technical and social WDM interventions. Of particular note are the following:

- Sectorisation to enable proper management of the reticulation system -
- Consumer metering and billing as a first step to proper billing;
- Community awareness with particular reference to garden watering;

- Pressure management at district level (<3000 properties) to gain further savings in low lying areas;
- Continuous monitoring of control points to assist with system management
- Development of an asset register as first step to full asset management system.

Sustainability of Savings

One of the key problems to many WDM interventions is the problem of maintaining the initial savings after the project has been completed and the project team has been paid for its efforts. In the case of the Sebokeng/Evaton PPP, the Project Team is responsible for all maintenance and operation for a period of at least 5 years. Since the Project Team receives payment in accordance with the savings generated (up to an agreed limit after which 100% returns to the Municipality) it is essential that the project continues to operate properly until such time that the Municipality takes over or extends the period of the contract. The percentage of the overall savings retained by the Project Team is approximately 15% based on the first two years of operation and is sufficient motivation for the team to ensure that the project is fully functional at all times. In this manner, the savings initially achieved are still being achieved two years after the project was completed. This is one of relatively few WDM projects where the savings are audited carefully on a continuous basis and this is considered one of the key elements of a successful WDM project.

Conclusions

While the Sebokeng and Evaton Public Private Partnership is clearly one of the most successful small scale PPP's to be completed in South Africa, the real benefits of the project are only now materialising two years after the project was commissioned. Both the Project Team and the Municipality are very happy with the outcome of the project and are continuing to work together to build on the initial success. While the financial savings generated exceed all initial expectations, the hidden and often less tangible benefits greatly outweigh the obvious and tangible benefits.

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Performance based Non-Revenue Water Reduction Contracts

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Introduction

One of the major challenges facing water utilities in the developing world is the high level of water losses either through physical losses (leakage) or theft of water from the system, or from water users not being properly billed. This difference between the amount of water put into the distribution system and the amount of water billed to consumers is known as “non-revenue water” (NRW).

The total cost to water utilities caused by NRW worldwide can be conservatively estimated at $15 billion/year. More than a third of that occurs in the developing world, where about 45 millions m3 are lost daily through water leakage in the distribution networks—enough to serve nearly two hundred million people. Similarly, close to 30 millions m3 are delivered every day to customers but are not invoiced because of factors like pilferage, employees’ corruption, and poor metering. These challenges seriously affect the financial viability of water utilities through lost revenues, lost water resources, and increased operational cost, thus reducing their capacity to fund necessary expansions of service, especially for the poor.

Box 1:
The three components of non-revenue water:

Physical (real) losses consist of leakage from the system and overflows at the utility’s storage tanks. They are caused by poor operations and maintenance, inadequate leakage control, and poor quality of underground assets.

Commercial (apparent) losses are caused by customer meter under-registration and data handling errors, as well as thefts of water in various forms.

Unbilled authorised consumption includes water used by the utility for operational purposes, water used for fire fighting, and water provided free to certain consumer groups.

A high NRW level normally indicates a poorly run water utility that lacks the governance, autonomy, accountability, and the technical and managerial skills necessary to provide reliable service. The private sector, through well-managed performance-based service contracting, can help water utilities with the technical and managerial skills to carry an effective NRW reduction programs.

The case for reducing NRW

Table 2: Estimates of worldwide NRW volumes (billions of cubic metres/year)

	Supplied population, millions (2002)	System Input l/capita/day	ESTIMATES OF NRW					
			NRW as share of system input (%)	Ratio (%)		Volume, billion m^3/year		
				Physical losses	Commercial losses	Physical losses	Commercial losses	Total NRW
Developed countries	744.8	300	15	80	20	9.8	2.4	12.2
Eurasia (CIS)	178.0	500	30	70	30	6.8	2.9	9.7
Developing countries	837.2[2]	250	35	60	40	16.1	10.6	26.7
				TOTAL		**32.7**	**15.9**	**48.6**

Source: World Health Organisation, IB-Net, and authors' estimates.

Research by international institutions is helping us understand the true magnitude of the losses from NRW, since utilities responsible for the losses have proven either unwilling or unable to provide such information. The World Bank database on water utility performance, known as IB-Net (www.ib-net.org), includes data from more than 900 utilities in 44 developing countries. The average figure for NRW level in developing countries utilities covered by IB-Net is around 35 percent (table 1) - representing a value of $5.8 billion (table 2).

Table 3: Estimated value of NRW and its components

	Marginal cost of water (US$/m³)	Average tariff (US$/m³)	Cost of physical losses	Lost revenue due to commercial losses	Total cost of NRW
			Estimated value (US$ billions/year)		
Developed countries	0.30	1.00	2.9	2.4	5.3
Eurasia (CIS)	0.30	0.50	2.0	1.5	3.5
Developing countries	0.20	0.25	3.2	2.6	5.8
		TOTAL	**8.1**	**6.5**	**14.6**

Source: Authors' calculations.

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[2] Based on a total population having access to safe water supply of 1,902.7 million people, with 44 percent of these receiving water through individual household connections.

What are the sources and costs of NRW? The principal components are leaks and unbilled consumption.

Water leaks. Every year 33 billion cubic metres of treated water physically leaks from urban water supply systems around the world, while 16 billion cubic metres are delivered to customers for zero revenue. Half of these losses are in developing countries, where public utilities are starving for additional revenues to finance expansion of services, and where most connected customers suffer from intermittent supply and poor water quality. It is estimated that US$15 billion is lost every year by water utilities around the world, more than a third of that by water utilities in developing countries. The scale of the problem is obvious and cannot be ignored.

Commercial losses. The value of water lost every year in developing countries through commercial losses—water actually delivered but not invoiced—is estimated at US$ 2.6 billion. This is about a quarter of the total yearly investment in potable water infrastructure in the developing world. It is also more than the World Bank, the biggest water financier among international financial institutions, lends every year in aggregate for water projects in developing countries.

Box 2:
Clear benefits from reducing NRW

Reducing NRW to just half the current level in the developing world would deliver the following benefits:

- Every year 8 billion m3 of treated water would be available to service customers.
- 90 million people more could gain access to water supply, without increasing demand on endangered water resources.
- Water utilities would gain access to an additional US$2.9 billion in self-generated cash flow, equivalent to more than a quarter of the amount currently being invested in water infrastructure in the developing world, and this without affecting in any manner the debt capacity of those countries.
- Fairness among users would be promoted by acting against illegal connections and those who engage in corrupt meters reading practices.
- Consumers would have improved service from more efficient and sustainable utilities.
- New business opportunities would be created for NRW reduction activities, with thousands of jobs created to support labor-intensive leakage reduction activities.

Although more analysis is needed, it is already clear that a sizeable portion of this commercial loss is likely to come from fraudulent activities and corruption—such as illegal connections, fraudulent meter reading, or meter tampering. This should be cause of concern for both developing countries' governments and the donor community alike. The benefits of reducing NRW are clear (see box 2).

Why utilities struggle with NRW—and how the private sector can help

NRW reduction is not a simple matter to implement, which explains why so many water utilities fail to address it effectively. New technical approaches have to be adopted and effective arrangements established in the managerial and institutional environment—often requiring attention to some fundamental challenges in the utility.

Not understanding the magnitude, sources, and cost of NRW is one of the main reasons for insufficient NRW reduction efforts around the world. Only by quantifying NRW and its components, calculating appropriate performance indicators, and turning volumes of lost water into monetary values, can the NRW situation be properly understood and required action taken. Other issues concern the inherent weaknesses of water utilities in developing countries. Utilities in developing countries:

- Often operate under a weak governance and financial framework, with utility managers having to face multiple political and economic constraints.
- Must provide service to customers on a daily basis using deteriorated infrastructure.
- Often lack the proper incentives and the specialised management and technical expertise necessary to carry out an effective NRW program.
- Operate under an inadequate incentive framework.

Because the water utilities in developing countries typically lack the capacity, incentives or governance to put in place NRW reduction programs, they need external assistance.

Potential for private sector involvement

A potential source of assistance is the private sector, where involvement can take many forms, ranging from long term-public-private partnerships (PPP) to service contracts or subcontracting of certain tasks. Depending on the option chosen, the private sector can bring:

- New technology and the know-how to use it efficiently
- Better incentives for project performance
- Creative solutions for the design and implementation of the program
- Qualified human resources
- Flexibility for field work (e.g. night crews)
- Investment, under certain conditions.

The key message, one too often overlooked, is that NRW must not be considered in a vacuum, but within the broader context of utility reform. The design of any NRW program needs to take into account the incentives open to the managers and staff of the program, as well as the other parties involved. Any program should ensure, as far as possible, that incentives are properly aligned with the objective of developing an efficient and effective utility that meets the needs of its consumers.

The paper excerpted here deals with *performance-based service contracting* (PBSC), a relatively new and flexible approach to the NRW challenge. Under PBSC, a private firm is contracted to implement an NRW reduction program. It is paid for services delivered and provided incentives to meet contractually enforced operational performance measures. With the proper balance of government oversight and private sector initiative, PBSC can provide an enabling environment and incentives conducive to reducing NRW, with immediate operational and financial benefits. But it is not a substitute for carrying out the broader institutional reforms necessary to promote the sustainability of the sector.

In practice, the applicability of PBSC to an NRW reduction program depends on the level of risk that the private sector is willing or able to take. Although PBSC is a relatively new concept for the water sector in the developing world, it is increasingly contemplated in other sectors as a way to improve efficiency and accountability of contracts with private providers. This is the first full study of large NRW reduction performance-based service contracts, and it considers key issues in contract design, management practices, outsourcing options, technical assistance, risk management, and other lessons learned.

Case studies: reducing lost water and increasing revenue

To date only a small number of large contracts have been let, and little information has yet become publicly available. However, the authors were able to study four significant and diverse projects. In Selangor, Malaysia, a large contract for reducing physical and commercial losses has been in place since 1998 between the water utility (at that time state owned) serving Kuala Lumpur and its surroundings, and a consortium led by a Malaysian company. In Thailand, the Metropolitan Waterworks Authority (MWA) that supplies Bangkok outsourced physical loss reduction to private contractors from 2000 to 2004. In Brazil, SABESP, the water utility that serves the São Paulo Metropolitan Region, experimented with different contractual approaches with the private sector for reducing commercial losses. And in Ireland, the Water Division of the Dublin City Council contracted in 1997 an international private operator to implement a two-year contract for reducing physical losses.

The following six key factors were used to evaluate these contracts:

- **Scoping.** What is the role of the private contractor? What are the NRW reduction targets?
- **Incentives.** How is the performance-based element of the contract structured?
- **Flexibility.** To what extent does the contract allow the private sector to be creative in the design and implementation of the NRW reduction activities?
- **Performance indicators and measurement.** How is NRW reduction measured?
- **Procurement/selection.** How was the private contractor selected?
- **Sustainability.** What happened after the performance-based service contract was completed? Does the contract include specific clauses to ensure transfer of know-how to the public utility?

Selangor, Malaysia: the largest NRW reduction contract to date

In 1997 the population of the Malaysian State of Selangor (and the Federal Territory of Kuala Lumpur) experienced a serious water crisis caused by the El Niño weather phenomenon. The crisis provided the trigger for the government to start dealing with the high level of NRW that had affected the water utility for many years. An estimated 40 percent of the water produced was not invoiced, with leakage estimated at 25 percent, or around half a million m^3/day. Halving the amount of physical losses would provide sufficient water to serve the equivalent of 1.5 million people and thereby avert the water shortage in Kuala Lumpur.

Faced with this crisis, the State Waterworks Department accepted an unsolicited proposal from a consortium led by a local firm, in joint venture with an international operator. The contractor committed to reduce NRW by a specified amount agreed in advance, in a given time. The contractor had full responsibility for designing and implementing the NRW reduction activities with its own staff, in exchange for an agreed lump-sum payment.

The incentives for achieving the targets included (i) penalties for non-compliance of up to 5 percent of the total lump sum, and (ii) a performance guarantee of 10 percent of the contract value. The lump sum included all necessary activities like establishment of district metered areas (DMAs), pressure management, leak detection and repair, identification of illegal connections and customer meter replacement as well as the supply of all equipment and materials, The contractor was free to select the zones within the network in which to conduct its NRW reduction activities.

Phase 1 of the contract demonstrated that the concept worked: a private firm can be contracted to efficiently reduce NRW level to specific targets, provided it has the flexibility to conduct the NRW activities and the payment arrangement covers all necessary work and materials. One of the technical innovations in this case was the universal use of pressure-reducing valves (even in very low-pressure situations) not only to reduce leakage through the reduction of excessive pressures but to also protect the already repaired DMAs from upstream pressure fluctuations.. The performance of Phase 1 actually exceeded the target (18,540 m^3/d), achieving savings of 20,898 m^3/d (approximately equally between commercial and physical losses). Twenty-nine DMAs were established with average savings of 400 cubic metres per day in each DMA and around 15,000 meters were replaced. The cost to the State Waterworks Department was equivalent to US$ 215 per m^3/d.

The Phase 2 contract had a number of shortcomings but was significant in its size—the contractor was committing to an ambitious target of around 200,000 m^3/d NRW reduction, something that had never been done under a PPP arrangement.

The long term sustainability of the project is not clear. The Phase 1 contract included training of the client's staff. Training on its own, however, proved insufficient for the client to maintain the improvements, and the Phase 1 zones were handed back to the contractor to operate in Phase 2. Obviously, any NRW strategy must address the issue of what to do once the contract ends.

Bangkok: plugging leaks

Water services in Bangkok are provided by a public utility, the Metropolitan Waterworks Authority (MWA). Like most water utilities operating in the megacities of Southeast Asia, MWA has been struggling for years to cope with demand from a fast-growing population. Major investments were made to increase production capacity, with production raised from 1.7 to 3 millions m^3/day between 1980 and 1990. It seemed that NRW was also reduced from 50 percent in 1980 down to about 30 percent in 1990. However, the reduction in percentage terms was mainly the result of the substantial increase in production capacity. Despite significant efforts, the volume of NRW remained stable during this period, at a high of about 900,000 m^3/day.

During the 1990s, as the system's supply swelled from 3.0 to 4.5 million m^3/day, NRW rose dramatically, both in percentage and in volumetric terms, reaching a peak in 1997 (1.9 million m^3/day. or 42 percent), presumably caused by supply improvements and pressure increases. System input was then again reduced to below 4 million m^3/day, and NRW consequently decreased and stabilised in 1999, albeit at a rather high level of 1.5 million m^3/day.

Subsequent efforts have resulted in NRW reduction by 200,000 m^3/day (to 1.3 million m^3/day, or 30 percent) even as the system input increased to 4.2 million m^3/day. A significant part of the reduction in NRW can be traced to performance contracts, which the MWA decided to award to private contractors in 2000. The objectives of these contracts were to reduce physical losses in three of the fourteen service branches of Bangkok (each representing around 100,000 customers). The duration of the contracts was four years. They were competitively bid, but only two companies were prequalified and submitted proposals. Both received contracts.

The design of these contracts was significantly different from the case of Selangor. There was no fixed target for leakage reduction, and payment was based in part on the actual water savings achieved through leakage reduction. While each contractor was free to carry out leakage reduction activities (such as detection, pipe repairs, main replacement, installation of hydraulic equipment) as they saw fit, this was done through the use of local firms based on reimbursables (on a cost-plus basis). Instead of a lump-sum payment, as used in Selangor, the remuneration of the contractor comprised three elements: (i) a performance-based management fee to cover overhead, profits, and foreign specialist staff, (ii) a fixed fee covering essentially the cost of local labour, and (iii), the largest part of the project's cost, reimbursables for all outsourced services, work, and materials performed in the field.

In terms of technical performance, the contracts can be considered a success – but the cost efficiency of the three contracts varied widely (between US$ 246 – 518 per m^3/d water loss reduction). Physical losses in these three areas were reduced by 165,000 m^3/day. To give some sense of perspective, the amount of water saved is equivalent to the volume needed to serve an additional half-million inhabitants.

It is interesting to compare the three Bangkok contracts to the Selangor contract:

Advantages compared with Selangor. There were neither arbitrary targets nor lump-sum remuneration, but instead a true performance-based element, based on the actual volume of NRW saved. In addition, the fact that two different contractors were in place simultaneously allowed for some useful benchmarking.

Disadvantages compared with Selangor. The high proportion of reimbursables transfers a substantial amount of risk from the private to the public partner. Basic activities, such as leak detection, should have been included in the performance fee.

In terms of sustainability, it does not seem that the contractors put proper control and management systems in place, which the MWA staff could then continue to use. However, MWA is aware of the problem and has recently tendered a project for advanced network monitoring, DMA establishment, and so on.

Sao Paulo: payments and collections

SABESP, the utility that serves the São Paulo Metropolitan Region, is one of the largest public water utilities in the world (supplied population: 25 million). It has put in place a proactive approach to water loss reduction with the help of the local private sector. Leakage reduction is routinely carried out by a series of leak detection contractors that are paid per length of distribution network surveyed. Some 40 percent of the 26,000 km network is surveyed every year.

However, customer metering and billing, including identifying and replacing under-registering meters, had been traditionally left to in-house crews. In 2004, it was estimated that SABESP was incurring daily revenue losses equivalent to one million cubic metres per day. Faced with this situation, SABESP decided to experiment with performance-based arrangements with the private sector. One of the contracts discussed below dealt with the reduction of bad debts (which is not, strictly speaking, part of NRW but has a similar negative impact on the utility's financial equilibrium); the other focused on meter replacement.

The concept of the first contract was simply to contract local private firms to negotiate unpaid invoices and collect the agreed amount. The scope of the contracts was limited to domestic and commercial customers; SABESP still dealt directly with public institutions. Several contracts were awarded covering all of SABESP's branches. The initial contracts started in 1999 for a two-year term. The contractors were remunerated by retaining a percentage of the debt collected. That percentage was bid on by the contractors; the winning bid in each branch offered the lowest percentage figure.

The São Paulo Metropolitan Region is the industrial heartland of Brazil and industrial and large commercial customers and large condominium buildings account for a major portion of SABESP's revenues. In fact, 28 percent of total billed metered consumption and 34 percent of all revenues come from just 2 percent of SABESP customers. Because meters were suspected of under-registering true levels of consumption, the strategy of the second contract was to upgrade and optimise the metering system.

SABESP came up with an innovative solution to this problem by tendering a series of turnkey contracts for meter replacement. The project target was to replace the meters of 27,000 large accounts identified by SABESP. Five 36-month contracts were put in place, and the contractor was responsible for the analysis, engineering and design, supply and installation of the new meters. There was no upfront payment, and the contractor had to pre-finance the entire investment. The contractor was entitled to a payment based on the average increase in billed volume, through a complex formula.

The concept of performance payments, rather than just paying for supply and installation, was chosen because resizing and flow profiling of the meters were the most critical activities in the contract. Given the high daily consumption of the large customers concerned, proper calibration could significantly increase metered flows and billing. By linking payments to the improved billed volumes, SABESP ensured that the contractor would focus on these critical issues.

The results of the contract were remarkable. The total volume of metered consumption increased by some 45 million m^3 over the contract's three years, while revenues increased by US$72 million. Of this, US$18 million was paid to the contractors, with a net benefit to SABESP of US$54 million.

In terms of sustainability, the contracts for reduction of bad debt have now become standard practice for SABESP. With new, properly sized, commercial customer meters installed it should now be easy for SABESP to maintain the accuracy of these meters and thus maintain the higher billed volumes from this customer category..

Dublin: upgrading a very old system

In January 1994 the City of Dublin had to deal with a severe water shortage caused by decades of underinvestment in the distribution network, combined with the absence of systematic leakage control, which had allowed physical water losses to reach very high levels. Several areas of Dublin had only intermittent water supply.

The first reaction was to ask for funds to build new treatment plants and expand existing ones. However funding was not made available because of the high level of leakage. A comprehensive study then identified, for the first time, the volume of water being lost: every day some 175 million litres of water, more than 40 percent of the existing treatment capacity, was estimated to be leaking away from the distribution network. The European Commission was approached, and the request for co-financing of the planned Dublin Region Water Conservation Project was approved, with a focus on reducing physical water losses.

The project target was very ambitious: to reduce leakage over a two-year period from 40 percent to 20 percent (in volumetric terms from 175,000 to 87,000 m^3/day). Given the aggressive nature of the reduction program, there was no alternative but to engage an experienced contractor to assist the city.

In November 1996 eight consortia were invited to submit bids. The contract was of limited duration—only two years—and focused on physical loss reduction. The contractor was responsible for establishing DMAs throughout the network, locating and repairing leaks, installing pressure-reducing valves, rehabilitating parts of the network, and training the Dublin water utility staff. It was designed essentially as a target cost contract expressed in monetary terms. It included a bonus-and-penalties mechanism to provide some incentive for performance, based on a complex methodology combining actual project expenses with the marginal cost of physical losses.

The contract was won by a UK water utility on a quality/cost basis. Significant details were left to be resolved during contract negotiations. The contractor’s remuneration in the winning bid covered a management fee, technical labour, and all leak-detection equipment. This did not include the cost of leak repairs, repair materials, or network rehabilitation, which were carried out through local subcontractors and covered separately as reimbursables under what were known as “compensation events.” The contract's accomplishments were significant:

Establishment of a total of 500 small DMAs (less than 1,000 connections each), covering the whole distribution network. Some 15,000 leaks were repaired and about 20 km of mains replaced. Total leakage was reduced from 175,000 m^3/day to about 125,000 m^3/day, and although the 20 percent leakage target was not achieved, the project was considered a success. (There was broad consensus that the original 20 percent target was not realistic given the short duration of the contract). The savings made were sufficient to end the water crisis.

In terms of sustainability, training and capacity building were components of the contract and were taken seriously by both parties. Substantial transfer of technology took place in practice, and the Dublin water utility now controls leakage as a regular part of its day-to-day operations.

Lessons learned

It is not feasible to eliminate all NRW in a water utility, but reducing by half the current level of losses in developing countries is a realistic target. Figures of such magnitude, even though based on estimates, should be enough to capture the attention of donors and developing country governments. In practice, good paybacks are possible with well-designed NRW reduction programs; therefore, if nothing else, NRW reduction makes business sense, although each opportunity has to be assessed in terms of its particular cost-benefit ratio.

Performance-based service contracts appear a viable way of reducing NRW losses. However, successful project implementation requires two essential and related elements: preparing good contracts and setting realistic baselines.

The case studies show various levels of quality in contract preparation, baseline setting, and—as a consequence—project effectiveness. Contract design must be clear about what the utility expects from the contractor and how it envisions success. All NRW reduction contracts should include basic guidelines concerning risk transfer, an indicator for leakage, and provisions for effective oversight by utility managers. Contracts should set viable targets and allow for flexibility in responding to challenges and opportunities.

To be successful, however, the study shows that good preparatory work is required. The starting point is to develop a strategy based on a sound baseline assessment of the sources and magnitudes of the NRW. Such a strategy needs to consider both the short and long terms (for example, the achievement of short-term reductions versus how to maintain lower levels of NRW over the long term). It is during strategy development that opportunities for teaming with the private sector can be identified. Once those opportunities are known, policy makers must create an incentive framework that will encourage the private sector to deliver reductions in the most cost-effective manner, allocating risk appropriately between the parties.

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Understanding the components of your Infrastructure Leakage Index (ILI) is necessary to develop a successful strategy to reduce the overall ILI value – especially in systems with a low ILI

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Keywords: Infrastructure Leakage Index (ILI); Component Analysis; Target Setting

Abstract

The Infrastructure Leakage Index (ILI) is gaining widespread acceptance as a benchmarking indicator for leakage management initiatives and is being used in many countries around the world. The calculation of the ILI requires a robust water audit, using standard definitions as set out by the International Water Association. However, once the ILI was calculated the next step for a water utility is to ask “how low can we go”. This paper discusses the various components of the UARL/ILI and their impact on the overall ILI.

The Concept of the ILI

The ILI is a dimensionless performance indicator, calculated by dividing the volume of Current Annual of Real Losses (CARL) by the volume of Unavoidable Annual Real Losses (UARL). The volume of CARL is obtained through the results of a standardized water balance using IWA terminology and the volume of UARL is determined by applying individual parameter values for three components of losses on three components of the water supply system (see Table 4).

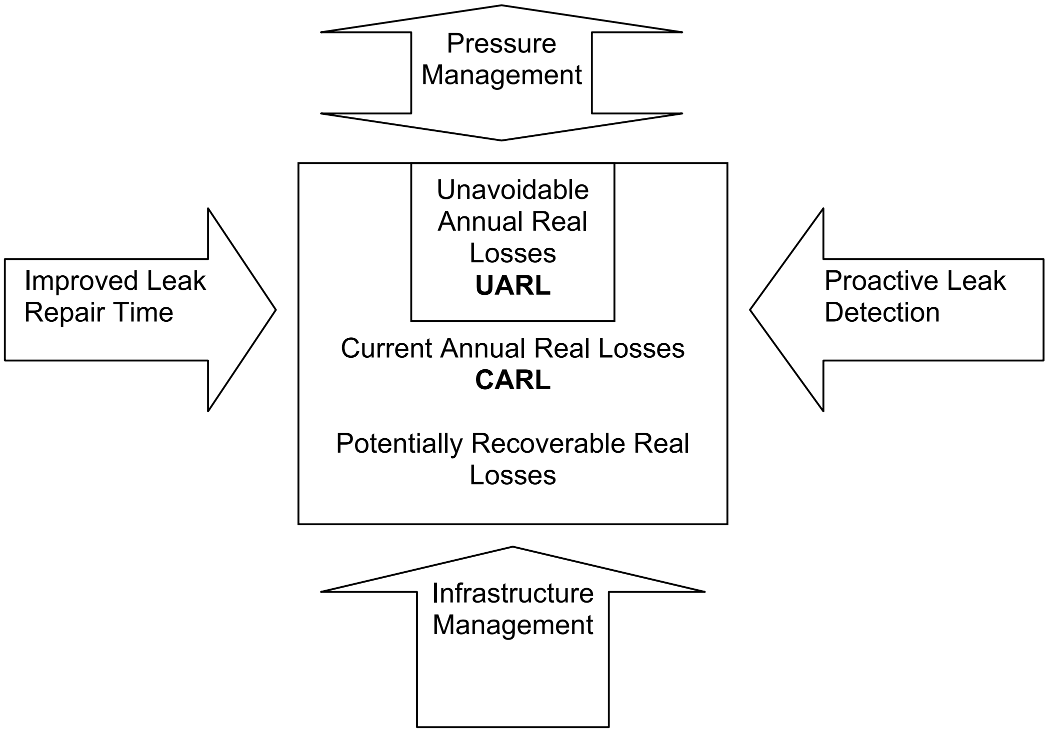
ILI = CARL / UARL

It is a well known fact among leakage practitioners that real losses cannot be eliminated completely, and even in newly commissioned distribution networks, there is a minimum level of leakage. The volume of UARL - the lowest technically achievable annual volume of real losses for a well-maintained and well-managed system at the current pressure – is represented by the smaller inner rectangle in Figure 9. The larger rectangle represents the volume of CARL and the ratio between the large rectangle and the small rectangle represents the ILI.

Equations for calculating UARL for individual systems were developed and tested by the IWA Water Losses Task Force (Lambert et al. 1999), allowing for:

- **Background leakage** – small leaks with flow rates too low for sonic detection if non-visible. They comprise of small pinholes, leaking joints, etc. As they are “undetectable”, their presence, it can be argued, is not as a result of the lack of leak detection activity. They are a function of the type of infrastructure, ground conditions and age of system.

- **Reported leaks and breaks** – based on frequencies, typical flow rates, target average durations. Reported breaks and leaks are those events reported by customers and the general public because they have a nuisance value. They are typically catastrophic failures that have high flow rates but short durations. These events will arise whether leak detection is carried out or not.
- **Unreported leaks and breaks** – based on frequencies, typical flow rates, target average durations. They are those leaks that can be detected but are too small or in a location that does not cause nuisance to the customers and general public. They can run undetected for long periods of time and their duration is a function of the method and intensity of leak detection activities. Increasing leak detection activity will reduce the overall duration of unreported bursts and so reduce the annual volume of water that is lost through these types of leaks.
- **Leakage-pressure** relationships (a linear relationship being assumed for most large systems).



Source: Adapted from IWA Water Loss task Force

Figure 9 The UARL and the four basic methods of managing real losses

UARL is determined by applying individual parameter values for three components of losses on three components of the water supply system. The nine component values, as published by IWA are as presented in Table 4.

Table 4 Standard Unit Values Used for the Calculation of Unavoidable Annual Real Losses, UARL

Infrastructure Component	Background Leakage	Reported Bursts	Unreported Bursts	UARL total	Units
Mains	9.6	5.8	2.6	18	litres/km of mains/day/metre of pressure
Service connections main to curb stop	0.6	0.04	0.016	0.80	litres/service connection/day/metre of pressure
Service connections curb stop to meter	16	1.9	7.1	25	litres/km of service connection/day/metre of pressure

Source: Adapted from IWA/Aqua 48

It should be noted that no allowance is made for any losses from service reservoirs, break-pressure tanks or other forms of water storage within the water supply system. ILI is strictly limited to pipes only.

The equation used for calculating UARL is based on clearly stated auditable assumptions for background losses, for frequencies and durations of the different types of leaks, and their typical flow rates related to pressure. The UARL equation requires data on four key system-specific factors:

- Length of mains (Lm)
- Number of service connections (Nc)
- Location of customer meter on service connection (relative to property line, or curb-stop in North America) (Lp)
- Average operating pressure when system is pressurized (P)

The UARL equation as shown below assumes a linear leakage-pressure relationship and this assumption works best for large systems with mixed metal and non-metal pipe work.

UARL = (18xLm + 0.8xNc + 25xLp) x P

Use of ILI for Target Setting

The conventional approach to using ILI is to consider ILI as a single value for a system and to compare this against the ILI values for others systems or against a set of guidelines. Well managed systems are expected to have low values of ILI – close to 1.0 – while systems with some deficiencies in infrastructure management will have higher values.

An example of guidelines that are being developed using ILI are those prepared by the American Water Works Association (AWWA) Water Loss Control Committee and presented in their committee report “Applying worldwide BMPs in water loss control” (AWWA Water Loss Control Committee, 2003) (see Table 5).

This approach works well and provides the user with a clearly defined performance indicator that can be used as a general comparator for decision making.

Like all performance indicators, however, the ILI does have some limitations. The most notable limitation is in relation to one of the most effective leakage management strategies, namely pressure management. The linear leakage-pressure relationship used in this equation is an important feature that is often overlooked. The inclusion of the leakage-pressure factor is designed to take into account the fact that systems operated at higher pressure will intrinsically have higher leakage rates than those operated at lower leakage levels. Without the inclusion of pressure as a factor, systems with higher pressure would tend to have higher ILI than those with lower pressure. However, the corollary of this is that changing the operating pressure in any given supply system will affect both the UARL and CARL equally, and therefore the ILI, being the ratio of UARL to CARL, will remain unaffected. In other words, the assumption is that pressure management will not impact on ILI. If ILI is used exclusively as the target indicator for leakage management, then there is little point to look towards pressure management as a tool for achieving a lower ILI.

Table 5 AWWA Guidelines for setting a target ILI

Target ILI Range	**Water Resources Considerations**	**Operational Considerations**	**Financial Considerations**
1.0 – 3.0	Available resources are greatly limited and are very difficult and/or environmentally unsound to develop.	Operating with system leakage above this level would require expansion of existing infrastructure and/or additional water resources to meet the demand.	Water Resources are costly to develop or purchase; ability to increase revenues via water rates is greatly limited because of regulation or low ratepayer affordability.
3.0 – 5.0	Water resources are believed to be sufficient to meet long-term needs, but demand management interventions (leakage management, water conservation) are included in the long-term planning.	Existing water supply infrastructure capability is sufficient to meet long-demand as long as reasonable leakage management controls are in place.	Water resources can be developed or purchased at reasonable expense; periodic water rate increases can be feasibly imposed and are tolerated by the customer population.
5.0 – 8.0	Water resources are plentiful, reliable and easily extracted.	Superior reliability, capacity and integrity of the water supply infrastructure make it relatively immune to supply shortages.	Cost to purchase or obtain/treat water is low, as are rates charged to customers.
Greater than 8.0	Although operational and financial considerations may allow a long-term ILI greater than 8.0, such a level of leakage is not an effective utilization of water as a resource. Setting a target level greater than 8.0 – other than as an incremental goal to a smaller long-term target – is discouraged.		

Source: Adapted from AWWA Water Loss Control Committee

Component-based approach to using ILI

Having calculated the ILI, most utilities will ask the questions “how low can I go?” or “what steps should I take to reach the guideline target ILI ?”.

In answering these questions, it is important to step back and look at the various components that comprise both the UARL and CARL values. From Table 4 we saw that there are essentially nine components to the UARL value comprising three types of leakage (background leakage, reported bursts and unreported bursts) on three classes of infrastructure (mains, service connections from main to curb-stop and service connections from curb-stop to customer meter). The next step is then to consider what factors will affect each of these components and what actions can be taken to reduce the level of losses from each component. For the purposes of this paper, we have ignored pressure management as an option for reducing ILI on the basis of the reasons explained in the earlier section.

What Influences the ILI?

Background losses comprise the “undetectable” leaks; small pinholes, leaking joints etc. As they are “undetectable”, their presence, it can be argued, is not as a result of the lack of leak detection activity. They are a function of the type of infrastructure, ground conditions and age of the system. The primary tool for reducing background leakage, other than pressure management (which has already been shown to be of no assistance in reducing ILI), is infrastructure replacement.

Reported bursts are those leaks that are reported by customers and the general public because they have a high nuisance value. They are typically catastrophic failures that have high flow rates but short duration, such as a circumferential break on an iron main or total failure of a service connection. They may also be caused by third party damage such as contractors excavating for other utility companies. In most cases, reported bursts cause a loss of supply to one or more customers and can be characterised as ‘sudden and short-lived’ events. It can also be argued that many of these events also arise whether leak detection is carried out or not. For example, the amount of leak detection carried out by the water utility will have no affect on the amount of third-party damage caused by other utility companies and their contractors. Likewise, the number of catastrophic failures caused by pressure surge events will not be affected by the amount of leak detection that takes place.

Unreported bursts are those leaks that can be detected but are too small or in a location that does not cause nuisance to the customers and general public. They can run undetected for long periods of time and their duration is a function of the method and intensity of leak detection activities. Increasing leak detection activity will reduce the overall duration of unreported bursts and so reduce the annual volume of water that is lost through these types of leaks.

The frequency with which leaks occur on water mains and service connections is affected by many factors. These may include pipe material, pipe specifications, installation workmanship, operating pressures, ground conditions, soil types and so on.

A United Kingdom Water Industry Research (UKWIR) project (MacKellar and Pearson, 2003) collected mains failure data from water companies in United Kingdom. The data set is comprehensive and is a good source of comparators, especially given the UK Water Industry focus on optimizing leakage levels. The UK Water Industry average mains failure rates for various pipe materials for the periods 1998 to 2001 are presented in Table 6. This clearly shows that different pipe materials have differing failure rates. Table 7 also demonstrates that the failure rate is also affected by pipe size. Since any given network will have a unique combination of pipe materials and pipe sizes, it is understandable that the burst frequency will vary from network to network.

Table 6 UK water industry average mains failure rates for various pipe materials

Material	**Mains Failure Rate per 100 km per year**			
	1998	**1999**	**2000**	**2001**
Asbestos Cement	16.4	17.1	15.1	15.8
PE	3.5	2.9	3.3	3.1
PVC	9.6	9.1	7.2	7.4
Ductile Iron	5.0	5.3	4.8	4.8
Cast Iron	23.7	23.7	19.1	21.7

Source: constructed from data contained in the UKWIR National Database of mains Failures

Table 7 UK water industry average mains failure rates for 2001 for two different pipe materials

Nominal pipe diameter (mm)	Mains Failure Rate per 100km per year	
	UK Water Industry Average for 2001 for Asbestos Cement	UK Water Industry Average for 2001 for Ductile Iron
200	8.0	3.9
150	16.7	6.8
100	23.8	7.4
80	26.0	26.3

Source: constructed from data contained in the UKWIR National Database of mains Failures

The UARL values for mains bursts is based on an assumption of 13 bursts per 100 kilometres per year with 95% of bursts being reported (12.35 bursts/100km/year) and 5% being unreported (0.65 bursts/100km/year) (Lambert and McKenzie, 2002). If the network being considered is predominantly asbestos cement pipes of nominal diameter of 150mm or less, then it could be expected that the natural burst frequency will be higher than that assumed in the UARL calculation and that the network, all other things being equal, will naturally have an ILI higher than 1.0.

This demonstrates that is essential to look beyond the ILI as a single value when considering what range of ILI can be achieved for any given water utility. As an example we might consider a water utility with an overall ILI of 2.0. Is the amount of water being lost from each of the Real Loss components (background losses, reported bursts and unreported bursts) and on each of the infrastructure components twice the UARL volume for each component? Or is it possible that the CARL volume on the background losses component is equal to the UARL value (ILI=1.0) and some other multiplier on the other components? And what impact will this have on working out how low we can go?

Conversion of UARL values into standardised units

The first step in looking at ILI on a component basis is to convert the UARL into standardised units for all the nine components so that they can be readily compared against one another. It can be seen from Table 4 that the UARL for the three infrastructure components are in different units. To convert the UARL values into standardised units requires the connection density of the network (number of service connections per kilometre of main) and the assumed average length of the service connection from curb-stop to customer meter expressed in km.

The following example is for a water utility with 100 service connections per kilometre of main and an average of 4 metres of service connection length between curb-stop and customer meter. The UARL units for the example water utility are presented in Table 8 and have been converted into litres/service connection/day/metre of pressure using the following methodology:

For mains, UARL (litres/km of main/day/metre of pressure) divided by service connection density (service connections per km of main) equals UARL (litres/service connection/day/metre of pressure);

For service connections, curb-stop to meter, UARL (litres/km of service connection/day/metre of pressure divided by the assumed average length of the service connection from curb-stop to customer meter (km) equals UARL (litres/service connection/day/metre of pressure).

Table 8 Standardised UARL values for a specific water utility example

Infrastructure Component	Units	Background Leakage	Reported Bursts	Unreported Bursts	UARL total
Mains	litres/service connection/day/metre of pressure	0.096	0.058	0.026	0.180
Service connections main to curb stop	litres/service connection/day/metre of pressure	0.600	0.040	0.016	0.800
Service connections curb stop to meter	litres/service connection/day/metre of pressure	0.064	0.008	0.028	0.100
TOTAL	**litres/service connection/day/metre of pressure**	**0.760**	**0.106**	**0.214**	**1.080**

By standardising the parameter values to the same units for each component in this way, it is now very easy to compare the relative contribution that each component makes to the overall UARL total. In the example given it can be seen that background losses on services connections dominate the UARL value. The UARL for background losses on service connections is 0.664 litres per service connection per day per metre of pressure out of a total UARL value of 1.080 litres per service connection per day per metre of pressure – almost 70% of the UARL value is associated with the single component of background losses on service connections! It should be remembered that standardisation of the UARL components into similar units is system specific and that the example shown is for an urban situation with relatively high service connection density.

Applying the calculated ILI value to the UARL values

The next step is to consider how the single ILI value relates to each component of the UARL total. In our example utility, the CARL value has been calculated from an annual water balance and an ILI of 3.0 has been calculated. How should the ILI value be applied across all of the UARL components and what does this mean for possible leakage management strategies?

In Table 9, the ILI of 3.0 is assumed to be constant across all the UARL components. If a standard leakage management strategy of increasing the amount of active leakage control activity that is carried out is considered, such that the duration of unreported bursts is reduced, then it can be assumed that the losses from background leakage and reported bursts would be unaffected. Figure 10 presents the active leakage control cost curve that would result from the application of the selected leakage management strategy of reducing the duration of unreported bursts by increasing active leakage control activity for an ILI of 3.0 assuming this is uniformly applied across all UARL components. It can be seen from the cost control curve, that it would be necessary to increase the expenditure on active leakage control six-fold to reduce the overall ILI from 3.0 to 2.5.

In Table 10, the ILI of 3.0 is not assumed to be constant across all the UARL components. The utility keeps careful records of the number and type of bursts that take place and the length of time that is taken to locate and repair those bursts and the individual ILI values for each of the reported and unreported burst components have been calculated. The background leakage component is allocated an ILI of 1.0 and is inferred from the difference between the other components and the total overall ILI of 3.0.

Note that both examples have the same overall ILI but with quite markedly different CARL values for each of the nine components.

Table 9 Standardised UARL values for an ILI of 3.0 assuming uniform application (Example 1)

Infrastructure Component	**Units**	**Background Leakage**	**Reported Bursts**	**Unreported Bursts**	**Total**
Mains	UARL litres/service connection/day/metre of pressure	0.096	0.058	0.026	0.180
	Component ILI	**3.0**	**3.0**	**3.0**	**3.0**
	CARL litres/service connection/day/metre of pressure	0.288	0.174	0.078	0.540
Service connections main to curb stop	UARL litres/service connection/day/metre of pressure	0.600	0.040	0.016	0.800
	Component ILI	**3.0**	**3.0**	**3.0**	**3.0**
	CARL litres/service connection/day/metre of pressure	1.800	0.120	0.480	2.400
Service connections curb stop to meter	UARL litres/service connection/day/metre of pressure	0.064	0.008	0.028	0.100
	Component ILI	**3.0**	**3.0**	**3.0**	**3.0**
	CARL litres/service connection/day/metre of pressure	0.192	0.023	0.085	0.300
TOTAL	UARL litres/service connection/day/metre of pressure	0.760	0.106	0.214	1.080
	Component ILI	**3.0**	**3.0**	**3.0**	**3.0**
	CARL litres/service connection/day/metre of pressure	2.280	0.317	0.643	3.240

Table 10 Standardised UARL values for an ILI of 3.0 assuming non-uniform application (Example 2)

Infrastructure Component	**Units**	**Background Leakage**	**Reported Bursts**	**Unreported Bursts**	**Total**
Mains	UARL litres/service connection/day/metre of pressure	0.096	0.058	0.026	0.180
	Component ILI	**1.0**	**3.0**	**11.0**	**3.11**
	CARL litres/service connection/day/metre of pressure	0.096	0.174	0.286	0.559
Service connections main to curb stop	UARL litres/service connection/day/metre of pressure	0.600	0.040	0.016	0.800
	Component ILI	**1.0**	**3.0**	**11.0**	**3.12**
	CARL litres/service connection/day/metre of pressure	0.600	0.120	1.760	2.496
Service connections curb stop to meter	UARL litres/service connection/day/metre of pressure	0.064	0.008	0.028	0.100
	Component ILI	**1.0**	**3.0**	**3.5**	**1.86**

	CARL litres/service connection/day/metre of pressure	0.064	0.023	0.099	0.186
TOTAL	UARL litres/service connection/day/metre of pressure	0.760	0.106	0.214	1.080
	Component ILI	**1.0**	**3.0**	**10.1**	**3.0**
	CARL litres/service connection/day/metre of pressure	0.076	0.317	2.164	3.241

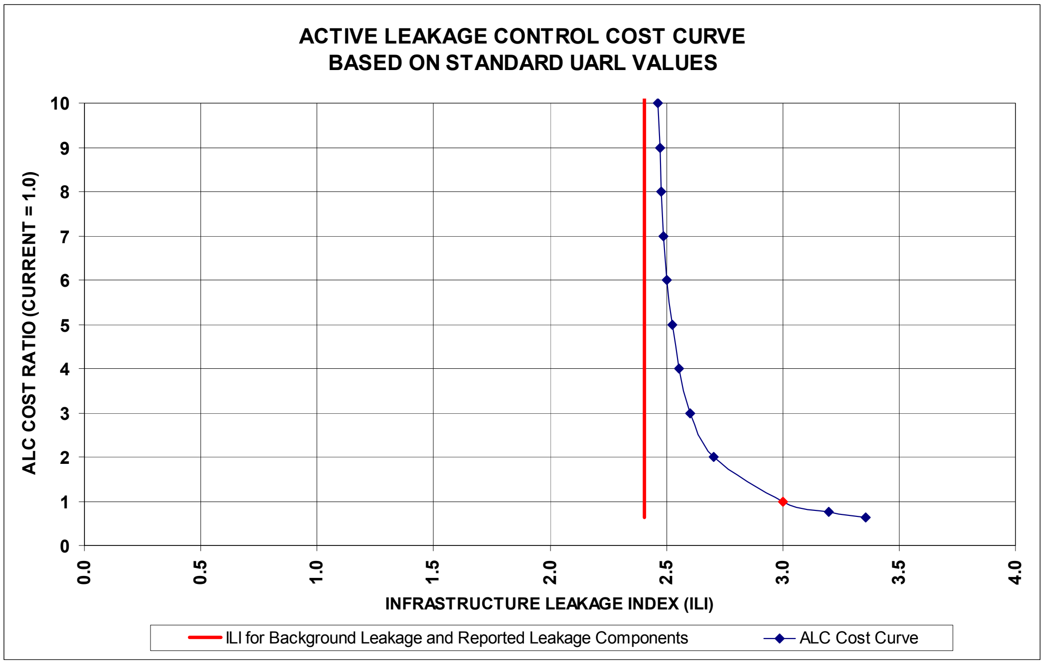


Figure 10 Active leakage control cost curve for example 1 with ILI = 3.0

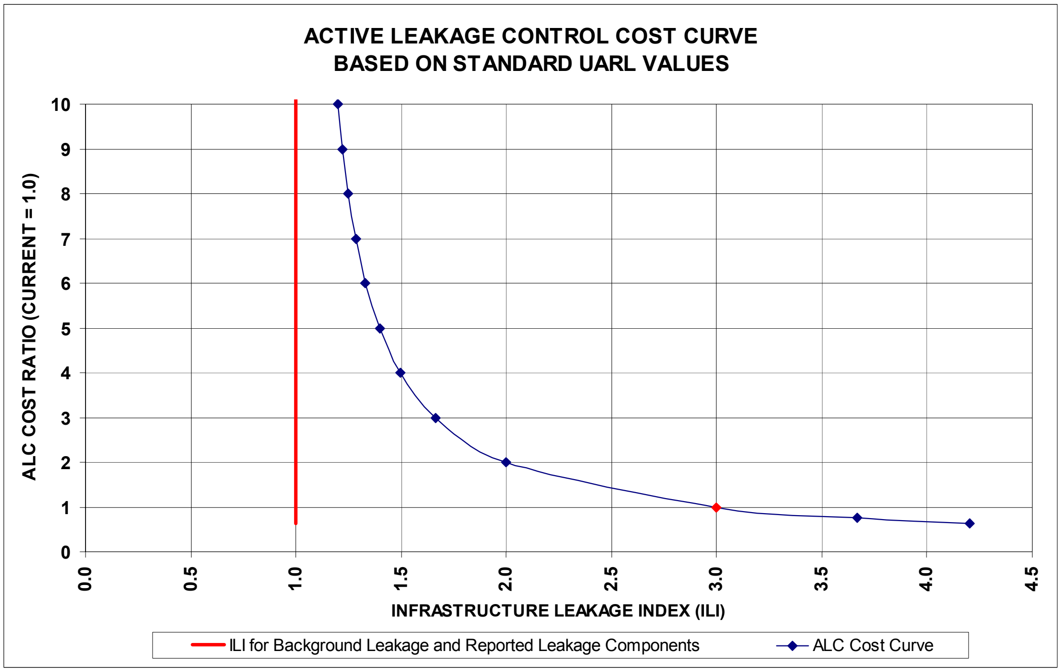


Figure 11 Active leakage control cost curve for example 2 with ILI = 3.0

Figure 3 presents the active leakage control cost curve that is developed for example 2 using the same leakage management strategy as used in example 1. The difference between example 1 and example 2 is immediately obvious. Although both examples have a starting ILI of 3.0, it is clear that in example 2, a much lower ILI can be achieved for any given increase in leakage management activity. For instance, in example 2, a six-fold increase in leakage management activity would result in an overall ILI of 1.3 compared to an overall ILI of 2.5 in example 1.

Alternatively, the utility in example 2 could achieve an ILI of 2.5 for a relatively modest 50% increase in active leakage control compared to the six-fold increase required by the utility in example 1.

Summary and Conclusions

The paper has discussed the various components that comprise the UARL and CARL values and how ILI, the ratio of CARL to UARL, is now being used as a target setting tool for water utilities. It has been shown that the main limitation of ILI as a target setting tool for leakage management is that pressure management, one of the most effective methods of leakage reduction, is known not to reduce ILI.

It has also been shown that for any given value of ILI, the CARL value may be made up of many different variations in the component ILI values for the nine stated components of real losses. This demonstrates that although ILI is a useful performance indicator, it is still necessary to develop a thorough understanding of the volume of real losses by component for each specific water utility so that the most appropriate leakage management strategy is selected. A water utility with an overall ILI of 3 and where the CARL values of each of the nine components are assumed to be 3 times the UARL values will have the majority of real losses arising from background leakage. In this case even very large increases in expenditure on active leakage control or repair duration will have very little impact on the overall ILI. By comparison, a water utility with an ILI of 3 and where the CARL values of each component are such that background leakage is 1 times the UARL and unreported bursts on mains and service connections in the street are 11 times the UARL value, will be able to achieve a much lower ILI with the same leakage management strategy and the same increase in expenditure.

It can therefore be concluded that water utilities with similar ILI do not necessarily have similar ratios of real losses across all of the nine identified components used in the UARL calculation. It also follows that water utilities with similar amounts of infrastructure may need to apply quite different leakage management strategies and spend quite differing sums of money to achieve the same overall ILI.

Only by carrying out the detailed component based analysis of real losses is it possible to determine whether any particular leakage management strategy will be effective in achieving a target level of ILI.

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Studies of reference values for the linear losses index in the case of rural water distribution systems

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Context of the studies

The Linear Losses Index (LLI) used in France

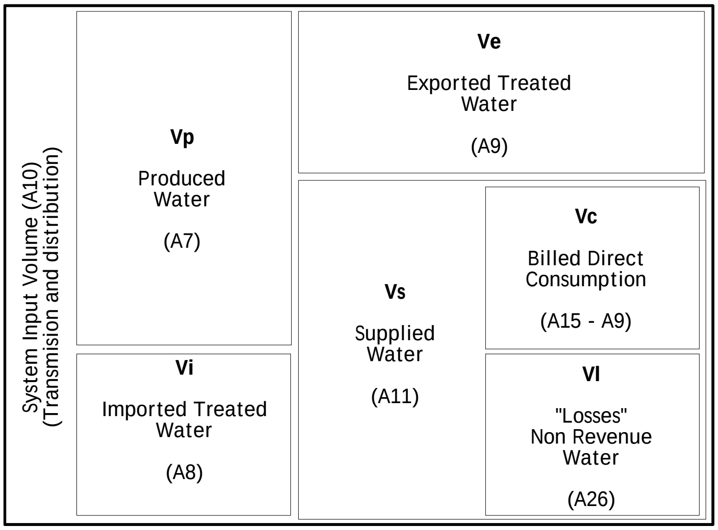
In France, the main indicator used to quantify the evolution of leakage levels in water distribution systems is the Linear Losses Index (LLI), which has been defined and recommended by governmental environment agencies (Dumont et al., 2005).

LLI = (Vs – Vc) / (365 x M) m^3/km/day

Where: Vs is the annual volume of supplied water; Vc the annual billed direct consumption and M the total transmission and distribution mains length.

Using standard terminology for Water Balance (Alegre et al., 2000) and considering that the system input volume is the transmission input : LLI = A26 / (365 x C3)

Table 1 Conformity with the components of Water Balance



Reference values for the Linear Losses Index

There are several sources of reference values of LLI, all of them take in consideration the rural or urban character of the network. However, two ways to define this character are proposed :

- For public agencies, the urban/rural character is defined according to the customers' density D = N/M customers per kilometre where N is the total number of registered customers (Agence de l'eau Adour Garonne, 2005) (Laboratoire GEA, 2006)

- For water companies the urban/rural character is defined according to the Linear Consumption Index LCI = Vc / (365 x M) m3/km/day.

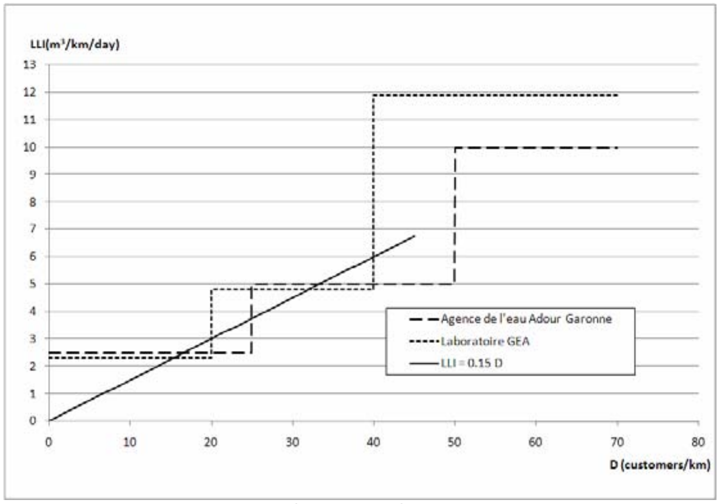


Figure 1 Public agencies references of acceptable LLI according to D

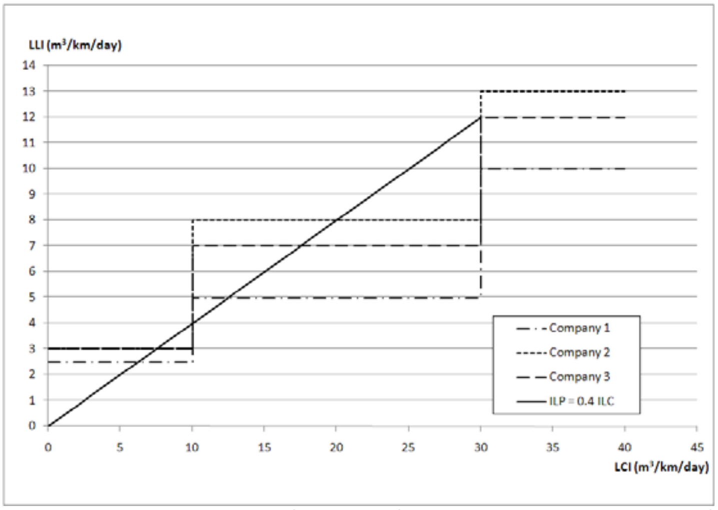


Figure 2 Water companies references of acceptable LLI according to LCI

Due to the variation in the references, the same water distribution system can be considered as urban in one reference system and intermediate in an other. A LLI value can also be considered acceptable in some reference systems and not acceptable in others.

In this context, the main aim of our work is to verify and compare the validity of the different reference systems using data gathered by decentralised services of the French Ministry of Agriculture. These data mainly concern rural water suppliers.

Collection of the data

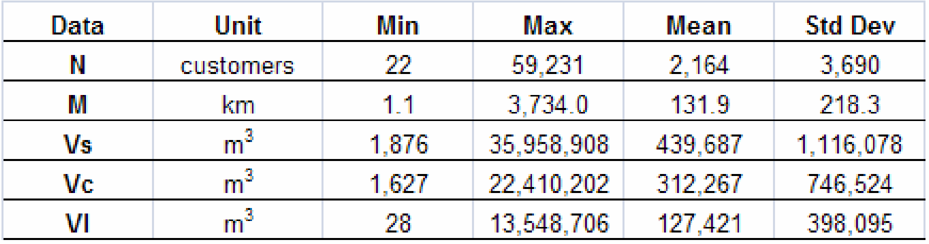
The French Ministry of Agriculture provides access to decentralised services for the management of public water distribution systems in rural areas. The data gathered by these organisations are validated by means of a purpose-built computer system. An investigation was carried out with all the directorates of the 101 French counties. 86 answers were received, 69 of them could be used with data from around 2 000 water suppliers.

The data covered different time periods with 2004 being the most recent year of available data. The main data collected were:

- Annual volume produced, imported and exported (Vp, Vi, Ve)
- Annual volume consumed (Vc)
- Number of consumers (N)
- Network lengths (M)

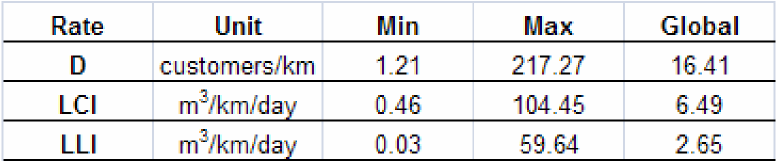
In total, 15 296 LLI values could be calculated. After cleaning (deletion of aberrant data), the database finally contained 14 987 rows.

Table 2 Simple statistics



Data	Unit	Min	Max	Mean	Std Dev
N	customers	22	59,231	2,164	3,690
M	km	1.1	3,734.0	131.9	218.3
Vs	m^3	1,876	35,958,908	439,687	1,116,078
Vc	m^3	1,627	22,410,202	312,267	746,524
Vl	m^3	28	13,548,706	127,421	398,095

Table 3 Rates



Rate	Unit	Min	Max	Global
D	customers/km	1.21	217.27	16.41
LCI	m^3/km/day	0.46	104.45	6.49
LLI	m^3/km/day	0.03	59.64	2.65

Extreme values shows that the data concern very different networks. However, the values of the means show that the majority of them are rural.

The repartition of density and LLI values confirm the predominance of rural networks :

- Density is fewer than 25 customers per kilometre for about 70 % of the values
- LCI is fewer than 10 m^3/km/day for about 75 % of the values

Studies of the links between data or indicators

Correlations between basic data

Table 4 Correlations between basic data

	Vl	M	N	Vs	Vc
Vl	1	0.45072	0.71425	0.95296	0.89145
		<.0001	<.0001	<.0001	<.0001
M	0.45072	1	0.7591	0.53679	0.56217
	<.0001		<.0001	<.0001	<.0001
N	0.71425	0.7591	1	0.82003	0.84508
	<.0001	<.0001		<.0001	<.0001
Vs	0.95296	0.53679	0.82003	1	0.98685
	<.0001	<.0001	<.0001		<.0001
Vc	0.89145	0.56217	0.84508	0.98685	1
	<.0001	<.0001	<.0001	<.0001	

All given data are correlated.

The stronger correlation with non-revenue water volume (Vl) is the one with supplied water volume(Vs).

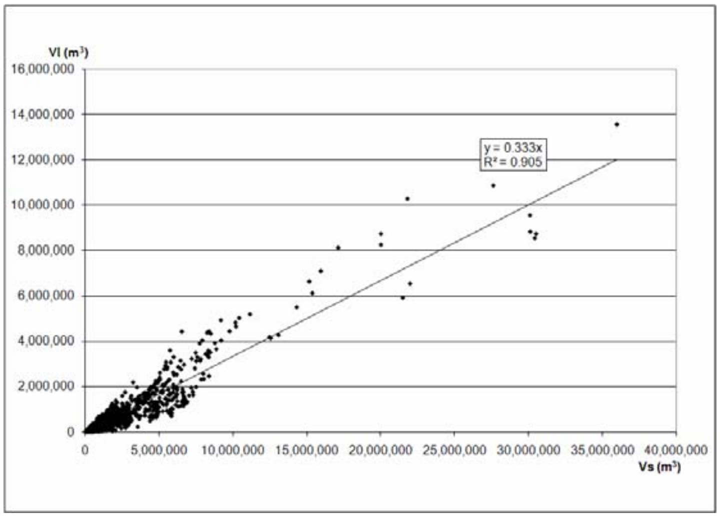


Figure 3 Non revenue water as a function of supplied water

The linear regression without intercept shows that non-revenue water correspond to about 1/3 of supplied water. This result is close to the Portuguese value, 34.9 % (Cunha Marques & Monteiro 2003).

In fact, despite a lot of studies, results have shown that non-revenue water volume as percentage of system input volume is an unsuitable performance indicator for assessing the efficiency of management of distribution systems (Lambert & Hirner 2000; Carpenter et al 2003). Non-revenue water volume as percentage of supplied water volume can be a useful performance indicator to identify networks, justifying complementary investigations.

Analysis by classes

Data are many but dispersed, so we analyse data by classes of density or linear consumption index.

Data are grouped by class of density, (interval of 5 customers/km) or by class of LCI, (interval 5 m^3/km/day).

Because of the lack of values, densities above 45 customers/km and LCI above 30 m^3/km/day are not taken into account.

For each group, the values of the indicators are not the mean of the values but the value corresponding to a theoretical large network, resulting from merging all the networks of the group :

$$D = \frac{\sum_{i=1}^{n} N_i}{\sum_{i=1}^{n} M_i} \qquad LCI = \frac{\sum_{i=1}^{n} Vc_i}{365 \times \sum_{i=1}^{n} M_i} \qquad LLI = \frac{\sum_{i=1}^{n} Vl_i}{365 \times \sum_{i=1}^{n} M_i}$$

Link between Density of costumers and Linear Consumption Index

The urban/rural character of a network is determined as a function of density by public agencies and as a function of LCI by water companies. To verify the coherence of these two ways we consider LCI as a function of D.

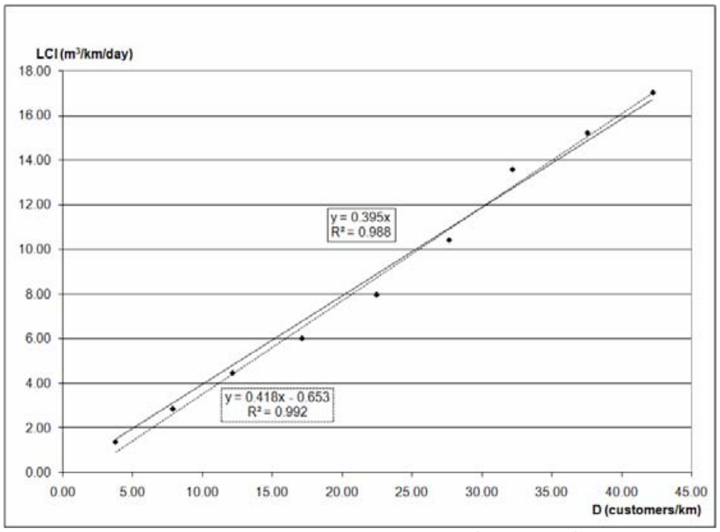


Figure 4 Linear consumption index as a function of density

The linear regression shows a strong link between these two indicators. Without intercept (There is no consumption without customers), the function is : *LCI = 0.395 x D*

This relationship allows us to compare the different points of view of urban/rural character according to the same scale.

Table 5 Urban/rural character

D (cutomers/km)	Laboratoire GEA	Agence de l'eau AG	Water companies
Rural	0 to 20	0 to 25	0 to 25
Intermediate	20 to 40	25 to 50	25 to 76
Urban	Above 40	Above 50	Above 76

In fact, water companies and Agence de l'eau Adour Garonne have concordant definitions of a rural distribution system but diverge about urban character (1.5 greater value of density for water companies).

Linear losses index as a function of density of customers

The linear regressions confirm a close relationship between LLI and D.

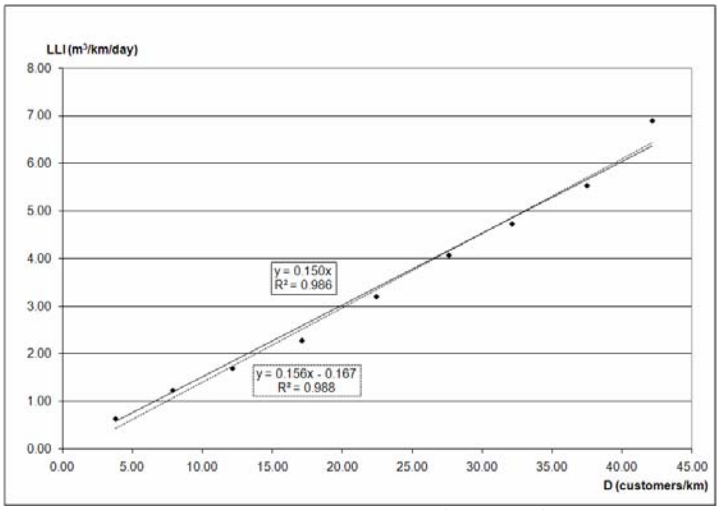


Figure 5 Linear losses index as a function of density

The negative intercept of the linear regression with intercept is surprising. Actualy, the function $LLI = (0.156 \times D) - 0.167$ can be written :

$Vl = 365 \times (0.156 \times N - 0.167 \times M)$

(Vl annual non-revenue water volume; N number of customers; M length of mains)

At first glance, this relation means that non revenue water volume decreases when length of mains increase.

We suppose that this anomaly is due to the positive correlation between M and N, the weight of customers in the explanation of losses includes a length of main associated to each one.

Because of this, we prefer the linear regression without intercept : $LLI = 0.150 \times D$

It follows that the length of mains isn't directly taken in account to explain the level of non-revenue water.

Linear losses index as a function of Linear Consumption Index

We can also observe a close relationship between LLI and LCI.

We make the same finding of a negative intercept (with the same meaning).

The function deducted of the linear regression without intercept is : $LLI = 0.400 \times LCI$

The R^2 of this linear regression (0.994) is a little better than the R^2 found for LLI according to D (0.986)

In fact, after simplifying the equation by the length of mains, this function expresses non-revenue water volume as a function of billed direct consumption volume : $Vl = 0.400 \times Vc$

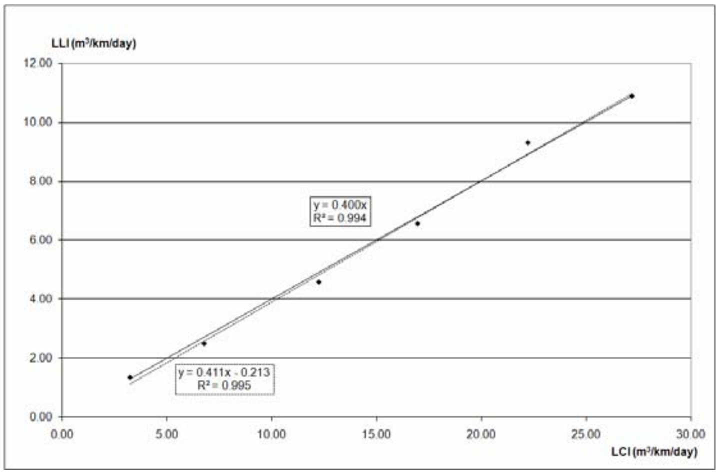


Figure 6 Linear losses index as a function of linear consumption index

Comparison with existing references

All the references are more or less coherent with the linear regressions without intercept for their range of validity (densities under 45 and LCI under 30). Nevertheless, a trend of overestimation for extensive networks can be observed.

Regarding our results we can ask questions about the advantage of step-by-step references.

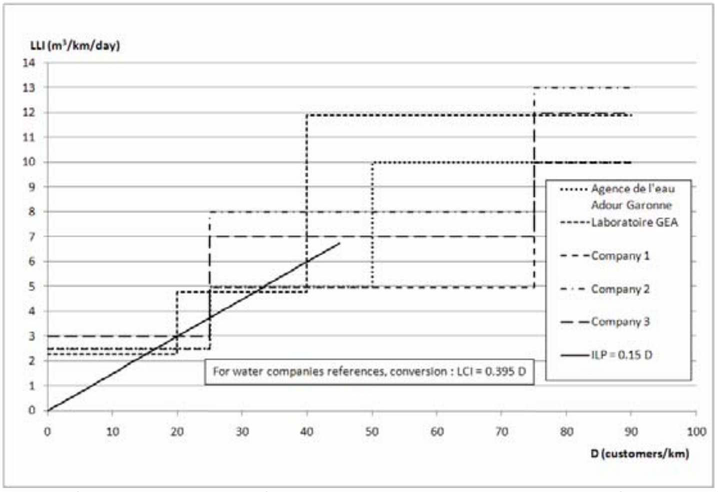


Figure 7 Comparison : All references / Regression line LLI as a function of D

Comparison between Linear Losses index (LLI) and Unavoidable Annual Real Losses (UARL)

Unavoidable average real losses (UARL) can be estimated with the following equation (Lambert & Hirner 2000; Alegre et al 2000) :

$$UARL = (18 \times \frac{Lm}{Nc} + 0.8 + 0.025Lp) \times P$$

Litres/service connection/day when the system is pressurised where : Lm is the length of mains (km); Nc the number of service connections; Lp the average length of service connections (m) and P the average operating pressure (m).

Gathered data aren't sufficient to calculate a precise value of UARL. However, an estimation can be made under the following assumption :

- Nc = N (For rural networks, number of connection is close to number of customers)
- Lp = 8 m
- P = 50

After conversion of units, under these hypothesises, an estimation of unavoidable annual real losses can be express in cube metre/kilometre of mains/day :

$$UARL = 0.9 + 0.05 \times D$$

This expression of UARL as a function of the density of customers is homogeneous with LLI, whitch allows comparisons.

Current annual real losses (CARL) can also be express as the real losses cube metre/kilometre of mains/day. By definition, real losses volume is included in non-revenue water volume so, CARL < LLI.

The infrastructure leakage index (ILI) is defined as CARL/UARL (Alegre et al 2000). For each network we calculated the ratio Non-revenue water volume/UARL <ILI

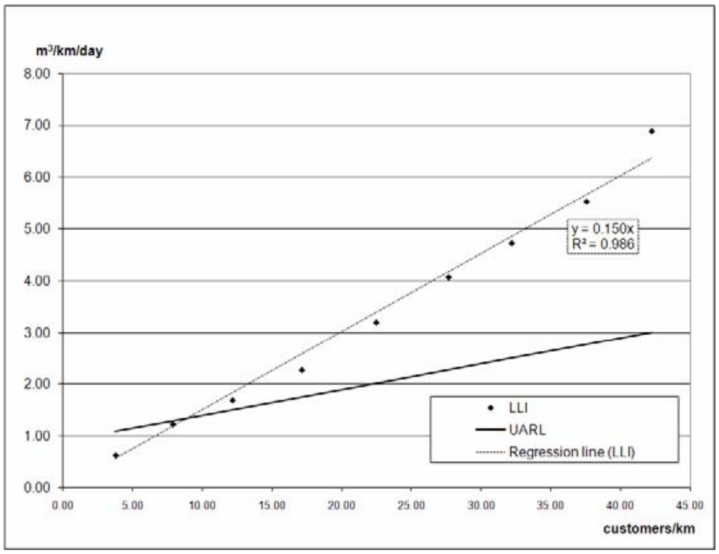


Figure 8 Comparison : UARL and LLI as a function of D

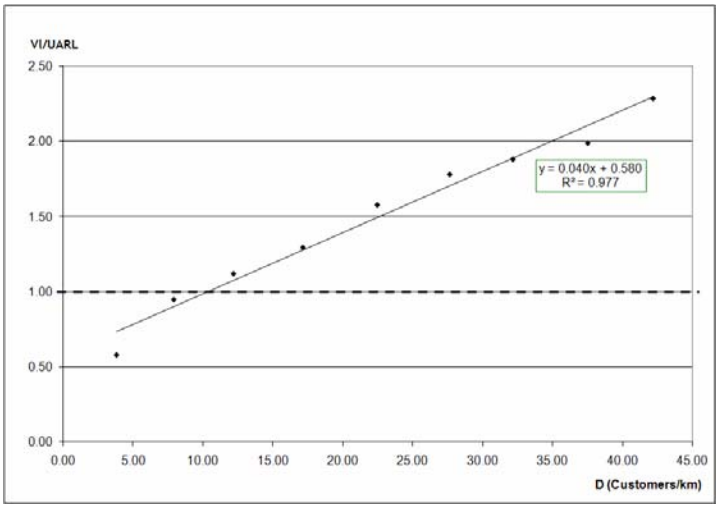


Figure 9 VI/UARL as a function of D

We can observe that for low values of D, LLI is less than UARL, therefore CARL is less than UARL. This observation provided evidence of the unsuitability of the IWA's calculation when used to determine UARL in the case of extensive networks.

Of course because of the approximation used we must be careful with the results of the estimation of UARL's values. Despite everything, it is clear that our assumptions are independent of the density of the network, so the link between VI/UARL and D is real. It mean that the value of this indicator for a network cannot be interpreted independently of the density of customers.

Conclusion

On the basis of data gathered by French agriculture ministry decentralised services, we could build a database of 15 000 rows with basic annual volumes, number of customers and length of transmission and distribution mains values.

A first analysis of the correlation between basic data showed that volumes of non-revenue water and supplied water are strongly correlated : Vl = Vs/3

Merging the data by classes, linear regressions without intercept gave the following connections when D is under 45 and LCI under 30 :

- LCI = 0.395 x D
- LLI = 0.150 x D
- LLI = 0.400 x LCI

Where : LCI represents the Linear consumption index (m^3/km/day); D represents the density of customers (customers/km) and LLI represents the linear losses index (m^3/km/day).

The negative intercept of the linear regressions of LLI as a function of D and LLI as a function of LCI showed that, compared to the number of customers or to the volume of supplied water, the length of transmission and distribution mains doesn't have a significant effect on the level of losses.

Using the relationships between LLI, LCI and D we can observe that the different reference values used in France by the water companies and by public agencies to judge the acceptable level of leakage are coherent with our results despite a trend of overestimation for extensive networks. Systems of references by brackets do not show any advantage, proportional references would be better.

Under simplifying assumptions we estimate the unavoidable annual real losses (UARL) according to D using IWA's reference values. For network with a low density of customers, UARL values are above non-revenue water volume, providing evidence of the unsuitability of IWA's reference values when used to determine unavoidable annual real losses in the case of rural networks.

A practical application of the results obtained in the study will be realised in the Département of Gironde in the South West of France. The main water resource in this region is from deep groundwater resources. These are generally over-exploited and a general decrease in the piezometric head is observed. To counter this situation, the public agencies have implemented a scheme for managing the distribution cycle with the reduction of abstraction from these deep resources being the primary objective. One of the important steps in water economy is the reduction of losses in water distribution systems. The SMEGREG, a public structure in charge of the management of water saving, will use the links between non-revenue water volume and other data and indicators resulting from this study, to make progress in quantifying potential reductions in water loss and to prioritise the battle against leaks in the hierarchy of water distribution management.

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Calculation, Estimation and Uncertainty in the Apparent Losses Volume in the Water Supply System of Canal de Isabel II.

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Keywords: Apparent losses; water balance; uncertainty.

Introduction.

The practicality involved in the Water Balance is proven by the experience of a multitude of utilities companies and water management representatives throughout the world. Both drafting of this balance, as well as calculation of the performance indicators derived from the same, orientate the efforts dedicated to obtaining increased efficiency in the water distribution network and in its operation.

The IWA has disseminated a format for editing of this balance in the shape of a summary chart, which responds to the typical needs of the vast majority of suppliers, managers and administrations related with water supply and distribution.

Canal de Isabel II is responsible for the management of Region of Madrid's whole water cycle. In relation to supply, this represents operating 14 dams, 12 treatment plants, 77 wells and close to 15,000 km of mains. It counts with 1,110,000 water supply contracts, each one with its own individual water metering device. These users are connected to the network by means of 600,000 water supply connections. Supply reaches a population of 6,000,000, shared out between 177 municipalities.

Canal de Isabel II is quite aware of the advantages that the balance provides, as an analysis of correspondence between supplies and consumption and, not in vain it has been applying the audit techniques for more than 15 years to control and improve efficiency in the supply service for which it is responsible. A series of changes and particularities have been introduced over this period of time so that the water balance summary chart will exactly respond to the different realities that make up the Region of Madrid's water cycle management.

Once calculation and estimate of the different chapters that make up the water balance to which we refer have been assumed, the need arises to evaluate the precision of the different volumes that are assigned to these chapters or water destinations. Inaccuracies that accompany each one of these partial results will turn into an uncertainty when it comes time to orientate tasks aimed at their reduction and to determine the real volume of losses involved in hidden leaks.

The methods for calculation and estimate of the apparent loss components of the balance in Canal de Isabel II's supply system are presented in this article, outlining the uncertainty points in which we incur and the indecision represented by the same, in view of evaluation of the volume of real losses.

About Canal de Isabel II's balance.

Canal de Isabel II elaborates water balances over the entire scope of its competence, over differentiated supply areas and over network sectors. Apart from this, different periods of analysis are contemplated: monthly, quarterly, half-yearly and annual. The most significant work is carried out for the global scope of influence and the annual period. From now on this is the balance to which we will refer to.

The advantage in this frame of operation is that we count with all the necessary information to elaborate the water balance within the scope of responsibility that results from the same company's different departments, which facilitates obtaining of the information to be processed.

The main water volume (at least 77% of the total input) reaches the private users and is recorded by the individual water meters. This volume is shared out between metered billed and metered unbilled (metered unbilled results approximately 1% of the total metered in users).

The volume of supplied water that is not metered in users is divided into two important concepts: apparent losses and real losses.

Apparent losses always include destinations of water that without having been metered is used in one way or another in the network's operation and its elements, or in authorised or non-authorised real unmetered consumption. In the case of Canal de Isabel II, in view of the importance given to separate analysis of consumption metered in users, the volume of billed water that is not recorded by meters is also included in apparent losses. Origins of said billing are the result of the agreement with certain municipalities with regards to cleaning of streets, and collections made from third parties when they cause bursts with water losses.

Apart from unmetered billed water, apparent losses are composed by pipe draining and cleaning volume, fraudulent use, submetering of individual water metering devices and unmetered exterior municipal uses.

All that described is presented in a balance summary chart, vertically read, in which the chapters are disaggregated until the detail we are commenting is actually reached. Finally a series of utility regroupings are added for analysis of the water and corporate results.

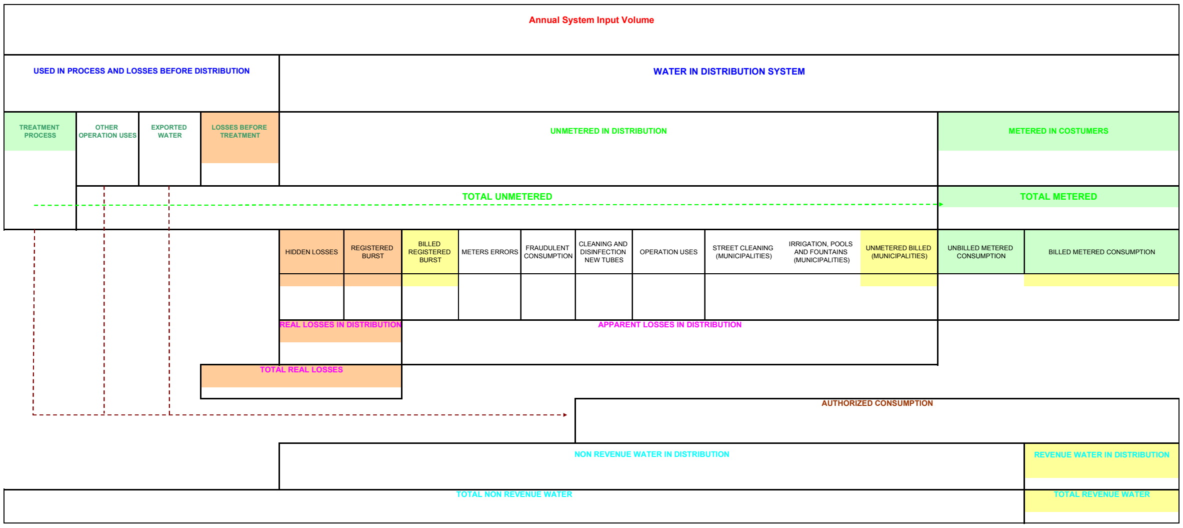

Figure 1 Balance Chart with Canal de Isabel II Format.

Apparent losses in Canal de Isabel II’s balance.

This is the volume of non-accounted water in individual contracts, although it does not represent a real loss, given that it has some sort of individual, collective or network operational use, or because its value has been economically recuperated even if its destination is not a useful one.

It includes unmetered volumes due to submetering in meters, unmetered consumption (includes unmetered billed water to the Capital City of Madrid), possible fraudulent connections and uses involving operation and distribution network processes, plus water that is lost (although billed) in the case of bursts caused by third parties. The total annual volume of apparent losses in the Canal de Isabel II has represented approximately 12% of the total annual system input.

Apparent losses are currently reason for particular concern for Canal de Isabel II, which is carrying out important efforts to improve the results in terms of total metered water. This makes calculation of the chapters making up the same and precise knowledge of the error margins that are being handled far more important. It is only through the same that the sources causing the apparent losses (and its characteristics) in the system can be identified so that work can then be carried out on its reduction or elimination.

Each one of the destinations for the water that have been considered as apparent losses are described here below.

Exterior municipal use: public parks and garden irrigation.

This chapter represents the non-metered volume employed by municipalities in irrigation of green spaces, public swimming pools and operation of decorative fountains.

A series of irrigation outlets that do not have water metering devices installed exist in many municipalities included in Canal de Isabel II’s scope of responsibility. This configuration is a legacy of the past, during which agreements with criteria that was different to that currently applied were formalised. Municipal employees also have access to the hydrants, from which the public roads are sluiced and cleaned.

The volume that corresponds to the water needs involved in these public green spaces, fountains and swimming pools is calculated by applying the unitary water dosage over the surfaces according to characteristics and the month of the year. The volume metered in the municipal water metering devices with irrigation tariff, for both billable and non-billable contracts, is discounted from this necessary amount, disaggregated per municipality.

Unitary water dosage was determined in an agronomic study over green plots of land in the Community of Madrid, in which spaces for irrigation were grouped into three categories: trees, bushes and grass. The water dosage chart, in cubic meters per month and hectare for each category, was calculated in accordance to real plantations and their representative level in green urban areas. In like manner in the case of swimming pools and water layers in decorative fountains the water dosage that is considered is the one representing the depth of the element. Thus the same methodology is applied over surfaces and over irrigation areas. Water dosages include climatic factors to adapt to the real conditions of the period under study.

Publicly owned green urban surfaces over which irrigation takes place are determined as of the areas that were localised by means of a fly-over in the year 2004, with which the inventory of parks, gardens, swimming pools, fountains and public ponds within the complete scope was updated.

Precision of the measurements of green surfaces, swimming pools and public fountains is intra-cadastre. All information generated in the study was dumped into the geographic corporate information system to facilitate the task involving monitoring of its suitability. Opportune comparison was executed with the land register maps, likewise integrated in Canal de Isabel II's SIG system, to then assign public or private ownership.

Currently influence of the change originated by the severe drought period suffered by the Region of Madrid (September 2005 to May 2006) in the irrigation infrastructure in that pertaining to new unitary water dosage associated to more efficient methods is under study. Likewise improvements over the study of surfaces provided by the satellite image photo-interpretation systems are also being evaluated. This system could provide more agility and increased precision in the calculation than the photogrammetric flights, to the point of even eliminating errors that are generated in the estimate of over-irrigation, given that surplus or defects produced in irrigation of the vegetable species (and their reference plots) would be directly analysed.

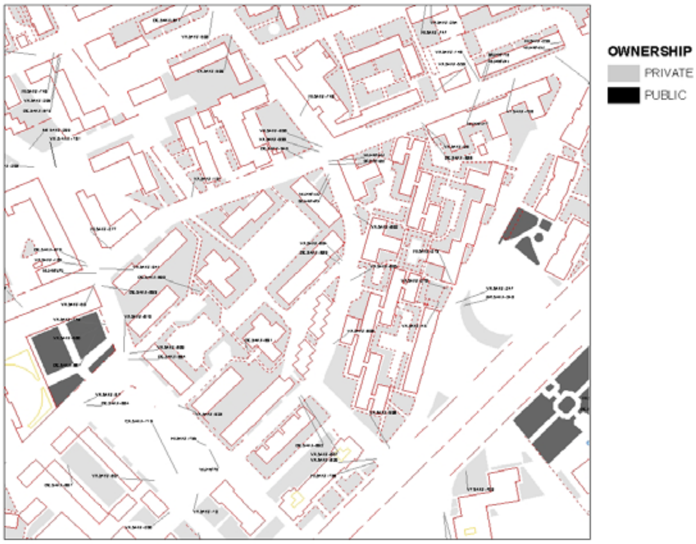

Figure 2 Exterior uses in Canal de Isabel II's corporate GIS (Geographic Information System), GAMBA.

Behaviour of the different green areas and parks that do count with water metering devices for their control was analysed in detail to determine over-irrigation, extrapolating the conclusions that were obtained. This is the operation that has rendered the greatest uncertainty in the calculation, although the methodology does not actually depend on this fact, that is to say, zone and even local over-irrigation factors will be applied in the future, thus gaining the precision that has been lost.

In fact, monitoring of metered water consumption behaviour for irrigation of public green areas is fundamental to determine the irrigation water dosages, as a generalised change in the irrigation techniques and guidelines in large extensive municipalities has a decisive influence on calculation of the volume for this chapter. During the last few campaigns work has been carried out along the line of water reclamation for these uses. This concept is already included in the calculation processes for this chapter, subtracting volume from the non-metered need for the same use.

The error margin in unmetered public irrigation results in 15%, and in greater measure depends on the estimate of over-irrigation rather than on water dosages, and in less measure on the surface per type of use:

Unmetered municipal irrigation error (%) = f (over-irrigation; dosage; surface) = 15%

In this way, in the annual balance of Canal de Canal de Isabel II, the unmetered volume used by municipalities for irrigation results 2,9% of total input volume, with min and max between 2,4% and 3,3%.

Exterior municipal uses: sluicing and cleaning of public roads.

Methodology in this case is identical to that described for the green areas, even in the origin of the data and its treatment. The surface taken into consideration now corresponds with the public road system that is susceptible to be sluiced.

Water dosages are assigned with reference to uses and customs involving urban services, which are generalised for all the municipalities.

When it comes time to discount the volumes metered with this aim in each municipality from the necessary water, that described in the water concept that is billed unmetered in municipalities is of notorious influence. This non-metered billed water is actually located in the municipality of Madrid, location where the amount is discounted to avoid duplication in the calculation. Incorporation of regenerated water resources for street sluicing in Madrid is also particularly active.

The error margin in unmetered use for street sluicing results in 20%, depending both on the water dosage and on the surface over which actually it takes place:

Unmetered municipal sluicing error (%) = f (water duty; surface) = 20%

In the annual balance of Canal de Isabel II, this represents 0,23% of total input volume. Min and max are then 0,18% and 0,28%.

Unmetered billed to municipalities.

According to the agreement signed between Canal de Isabel II and the Capital City of Madrid, billing is per estimate of a certain annual water volume pertaining to the concept of use of public road hydrants that do not have a water metering device for recording of water supply. It has been verified that the supposed amount adjusts correctly to the necessary volume for street sluicing in the capital, although it does not reach the total annual volume that is calculated in accordance to the methodology described above. At any rate the uncertainty is transferred to the chapter for sluicing of public roads, in which it holds a discount role with regards to the water needs for these cleaning tasks. Given that the estimate is made for the complete period, the advantage of distributing it according to the seasonal nature of the need that is detected for the road sluicing task is assumed.

This means that an error could exist in the distribution of amounts between these two chapters, but that its sum does not necessarily imply more uncertainty than the one corresponding to the methodology that is assumed for calculation of the water needs for these municipal tasks.

One of the works that is underway at Canal de Isabel II is focused on installation of individual water metering devices in the hydrants and irrigation series in the city of Madrid and other municipalities. Recorded metering of all operations involving the public road system will be available in the medium term.

Also, incorporation of reclaimed water will be noticeable precisely in these sluicing tasks, origin of which will be in the tertiary treatment of the waste water purifying plants.

This chapter represents 1,7% of total annual system input volume.

Draining.

Draining includes those volumes employed in network operation, covering both programmed draining and draining executed to cover unexpected needs.

Data for calculation is obtained from the Warning and Incidents Management System (GAYTA), in which date, start and end time of the action is recorded, element by which the same is executed (draining diameter or metering set faucet, if the case was to be as such), along with the opportune comments. Volume of each draining executed on the network during the annual analysis period is thus obtained with the mentioned data, the same by means of traditional water formulas.

When the period of operation or the diameter is not recorded, then the mean of data that is available for equivalent operations is applied, distinguishing mainly between drainage per outlet and drainage executed in the whole of metering of users (obviously with smaller diameters). The water pressure to be considered is estimated per each zone or sector, using 60 meters of pressure by default, which is the mean pressure in the nodes of Canal de Isabel II's networks.

Thus in this manner the uncertainty resides in filling-out the necessary data for calculation of the flow and effective draining period. Said uncertainty is quantified as of the number of estimated datum, and by taking into consideration dispersion of field data with which work is carried out to backfill the lagoons.

The error margin for the drainage volume in the distribution network results in 15%, mainly depending on the effective diameter of the drainage elements and the water pressure:

Draining error (%) = f (diameter; pressure; time) = 15%

The operation uses for network drains represent 0,10% of total volume, with min – max of 0,09% - 0,11% of total volume.

Cleaning and disinfecting of new pipes.

Calculation of this concept is executed by assuming the protocols that are described in Canal de Isabel II's standards, applied to each span of newly installed piping corresponding to network extension, per receipt facility corresponding to new urban development or due to renovation of the existing network. Individualised application of each pipe span guarantees correct evaluation of the lengths in question with their respective diameters.

The number of kilometres of new piping employed in the calculation is consulted in the geographic information system, in accordance with the field in which the date of pipe installation is recorded. Given that uses involving disinfection are exclusive to new receipt piping, these must be differentiated. The GIS tools are used for this, assigning spans designated as "new" in accordance to the period under analysis, the consolidated urban centre or new urban developments.

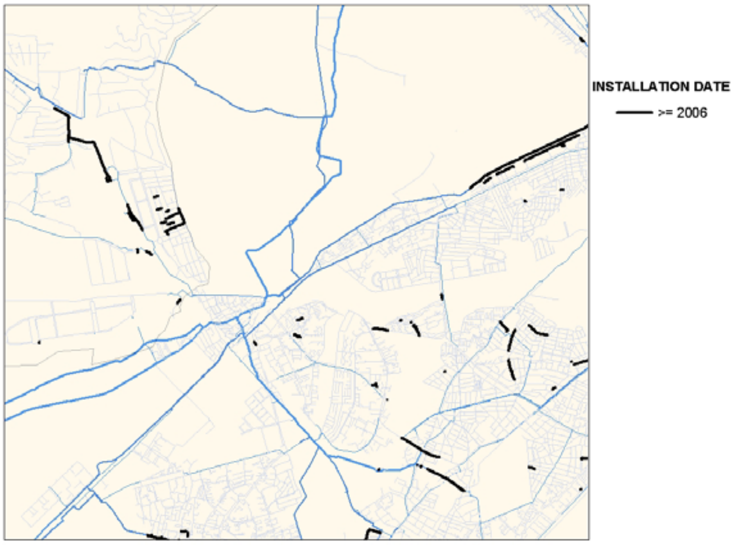

Figure 3. Period of service of the pipes in Canal de Isabel II's GIS, GAMBA.

The volume of water used for cleaning corresponds to circulation of flows at certain speeds during periods that suffice for cleaning of pipes and picking up of sediments.

For incorporation of new networks to Canal de Isabel II's supply system, disinfection represents a water use that is calculated by the number of times the system is filled and emptied, with disinfection executed by means of chemical disinfecting products.

Uncertainty in this chapter is greater, as it depends on the field operation involving drainage. This task is not always executed by company personnel, but is in fact carried out by the construction contracts pertaining to new urban developments, if the case was to be as such, thus there is no record of operations available. Experience indicates existence of great dispersion in terms of draining periods for cleaning, in particular to the rise. A metering project covering volumes employed in cleaning and disinfection of the networks for new developments has been activated. Results will be available as of the second quarter of the year 2007, although quite probably it will not be until spring of 2008 when sufficient information will be available to reconsider the opportunities that exist to improve efficiency of these tasks.

The error margin in the calculation of volume employed in cleaning and disinfection of new pipes is 60%, and above all it depends on the period of time employed in cleaning:

Cleaning and disinfection error (%) = f (time; pressure and drainage elements; pipe length and diameter) = 60%

This volume of operation uses for cleaning new mains represents 1,34% of annual system input volume, with min 0,54% and max 2,14% of that volume.

Estimate of fraud.

This is the volume that is consumed in fraudulent connections and in properties that have some sort of irregularity that allows the owner to extract more water from the distribution network than that recorded in the whole of said owner's contract metering. Given its nature it is not possible to calculate this volume, having to estimate it.

The volumes appearing in the balances corresponding to the last few years is derived from extrapolation of the results obtained in a report on detection of fraud in certain municipalities of the Region of Madrid, which was drafted throughout the year 2005. Extrapolation to the whole of the network is executed in accordance to the typology of the municipalities, length of the network and number of connections.

Fraud is now starting to be known thanks to the advance experienced in sector works, including its repercussion on the non-recorded water volume that passes over to the concept of recorded in users once fraudulent consumption is regulated.

At any rate the degree of uncertainty is important, as characterising the impact of fraud in municipalities with diverse extensions, populations, consumption typologies, origin of the distribution network, etc. is anything but easy. The percentage of errors in relative terms is high, although impact in the calculation per differences in the real loss volume due to hidden leaks is not quite as high.

The error margin in the estimate of fraud results in 25%, figure that mainly depends on the number of real fraud cases that exist in smaller municipalities:

Fraud error (%) = f (number; period; water duty) = 25%

These unauthorized consumptions represent 1,6% of annual system input volume, with an uncertainty between 1,2% and 2,0% of annual system input volume.

Meters errors.

This concept reflects the fact that the water metering devices do not actually display an exact readout of the water volume that passes through them. It is well known that submetering at low speeds can cause a certain error rate in accumulated metering during the readout periods.

Until results corresponding to the water metering devices error curve for the Region of Madrid are available, study that is currently under elaboration, calculation of metering errors is solely executed in accordance to the age of the metering device.

Inaccuracy is determined for each metering device in a % over its readout, based on the age of the device counted since the date of its installation. The error percentage, which is then multiplied by the recorded amount (metered billed and/or metered unbilled in the period of analysis), is assigned to the meters per linear distribution, between the minimum and maximum error taken into consideration, depending on the number of days that the device is under operation. In the case of metering devices that were renovated during the course of the year of analysis, weighting is executed with the period of time that each one of the devices has been installed (replaced and new).

All contracts count with reliable information on their metering device and its date of installation, including any replacements that take place.

Obviously the uncertainty that is assumed in this case is based on simplification to an error percentage tangent on which each metering device incurs, likewise without taking into account the histogram of real demand flows. It will be possible to precisely quantify the error in executed metering, and the margin of reliability that the new calculation can offer once the above-mentioned study is available and after its comparison with consumption guideline monitoring studies for the different consumption categories in the Region of Madrid.

Quantified as a margin associated to the field readout system is the fact that a certain number of records exist that are actually filled-out per estimation during each meters consumption readout campaign, apart from the fact that there are always a series of claims from consumers who do not agree with the readout that is made. The error involving manipulation of data produced in the monthly ratio of the bimonthly readouts is also taken into consideration.

The error margin in calculation of the metering devices' precision results in 40%. Data that is used is of great quality; the uncertainty resides in the simplified methodology:

Metering device measurement precision error (%) = f (simplification to the age – error tangent, estimation, monthly ratio, age) = 40%

In the annual balance of Canal de Isabel II it represents 2,4% of total volume, with an error margin between min 1,5% and max 3,4%.

Unmetered billed in bursts.

Part of the water that is lost due to bursts that are recorded and repaired throughout the period of analysis is included in the apparent losses, as has already been explained. These are bursts caused by third parties, that is to say, non-fortuitous bursts. When a party that is responsible for the burst is identified, billing of the volume calculated as loss in the incident is proceeded with. Thus in this manner, although this volume of water does not generate any gain as a natural resource, at least its economic value is actually recuperated, thus in some way being comparable to negligent use of the water by certain users.

Calculation of each one of these bursts is made by means of water formulas, in which the loss periods are more exact than they are in fortuitous incidents, as bursts are produced in a specific and known action. In fact the information of origin that is used by the team executing calculations resides in the database of the Warning and Incidents Management System (GAYTA), in which the condition involving the burst caused by third parties is explicitly annotated, while it is not registered in the commercial systems.

Thus uncertainty is actually based on assigning of parameters for water calculation of the lost flow, such as broken pipe surface, water pressure at the location and moment of burst. Typically these bursts take place at urban service facilities sites. Some photographic records executed by the company inspectors actually exist, which are useful to assign a water leak surface corresponding to the pipe in question. In that pertaining to output pressure, in those areas where no information is available, the mean of the measurements over replenishment network sectors is applied, resulting in a pressure of 60 meters.

The margin of error in the calculation of the volume that is billed per provoked bursts is 15%, and above all it depends on the gap that is produced in the pipe and on the pressure of the water:

Provoked burst error (%) = f (broken pipe surface; pressure; time) = 15%

This kind of unmetered billed use, apparent losses in annual balance of Canal de Isabel II, represents 0,58% of annual system input volume. It has an error margin between min 0,49% and max 0,67% of total annual system volume.

Uncertainty in Canal de Isabel II's balance.

The following error margins result by applying that described above to the annual figures on Canal de Isabel II's supply system:

Concept	*Volume (%)*	*Error Margin*
Total system input	100	1,0 %
Metered in users	77.6	-
Apparent losses	11.6	20,6 % (between 9,2 and 14,1)
Real losses	10.8	15,9 % (between 9,1 and 12,5)
Non-revenue water	21.1	4,7 % (between 20,1 and 22,1)

Table 1 Uncertainty in the calculation of Canal de Isabel II's balance

Conclusions.

It is quite convenient to have disaggregated information available on the origin of the data, this to determine uncertainties incurred upon with elaboration of the system's balance, thus fundamentally:

- Record of flows and network pressure, including precise location and condition of the installed metering devices.
- Record of user consumption, with information on the installed meters.
- Record of network operations and repairs, with dates and times covering each action.

The databases that feed the necessary calculations for elaboration of the balance must count with a data structure such that the individual state of the operations that are represented can be evaluated. Thus in this manner, per aggregation of particular cases, it will be possible to easily analyse the uncertainty. This is also of great use when voids in information are to be covered, when the same proceeds.

In the case of events that are difficult to record, a series of clear and executable action protocols should be established, and field operators should have sufficient sensitivity to assume the same.

The calculation process of the components that make up the balance should respond to the physical facts they represent, in such a way that it will be possible to clearly identify during which phase of the events under study action should actually take place. It will also be possible to analyse the sensitivity to quality of the data that is used and the dispersion that is produced by each one of the explanatory variables pertaining to the event under analysis.

Canal de Isabel II considers that reduction of uncertainty in the calculation of the components that make up apparent losses should be a strategic objective, and in this sense it is developing a series of Research, Development and Innovation projects that correspond to different natures. The final objective is to reduce the annual volume in concepts involving apparent losses, for which veracity of the annual supply and consumption balance must be defined, while the weak points in the operation and management of the supply system must be detected.

The following is to be highlighted:

- Project on use of satellite images for calculation of water volume used in public green areas irrigation.
- Study of the precision of the park of individual water meters installed in the Region of Madrid.

- Project on analysis of efficiency in practices involving cleaning and disinfecting of new piping in the network.

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Trials to Quantify and Reduce in-situ Meter Under-Registration

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Keywords: Apparent Water Losses, Meter under-registration, Automatic Meter Reading, Zigbee, Unmeasured Flow Reducer, Magnetic Water Inlet Valve.

Introduction

The paper shall look at three sequential issues: 1) the theory behind Apparent Water losses; 2) modern techniques for measuring Apparent Losses and revenue water meters; and 3) two alternative solutions for controlling consumer meter under-registration. The theory in question relates to the International Water Association's methodology of dividing Apparent Losses into four components, allowing for a systematic solution to the problem. Techniques for measuring meter readings shall look at an innovative ZigBee based automatic meter reading system. Finally, the paper shall look at two potential in-situ solutions to meter under-registration; the magnetic water inlet valve and the unmeasured flow reducer. Both solutions are viable, according to the specific situation a water utility faces.

The Theory behind Apparent Water Loss Control

The IWA groups water losses into two types; Real Losses, which are the physical losses (or leakage) and Apparent Losses, which are caused by revenue meter under-registration, water theft and billing errors. Real Losses are an expense to a water utility for a number of reasons: the leaking water costs money to produce; maintaining the water network to avoid further losses is expensive; and additional capital expenditure may be required in the form of new production plant, and as a result of the losses. Apparent Losses are not so much an expense to the water utility as they are a loss of potential revenue. Apparent Losses relate to water that is being consumed, but not being paid for. Thus for every cubic metre of water unbilled as a result of an Apparent Loss, the water utility loses the opportunity of collecting money for that cubic metre of water. Whilst the concept of Real Losses is fairly easy for one to understand, that of Apparent Losses is more complex for a number of reasons: First of all Apparent Losses are somewhat more subtle or intangible, when compared to Real Losses. Imagine comparing a leaking valve (a Real Loss) with a billed consumption for a household that is being under-estimated due to an inaccuracy in a water utility's billing system (an Apparent Loss). As a second example, compare a weeping (slight leakage) service pipe (a Real Loss) to a well hidden illegal service that is being used intermittently and intelligently (an Apparent Loss).

A second reason is in the multidimensional nature of Apparent Losses. Four types of Apparent Losses exist, as shown in Figure 1 on the following page. The first loss, water theft, can occur in a variety of ways: Water can be stolen from an illegal connection, from a bypassed water meter, from a damaged water meter, or simply from the neighbour's plumbing system! The second loss, meter under-registration, consists of a situation where the consumer meter is incapable of measuring all the flows passing through it. Flows below the accurate starting flow of the meter are a particular problem. The third and forth losses, meter reading errors and billing errors go hand in hand. Meters can be misread or alternatively wrongly computed in a utility's billing system. Also, certain Apparent Loss components can be both positive or negative, even going to the extent of cancelling out the effect of other components. As an example a water utility may be over-billing substantially due to an incorrect 'closed premises' estimation policy, whilst at the same time substantial meter under-registration in the locality exists.

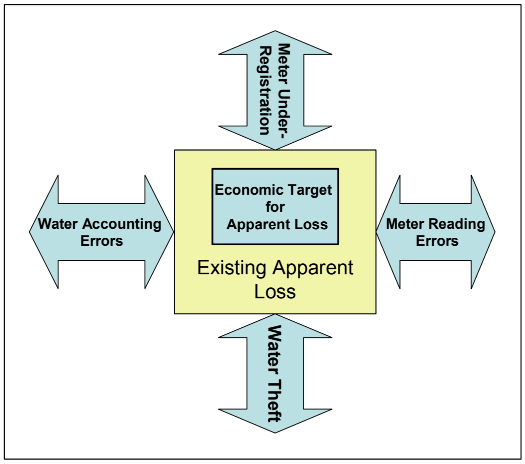

Figure 1: The IWA Apparent Water Loss Control Methodology

Thus, due to the complexity of the problem at hand, it is vital that every water company has a strategy for tackling Apparent Losses. Figure 2 below describes a strategy advocated by the authors for managing Apparent Losses in an integrative fashion. The multidimensionality of the model is result of the various levels in which Apparent Losses impact upon a water utility. Whilst to a certain extent Real Losses can be managed by a single, functionally organized section, the same cannot be said for Apparent Losses. Policy decisions on water tariff structures may impact upon the amount of water theft taking place. Purchasing policies may impact upon the quality and availability of water meters. Finance and budgeting decisions may impact upon the means being utilized to read or estimate meter readings. Oversight agencies or institutions may demand reduced interference to certain key consumers, etc, etc. For this reason an Apparent Loss control strategy must relate to the various hierarchies and decision-making levels within a water utility. It must be applied as a centralized initiative, taking the form of a project that may one day evolve into an operation when running efficiently enough. As in all projects, all changes need a champion! The main challenges lie in management; managing the human resources (employees), the physical resources (instrumentation and equipment) and the organizational resources (such as quality procedures). Hence, for effective Apparent Loss control, one must have a focused, dedicated and well led management team.

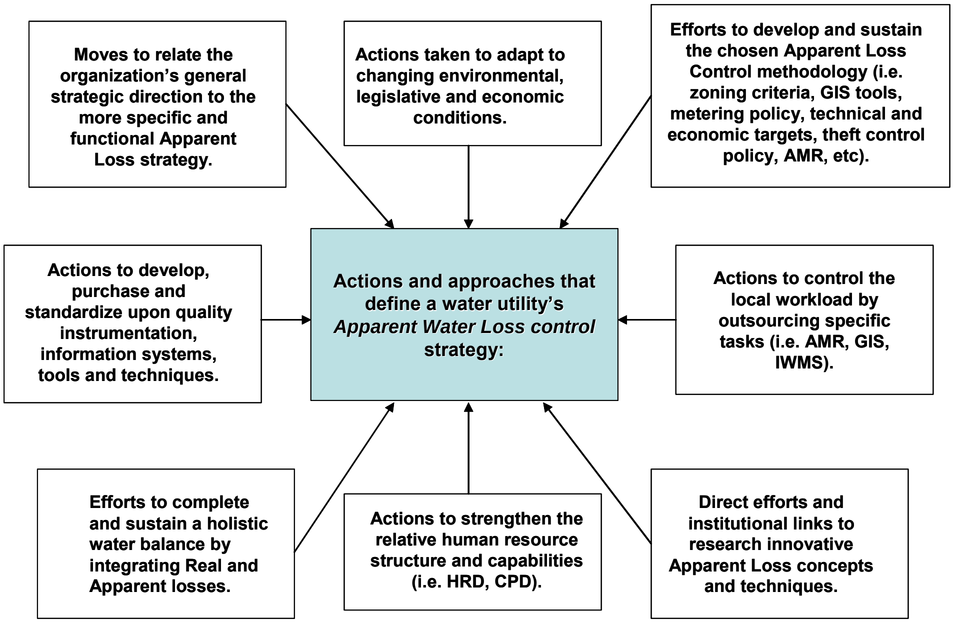

Figure 2: Strategic Control of Apparent Water Losses

Measuring Water Meter Readings and Computing Apparent Losses

The authors have been through the experience of developing and successfully commissioning two automatic meter reading (AMR) systems on the Island of Malta. The paper will look at the transition from an earlier radio frequency (RF) system to the more modern ZigBee-based typology. Various organizations and utilities world-wide are turning towards ZigBee after finding out that AMR systems such as '*power line carrier (PLC)'* or '*GSM-based AMR'* come at a huge expense and a compromised functionality. A further issue is one of flexibility: The ideal system will allow the user to be able to acquire data in a fully automated fashion, if need be, or alternatively to download data on site. Furthermore the system must allow for a transition from reading to data logging, this requiring enhanced memory and more varied data input channels.

Radio Frequency AMR

Radio frequency, or RF, automatic reading is possibly one of the most commonly used and popular AMR systems around. The popularity is a result of low cost and robustness, with hundreds of thousands of units sold yearly by companies such as Ramar, Itron and Schlumberger worldwide. RF units are usually low power, at below 8w, work at standard telemetry frequencies (usually at around 400MHz), and boast lithium batteries that provide a 5-year lifetime. The units are sealed, tamper-proof, and disposable. RF-based AMR in Malta has been around since 2003, serving its purpose to acquire data and transmit for the 100 metre range that the system allows. Of course repeaters can be used to gather and boost data to a further point, but at an expense.

The ZigBee Typology

It is this limited range of standard RF systems that brought about the concept of Zigbee. The ZigBee wireless-personal-area-networking (WPAN) technology has been designed from the ground up with one application in mind; low speed, low data rate sensors. The ZigBee Alliance specified the ZigBee foundation as per the wireless standard IEEE 802.15.4, which defines and handles the radio section (PHY and MAC) of the ZigBee technology.

What does ZigBee stand for, anyway? The name was coined from the zigzag dance which certain African honeybees use to relay information, related to location and distance of sources of nectar, to other bees. Likewise, the ZigBee specification enables data packets to propagate through nodes in a mimic of the honeybees' dance.

Low-power wireless sensors typically source their power either from a battery or parasitically. The latter employs age-old principles based on magnetic and electrical energy coupling. This may take the form of a coil of wire wound around a current carrying conductor which induces a tiny E.M.F. in this coil. The energy thus gleaned is harvested in super-capacitors for subsequent use during RF data transmission and reception. In the former case, the battery has to be used so sparingly that its lifetime should equal the listed shelf-life. This may be as high as ten years (alkaline batteries). The only way to achieve such performance is to have the ZigBee node in sleep mode for 99% of its lifetime.

A further characteristic of the ZigBee technology which enables compliant nodes to achieve such an enviable performance is the use of the 2.4GHz frequency band as stipulated in the IEEE 802.15.4 standard and which allows nodes to exchange data at a maximum of 250kbps. Such a high data rate ensures that nodes are awake for a couple of milliseconds only.

A ZigBee network may have any of the following topologies;

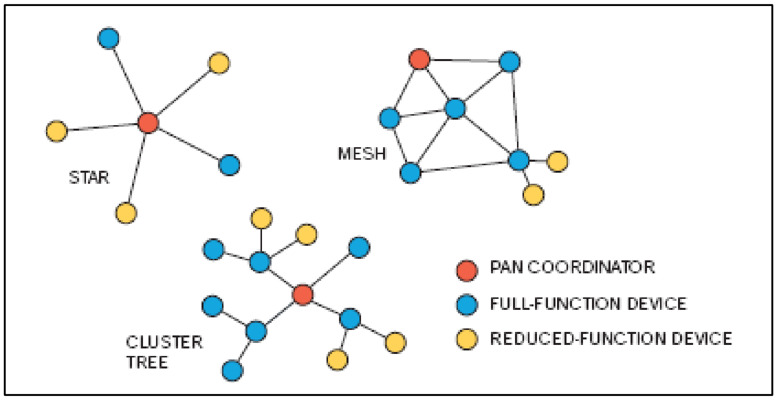

Figure 3: Various ZigBee Topologies

The Star topology involves only end devices (the sensors which are mostly in sleep mode and represented by the reduced function devices) and the coordinator, which manages the network. The tree involves another device, called a Router (shown as a full function device). A Router is typically run from an AC point and acts as a relay, a messenger, of data packets between nodes which are out of each others radio range. This is feature gives the ZigBee technology its much vaunted feature of data hopping where spatially distant nodes may communicate with each other via such hopping. The mesh topology allows full peer-to-peer communication. The technology has been deployed locally in an AMR pilot project for the Water Services Corporation. In both instances, the ability of the ZigBee network to manage the propagation of data along the most appropriate path ensured the robustness of the network. ZigBee automatic reading is essentially an ideal compromise between a conventional RF system that transmits to a 100 metre, or so, distance, and a fully automated system that relays data to a base PC. With a ZigBee system data can be relayed to a pickup point, it can be downloaded in the vicinity of the transponder, or it can alternatively be relayed all the way to the base PC. The fact that the Zigbee system builds its own network as it propagates data allows for reduced costs and enhanced flexibility. The transponders also have the reliability and ruggedness of the earlier RF based modules, and are self-powered and disposable (if need be). It is understandable that major water utilities worldwide are now turning towards Zigbee to find a solution to their AMR requirements. AMR essentially solves two Apparent Loss components; meter reading errors and billing errors. It also allows for effective water accounting exercises to be implemented. By choosing a hydraulically encapsulated zone, monitoring the summated water consumptions via AMR, and comparing these values to the water intake into the zone, one can accurately compute both Real and Apparent water losses. Through AMR, comparisons can be made every few minutes, and precise computation can be made regarding the zone's meter under-registration value.

Figure 4: A ZigBee AMR Transponder Latched onto a Water Meter

Two Alternative Solutions for Controlling Water Meter Under-Registration

The last section of the paper shall look at two unique ways of reducing meter under-registration for revenue water meters that are already installed and functioning. The authors are of the opinion that three options should be in fact available, and not two. The first option is the utilization of the ideal water meter that registers all the flows passing through it, and at 100% accuracy. This water meter does not yet exist, and metering experts have serious doubts that water meters will ever measure flows down to zero litres per hour. The second option, available for indirect plumbing systems, is to utilize a roof tank valve that has an immediate closure. The third option, for both direct and indirect plumbing systems, is to utilize a flow manipulation valve.

Before looking for a solution, one must first understand the problem. All water meters have a starting flow (Q_s) at which the meter starts to register, albeit inaccurately. In Figure 5 below, this would be at around 3.75 litres per hour. The meter also has a minimum accurate flow value Q_{min}, at which the meter starts to measure fairly accurately (up to 5% inaccuracy). At around 150% of Q_{min} the water meter moves into the accurate measurement range, called the transitional flow, or Q_t. The value for the meter depicted below is 11.5 lt/Hr, at which the meter will achieve a maximum accuracy of below 2% error. This is normally retained until Q_{max}, which is double the nominal, or mid-value flow Q_n. As a meter ages its accuracy curve deteriorates, and especially flows below the transitional flow will be measured with difficulty, if at all. The challenge of both meter under-registration solutions is thus to induce flows through the consumer's meter that are above this transitional flow. Indirect plumbing systems, that is consumers with roof tanks, cause an additional problem due to the low flows generated by the tank's ball valve.

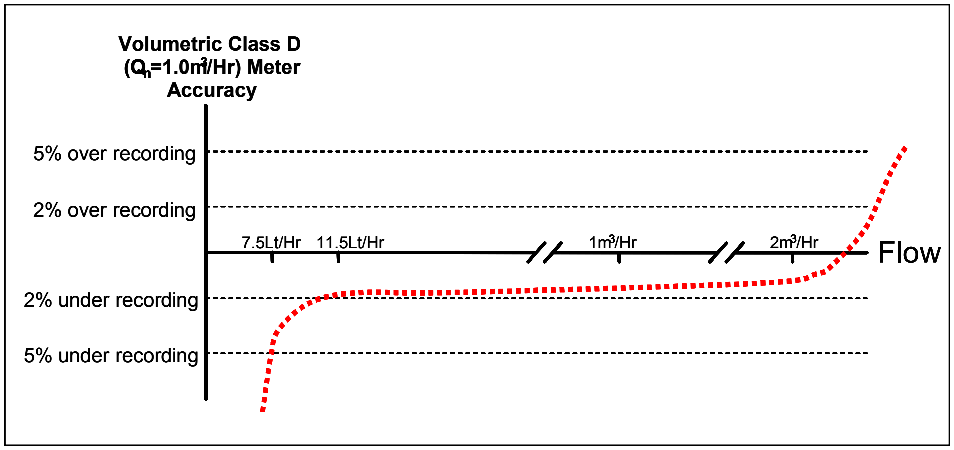

Figure 5: Accuracy profile for a Q_n = 1.0m^3/Hr Water Meter

The First Meter Under-Registration Solution: The Magnetic Water Inlet Valve.

Figure 6 depicts a magnetic water inlet valve. For consumers with an indirect plumbing system (roof tanks), this valve is the ideal engineering solution. The valve is installed within the rooftank instead of the standard ball valve. The valve features a mini float with an embedded magnet that operates a diaphragm inside the valve, not unlike that of a diaphragm-type pressure reducing valve. The rising water level lifts the valve's float, and the magnet of this float moves a stainless steel pilot plunger. This plunger, in turn, induces the diaphragm to close (or open if the water level is receding). The shut-off of the valve is almost instantaneous, allowing for either a flow in excess of 100 lt/Hr, when open, to zero litres per hour, when closed. There is simply no intermediate flow. The valve thus ensures that all flows that pass through the revenue meter are way in excess of the transitional flow. The main limitation that must be overcome with the valve is in the accessibility of consumer roof tanks. Two solutions exist; 1) legislation that enforces the use of the valve, and 2) incentives by water utilities that subsidize the valve and promote its advantages (such as its very low failure rate).

Figure 6: Two Solutions to Meter Under-Registration: The Magnetic Water Inlet Valve (Left) and the Unmeasured Flow Reducer (Right)

Possibly the most interesting aspect about the magnetic water inlet valve is in the economic dimension. The valve is inexpensive, at around €5 per valve. The cost of the valve plus that of a single jet meter roughly equals the cost of a more accurate volumetric meter. Modelling the economics of the valve for the Island of Malta (roughly a half million inhabitants), if the valve were to be installed on all consumer tanks, the local water utility would stand to gain in the region of €1.4M yearly, at no initial expense (i.e. by buying valve plus jet meter instead of volumetric meter). This would be due to the reduction in meter under-registration from a conservative value of 6% to close to 0%. The economics show the huge potential of the valve for systems with indirect plumbing. The valve is small, easy to install, and maintenance free.

The Second Meter Under-Registration Solution: The Unmeasured Flow Reducer (UFR).

For the problem of access to roof tanks, or for consumers who have direct plumbing systems, a different solution exists. The solution lies in manipulating the flow pattern of the water through the meter so that all flows that are registered are in excess to the minimum accurate flow of the meter. This solution is the unmeasured flow reducer, or UFR. At low flows the UFR causes water to pulse through the meter at flows above the minimum accurately measured flow for that meter. At higher flows the UFR opens up, allowing water to pass unobstructed. The valve works through a differential pressure concept. The valve will remain closed until the water pressure downstream of the valve is at least 0.4 bar less than the water pressure upstream of the valve. This will happen as the consumer draws water within the household. At that point in time the valve will open up, resulting in a negligible 0.1 bar head loss. Once the internal consumption stops, the differential pressure will disappear and the valve will close down again. This closing and opening of the valve occurs in bursts, or batches. Thus, in effect, the valve induces water to pass through the water meter in pulses that are above the Q_{min} of the water meter. Figure 7 on the following page shows the effect of a UFR on the accuracy of a water meter. The UFR can be installed directly upstream or downstream of the consumer meter.

In a bid to test the effectiveness of the UFR valve, the national water utility in Malta identified a small zone for pilot study purposes. The zone was chosen in accordance with the ages of the water meters in the zone, allowing for a normal distribution of meter ages with an average of five years in age.

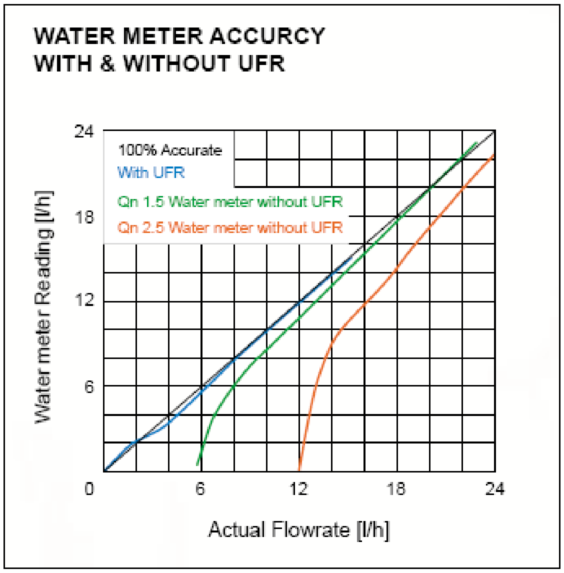

Figure 7: The Effect of the Unmeasured Flow Reducer on Water Meter Accuracy

By installing UFR devices in series with each consumer meter, the water utility could study the meter under-registration value for the zone without UFR's (i.e. by opening their bypass valve) and then with UFR's. In the pilot zone in question, application of the UFR units increased the metered volume of water by a substantial 5.5% to 6% of the water supplied to the zone (Table 1 below). If the results of applying UFR on the pilot zone are extrapolated over the complete jurisdiction of the relevant water utility, an increase in annual revenue to the tune of €1.3M would be gained.

Test	Global % Under-registration vs. Master Meter		% Overall Improvement
	UFR Bypassed	UFR Active	
1	18.1	12.1	6.0
2	26.7	21.2	5.5
3	28.0	22.2	5.8

Table 1: Effect of UFR's on Water Meter Under-Registration

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UFR – an innovative solution for water meter under registration – Case study in Jerusalem, Israel

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Abstract:

The paper will give a brief overview of how the UFR works, and will describe the trials that have been conducted in Jerusalem, before a decision was reached to install UFRs in the city. One reason for water meter under registration is the inability to measure low flow rates. The innovative **Unmeasured-Flow Reducer (UFR)** reduces the amount of water that flows below the measurement threshold (the starting flow rate) by means of changing the flow regime through the water meter at low flow rates. The UFR actually changes the flow regime through the water meter in such a way that a greater quantity of water passes through the water meter above the measurement threshold . The UFR does not affect the actual amount of water flowing through the water meter. The installation of the UFR, either upstream or downstream to the water meter, is likely to improve the measurement of water flow in that low flow rate region of measurement that, to date, water meters failed to measure (below the measurement threshold).

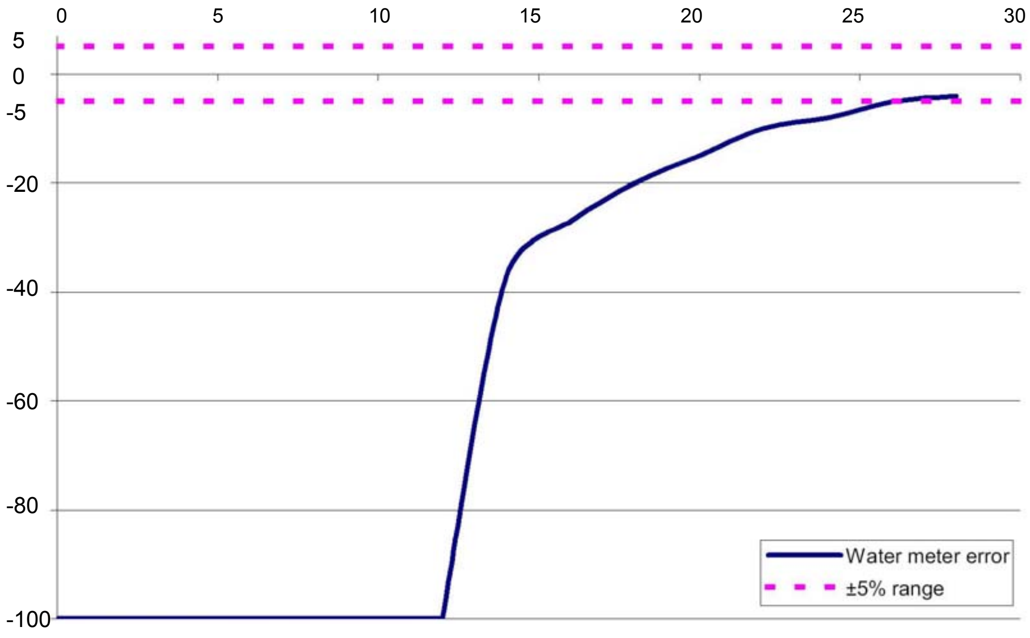
Unmeasured-Flow Reducer (UFR)

Keywords: UFR, water meter, under registration, apparent losses, measurement threshold.

Water meters at low flow rate

All water meters have difficulty in measuring very low flow rates. ISO standard (ISO 4064-1, Second edition, 1993) defines Qmin, Qt, Qn, Qmax, but it doesn't mention anything regarding the measurement threshold or starting flow rate, below which the water meter does not register flow. In the recent years, some of the water meter manufacturers have started to supply this data, but still, it relates only to water meters when they are new and doesn't take into account water meter aging. The water meters installed in the Ein Karem (Jerusalem, Israel) district meter area (DMA), are multi jet type, Qn 2.5, class B. A new multi jet, Qn 2.5, class B water meter, of the same type used in Ein Karem, was tested in the Aran laboratory: the results are shown in the graph below. The water meter doesn't measure at all up to 12 l/h so the measuring error is 100%. At 26 l/h the error curve reaches the ±5% range, defined by the aforementioned ISO standard as the permissible error between Qmin and Qt. According to this standard, for this particular type of water meter, Qmin is 50 l/h and so below this value no measuring error is defined. An innovative solution to the problem of under measurement of flow at very low flow rates is the Unmeasured-Flow Reducer (UFR), a product of A.R.I. Flow Control, in which its main goal is to reduce the unmeasured flow through the water meter and hence reduce apparent losses.

Multi-jet Qn 2.5, low flow rate error curve Flow rate [l/h]



Purpose of the study in Ein Karem, Jerusalem

The main purpose of the study was to answer the following questions:

1 Is there in fact water flow, within the DMA, that the multi jet Qn 2.5, class B water meters, as described above, can't measure?

2 Can the UFR reduce the unmeasured flow?

3 Is the contribution of the UFR to the flow registration of the water meter significant?

Unmeasured flow through the multi jet, Qn 2.5, class B water meters of the DMAs in Ein Karem, Jerusalem

In order to find out if there is unmeasured flow, passing through the multi jet, Qn 2.5, class B water meters of Ein Karem, at low flow rates, a statistical test was conducted. The test procedure is as follows:

1 Verify that the water meter leak detector is stationary.

2 Close the shut off valve, before or after the water meter. (If the valve is old or faulty it might not seal, in that case this procedure is not reliable enough to determine if there is unmeasured flow or not).

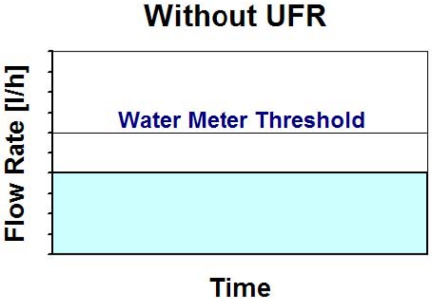
3 Wait for about 60 seconds (during this time water would have drained from the household pipework, or as a leakage or as a low flow rate flow, for instance through a float valve into a water tank).

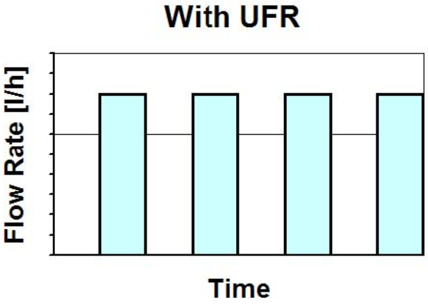
4 Open the shut off valve while watching the leak detector carefully. If there is a leakage in the household, the volume, equal to that of the drained water, will flow with enough energy to activate the water meter, and it will be seen on the leak detector. If there is no leakage, the leak detector will remain stationary.

Remark: This test is possible only for water meters with integral leak detectors.
This test showed that a large number of households have unmeasured water flows at low flow rates.
The author of this paper recommends that every water utility make this simple test, in order to understand if they have such a problem or not.
<u>Possible reasons for low flow rates</u>
Leakages from: Taps, toilet tank seals, faulty pipework.
Filling (at a low flow rate) of: Water storage tanks, toilet tanks.

Unmeasured-Flow Reducer – (UFR) What does the UFR actually do?

The UFR works by changing the way that the water flows through the water meter at low flow rates. Normally there is not enough energy in the flow to activate the water meter register at low flow rates. With a UFR installed the same flow is divided into measurable quantities of water that pass through the water meter at certain intervals, these quantities of water have enough energy to activate the water meter register and hence the flow is finally measured.

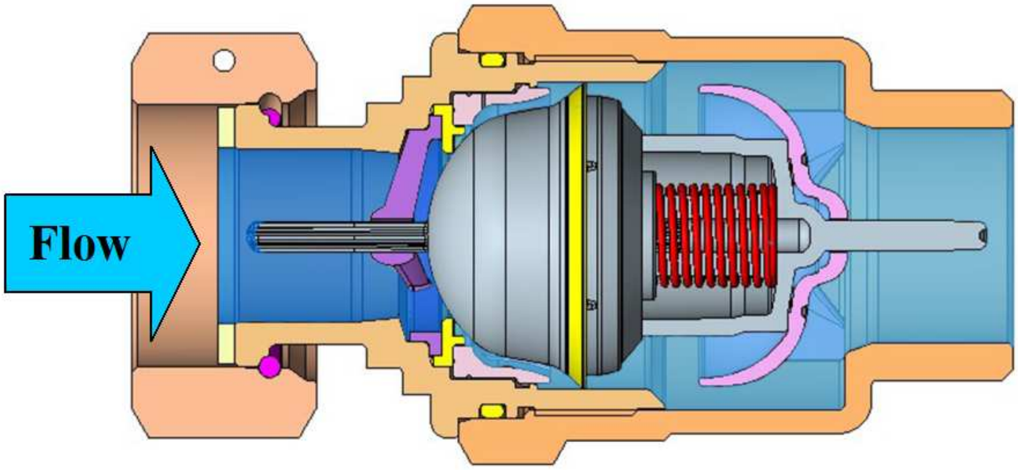




With a UFR installed, the water meter operates in cycles at low flow rates, where the water meter flow indicator (leak detector) is stationary most of the time and then rotates at regular intervals. At higher flow rates (where the water meter operates satisfactory without the aid of the UFR), the UFR detects the higher flow rate and automatically goes into the override mode (i.e. the water meter measures flow as though the UFR was not installed in the water system). In the override mode the UFR (as in the low flow rate mode) also acts as a non-return valve, i.e. it closes when the downstream pressure exceeds the upstream pressure.

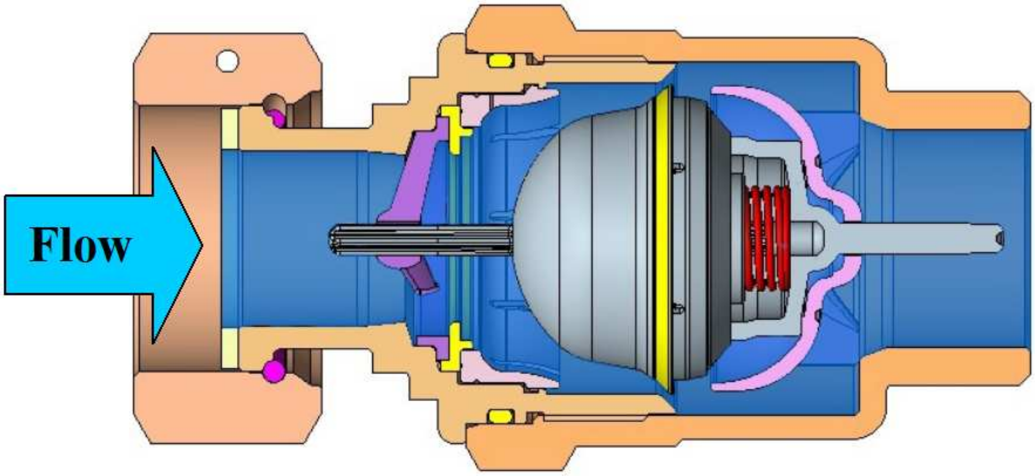
How does the UFR work?

The UFR is a differential non-return valve, designed in such a way that the pressure difference required to open it is more than that required to keep it open. The pressure difference to open the UFR is 0.4 bar, whilst the pressure difference to keep it open is 0.1 bar. When a leak develops the downstream pressure drops.



UFR closed; downstream pressure decreases because of leakage

When the downstream pressure drops below 0.4 bar of that of the upstream pressure, the UFR opens and allows for a flow rate above that of the measurement threshold.



UFR opens; downstream pressure equals that of upstream

The free flow of water through the UFR equalizes the pressure across the UFR and allows it to close. The continuing leak downstream to the UFR will make this operation repeat itself over and over again.

Every time the UFR opens, a quantity of water passes through the water meter at a flow rate above the measurement threshold of the water meter and so the flow is measured.

Installation of UFRs in Ein Karem, Jerusalem

In March 2005, 120 UFRs and 360 UFRs were installed in two separate DMAs in Ein Karem, Jerusalem. The water meters in these DMAs are multi jet, Qn 2.5 class B.

The under registration percentage was recorded prior to and after the installation of the UFRs and is a comparison of the sum of the domestic water meter readings to that of the main water meter of the DMA. The table below summarizes the results.

Location	**No. of Consumers**	**With or Without UFR**	**Period of Time [months]**	**Under-Registration [percent]**
Ein Karem First DMA	120	Without UFR	8	16%
		With UFR	6	6.1%
		Contribution of UFR		9.9%
Ein Karem Second DMA	360	Without UFR	8	26%
		With UFR	6	18.8%
		Contribution of UFR		7.2%
		Average Contribution of UFR		**8.50%**

With respect to revenue calculations, it is very important to take into consideration the following:

Per litre, the reduction of apparent losses, after the water meter, is more cost effective than the reduction of real losses before the water meter. The reasons for this are:

- The loss of revenue due to one litre of water being unregistered by the water meter is substantially higher than the cost of supplying that same litre of water into the system.
- If water is billed according to a tier system, then the loss in revenue due to water meter under-registration will be according to the highest tier billed.

Conclusions:

Leakages and other unmeasured water flows at low flow rates were found in many of the households tested.
The UFR succeeded in the reduction of unmeasured flow and was found very effective in reducing apparent losses in Ein Karem, Jerusalem.
The contribution of the UFR to the flow registration of the water meter was very significant (8.5%).
In response to this study, the water company of Jerusalem decided to install UFRs in the city.

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Meter Under-Registration caused by Ball Valves in Roof Tanks

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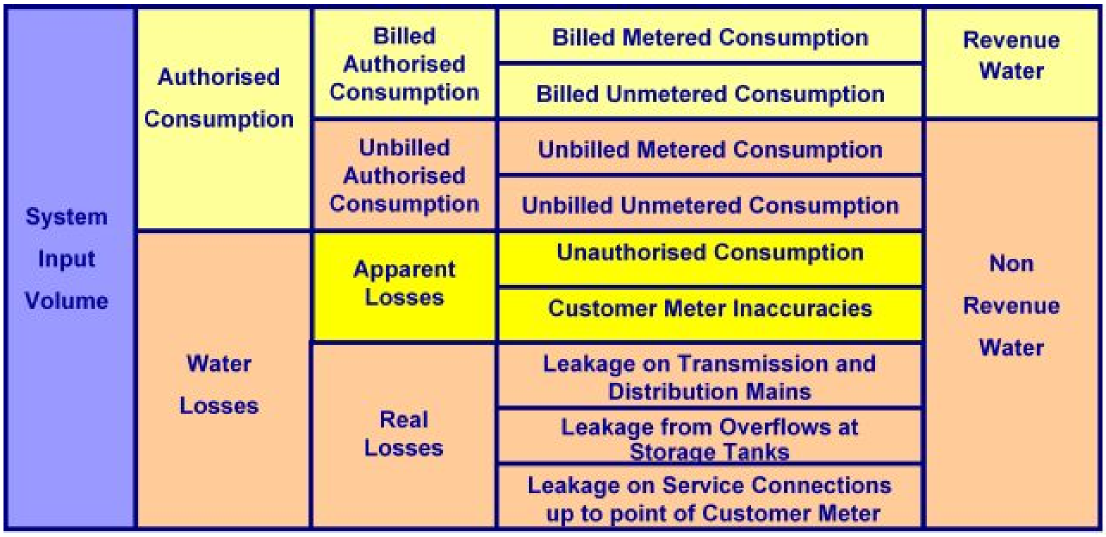
Abstract

Water Losses are made up of the Apparent and Real Losses. The Water Board of Lemesos considers that the reduction of Apparent Losses is as equally important as the reduction of Real Losses. This paper provides a detailed account of two studies that were carried out at the Water Board of Lemesos in order to quantify meter under-registration caused by the use of ball valves in customer roof tanks and the results obtained and conclusions reached are presented and discussed. The first study aimed at proving beyond any reasonable doubt that the ball valves used in customer roof tanks result in meter under-registration especially at the very low flows. The second study involved the testing of a flow manipulation device which would assist in the reduction of the un-measured flow through the customers' meters at low flows.

Introduction

The Water Loss Task Force (WLTF) of the International Water Association (IWA) has established a Standard Water Balance, which traces water from its source right through the system and derives at the end the revenue and non-revenue component. The Water Losses component of the Water Balance from urban distribution systems are made up of Apparent Losses and Real Losses. These losses together with the Unbilled Authorised Consumption comprise the Non-Revenue Water (Figure 1).

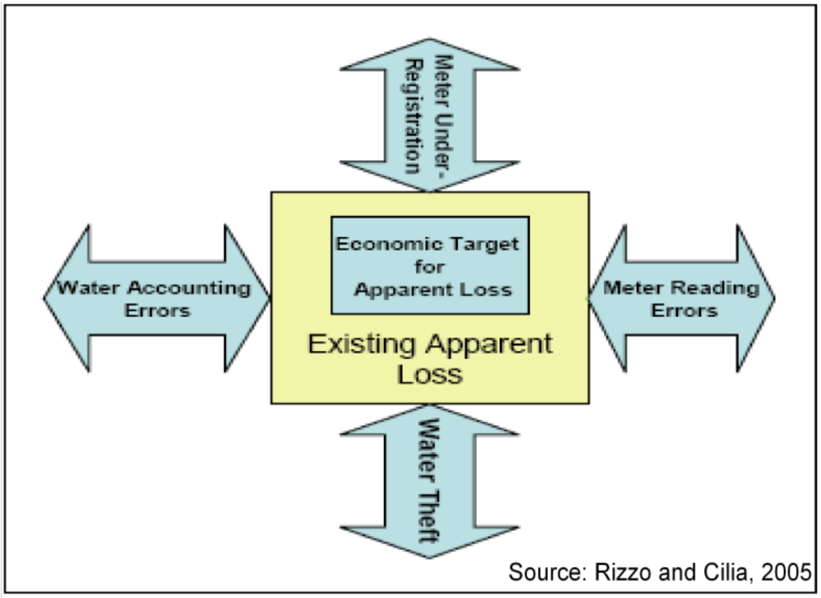


System Input Volume	Authorised Consumption	Billed Authorised Consumption	Billed Metered Consumption	Revenue Water
			Billed Unmetered Consumption	
		Unbilled Authorised Consumption	Unbilled Metered Consumption	Non Revenue Water
			Unbilled Unmetered Consumption	
	Water Losses	Apparent Losses	Unauthorised Consumption	
			Customer Meter Inaccuracies	
		Real Losses	Leakage on Transmission and Distribution Mains	
			Leakage from Overflows at Storage Tanks	
			Leakage on Service Connections up to point of Customer Meter	

Figure 1. IWA Water Loss Task Force Standard Water Balance

Apparent Losses or commercial losses as sometimes are referred to, are valued at retail billing rates whereas the Real Losses are valued at the variable cost of water production and distribution (Thiemann, R. and Henessy, S. 2005). Apparent Losses consist of water which has been produced, distributed and ultimately consumed but not paid for by the user. Members of the WLTF dedicated considerable time in the last few years working on this issue which is considered to be of the utmost importance for any water utility. A major finding was that the Apparent Losses comprise four components, namely:

- Water theft
- Meter reading errors
- Accounting errors
- Customer meter under-registration



Source: Rizzo and Cilia, 2005

Figure 2. The Four Components of Apparent Losses

The four components can act independently but at the same time interact with each other as shown in Figure 2 in either reducing or increasing the overall value of Apparent Losses. Meter under-registration is a result of the customer water meter not registering correctly the flows which go through the meter especially at very low flows. In addition, as the meter gets older the ability to register accurately degenerates (Rizzo et al, 2007).

The Water Board has developed policies for water meter management, which include the following:

- periodical checking and replacement if necessary of all source, storage and DMA meters,
- use of high accuracy positive displacement domestic meters.

The domestic meters are inspected every four months, when they are read by the meter readers, and malfunctioning or damaged meters are reported and replaced with new ones. In addition a replacement programme for ageing domestic meters is effected every year with the aim to replace meters which are over 10 years old. This means that every year about 6000 meters are replaced but a considerable number of meters over 10 years old are still in use.

Domestic Roof Tanks

In Cyprus and generally in arid and semi-arid regions, where intermittent or unreliable water supplies due to water shortage can occur all households have storage tanks of approximately I m^3 capacity. In addition advantage is taken of the abundance of sun to provide hot water to the household through the use of solar panels (Figure 3).

Figure 3. Typical Installation of Domestic Roof Tanks in Houses

The wide use of roof storage tanks by all customers appears to be a major cause of meter under-registration especially at low flows. International experience in this field indicates that even with the most accurate of domestic water meters the percentage of under-registration at low flows is significant. It was reported by Rizzo and Cilia, 2005 that *"the average under-registration of domestic meters, class "D" resulting from low flows induced by the tank ball valve was found to be at around 6% of the total household consumption".* Further evidence provided by Cobacho, et al, 2007 showed that in the case of a system using roof tanks and employing Class "B" water meters the overall under-registration was close to 20% during the initial years becoming much worse after 6-8 years with the under-registration reaching 30% and more.

The problem of meter under-registration is normally caused at extremely low flows through the meters, flows which are lower than the minimum flow which can accurately be registered by the meter. For metrological class "D" the minimum flow that can accurately (+/- 2%) be registered by the meter is 7,5lt/hr.

The roof tank is used to store water which subsequently is used for toilet flushing, showering, cooking, clothes and dish washing, house cleaning and for personal hygiene. For most of these uses the quantity used is relatively small causing a minute drop in the ball valve arm in the roof tank thus allowing water to drip into the tank at flows lower than the minimum flow that can be registered by the meter. Two shapes of roof tanks are in use; square (1m x 1m x 1m) which is made of galvanized mild steel and cylindrical (0,8m diameter x 1,5m long) which is made of polypropylene. For the square tanks a drop of 1mm in the level of water in the tank is equivalent to 10lt of water being used. Similarly, a drop of 1mm in a cylindrical tank is equivalent to 5lt. For most households uses the quantity of water used is less than 10lt or even 5lt and therefore the drop in the ball valve arm is not sufficient to allow flow conditions through the meter which can be registered.

The galvanized mild steel tanks proved problematic in that they were showing signs of corrosion soon after they were installed and had very limited life span. The galvanized mild steel tanks are no longer produced and are gradually being replaced by polypropylene tanks (Figure 4).

Figure 4. Polypropylene Roof Tank

The studies undertaken at the Water Board of Lemesos were inspired by the work carried out in this field by the Apparent Losses Team of the WLTF and aimed at quantifying meter under-registration caused by the ball valves in roof tanks. These were:

- Study A: Continuous monitoring of inflows and outflows from roof tanks.
- Study B: Installing flow control devices (unmeasured flow reducers).

Study A: Continuous Monitoring of Inflows/Outflows from Roof Tanks

The aim of the study was to:

- Establish that the use of the ball valves in roof storage tanks causes meter under-registration, especially at the very low flows.
- Quantify meter under-registration under several supply regimes.
- Determine the loss of revenue due to meter under-registration.

This study was carried out based on the methodology and principles of a similar study carried out by Rizzo and Cilla, 2005, in Malta where roof tanks are also used in much the same way as in Cyprus. Rizzo and Cilla, 2005, in their study proved that by using an innovative solenoid system to replace the traditional ball valve, all flows into the roof tank were accurately registered by the domestic meter without any indication of under-registration. Therefore, this aspect was not investigated in the Water Board study since the results of the Maltese study are considered universally applicable. It was therefore considered important to investigate and quantify meter under-registration for the typical roof tank installations encountered in Cyprus and particularly in the town of Lemesos which are similar to those found in other countries.

Selection of Project Site

It was considered important to investigate meter under-registration for different types of customers in order to establish whether the different patterns of customer usage are related to varying degrees of meter under-registration. To this end a Shop, a Residential Flat and an Office all located in the same multi storey building were chosen (Figure 5).

Figure 5. Project site

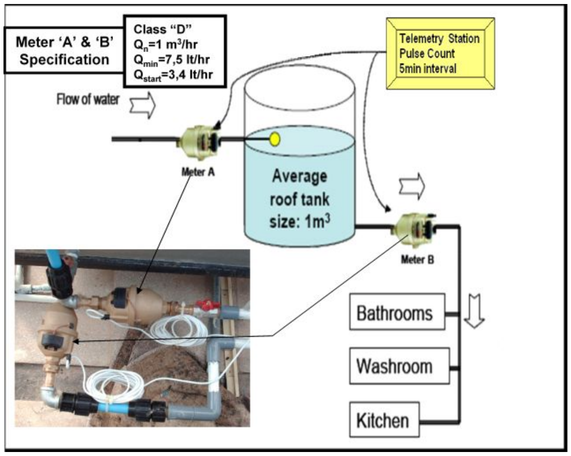


Figure 6. Test layout

Test Methodology

The methodology adopted was to install brand new class"D", Q_n=1m^3/hr, volumetric meters at the inlet and outlet to the tank as shown schematically in Figure 6 in order to measure the inflows and outflows from the tank. The inflows and outflows were logged at every 5 minute intervals and the data stored in a programmable controller which was set up on site on the roof of the multi storey building. This arrangement was adopted for each customer type under investigation, namely: shop, residential flat and office.

Results

The results obtained from the continuous monitoring of inflows and outflows are shown in Table 1 below. The measurements were taken during the period 20/11/2006 and 4/2/2007 and it is obvious from the results that the degree of meter under-registration varies according to the type of dwelling which of course is directly related to the customer's consumption pattern. The highest under-registration percentage was that of the water meter supplying the Office.

Table 1. Results of inflow and outflow readings

Roof Tank	Type of Dwelling	Period of measurements: 20/11/2006 – 4/2/2007			
		Inflow (m^3)	Outflow (m^3)	Difference(m^3)	% Difference
1	Office – 2nd floor	0,9525	1,2080	0,2555	21,15
2	Shop – ground floor	7,2800	7,5276	0,2476	3,29
3	Residence- 1st floor	20,7740	21,0990	0,3250	1,54

It should be noted that the actual water consumption for the Office was extremely small compared to the other two which show a much lower percentage of meter under-registration. The low consumption for the Office was due to the fact that water in the Office was basically used for toilet flushing and/or hand washing which meant that the quantity of water used at any time was small causing only a minute movement of the ball valve arm thus water going through was below the minimum volume that the meter was capable of accurately registering. In contrast to the other two cases where water was used in larger quantities and under different consumption patterns resulting in much lower figures for under-registration.

The above tests proved that the use of the ball valves in roof storage tanks cause meter under-registration, especially at the very low flows, and that the extent of the under-registration is closely related to the customer consumption pattern. On the basis of the above tests the average figure calculated for under-registration was 2.8%.

Study B: Installing Flow Control Devices (Unmeasured Flow Reducers)

The aim of this study was to:

- Install and test innovative flow control devices, Unmeasured Flow Reducers (UFR), based on an agreed methodology, in order to establish their functionality under field conditions.
- Investigate if the use of UFRs creates transient pressures in the network.
- Measure consumption before and after the installation of the UFRs.
- Quantify volume and cost of meter under-registration caused by the low flows induced by the ball valves in the roof storage tanks and calculate the benefit using the UFR.

In order to have a common base for comparing results from different case studies a standard test methodology established by Alex Rizzo, leader of the Apparent Losses team of the WLTF was followed. This involved a total of 4 sequential steps:

1. Selection of project site and consumer audit.
2. Pilot zone main meter.
3. Elimination of all other Apparent Loss components.
4. Installation of UFRs.

The above steps are analysed below giving details of the work which was carried out and the results obtained.

Selection of Project Site and Consumer Audit

A trial zone with 69 metered customers was chosen. This area was hydraulically encapsulated with a single entry point into the zone which was metered. The area comprised mainly large detached houses with an average daily consumption of approximately 1 m^3 per household.

Pilot Zone Main Meter

The flow into the pilot zone was initially measured by a Kent 50mm Waltmann type class ‘B’ meter. In order to ensure that the main meter was correctly sized the meter was logged for a period of time and the flow data analysed. As it can be seen from Figure 7 the meter was oversized and as a result the meter was registering zero for minimum night flows. Arrangements were made and the main meter was replaced with a Kent 25mm PSM class ‘C’ meter. The flows were logged for a similar length of time immediately after installation and as it can be seen from Figure 8 the meter was measuring correctly, within the minimum and maximum limits of the meter. From these measurement it was also concluded that there were no leaks that had to be removed as the minimum measured night flow of 0,2 m^3/hr was very close to the minimum target night flow for this particular trial zone.

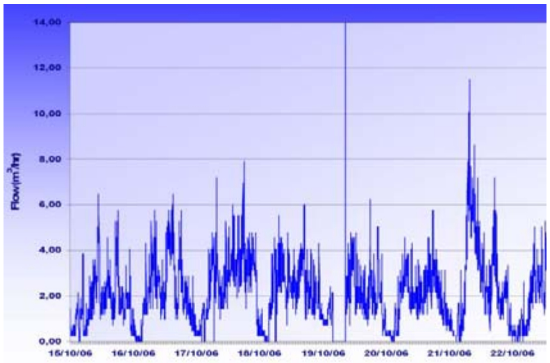


Figure 7. Kent 50mm waltmann type type ‘B’

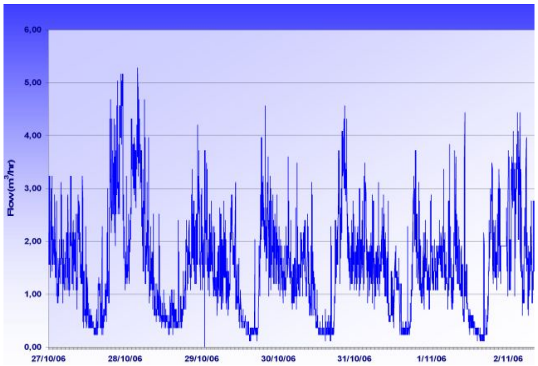


Figure 8. Kent 25mm PSM class ‘C’

Elimination of All Other Apparent loss Components

It was extremely important to remove all other apparent loss components, namely: water theft, billing errors and meter reading errors prior to carrying out the test. Before installation of the UFRs all meters were checked and the ones found malfunctioning were replaced. Furthermore, it was confirmed that there were no illegal connections. In order to avoid any billing errors it was decided that a separate recording system will be used which will be independent of the main billing system. Unfortunately it was not possible to install an automatic meter reading system so that all customer meters to be read manually. In order to avoid meter reading errors the meters were read simultaneously by two people cross checking their records after each meter was read.

Installation of UFRs

Great emphasis is now placed on minimising surges in distribution systems so it was thought important to have independent pressure measurements on the distribution mains to determine the size (if measurable) and frequency of any surges that may be induced by the UFRs. An appropriate fire hydrant location in the trial zone area was chosen and pressure measurements at 0,1 second intervals were taken before and after the UFRs' installation. Figure 9 shows the measurements after the UFRs were installed and as it can be seen no pressure surges were recorded. Therefore, it can be safely assumed that no surges are induced in the network by the use of the UFRs.

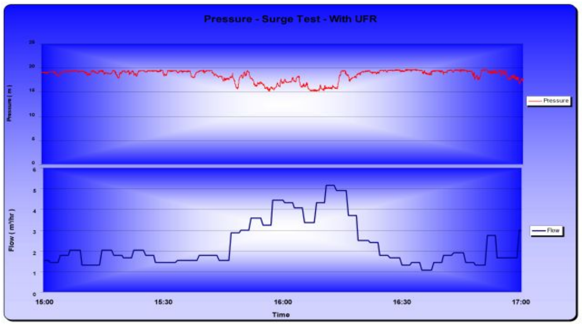


Figure 9. Pressure measurements after installation of UFR

Figure 10. UFR installetion

The UFRs were installed immediately downstream of the water meters as shown in Figure 10. During the installation of the UFRs some difficulties with limited space were encountered especially in cases where the inlet and outlet pipework were fixed in concrete paving slabs. Of course the problem of space will not be an issue with new installations.

Results

The flows through the main meter were recorded automatically, stored in a programmable controller on site and transferred to a computer in the main Offices of the Water Board at preset times or on request. The domestic meters were manually read three times a week, Monday, Wednesday and Friday, for two consecutive weeks with the UFRs and for another two consecutive weeks without the UFRs. The difference between inflows and outflows without and with UFRs are shown diagrammatically in Figures 11 and 12 respectively. It is evident from the graphs that the difference is a lot less with UFR than without.

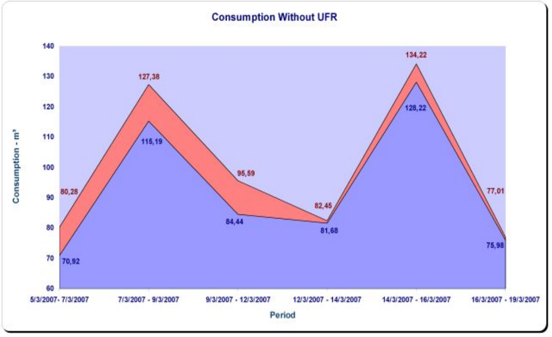


Figure 11. Difference in registered flows without UFR

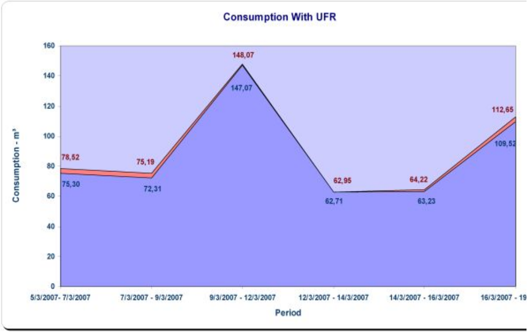


Figure 12. Difference in registered flows with UFR

The above results are shown in tabular form in Table 2 below. Without the UFRs the meter under-registration was 6,79% whereas with the UFRs this figure was reduced to 2,12%. The use of the UFRs increased the volume of water which was registered by the water meters by 4,67%.

Of course the above percentage may vary according to the type and age of meters that are installed. Low percentages are usually encountered with positive displacement meters than with multi jet or single jet meters. Similarly the percentage under-registration is lower with newer meters as compared to older ones.

An analysis of the trial zone meters was carried out which revealed that all 69 meters were positive displacement meters, 43 (62%) Class ‘D’ and 26 (38%) Class ‘C’. Also the age of the meters was analysed and showed that 26 (38%) were 1-3 years old, 7 (10%) were 4 -7 years old, 16 (23%) were 8 -11 years old and 20 (29%) were more than 11 years old.

Table 2. Results of inflow and outflow readings

Period	Main Meter consumption (m^3)	Customers’ usage (m^3)	Under-registration	
			(m^3)	%
Without UFR 21/3/2007- 2/4/2007	596,93	556,43	40,50	6,79
With UFR 5/3/2007 – 19/3/2007	541,60	530,13	11,47	2,12
			Additional Registration with UFR	4,67

Therefore, bearing in mind the above figure for meter under-registration without UFRs it can be safely assumed that the Apparent Losses figure for the Water Board of Lemesos is of the order of 7% of the measured inflows into the network. This figure is extremely is extremely important in the calculation of the Standard Water Balance and in the accurate calculation of the Real Losses using the ‘top down’ approach.

Benefits

Table 3 shows the additional volume of water which will be registered in the trial zone in a period of 1 year based on the additional registration of 4,67% using the UFRs as well as the additional income based on the consumption tariff applicable to each customer on the basis of the 4 monthly bills issued in the year 2006. The cost of supply and installation of the 69 number UFRs was estimated at €1400 and the benefit due to the additional revenue was estimated at €2100 per year. Therefore, for the trial zone the payback period would be 8 months.

Table 3. Cost - benefit analysis for trial zone

Description	Value
Additional annual volume of water	950 m^3
Additional Annual Income	€ 2100
Supply and installation of UFR	€ 1400
Pay back period	8 months

Applying the above concept across the entire Water Board using an average consumption of 25 m^3/month at an average rate of 0,60 €/m³ it gives a payback period of 28 months per customer. In addition on the basis of 70.000 customers the Board will have an additional income of the order of €600.000 per year.

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Snapshot ILI - a KPI-based tool to complement goal achievement

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Abstract:

Successful goal attainment within specific timeframes will undoubtedly remain strongly a function of the quality of a number of decisions taken throughout a project's life. Given the severe monetary penalties that emanate following a failure to adhere to targets, it is imperative that the worth of each decision is capable of being assessed in real time. This will serve as a necessary guidance to allow for the adjusting of tactics and steering of the effort towards successful goal achievement within stipulated timeframes. The importance of such a strategy arises in multi dimensional scenarios requiring judgment and is further magnified when high inertia/slow response situations are being tackled as in the case of the water industry. The paper will attempt to clarify the requirements of leakage control and then build up to spell out the management problems that arise whilst attempting to see a project to completion. A solution as adopted by the Maltese water entity, referred to as the Snapshot ILI has been built around the concept of the Infrastructure Leakage Index as defined by Allan Lambert. The paper is intended to shed light on possible solutions that allow real time analysis of the worth of freshly taken decisions.

Keywords:

Goal attainment. Real time control. Leakage control. Infrastructure Leakage Index (ILI). Snapshot ILI.

Understanding the Problem:

The well-known 'four force' methodology is being displayed in Fig 1 overleaf, conveniently blown up into eight separately managed forces. Concentration of efforts in these separate forces has to be continually juggled by the water control manager in an attempt to finally achieve a pre-set state of leakage levels. Forces that are time sensitive are also being shown in purple as this phenomenon adds to the complexity of the task. It is obvious that any decision taken will positively or negatively affect the leakage outcome in the long run. Three key parameters for the attainment of goals are the structure setup, 'closed loop control', and facilitating tools as can be seen in Fig 2. These three parameters must be present to allow the Water Manager to effectively implement tactics he deems appropriate in attaining a desired future state of leakage.

Any managerial decisions are taken with the primary aim to attain corporate goals. These decisions are obviously guided by constraints as directed by the policies and ethics of the water entity. The blue line in figure2 represents the implementation process and has to allow for the 'easy' implementation of tactics as set by the water manager. Annual reports for the Maltese Water Services Corporation 2003/04/05/06 delve well into the strategy implemented to ensure that the proposed tactics are appropriately delivered with the minimum possible problems.

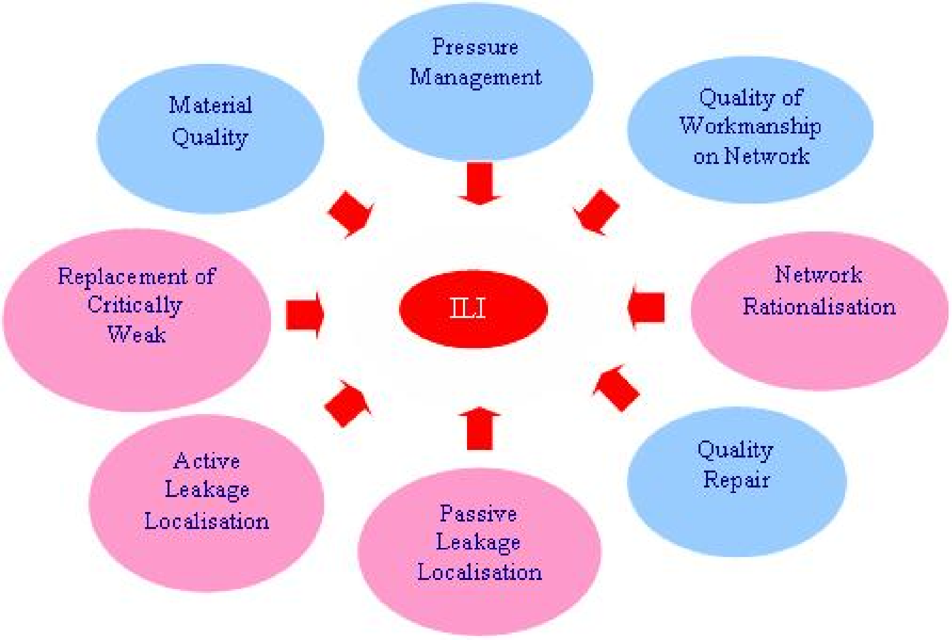


Figure 1 The eight components attributable to leakage reduction

Having the implementation line sorted out, a water entity is faced with the closed loop feedback problem. Any operator, whatever the size, is considered as resembling a slow responding, high inertia model. Having said that, results emanating from implemented tactics seldom reflect the purpose of that decision due to the time function. This occurs as a result of the complexity and interaction of the numerous problems that are synonymous with any water entity. A water manager may for example focus upon successfully reducing leakage in a particular zone but in the meantime experience an overall increase in leakage over the whole area under his jurisdiction. He may also decide to implement a wide network replacement program in a particular area, whereby the entity would have reaped better gains from a similar investment implemented on a different area. It is thus imperative that the worth of decisions taken along the period in question are easily assessed, so as to allow for adjustments in tactics as necessary for the successful attainment of the set targets before it is too late. Whereas loop 1 takes care of the latter problem, loop 2 as shown in Figure2 is strategically designed to address the issue of accountability and self directed teams. For goal theories to become effective, the staff responsible for any part on the network have to know the past, the present and the expected future of the water network as dictated by the stake holders. Without this, the desired state required to be achieved would remain unknown, and no astonishing results can be expected. It is the role of the water manager to design and supply the necessary facilitating tools to the hands on staff whereby the goals set for each zone and the difference between the present and the desired state of leakage be made tangible. The second loop thus comes into play and adjusts the efforts carried out at the lower levels of the entity’s structure. The network of even the smallest water entity is huge and complex, and this is where the need for the facilitating tooling component comes in. Calculating the ILI of a water entity may be ideal to compare to international standards, or to calculate the gains in leakage reduction over a period in question. However a yearly computation of ILI can only act as praise for decisions well taken or a quake when the leakage values do not follow the desired downward trend.

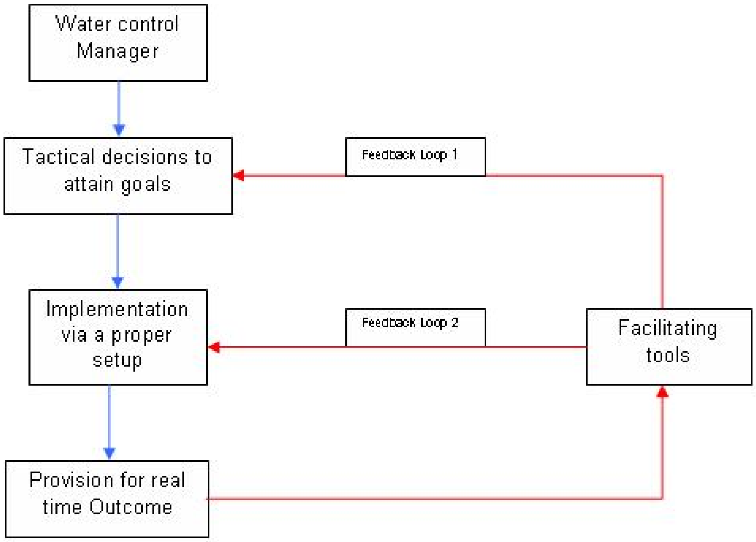
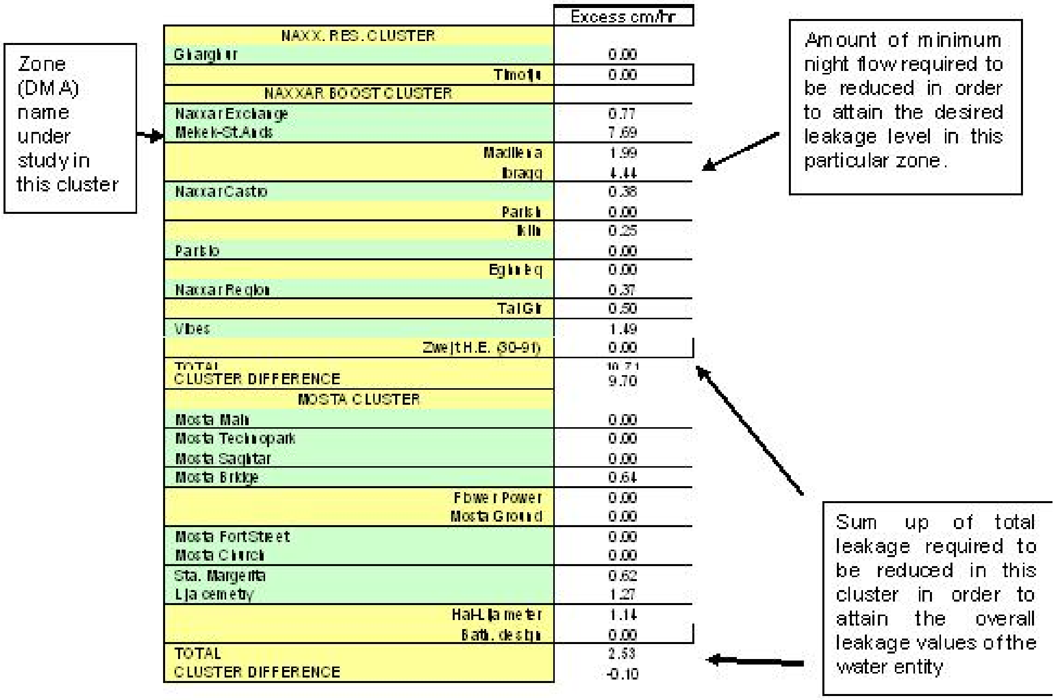


Figure 2 Implementation facility and control provision – key parameters.

Monitoring and control – the Strategy

A. Lambert's ILI concept has been extended in a manner to act as the tool that facilitates the complexity of the size of an entity's network and also to picture the past, present state and discrepancy between the present and the desired state of leakage. The complexity faced by any water operator originates mainly from the size of the network, so breaking down a complex problem into more manageable portions is a must if a water manager is to really manage the network proactively. The scope of this tool is that to calculate the instantaneous (hence nicknamed snapshot) ILI on a zonal basis. The global ILI will thus comprise the summation of all the zone snapshot ILI's. In the case of Malta, the whole island has been divided into 300 zones, each having their own respective snapshot ILI being calculated on a weekly basis. Along with this, the desired Minimum Night Flow (MNF) per zone and hence desired state of ILI is also available as can be seen in Fig5. In this manner the targets are not just set for the whole water network, but for each individual zone. In addition to this, there is a continuous plot of the MNF of each zone on a weekly basis, thus depicting the historical trend, the present MNF value, and the desired MNF required to achieve the targeted ILI. Depicted below is an extract of the tool used at the Maltese water operator. The zones, more commonly known as district metered areas (DMA's), that require prime attention can be easily identified and can be used as a tactical tool to identify priority areas and help focus efforts where the highest gains are most likely to be attained.



Zone (DMA) name under study in this cluster

	Excess cm/hr
NAXX. RES. CLUSTER	
Gharghur	0.00
Timoju	0.00
NAXXAR BOOST CLUSTER	
Naxxar Exchange	0.77
Mekek-St.Ands	7.69
Madliena	1.99
Ibragg	4.44
Naxxar Castro	0.38
Parish	0.00
Iklin	0.25
Parklo	0.00
Eghineq	0.00
Naxxar Region	0.37
Tal Gh	0.50
Vibes	1.49
Zwejt H.E. (30-91)	0.00
TOTAL	10.71
CLUSTER DIFFERENCE	9.70
MOSTA CLUSTER	
Mosta Main	0.00
Mosta Technopark	0.00
Mosta Sagittar	0.00
Mosta Bridge	0.64
Flower Power	0.00
Mosta Ground	0.00
Mosta Fort Street	0.00
Mosta Church	0.00
Sta. Margerita	0.62
Lija cemetry	1.27
Hal-Lija meter	1.14
Bath. destn	0.00
TOTAL	2.53
CLUSTER DIFFERENCE	-0.10

Amount of minimum night flow required to be reduced in order to attain the desired leakage level in this particular zone.

Sum up of total leakage required to be reduced in this cluster in order to attain the overall leakage values of the water entity

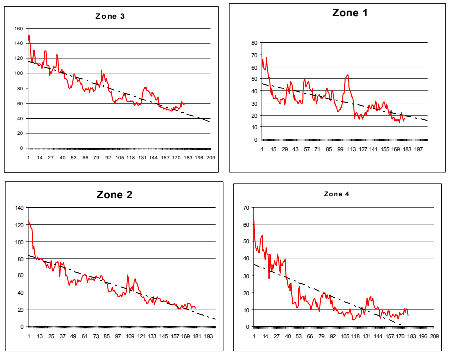
Figure 3 Quick visual tool highlights zones in need of urgent attention

A most common misconception amongst water managers is that zones that carry the highest MNF values should be in fact tackled with priority as this is where the largest gains can be achieved. Given the ‘allowances’ of legitimate night consumption and the unavoidable real losses in the ILI calculation, there have been surprising instances whereby zones carrying a high night line were in fact already very close to the required ILI targets. This is where sustained efforts in such zones becomes a waste of time and resources, as going far below an ILI of 1.5 becomes in itself a physical impossibility. An understanding of the snapshot ILI of each zone thus makes more sense in deciding which zone can benefit most with the least effort.

Strategically breaking down a complex problem into manageable tasks

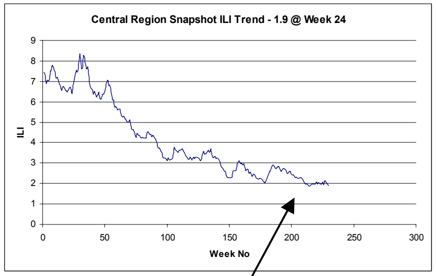
The trend plot of number of metered zones (DMA's) under the responsibility of one team leader. Together these make up the ILI of one cluster

Summation of the 4 clusters make up the total ILI of the area under jurisdiction



The area under Jurisdiction divided into 4 separately monitored clusters, each falling under the responsibility of one team leader and comprises a number of hydraulically metered zones

Overall ILI trend of area under Jurisdiction



Target line for an ILI of 1.5 depicts the planned target

Figure 4 The breaking down of a complex problem into more manageable chunks

All zones (DMA's) in a particular cluster

Snapshot ILI on a zonal basis

This zone is already below the required ILI value of 1.5, and so no effort is required at this point in time

Zone/Cluster Name	Mean Zone Cons m^3/hr	MNF (min) m^3/hr	MNF (median) m^3/hr	No of Accnts	Length of MAINS km	Night Cons l/conn/hr	Leak Qty (min) m^3/hr	Leak % of Mean %	Leak Qty in l/p/hr	Leak Qty (median) m^3/hr	Service Density		No of conn	ILI @ present	Target MNF(min) m^3/hr	Mnf to reduce m^3/hr
NAXXAR BOOST CLUSTER																
Naxxar Exchange	5.7	1.4	1.5	347	8.17	2	0.65	0.11	1.87	0.74	42.46		306	1.13	1.63	0
Mekek-St.Ands	25.2	8.6	9.1	1528	22.62	2	5.08	0.2	3.33	5.54	67.56		1088	2.73	6.11	2.49
Madliena	11.6	3.6	3.8	555	12.04	2	2.28	0.2	4.11	2.47	46.08		461	2.67	2.51	1.09
Ibragg	14.8	5.6	6	971	9.94	2	3.35	0.23	3.45	3.72	97.71		625	3.39	3.56	2.04
Naxxar Castro	17.4	6	6.3	1469	16.73	2	2.81	0.16	1.91	3.08	87.82		1133	1.59	5.82	0.18
Parish	10.6	3.2	3.4	959	10.26	2	1.18	0.11	1.23	1.36	93.45		730	1.05	3.75	0
Iklin	3	1	1.2	154	4.05	2	0.63	0.21	4.12	0.82	37.99		143	2.33	0.75	0.25
Parisio	21.5	6	6.1	1992	14.76	2	1.85	0.09	0.93	1.94	134.94		1460	0.88	7.4	0
Eghnieq	9.5	2	2.1	877	5.26	2	0.23	0.02	0.26	0.32	166.69		594	0.27	3.11	0
Naxxar Region	18.7	5.2	5.5	1371	16.4	2	2.25	0.12	1.64	2.53	83.61		1036	1.37	5.43	0
Tal Gir	8.3	2.1	2.3	526	5.84	2	0.96	0.12	1.83	1.14	90.04		470	1.37	2.2	0
Vibes	5.6	4.6	5.2	1399	9.23	2	1.65	0.29	1.18	2.2	151.62		901	1.28	4.91	0
Zwejt H.E. (30-91)	0	0	0	497	3.2	2	0		0	0	155.28		281	0	1.67	0
CLUSTER SUB-TOTALS:	94.1	31.8	33.7	8106	87.9	2	14.29	0.15	1.76	16.03	92.22		5925	1.55	31.3	2.68

The MNF of this zone has to be reduced by 2.04 cum/hr if it is to attain an ILI value of 1.5

MNF required to be reduced to attain an ILI value of 1.5 in this cluster

Total leakage in cluster (URL + ARL)

Snapshot ILI value of cluster under study

Figure 5 The tool's inputs and outputs

Creating the tool

Using A.Lambert's Key performance indicator for leakage

$$ILI = CARL / UARL,$$

Removing the A (as this stands for Annual) to attain a snapshot value we get

$$ILI = CRL / URL$$

Where CRL = (MNF – (Accounts x Legitimate Night Consumption)) x Leakage Factor

And URL = ((18 x KM of Mains)+(0.8 x Connections)) x Average Zone Pressure / 2400

Thus,
the MNF required to achieve a particular ILI can be calculated for each zone using the following equation:

$$MNF (ILIa) = (ILIa \times ((18 \times KM) + (0.8 \times C)) \times AZP / (2400 \times LF)) - (ACC \times LNC)$$

Where

MNF (ILIa) is the required MNF for a particular ILI
ILIa is the target ILI as set by the Water operator/authority
Km is the length in Km of mains in that Zone
C is the connections in that Zone
AZP is the average zone pressure
LF is the leakage factor
ACC is the number of accounts in that Zone
LNC is the legitimate night consumption.

Limitation and conclusions

The requirements necessary to implement such a tool will prove to be the major bottlenecks during the construction stage. A proper GIS system is necessary to be in place a priori whereby all the households and accounts are accurately geo-coded on a base map. Superimposed on this map should lie the boundaries of the zones together with the network infrastructure of the water operator. This would eventually permit an accurate analysis in calculating the Snapshot ILI formula requirements by performing a simple query. This zone data is inputted into the tool, and following weekly MNF entries, the snapshot ILI calculation of the leakage situation in every zone for every particular week is achieved. Although Lamberts calculations become somewhat doubtful for zones comprising accounts lower that 5000, this tool has proved ideal to guide the water Manager in accomplishing set targets.

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Practical experiences in applying advanced solutions for calculation of frequency of intervention with Active Leakage Control: results obtained

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Keywords: Flow Monitoring, Pressure Management, Active Leakage Control

Abstract

Due to the activities of the Water Loss Task Force, Utilities are becoming increasingly encouraged to measure inflows to small distribution systems, and to split larger systems into Zones with measured inflows. Sometimes, pressure measurements will also be taken in the Zones.

However, in the experience of the Authors, there are two common problems. Firstly, new potential users are often discouraged by the high initial cost of permanent metering and data transmission, and concerns that the water loss management opportunities may not justify the high initial costs. Secondly, new users are usually unaware that pressure measurements at a few carefully selected locations, taken together with the flow data, allows rapid quantification of water loss management opportunities without the need for setting up detailed network analysis models.

The paper explains, step by step, the type of broad conclusions that can be obtained from occasional reliable measurements of Zone inflows with portable equipment, over several days at carefully selected times of year, without any pressure measurements. Then, the additional conclusions and predictions that can be obtained from pressure measurements at a few selected specific locations (Inlet Point, Critical Point, Average Zone Point) are described, together with a simple test to assess the relationship between pressure and leak flow rate for the Zone. The paper also describes a low cost data transfer from the measurement sites by e-mail to any chosen recipient, and a software that allows users to quickly identify opportunities for water loss management by pressure control and/or active leakage control at an economic frequency of intervention.

Introduction

Non-revenue water (NRW) is a common problem to all utilities all over the world. In Italy NRW levels range from 15-60% of total system input volumes, the average being 42% (ISTAT 2003). Some European countries – notably the United Kingdom and Malta – have fully sectorised distribution networks, with continuous night flow measurements, and frequent interventions to locate unreported leaks. In Italy however, the majority of water utilities only repair 'reported' leaks, and do not practice any regular form of active leakage control or pressure management, except perhaps as an emergency response during droughts.

Minimisation of losses in the network is a key requirement, in particular in those countries where the water loss levels are very high.

The starting point

In an effort to better manage water loss from the networks, regulators are looking at new legislative measures to require water utilities to report their water loss. With these moves underway, there is an urgent need for water managers to gather information and to use tools for implementing such requirements.

Lack of information regarding the advantages and economic benefits, of a correct approach to water loss management using the most up to date concepts, technologies and software, often represents a barrier for the Utilities managers and delays necessary actions.

For this reason it is important to:

- Increase water utilities' awareness of the operational and economic benefits of improved pressure management to reduce new burst frequencies and leak flow rates;
- Disseminate the practical approach developed by the IWA Water Loss Task Force to a wide number of potential end-users, to encourage and motivate;
- Communicate and transfer available methodologies and innovative technologies for efficient water loss management, allowing end-users to make contact with each other and exchange ideas and experiences;
- Assist water utilities to identify both short and long term economic investment policies, using practical methodologies and accurate measuring instruments.

Economic Frequency of Active Leakage Control

A stated key objective of the present Water Loss Task Force (Liemberger and Farley, 2004) is 'to develop a quick and practical method for calculating economic intervention (for active leakage control to locate unreported leaks and bursts), and short-run economic leakage level (SRELL).

Clearly, there is little point in attempting to calculate, or to achieve, an economic level or real losses for a particular system, unless the Utility commits to undertaking (to an appropriate extend) all four components of real losses management (Speed and Quality Repairs, Pressure Management, Active Leakage Control and Rehabilitation). (Figure 1.1).

Pending the development of a method for calculating economic leakage levels, a practical approach successfully used by Utilities such as Malta Water Service Corporation and Halifax Regional Water Council (Canada) has been introduced to identify and implement a mixture of initiatives within the 4 components that individually have the highest benefit: cost ratio or shortest payback period. When no further economically viable initiatives can be identified, it can be reasonably assumed that an economic leakage level has been achieved – although it must be recognised that the economic leakage level will change with time.

Using assumption similar to economic stock control theory, Fantozzi and Lambert (2005) showed that, for a basic active leakage control policy based on regular survey, the economic frequency of intervention occurs when the cost of a 'full system' intervention (excluding repairs costs) equals the value of the unreported leakage volume. Thus the economic period between interventions (Te in days) and the necessary operational budget can be calculated accordingly.

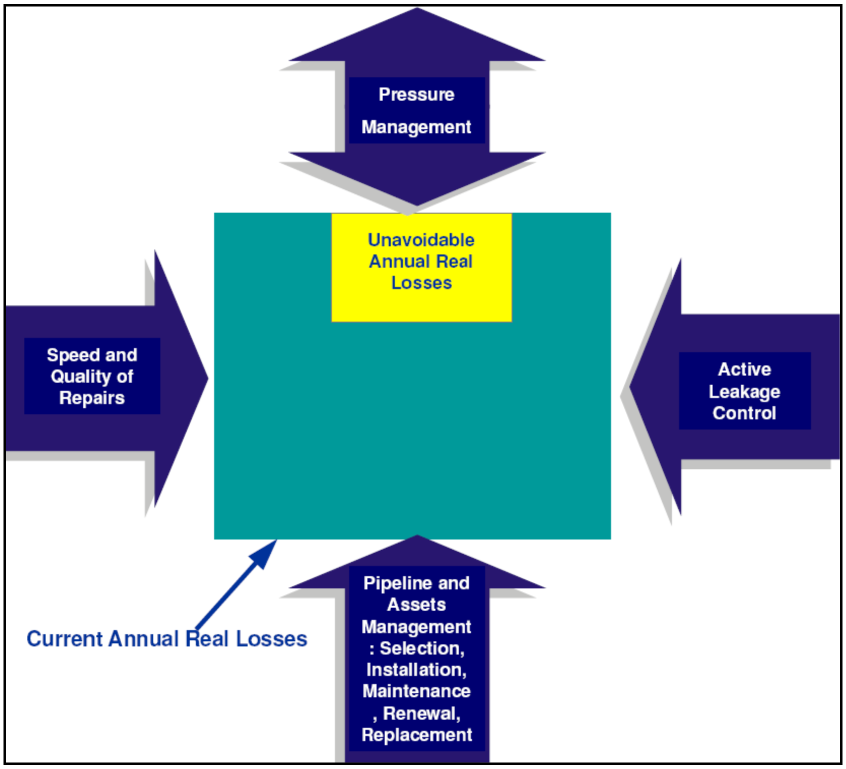


Figure 1.1 The Four Components Approach to Management of Real Losses.

A case study

This case study relates to a small system in Northern Italy with 1300 service connections, 23,2 Km of mains and with no permanent inflow metering (Fantozzi et Lambert 2006).

The following description clearly demonstrates that through the practical application of advanced methodologies and the use of the newest instrumentation for the acquisition and transmission of data from the field, 'hidden' leakage problems can be quickly identified and a significant improvement in the efficiency of this distribution system can be rapidly achieved. It is hoped that this case study will encourage Utilities in other countries to improve their performances using a similar quick, effective and low-cost approach.

Figure 2.1 shows the Montirone network and chosen specific monitoring points where temporary flow meters and pressure gauges were placed.

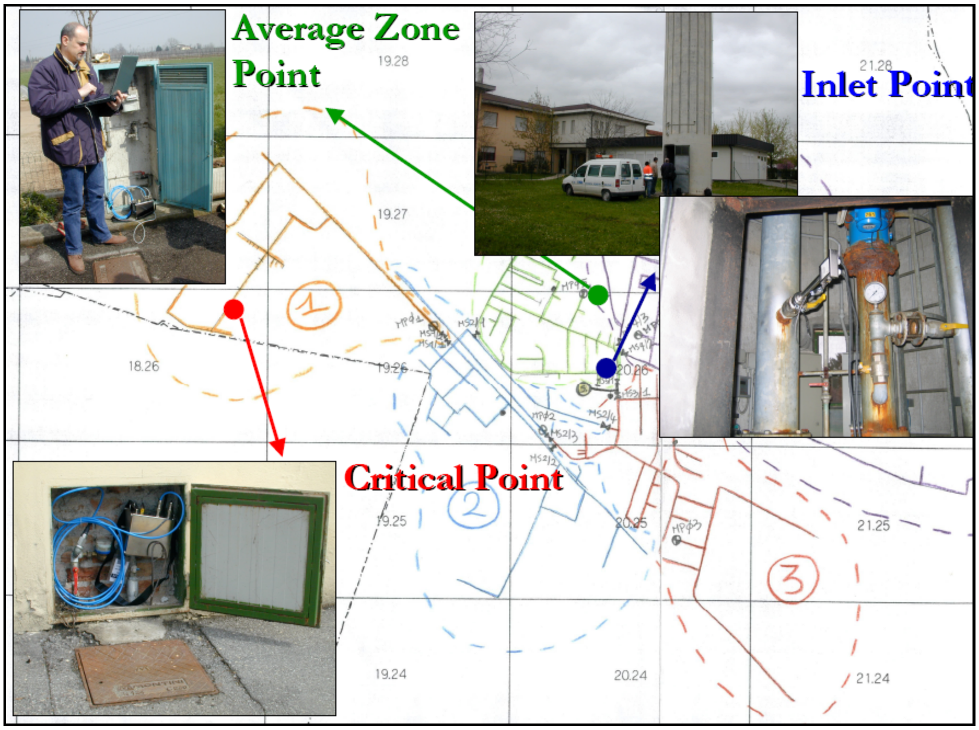


Figure 2.1 Montirone network and monitoring points.

Advantages of the technology used

In order to provide the user with a reliable analysis, data coming from the field must be of adequate accuracy for the purpose.

Different kind of flow-meters can be used, each of them characterised by different accuracy class and prices. The following are the components used:

- 1 battery powered electromagnetic flow-meter made of an insertion probe and a converter with an integrated Data logger for collecting data of flow and pressure and equipped with an internal GPRS module, which allow the sending of data wireless through e-mail directly to a remote personal computer. This mag-meter was installed at the Inlet Point (Figure 2.2);
- 2 battery powered electromagnetic converters coupled with 2 pressure transducers. Pressure data are sent by the converters through their internal GPRS modems. These 2 instruments must be installed at the Average Zone Point and Critical Point;
- the software Flowiz_Interface_Service able to extract the data excel files attached to the incoming e-mails and organise them into different folders identified by the instrument serial numbers;
- the Flowiz_Interface software, to present field data as tables and graphs, and identify the key parameters for the following economic intervention analysis (Figure 2.3);
- the WIZCalcs software, which uses occasional night flow measurements collected by the field instruments to decide when it is economic to perform a leak detection exercise. WIZCalcs uses the calculation methods described in Fantozzi & Lambert (2005) and Lambert & Lalonde (2005).

Figure 2.2 Electromagnetic flow-meter with GPRS modem and insertion probe.

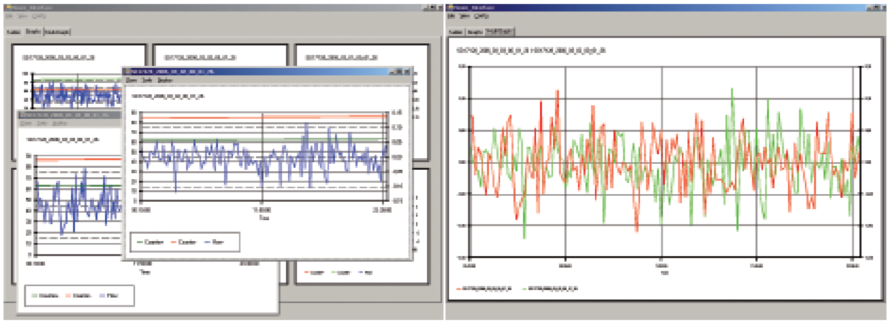


Figure 2.3 Example of graphs elaboration with the Flowiz_Interface.

The choice of using electromagnetic flow-meters reflects the high number of advantages provided by such equipment:

- High reliability granted by the absence of any mechanical moving part inside the instruments which avoids possible wear and tear;
- High accuracy also at low flow rates, essential for the reliable measurements of minimum flows.
- No significant pressure head losses, due to the absence of moving parts and absence of restriction within the pipe.

New features which have been integrated in these instruments have further increased their advantages:

- Pressure transducers with measurements driven, stored and elaborated directly by the instrument;
- Integrated Data Loggers collecting flow and pressure information at different selected sampling rates;
- GPRS Wireless communication protocol, enabling the transmission of data from remote sites directly to an e-mail account with no need of manual data collection by the users at sites where the instruments are installed.

Finally, also what was previously considered the highest limitations of the use of mag-meters in the water market has been overcome: battery power supply grants the independence of the instruments wherever it is installed.

But perhaps the real key factor in optimising this procedure in terms of savings of time and money is the way data are managed and analysed as an integrated part of the process.

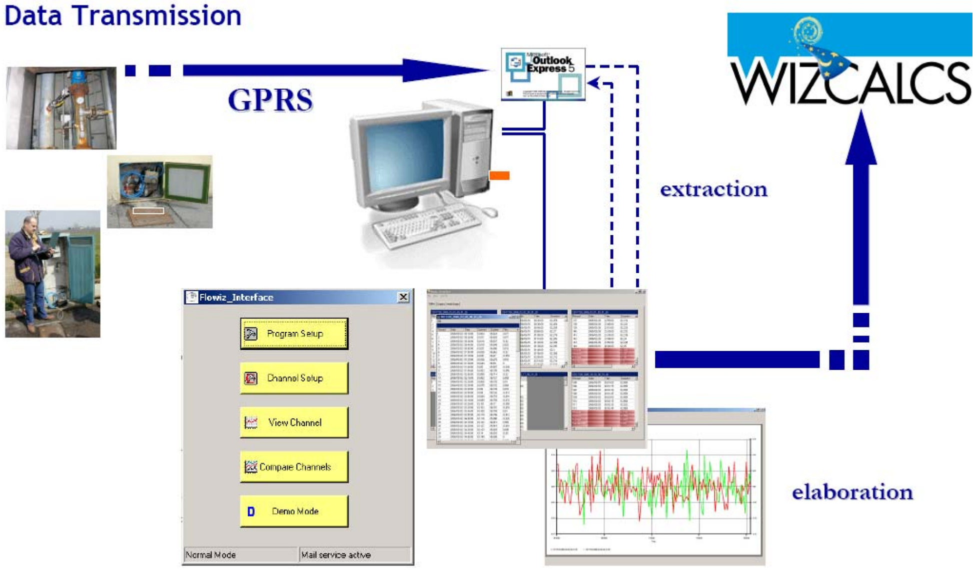


Figure 2.4 System data path.

Figure 2.4 shows the path followed by data from acquisition on site up to their loading in the final WIZCalcs software. The equipment is installed at the three specific locations (Inlet, Critical And Average Zone Points), collecting data of pressure (all three) and flow (only at the Inlet Point). Data are collected in the integrated Data Logger inside each instrument and finally sent to an e-mail address directly by the internal modem once a day.

The direct sending of data without any intermediate stage (i.e. pulses transmission to an external logger and subsequent flow rate derivation and transmission) guarantees the conservation of the flow-meter accuracy with no introduction of additional errors.

The GPRS (General Packet Radio System) protocol represents a very efficient way of transmitting data. Using a Packet commutation system, the GPRS collects data in packets made of sender address, information and receiver address. Once the packet has been sent through the net, there’s no possibility of losing the information.

The e-mails contain a CSV format file (Comma Separated Value), which is automatically downloadable in any editing software, e.g. Excel). Each record represents a complete acquisition of data (Date, Time, Positive and Negative Totalizers, Flow rate, Velocity and Pressure).

Speed in sending data (Kb/sec.) is very high, thanks to the use of best available communication channels in the network and to the optimisation of the net itself (data being collected in packets with reference to sender and receiver, with the network free to organise according to the current traffic conditions)

Cost for the user are low, as they are based on the actual amount of sent data and not on the connection time.

The software implementation developed for this application starts with the extraction of the data files from the e-mail account. All files coming from the same instrument during the period of measurements (typically 3 to 4 days) are collected in the same folder, automatically created by the software inside a chosen directory. Once the data are in the folder, the software allows the visualisation and elaboration of the same through tables and graphs, for single instruments or by comparison of several, for one single measure (i.e. flow) or with both flow and pressure values in the same sheet.

It is also worth mentioning that the kit of instruments above described, consisting of converters, pressure transducers and insertion sensor, has the advantage of being completely portable and usable many times in different locations to monitor a number of DMAs or water networks.

The total cost of the intervention, which comprehends the whole equipment to perform the analysis, the installation and the software for the evaluation of the Economic Intervention Frequency, is typically lower than 10.000,00 Euros.

Data analysis

Figure 2.5 shows the best achieved Minimum Night Flow after an active leakage control intervention done in 2003 and recent Minimum Night Flow in April 2006. It is possible to see that, without any further active leakage control since 2003, the night flow in the distribution system gradually increased with time, because of ‘unreported’ leaks and bursts, even though all ‘reported’ leaks and bursts have been promptly repaired. The actual night flow is checked against estimates of customer night use and background leakage, to calculate potentially recoverable losses.

The average rate of rise that occurred is system-specific, being influenced by several local factors. And, for reliable results in WIZCalcs, the occasional night flow measurements must be taken at times of year when industrial and irrigation use at night is considered to be minimal (typically early spring and late autumn)

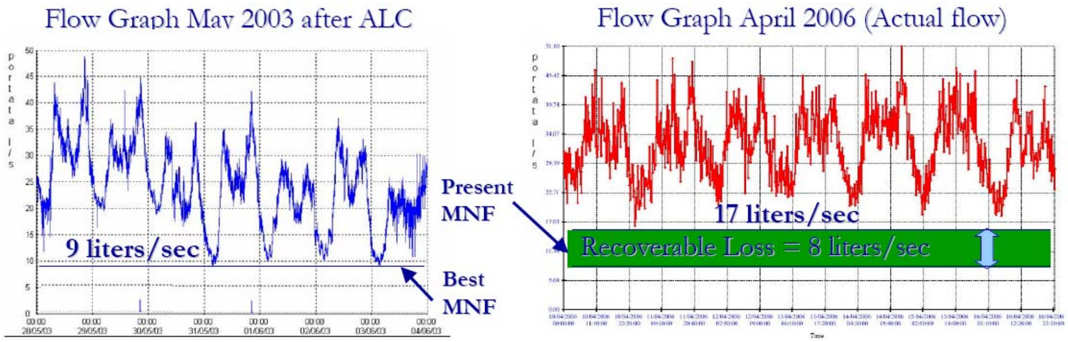


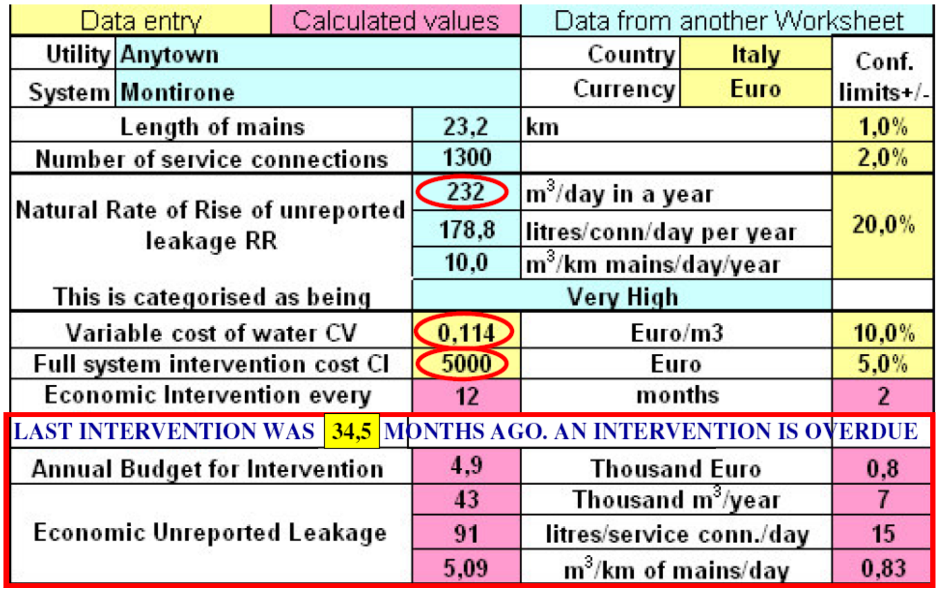
Figure 2.5 Comparison of night flows.

Figure 2.6 shows the results obtained from the WIZCalcs software. The applied method, presented in this paper, requires only three parameters:

- the average rate of rise of unreported leaks;
- the variable cost of water;
- the cost of intervention.

These are enough to determine the Economic Intervention Frequency with an Active Leakage Control, the Annual Budget for Intervention and the Economic Volume of Unreported Leaks.

In Montirone the calculated rate of rise of unreported leaks is very high and the previous intervention with active leakage control was 34 months before, WIZCalcs quickly shows that an intervention was overdue in April 2006. The annual budget for intervention and the economic volume of unreported leaks have also been calculated and reported in Figure 2.6.



Data entry	Calculated values	Data from another Worksheet	
Utility Anytown		Country Italy	Conf. limits+/-
System Montirone		Currency Euro	
Length of mains	23,2	km	1,0%
Number of service connections	1300		2,0%
Natural Rate of Rise of unreported leakage RR	232	m^3/day in a year	20,0%
	178,8	litres/conn/day per year	
	10,0	m^3/km mains/day/year	
This is categorised as being	Very High		
Variable cost of water CV	0,114	Euro/m3	10,0%
Full system intervention cost CI	5000	Euro	5,0%
Economic Intervention every	12	months	2
LAST INTERVENTION WAS	34,5	MONTHS AGO. AN INTERVENTION IS OVERDUE	
Annual Budget for Intervention	4,9	Thousand Euro	0,8
Economic Unreported Leakage	43	Thousand m^3/year	7
	91	litres/service conn./day	15
	5,09	m^3/km of mains/day	0,83

Figure 2.6 WIZCalcs software applied in Montirone.

Conclusions

Conclusions of this paper are the following:

- Starting a strategy in water loss control and in Active Leakage Control need not be expensive, or require permanent installations
- data collection from the field, to analysis and management decisions, can be completed in a matter of just a few days, and the equipment can then be moved to other sites
- the economical frequency of intervention, specific to each part of the network, can be quickly identified with little investment of time and instrumentation;
- flow and pressure values have to be considered together in order to reliably identify appropriate actions
- the data collected for economic intervention can be used off-line with another software (PreMOCalcs) to identify pressure management options , and predict benefits and payback periods for different types of pressure management
- the data transmission system explained in this paper is an optimum solution to manage network data: GPRS wireless transmission is safe, quick and low cost.

Acknowledgements

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Remote DMA Monitoring As a Useful Tool In Water Loss Control

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Keywords: remote monitoring; water loss control

Abstract

The possibility of continuous control of pressure and overnight flow is one of the key methods of water loss control. Thanks to combining new data communication technologies and the opportunity of WWW access in Biatel MS (Monitoring System) it is possible from any place in the world.

This system allows creating on-line balances for DMAs supplied from several sources. The autodiagnostic modules (e.g. Minimal Night Flow module) enable to trace the changes of parameters after opening/closing valves in the area. Due to its modularity the system can be extended freely according to needs. Using on-line monitoring, we can follow and judge the efficiency of leakage repair workers.

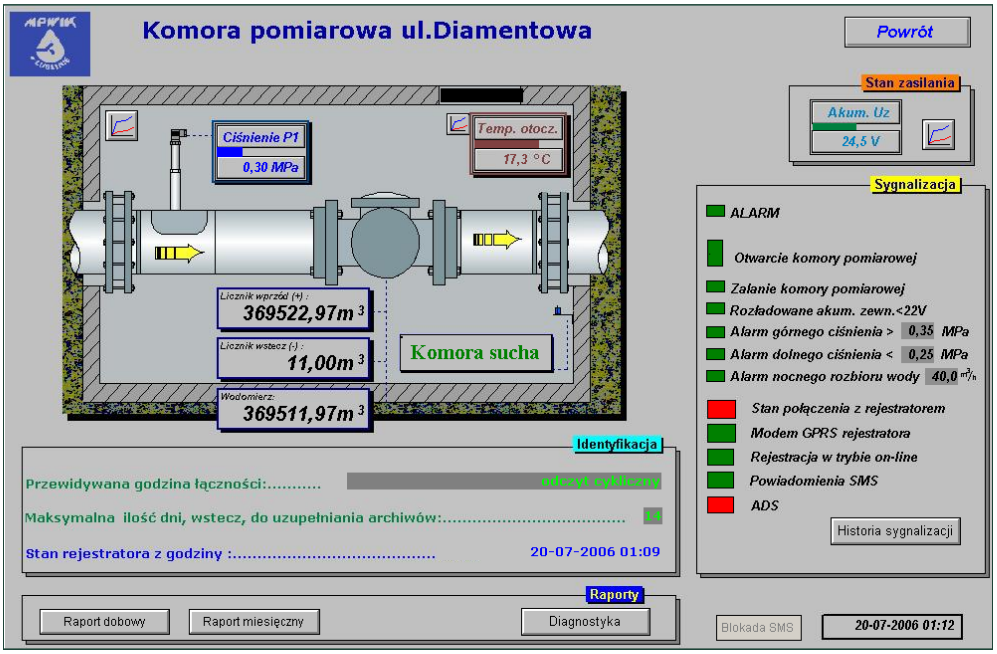
The paper shows examples of implementation of on-line monitoring systems for water networks in Poland and its effects.

On-line monitoring – Biatel-Monitoring System

The IWA standards according water loss management (Lambert at al., 2000) are indicating the advantages of closed DMA’s (District Metered Areas) with simultaneous night flow measurement and monitoring.

Not long ago there was no possibility to use on-line solution for remote reading, due to limited power sources. The fast GSM and IT technology development allowed to solve this problem by using energy saving CellBOX registers with GSM/GPRS modems and mobile batteries. The registers transmit their data through GSM/GPRS onto an encrypted WWW site were the SCADA system controls the measurement pits. This solution is called Biatel MS which is based on an application called HydraNET.

An example of a measurement chamber equipment with built-in information system of alarms, communication and reports is shown in Figure 12.



Komora pomiarowa ul.Diamentowa

Ciśnienie P1: 0,30 MPa

Temp. otocz.: 17,3 °C

Licznik wprzód (+) : 369522,97m ^{3}

Licznik wstecz (-) : 11,00m ^{3}

Wodomierz: 369511,97m ^{3}

Komora sucha

Stan zasilania

Akum. Uz: 24,5 V

Sygnalizacja

🟩 ALARM

🟩 Otwarcie komory pomiarowej

🟩 Zalanie komory pomiarowej

🟩 Rozładowane akum. zewn.<22V

🟩 Alarm górnego ciśnienia > 0,35 MPa

🟩 Alarm dolnego ciśnienia < 0,25 MPa

🟩 Alarm nocnego rozbioru wody 40,0 m³/h

🟥 Stan połączenia z rejestratorem

🟩 Modem GPRS rejestratora

🟩 Rejestracja w trybie on-line

🟩 Powiadomienia SMS

🟥 ADS

Identyfikacja

Przewidywana godzina łączności:........... odczyt cykliczny

Maksymalna ilość dni, wstecz, do uzupełniania archiwów:.................................... 14

Stan rejestratora z godziny :... 20-07-2006 01:09

Raporty

20-07-2006 01:12

Figure 12. Exemplary measurement pit equipment in Biatel MS (DMA ‘L1’)

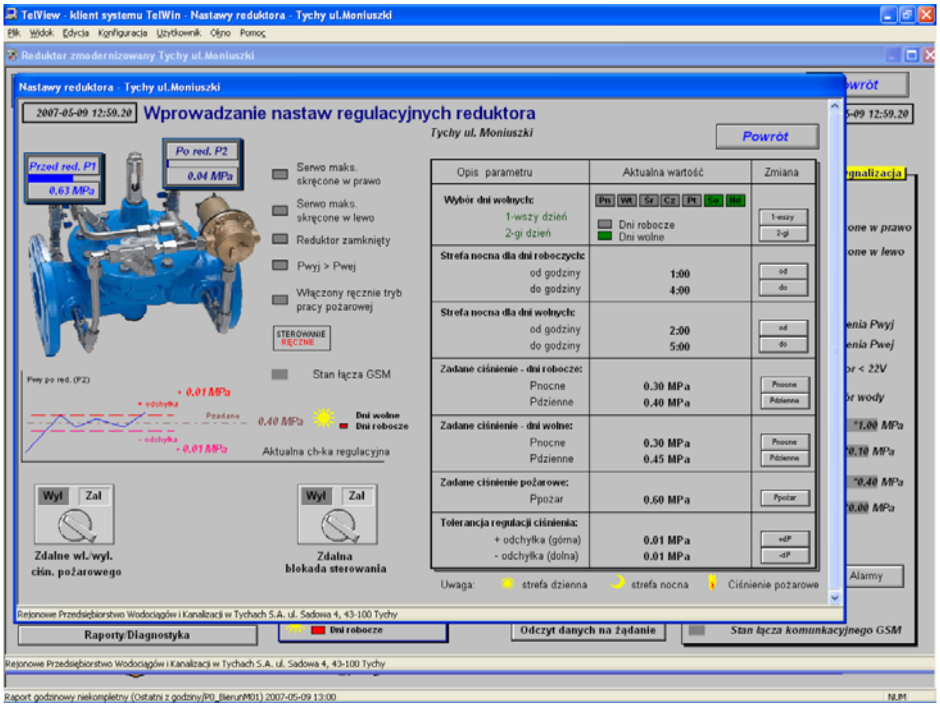
The standard equipment allows to measure the water flow in two directions, pressure (in case of PRV – before and after reduction); temperature, battery level, and pit/chamber opening and flooding. The modular construction of CellBOX devices enable to connect additional signals e.g. PWiK Gliwice (Municipal Water Network in Silesia) use them for turbidity monitoring; and there are plans of implementing conductivity monitoring (in order to define the range of water sources in network basing conductivity measurement).

Figure 13. DMA monitoring without measurement pit

When there is no possibility to build/adapt a measurement pit/chamber, there is a simple solution – buried underground battery supplied electromagnetic flow meters which are connected to the register in an underground fire hydrant or telecommunication column. Exemplary installation is shown in Figure 13.

The latest solution using GSM/GPRS technology is the remote battery operated PRV control. This solution enables controlling the pressure according to time or flow profile – and enables also remote control when it is needed in emergency situations (through WWW and GSM). The operator can change the mode by one click.

Sample control panel of PRV remote control is shown on Figure 14.



Opis parametru	Aktualna wartość	Zmiana
Wybór dni wolnych: 1-wszy dzień 2-gi dzień	Pn Wt Śr Cz Pt So Nd ▨ Dni robocze 🟩 Dni wolne	1-wszy 2-gi
Strefa nocna dla dni roboczych: od godziny do godziny	 1:00 4:00	od do
Strefa nocna dla dni wolnych: od godziny do godziny	 2:00 5:00	od do
Zadane ciśnienie - dni robocze: Pnocne Pdzienne	 0.30 MPa 0.40 MPa	Pnocne Pdzienne
Zadane ciśnienie - dni wolne: Pnocne Pdzienne	 0.30 MPa 0.45 MPa	Pnocne Pdzienne
Zadane ciśnienie pożarowe: Ppożar	 0.60 MPa	Ppożar
Tolerancja regulacji ciśnienia: + odchyłka (górna) - odchyłka (dolna)	 0.01 MPa 0.01 MPa	+dP -dP

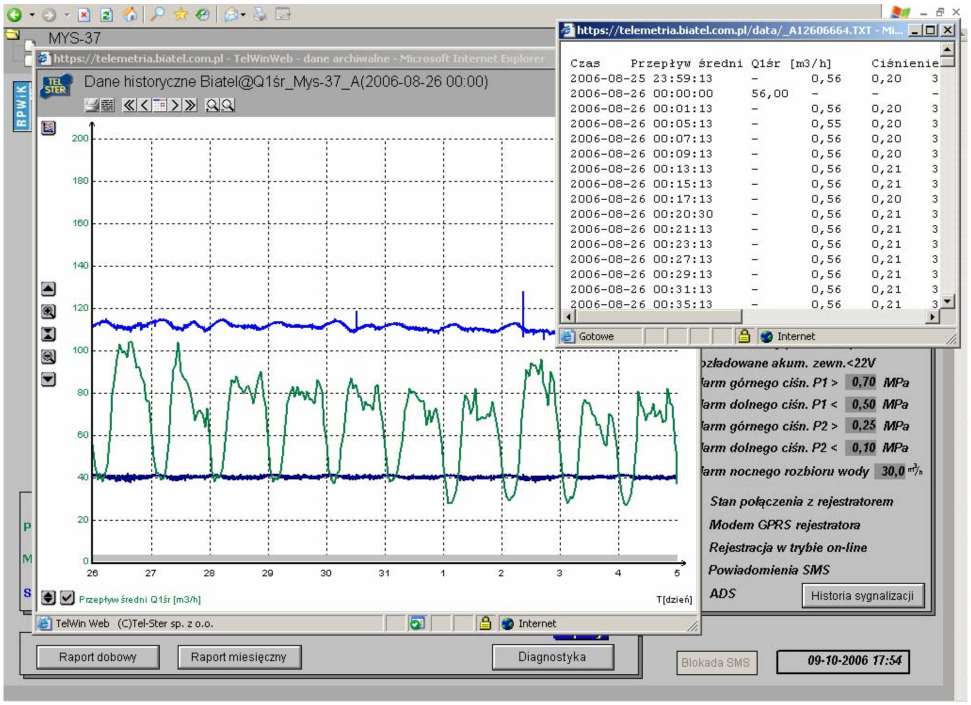
Figure 14. Example of pit with pressure reduction valve controlled remotely (DMA 'T')

Report modules and Autodiagnostics

The overnight flow monitoring and observations of its trends is one of the main tools of DMA diagnostics.

Therefore, one of the main components of Biatel MS are the report modules – daily and month report, MNR (Minimal Overnight Flows) and ADS (Autodiagnostics).

The report modules show in legible way the changes of every measured parameter in a selected timescale, on a graph, or in transitory/hour measurement table (Figure 15). But the real advantage is the on-line balancing of zones supplied from several sources, also by summation or subtraction in case of water export from the zone. It enables to decrease the range of search (so shortening its time) for leak survey and pinpointing. An example of this kind of balance is shown in Figure 16.



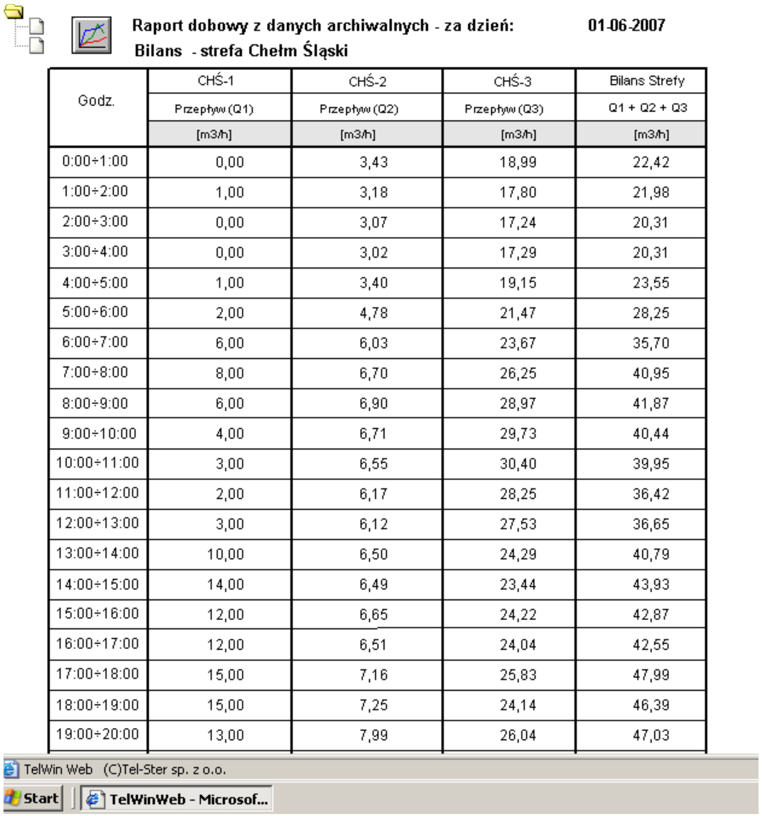
```
Czas    Przepływ średni Q1śr [m3/h]     Ciśnienie
2006-08-25 23:59:13     -         0,56    0,20    3
2006-08-26 00:00:00     56,00     -       -       -
2006-08-26 00:01:13     -         0,56    0,20    3
2006-08-26 00:05:13     -         0,55    0,20    3
2006-08-26 00:07:13     -         0,56    0,20    3
2006-08-26 00:09:13     -         0,56    0,20    3
2006-08-26 00:13:13     -         0,56    0,21    3
2006-08-26 00:15:13     -         0,56    0,21    3
2006-08-26 00:17:13     -         0,56    0,20    3
2006-08-26 00:20:30     -         0,56    0,21    3
2006-08-26 00:21:13     -         0,56    0,21    3
2006-08-26 00:23:13     -         0,56    0,21    3
2006-08-26 00:27:13     -         0,56    0,21    3
2006-08-26 00:29:13     -         0,56    0,21    3
2006-08-26 00:31:13     -         0,56    0,21    3
2006-08-26 00:35:13     -         0,56    0,21    3
```

ozładowane akum. zewn.<22V
larm górnego ciśn. P1 > 0,70 MPa
larm dolnego ciśn. P1 < 0,50 MPa
larm górnego ciśn. P2 > 0,25 MPa
larm dolnego ciśn. P2 < 0,10 MPa
larm nocnego rozbioru wody 30,0 m³/h

Stan połączenia z rejestratorem
Modem GPRS rejestratora
Rejestracja w trybie on-line
Powiadomienia SMS
ADS

09-10-2006 17:54

Figure 15. Graphs and tables of measured parameters (DMA 'I-CH')



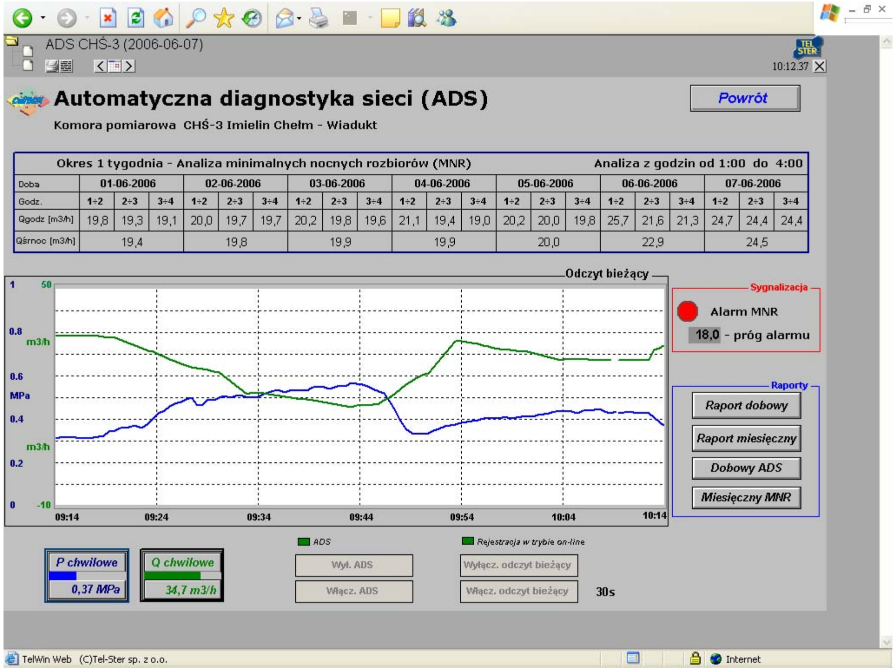
Raport dobowy z danych archiwalnych - za dzień: 01-06-2007
Bilans - strefa Chełm Śląski

Godz.	CHŚ-1	CHŚ-2	CHŚ-3	Bilans Strefy
	Przepływ (Q1)	Przepływ (Q2)	Przepływ (Q3)	Q1 + Q2 + Q3
	[m3/h]	[m3/h]	[m3/h]	[m3/h]
0:00÷1:00	0,00	3,43	18,99	22,42
1:00÷2:00	1,00	3,18	17,80	21,98
2:00÷3:00	0,00	3,07	17,24	20,31
3:00÷4:00	0,00	3,02	17,29	20,31
4:00÷5:00	1,00	3,40	19,15	23,55
5:00÷6:00	2,00	4,78	21,47	28,25
6:00÷7:00	6,00	6,03	23,67	35,70
7:00÷8:00	8,00	6,70	26,25	40,95
8:00÷9:00	6,00	6,90	28,97	41,87
9:00÷10:00	4,00	6,71	29,73	40,44
10:00÷11:00	3,00	6,55	30,40	39,95
11:00÷12:00	2,00	6,17	28,25	36,42
12:00÷13:00	3,00	6,12	27,53	36,65
13:00÷14:00	10,00	6,50	24,29	40,79
14:00÷15:00	14,00	6,49	23,44	43,93
15:00÷16:00	12,00	6,65	24,22	42,87
16:00÷17:00	12,00	6,51	24,04	42,55
17:00÷18:00	15,00	7,16	25,83	47,99
18:00÷19:00	15,00	7,25	24,14	46,39
19:00÷20:00	13,00	7,99	26,04	47,03

Figure 16. Balance of a zone supplied from three sources (DMA 'CH')

The next module is MNR: the measured overnight flows of the last week are shown in legible way to the operator. When the designated alert level of overnight flow is exceeded, the automatic alarm signal turns on.

These values can be also displayed on a combined graph for any time interval, so we can point out the trends of short and long term changes.



Automatyczna diagnostyka sieci (ADS)

Komora pomiarowa CHŚ-3 Imielin Chełm - Wiadukt

Okres 1 tygodnia - Analiza minimalnych nocnych rozbiorów (MNR) Analiza z godzin od 1:00 do 4:00

Doba	01-06-2006			02-06-2006			03-06-2006			04-06-2006			05-06-2006			06-06-2006			07-06-2006		
Godz.	1÷2	2÷3	3÷4	1÷2	2÷3	3÷4	1÷2	2÷3	3÷4	1÷2	2÷3	3÷4	1÷2	2÷3	3÷4	1÷2	2÷3	3÷4	1÷2	2÷3	3÷4
Qgodz [m3/h]	19,8	19,3	19,1	20,0	19,7	19,7	20,2	19,8	19,6	21,1	19,4	19,0	20,2	20,0	19,8	25,7	21,6	21,3	24,7	24,4	24,4
Qśrnoc [m3/h]	19,4			19,8			19,9			19,9			20,0			22,9			24,5		

● Alarm MNR

18,0 - próg alarmu

P chwilowe 0,37 MPa

Q chwilowe 34,7 m3/h

Figure 17. MNR and ADS modules (DMA ‘CH’)

From the practical point of view, the most advanced module is the ADS (Autodiagnostics). This module supervises the network in an automated way. Accordingly to the entered parameters for the DMA, when the overnight flow exceeds the alert level (even at daytime - in case of main waterline break) an automatic SMS message is sent to the operator and the CellBOX register turns on for permanent **on-line data transfer**.

This is very important function if we have to manage a network divided onto many DMAs. Thanks to this function the operator responsible for the maintaining the system, can immediately check the situation on the site from the beginning of the alarm and what are the current parameters. In case of big industrial users connected to the network, it can be checked if the big consumption is caused by industrial needs or not.

The system also includes the TeSt module (step testing) which is strictly connected with the ADS module. It enables to make a quick diagnosis of the DMA where the high consumption revealed. The TeSt module makes a regular step testing if needed (e.g. after every grow of overnight flow) in the night hours (2:00-3:00 a.m.) by remote closing and opening automated valves which divide the DMA into smaller areas. The ADS module records all of the changes and creates a complete report for the system operator. The report is available continuously in the system or in the morning as a printed report. Basing on this report the operator instructs the leak survey team.

At the moment the system works in half automatic mode (the valves are closed manually), but a project of implementing completely automated valves is in advanced progress at one of the waterworks in Poland.

The TeSt module of HydraNET system replaces the former step testing technology, where the measurement devices were built on Leak Test Vans.

Sample implementation of remote monitoring

Besides overnight flow diagnosis, remote monitoring is used in other applications. Some interesting examples of remote monitoring are shown below:

On line network regulation

The on-line monitoring is a very useful tool in checking the effects of pressure regulations made on the water networks, i.e. opening/closing of valves in order to improve the pressure situation in a detached area.

An example of this kind of application is shown on **Figure 18**.

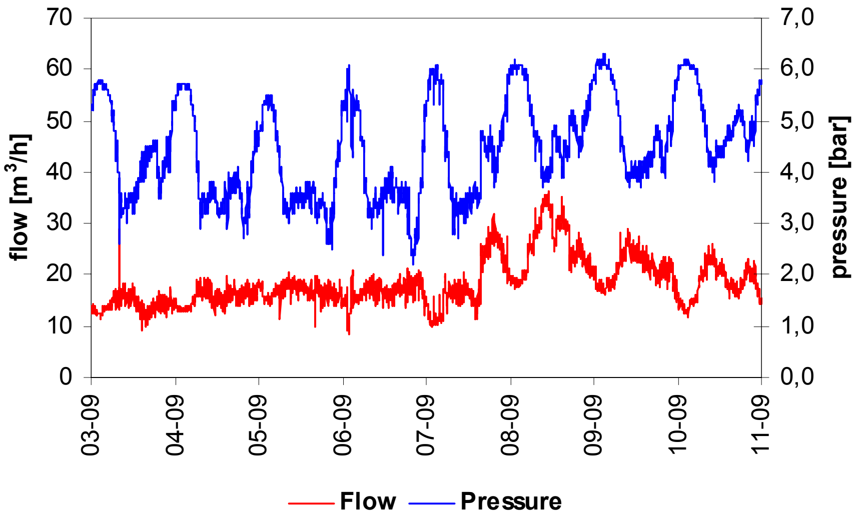


Figure 18. Network regulation– valve opening (DMA 'I-CH')

The graph shows the situation after opening a single valve on the supplied area, which caused pressure growth in the time of maximal consumption by almost 1.0 bar. Unfortunately, the overnight minimal flow also grew, despite the more pressure growth.

On-line step test

The automatic "step test" described above can be carried out by a specialist from any place with Internet connection. This kind of step test is shown on **Figure 19**, where Biatel SA water diagnostic specialist during leak survey made a step test alone in the daytime, by closing valves for 5 minutes in the areas. Without using any additional equipment he managed to narrow down the search area fast and precisely.

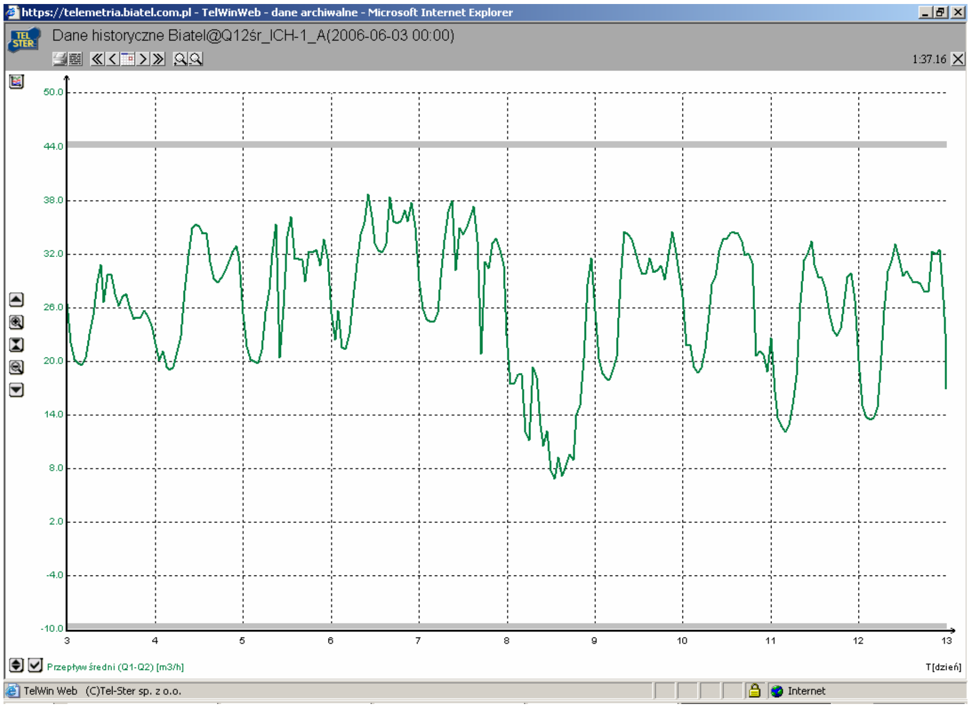


Figure 19. On-line step test (DMA 'I-CH')

Break reports and overnight flow trends

One of the most important applications of the remote monitoring system is the monitoring of effectiveness of leak survey and leak repair teams. It is possible by comparing the MNR and ADS data with leak repair reports. This kind of control is shown on **Figure 20** (S – water network break, P – attachment break).

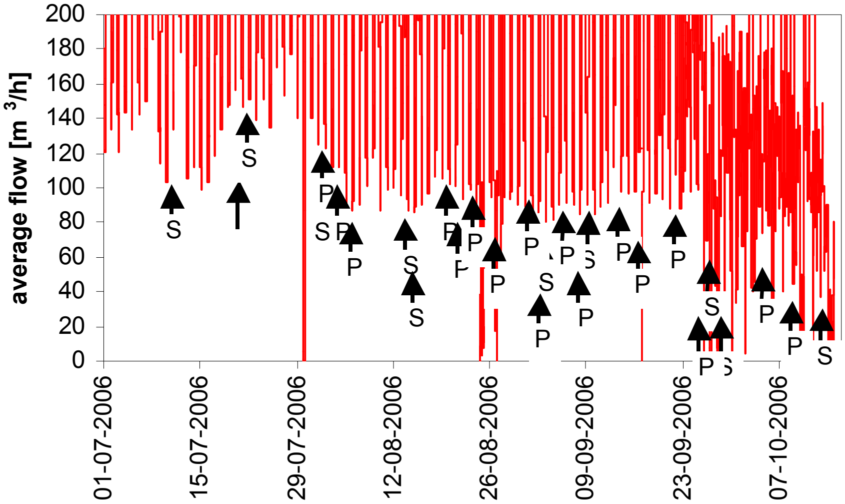


Figure 20. Connecting overnight flow with leakage repair reports (DMA 'SW')

Thanks to this connection it is possible to show precisely, the amount of saved water in the overnight flow, and check if the pressure growth after the leak repair didn't cause another leak in a short time.

Of course there is possibility to analyze the short and long term consumption changes e.g. weekly trend - **Figure 21**

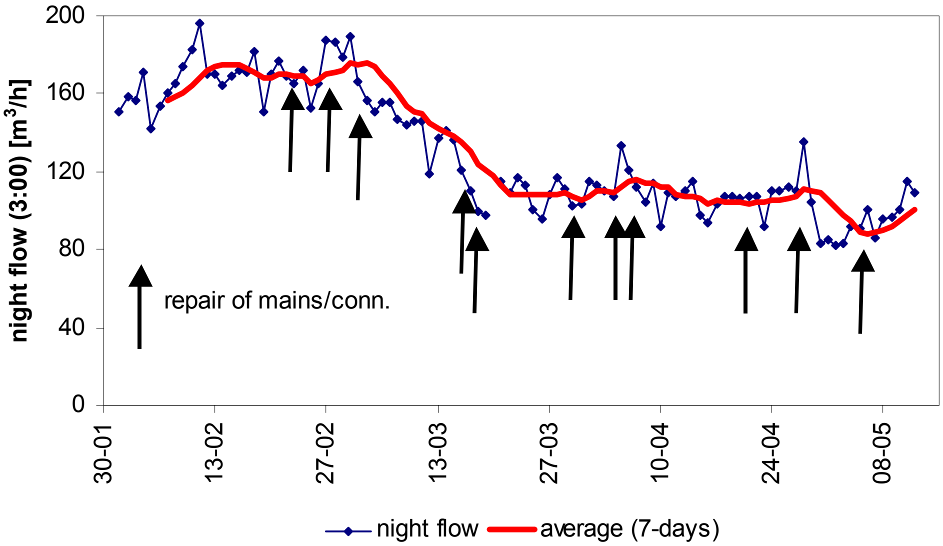


Figure 21. Long term overnight flow trend (DMA 'S')

Summary: using on-line monitoring, balancing and automatic diagnosis help to increase efficiency of management of Water Utility.

References

Lambert A. O.,Hirner W. (2000): „Losses from water supply systems. Standard terminology and recommended performance measures. IWA Blue Pages, www.iwahq.org

CellBOX documentation - www.cellbox.pl

Leakage Detection - Assessment of four different leakage control techniques

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Keywords: Active leakage control; leakage detection

Introduction

Three Valleys Water (TVW) is a water only company owned by Veolia Water, part of the Veolia Environnement group of companies, operating in the South East of England. TVW provides drinking water to about 3.1 million customers with a turnover of £213 million. In total 850 million m^3 of water is produced per day, 144 of which are recorded as leakage. The network has a length of approx. 14,500 km. The network is operated as 6 Water Resource Zones, divided into 33 Hydraulic Demand Zones (HDZ). These 33 HDZs are again sub-divided into 855 District Meter Areas (DMA) and 175 passive areas as shown in figure 1 below.

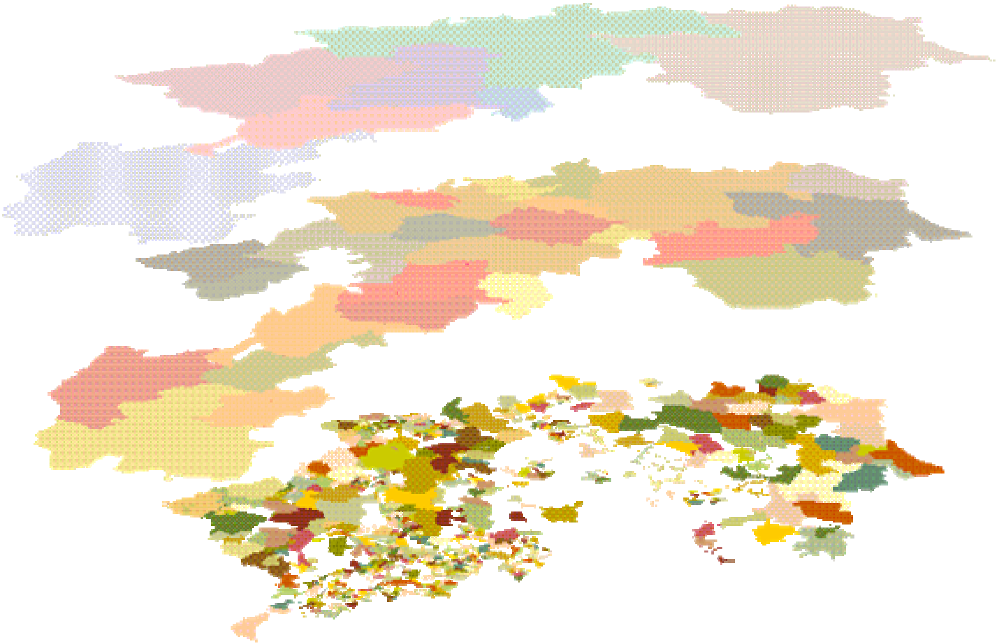


Figure 22: Three Valleys Water area: 6 Resource Zones, 33 Hydraulic Demand Zones and over 800 District Meter areas.

One of the key factors to effective management of leakage within District Meter Areas is to have a clear understanding of the network performance. There are a number of leakage detection techniques available to practitioners to locate leaks within a distribution network, generally falling into the four following categories: step testing, simple sounding surveys, noise logging and leak noise correlation.

The choice of detection method depends mainly upon the configuration and characteristics of the network, the water company's own policies, and the resources and equipment available. However, comparative information on the costs and effectiveness of the different techniques is not readily understood.

A study had therefore been designed to provide practical – company specific data from which a detailed assessment of the various leakage detection techniques was carried out.

The objectives of the Active Leakage Control (ALC) study were as follows:

- To measure the effectiveness of four different leakage control techniques (simple sounding, leak noise correlating, noise logging and step testing under controlled conditions.
- To establish from the information collected whether particular techniques or combination of techniques provide best results for particular District Meter Areas.

This paper describes the process followed to select a representative group of district meters areas, sets out the leakage control techniques or combination of techniques utilised in the study, the development of operational processes and tools to record and measure under controlled conditions the effectiveness of the different leakage control techniques and present preliminary results.

Method

DMA Selection

The leakage detection study was based upon a sample of DMAs, representative of TVW. All available DMAs within the company were analysed using on a matrix of:

- their calculated unavoidable annual real losses UARL (Lambert, 2003),
- the resource zone where they were located,
- a soil corrosivity index,
- a ground movement index.

This initial analysis allowed the DMAs to be divided into four distinct groups. A second selection process thought to influence leakage detection was then utilised using other factors:

- the number of fittings per km,
- the length of distribution mains,
- the average zone night pressure.

Four DMAs were then randomly picked in each of the four groups. From this analysis, 15 district meter areas were selected for the study. Additional operational information within the selected district meter areas was also collected to continuously record the minimum night flow, the average zonal pressure and the diurnal variations in pressure to allow hour to day factors to be reviewed and updated.

The following Table sets the main characteristics of the district meter areas.

Table 11: DMA selected and main characteristics: identification, number of properties, Average Zone night pressure, Soil type (Non-aggressive, slightly aggressive, Highly aggressive), Length of distribution mains (km) and Average age of distribution mains

Area ID	N of properties	Operability	AZNP	Soil Type	Length of mains (m)	Average age of pipes	% ferrous	% plastic	% AC
9421	346	B	79.75	Non-A/SA	7.762	1959	0.912	0.074	0.013
1110	460	A	23.79	Non-A	3.300	1968	0.917	0.083	0.000
9205	547	A	40.7	Non-A	3.442	1958	1.000	0.000	0.000
3804	571	A	60.52	SA	11.146	1972	0.768	0.232	0.000
6710	607	A	37.9	Non-A/SA	5.723	1936	0.880	0.120	0.000
5922	628	A	38.66	HA	4.121	1946	0.835	0.165	0.000
3402	753	A	48.54	Non-A	10.697	1965	0.773	0.227	0.000
7008	1167	A	37.69	Non-A/SA	9.116	1989	0.192	0.808	0.000
4201	1268	A	50	Non-A/SA	13.889	1969	0.631	0.031	0.338
2012	1410	A	33	HA	11.459	1958	0.946	0.054	0.000
9110	1568	A	60.06	HA	9.182	1947	0.852	0.128	0.020
8201	1663	A	32.52	Non-A/SA	15.468	1965	0.935	0.065	0.000
109	1934	A	10.71	Non-A	13.664	1972	0.856	0.144	0.000
2550	3349	A	37.57	Non-A/SA	30.582	1950	0.892	0.076	0.033
7812	6522	A	61.51	Non-A	39.234	1954	0.728	0.272	0.000

Selection of Leakage Detection Technique

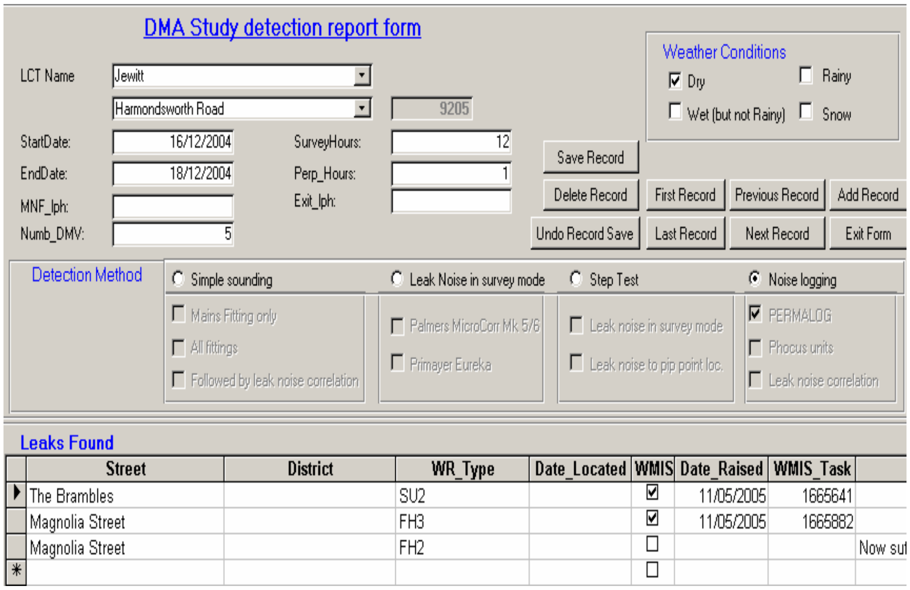
The four leakage detection methods are identified as follow:

- Simple sounding: listening sticks used to survey areas of suspected leakage.
- Leak noise correlating in survey mode: correlators used to pinpoint the leak location
- Noise logging: magnetised units are installed on a group of adjacent fittings and record the constant noise generated by a leak. Analysis of the readings is done by comparison of sound level and sound spread, recorded at each logger.
- Step Test: technique which require progressive isolation of sections of pipe by closing line valves, beginning at pipes farthest away from the meter and ending at pipes nearest from the meter.

Project Organisation

A programme of controlled sequence of leakage surveys using all four techniques was established for the 15 DMAs to minimise the impact of individual technicians' skills or knowledge or other uncontrollable parameters such as the weather.

Systems such as a study detection proforma and an Access database (cf figure 2) were also put in place to record in detail the costs and duration of each leakage detection intervention technique, the type and number of repairs located, the duration of the repairs and to measure the volume of water saved.



DMA Study detection report form

LCT Name: Jewitt

Harmondsworth Road — 9205

StartDate: 16/12/2004

EndDate: 18/12/2004

MNF_lph:

Numb_DMV: 5

SurveyHours: 12

Perp_Hours: 1

Exit_lph:

Weather Conditions: ☑ Dry ☐ Rainy ☐ Wet (but not Rainy) ☐ Snow

Detection Method: ○ Simple sounding ○ Leak Noise in survey mode ○ Step Test ● Noise logging

☐ Mains Fitting only
☐ All fittings
☐ Followed by leak noise correlation
☐ Palmers MicroCorr Mk 5/6
☐ Primayer Eureka
☐ Leak noise in survey mode
☐ Leak noise to pip point loc.
☑ PERMALOG
☐ Phocus units
☐ Leak noise correlation

Leaks Found

Street	District	WR_Type	Date_Located	WMIS	Date_Raised	WMIS_Task	
The Brambles		SU2		☑	11/05/2005	1665641	
Magnolia Street		FH3		☑	11/05/2005	1665882	
Magnolia Street		FH2		☐			Now su
				☐			

Figure 2 Study Detection Form

The leakage detection interventions(called also “sweep”) were repeated within each of these 15 DMAs with the four selected detection methods three times, until the level of leakage in the DMAs was “near” the calculated unavoidable annual real losses or it was deemed impossible to further reduce the level of leakage.

The technicians searching for leaks with the different techniques were allowed to spend one day per 250 properties. One extra day was added to the total number of days, in case of poor weather conditions, sickness and unpredictable events. A schedule was handed over to the Leakage Detection Team on a monthly basis. An example is shown in the table below.

Table 12: Example of work plan matrix handover to Leakage Detection team

Time allowance	2 days	2 days	2 days	2 days
DMA Name	DMA 1	DMA 2	DMA 3	DMA 4
Technique + Technician	LCT1 – SS	LCT1 – NL	LCT1 – CN	LCT1 – ST
	LCT2 – NL	LCT2 – CN	LCT2 – ST	LCT2 – SS
	LCT3 – CN	LCT3 – ST	LCT3 – SS	LCT3 – NL
	LCT4 - ST	LCT4 – SS	LCT4 - NL	LCT4 - CN

LCT = Leakage Control Technician
SS = Simple Sounding
NL = Noise Logging
CN = Correlating Noise
ST = Step Test

Method used for the volumetric saving calculation

The volume saved for each repair was calculated using flow and job data from Three Valleys Water Leakage Reporting System (TVLR).

The Three Valleys Leakage Reporter system provides means for collecting and presenting performance data. In order to solve the problem of lack of domestic metering, the network has been divided into several hundreds of District Meter Area (DMA). Flows entering DMAs are recorded with loggers (every 15 minutes and downloaded every night).

TVLR is a presentational tool that links to GIS and the TVW Work Management Information system (WMIS). Its thematic mapping capabilities can be readily put to use to identify relationships between activity and success. Its current use is:

- Outstanding leakage jobs in each DMA (output from WMIS),
- Completed leakage jobs in each DMA (output from WMIS)

It displays comparative night flow data for DMAs to improve the targeting of leakage detection.

For this project, TVLR was used to measure to calculate the volume saved for each repair is calculated by subtracting the flow before the repair to the flow after the repair.

Results

Comparison between techniques

The results displayed for each technique are summarised in table 3 and figures 2 to 4. No significant statistical differences were found between the four different techniques but some initial observation have been drawn :

- The most leaks per day were found through simple sounding but mainly at communication pipes and stop taps.
- The most volumetric saving per day was achieved through noise logging with leaks identified mainly at communication pipes, stop taps and fire hydrants. Noise Logging also identified the most main bursts.
- Leak noise correlating gave best results in rural areas, where long lengths of main without connections can be found and where traffic is less intense than in urban areas.

The results for the step test technique are not included in this summary table, as this technique was implemented on 3 DMAs only. The Step Test technique was abandoned due to network issues mainly water quality, terms and conditions (night work), and time consuming health and safety procedures such as the requirement to write method statements.

Table 13: Summary of results

Technique	Survey	Number of leaks	Number of MB	Volume saved (m3/h)	Time spent (h)	Number of leaks/day	Volume saved/day
Simple Sounding	No.1	100	3	49.78	567.50	1.41	0.70
	No.2	67	3	12.72	568.75	1.01	0.18
	No.3	78	1	12.31	290.25	2.15	0.34
Leak Noise Correlating	No.1	53	4	48.50	647.50	0.65	0.60
	No.2	26	4	3.90	458.25	0.45	0.07
	No.3	23	2	8.97	295.75	0.62	0.24
Noise Logging	No.1	60	5	56.57	566.00	0.85	0.80
	No.2	30	2	3.44	455.25	0.53	0.06
	No.3	10	2	3.29	253.75	0.32	0.10

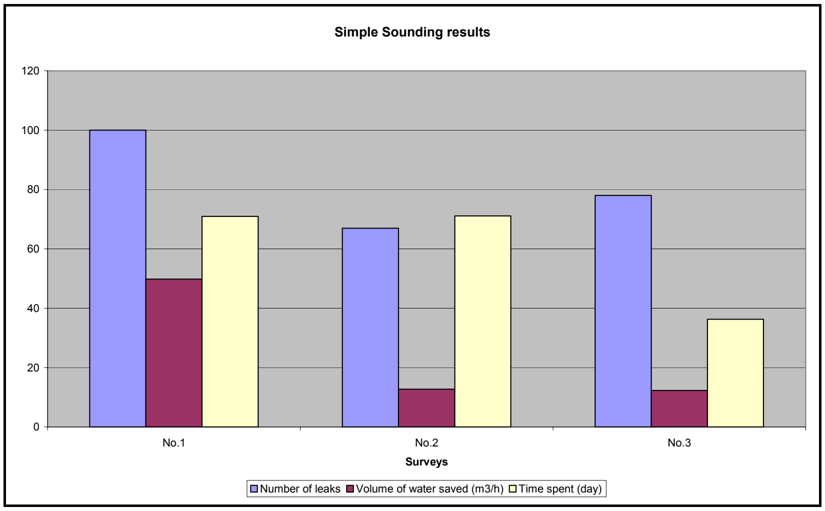


Figure 23: Simple Sounding Results

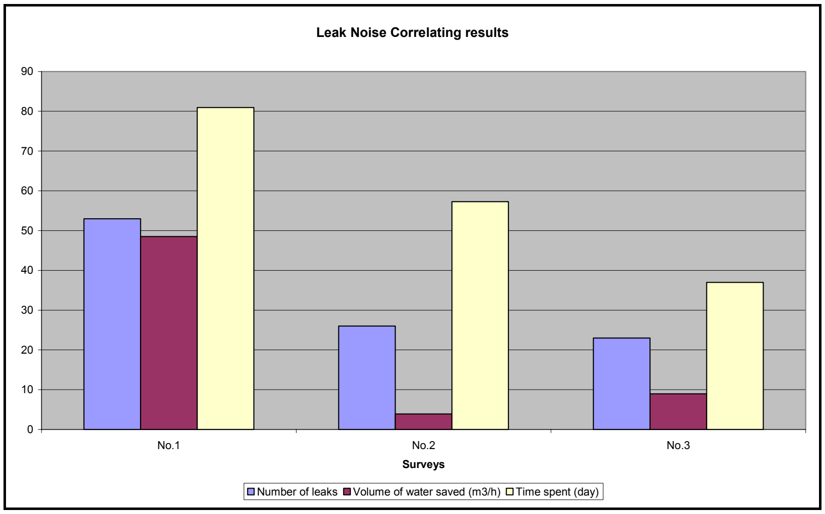


Figure 24: Leak Noise Correlating Results

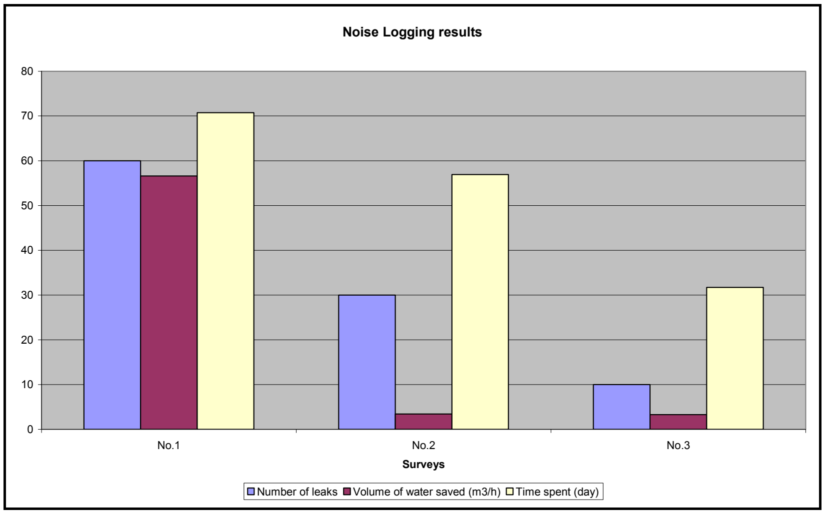


Figure 25: Noise Logging Results

For all techniques, statistical analyses showed that the number of leaks found, the volumetric savings or the time spent are linked to DMA characteristics such as its size (number of properties, length of mains) or pressure level . The table below summarises the different correlations found between the different variables for each technique.

However, the statistics analysis for this data sample did not highlight any correlation between the surveys and the soil corrosivity, the pipe material and the age of the mains.

Table 14: Correlation between leaks found and DMA characteristics

Technique	Survey	N leaks vs N properties	N leaks vs L mains	V saved vs N properties	V saved vs L mains	T spent vs N properties	T spent vs L mains	N leaks vs AZNP
Simple Sounding	No.1					regression R2 = 0.76		
	No.2		correlation	correlation	correlation	correlation	correlation	correlation
	No.3	regression R2 = 0.95	regression R2 = 0.92	regression R2 = 0.97	regression R2 = 0.93	regression R2 = 0.92	regression R2 = 0.82	
Leak Noise Correlating	No.1	correlation	correlation	correlation	correlation	regression R2 = 0.75	correlation	
	No.2					regression R2 = 0.81	correlation	
	No.3	correlation	regression R2 = 0.7			correlation	regression R2 = 0.72	
Noise Logging	No.1					regression R2 = 0.78	regression R2 = 0.74	
	No.2					regression R2 = 0.81	correlation	
	No.3	correlation	correlation			regression R2 = 0.73	correlation	

Comparison between surveys

The greatest number of leaks and volumetric savings were found during the first sweep. Volumetric savings during the first survey were around 7 times greater than during the second and third investigations. Less time was also spent to search for leaks during the second and the third surveys. The decrease in time spent between the first and the second survey could be explained for noise logging and leak noise correlating in part by the fact

- the distance between fittings for correlation survey may have been recorded on the initial survey;
- the location of valves, washouts and fire hydrants picked from GIS, where loggers can be placed on, may have not proved to be a suitable location to place a logger or correlate from. Once new suitable locations had been established, they would have possibly been recorded and used for the subsequent surveys.
- the productivity of technicians improved with their knowledge of the areas.

During the second leakage detection survey, TVW leakage technicians had to be replaced by Contractor leakage technicians. Concerns were then raised by the Leakage Manager in charge in regards to the technical competency of the contractors brought onto the project. This might have had an effect on the observed decrease in jobs found and volumetric savings between the first and the second survey.

Conclusion

Despite inconclusive statistical analyses in relation to the efficiency of the 4 techniques considered, this study has yielded some interesting initial results:

- The most leaks per day were found through simple sounding but mainly at communication pipes and stop taps.
- The most volumetric saving per day was achieved through noise logging with leaks identified mainly at communication pipes, stop taps and fire hydrants. Noise Logging also identified the most main bursts.
- Leak noise correlating gave best results in rural areas, where long length of main without connections can be found and where traffic management is less intense than in urban areas.
- The step test technique was the most difficult to apply due to water quality issues and was therefore only applied once to three DMAs.
- Leakage detection techniques are reliant on the technicians using them and it is therefore fundamental for all leakage technicians to be trained properly.

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An Economic Active Leakage Control Policy without a Performance Indicator is not a Myth

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Key Words: ALC; Performance Indicators; ILI

Abstract

City of Calgary - Water Services instigated a leakage study of certain parts of the city to estimate the time period to compete a survey of Calgary. This paper explains the technique used and the findings from the study.

The paper shows that regardless of the level of water losses reported, it is still possible to perform Active Leakage Control (ALC) and develop new techniques to increase water loss savings.

Some of the findings from this study are as follows:

- There is a correlation between an acoustic numeric value recorded on the ground microphone and the size/type of water leak; the size and type of leak can be estimated by the analysis of data sets and from confirmed leaks. In assessing the impact of leaks on the infrastructure fitments, indications within the Calgary distribution system are that the numeric value of 45 (using the Gutermann Aqua Scope 3 - AS3) and above is the ‘intervention’ level for conducting additional investigation on metallic mains and surrounding infrastructure. It should be noted that whenever a survey such as this is to be considered an acoustic calibration exercise should be completed.
- The propagation of sound values (using the AS3) on the non-metallic material mains and infrastructure appear to much less to that of metallic, therefore without the addition of supplementary data sets taken from confirmed leak types on this material type the methodology established cannot be supported.
- This technique has proved successful and more investigation is required to obtain a generic footprint for non metallic pipe-work. This shows that some form of ALC can be performed with success from an unskilled operator with minimum training.

Introduction

Hydrosave was commissioned by City of Calgary - Water Services to assist with the development of a bespoke leakage detection methodology anticipated to complement the existing detection resources with the addition of a robust procedure, which if proven successful could be utilised by an unskilled operative in order to successfully identify potential areas of loss from the below ground infrastructure.

These potential ‘areas of interest’ (AOI) could then be recorded and passed to a skilled operator for further investigation and the confirmation of leak position carried out with the assistance of specialist acoustic equipment.

This paper presents the findings of the survey undertaken by Hydrosave in association with City of Calgary - Water Services during 1st – 5th May and 9th – 23rd June 2006.

The real losses currently being estimated from its below ground assets in Calgary are recorded as a percentage of volumetric input, this is currently estimated as being some 12 per cent of the total water into supply. No calculation of the ILI has been completed for the city as a whole but has been for smaller temporary DMA's, and these results are provided later in this paper.

Although 'percentage of volume input' is the method currently being used by City of Calgary - Water Services to measure the real losses, it is important to note that the authors do not support the use of the 'percentage of volume input' as the sole PI for real losses but rather as one of several PIs that should be used collectively

The City of Calgary - Water Services also does not agree with "percentage of water input" being the sole PI for water loss, and will be undergoing a full IWA water audit in late 2007 and 2008.

The following PIs are suggested as being more reliable PI:

- litres/service connection day or L/km/day (if service connections density is > 20);
- ILI

Survey Scope

The survey scope was to develop a robust methodology utilising an acoustic listening device based upon a leak noise value represented by a numeric reading and where possible obtain an acoustic "footprint" for the supply area. The equipment of choice chosen for this purpose was the Aqua Scope 3 AS3 manufactured and supplied by Gutermann Limited the reason being it displays a numerical reading for the acoustic noise heard. The data captured during the process was analysed to identify the correlation between the numeric value and material type and, where possible, substantiated with recorded values taken from known leak positions. Survey times were also monitored and measured so that an approximate estimate of manpower, projected timescales and geographical coverage could be ascertained.

The survey methodology itself and subsequent data collection and assessment were aligned with the International Water Association's (IWA) & American Water Works Association (AWWA) principles and practices

Survey Team

The project survey team consisted of personnel from both Hydrosave and City of Calgary - Water Services whose skills were complementary in the significant identification of the project outcomes.

Survey Areas

The water supply network coverage within the chosen areas for the development of the methodology is approximately 120 square kilometres. These areas are sub-divided into city districts that are represented by a variety of topographic and demographic property types. Each district incorporates about 30 square kilometres of water supply network and integrates a proportionate mix of both material and fitment types. The network is generally constructed on a generic grid-based design with a singular strategic supplying main. In some instances the areas are independently fitted with localised pressure management facilities.

Four districts in total were chosen which concentrated principally upon the residential areas:

- Lake View;
- Glenmore;
- Bowness;
- Spy Hill.

The real losses for these survey areas were unknown as flow monitoring by area is currently limited, however flow data was gathered 12 months prior to the survey.

Fitments, pipe-work and service valves within these areas are mostly accessible and in some instances situated within unmade access routes sited toward the rear of properties. Above ground fire hydrants are generally positioned within the carriageway and are clearly visible with servicing valves installed controlling the fire hydrants.

Analysing the DMAs

One of the main goals of establishing a DMA is to answer these two important and prime questions of water management, "Where is the water coming from?" and "Where is it going to?" Once the DMA is isolated and the flow meter(s) installed, the 1st question can be answered. The 2nd is answered by monitoring the incoming flow and calculating the losses.

Snapshot ILI & basic cost analysis in Bowness

With the zone closed in the following was losses was recorded

UARL = 12m^3/hour

CARL =50m^3/hour

If all leaks are located then 38m^3/hour can be saved equating to an annual loss of 332.8 ML with a dollar value of $29,880 (based on $90/ML). Based on this information, a leakage survey should be cost effective however other factors also have to be considered.

CARL/UARL = ILI

50/12 = 4.1

Bowness ILI = 4.1

Snapshot ILI & basic cost analysis in Lakeview

With the zone closed in the following was losses was recorded

UARL = 1.5m^3/hour

CARL = 10m^3/hour

If all leaks are located then 8.5m^3/hour can be saved equating to an annual loss of 74.5 ML with a dollar value of $6,700 (based on $90/ML). Based on this information, a leakage survey would not be cost effective however other factors also have to be. CARL/UARL = ILI
10/1.5 = 6.3
Lakeview ILI = 6.6

This area was considered to small for the ILI calculation to be valid.

Based on this information it may not be worth while at this time to perform a leak survey of the area. The average cost for a repair is $8,000. If it was a single leak, it would pay for itself in a little over a year, however if the losses are coming from multiple small leaks, the cost may be prohibitive. This area has higher real losses than desired for such a small area, but the cost for multiple repairs may not be economically justified.

Equipment

The acoustic equipment utilised during the survey (Gutermann AS3) was selected by City of Calgary - Water Services and the methodology was to be developed using this brand of equipment.

Methodology - General Principle

The primary survey methodology was to incorporate the sounding of all available fitments. A secondary survey option was also identified which concentrated on the use of fire hydrants and fire hydrant controlling valves only.

Each survey option was completed under comparable circumstances by a two-man team comprising a team leader and an assistant with the data collection process defined by the following categories:

- Asset type;
- Asset number;
- Numeric reading;
- Material type;
- Geographical reference.

The primary survey option was completed by the sounding of every available fitment with the AS3. In some instances access to below ground servicing valves was partially obstructed and could only be made by the breaking and removal of the compacted debris around the proximity of the valve chamber and cover.

The secondary survey option was to concentrate on the sounding of the fire hydrant and fire hydrant controlling valve in order to determine if a correlation between the two methods could be obtained and if the survey option was successful in identifying high volume losses.

In both instances the survey equipment was configured with filters off, a volume setting of 75% of the total and held directly to the fitment surface for a minimum period of at least 20 seconds. No headphones were used and numerical values taken only, this was so no sound interpretation was taken into account from an experienced leakage engineer.

The numerical representation of generated system noise was captured from the digital indicator on the AS3 and duly recorded. In all instances the lowest numerical reading captured by the equipment was used.

Numerical Clustering

On the completion of each sub-division, the representative numerical data captured for each fitment surveyed would be overlaid onto a network schematic plan detailing asset position, size, type and distinct reference code. From this process a numerical footprint of the sub-division was obtained and an associated assessment completed. This highlighted any obvious numerical values in close proximity of the defined “intervention denominator” associated around a singular geographical point.

Prior to any further analysis, each high reading point was evaluated against an asset location. This was done as the asset may be in the location of a known noise, for example a pressure reducing valve or main incoming supply valve.

Each AOI would initially involve the re-capture of the numerical value recorded on the fitment at least 2 hours later or the following day at a different time within the day in case the value was that of domestic draw-off. Should the numerical values be comparable with those initially captured then a more robust leak location process should be instigated with the utilisation of standard leak noise correlation processes and above ground acoustic listening techniques.

The AOI's may only then be removed from the schedule by the confirmation of leak position or quantification of the numerical value as legitimate consumption.

The overall numerical value data range captured throughout the project is presented at table 1

Table 1: All Fitments Survey Results

Primary survey – All fittings sounded	
Total numerical range	Material type
25 - 43	PVC
26 - 70	Cast Iron
27 - 60	Ductile Iron

Intervention Denominators

The numeric data captured was generally found to be proportional across all material and fitting types particularly where no leaks were present. In places where several leaks were located, a significant increase in the numerical value was recorded. This increase in the numerical value can not indicate the size of the leak recorded for a known type of pipe material without further assessment of data captured from known losses.

In an attempt to validate the numeric values captured and establish a leak noise threshold by material type, a simulation of a known loss was replicated from the operation of a fire hydrant and the impact of the surrounding numeric values recorded on all valve types situated in close proximity.

Firstly a simulation of a known loss was replicated from the operation of a fire hydrant and the measurement of the subsequent impact of this upon surrounding numeric values recorded on all available valve types situated within a close proximity, the results of which are presented within table 2.

Table 2: Numerical values obtained during test

Asset Type	Material Type	Distance From Source (m)	Numeric Value (1)	Numeric Value (2)	Numeric Value (3)
Sluice Valve	Cast Iron	57	34	35	39
Sluice Valve	Cast Iron	57	31	37	39
Sluice Valve	Cast Iron	51	31	37	49
Fire Hydrant	Cast Iron	0	42	78	82
Sluice Valve	Cast Iron	38	31	39	61
Sluice Valve	Cast Iron	43	35	38	69
Sluice Valve	PVC	43	36	32	38

(1) Reading captured prior to test

(2) Reading captured during test at 0.2 litres per second

(3) Reading captured during test at 1.0 litres per second

From the above it was found that the amount of induced losses had little impact on the numeric values recorded on fitments directly situated on the non metallic pipe. Only those fitments situated on the metallic pipe recorded any “uplift” in numeric activity which was found to be both distance and flow proportionate.

Below are numeric values recorded during this exercise and any singular point identified by a numeric clustering greater than 40 (after calibration of the acoustic noise propagation in this area) was initially highlighted as an AOI and duly scheduled for further investigation, the outcome of which has been summarised below in table 3

Table 3: Area of interest summary

Area Reference	Numerical Profile Range	Investigative Outcome
Bowness	40 - 59	PRV location no leaks found
Bowness	37 - 57	Incoming supply no leaks found
Bowness	46 - 60	Burst service pipe at ferrule connection
Glenmore	56 & 69 & 70	3no leaking service valves found

Completion of the investigative outcomes identified a number of leaks, the predominant being from a numeric clustering of between 46-60 and over a 200m distance. These leaks arose from a combined failure of a copper service pipe and associated ferrule connection situated at a cast iron main. Following excavation and visual confirmation prior to repair, the loss was estimated at some 3 litres per second.

The remaining leaks identified were found to be general losses from service valves, predominantly from around the operating gland or the general corrosion to the body and bolt sets. These leaks shown readings of 56-70 when the ASA3 was placed on the fittings but there was little or no transmittal of these leaks noise the surrounding fitments.

Unfortunately the impact on the numeric values obtained from the confirmed leak position did not incorporate a proportional or representative amount of plastic pipe material within the immediate peripheral vicinity. Therefore additional information should be obtained before a definite conclusion can be drawn.

In considering the relationship between numerical value and material type it is suggested that the following intervention denominators at table 4 be considered.

Table 4: Numerical intervention denominators

Pipe Material	Numeric Intervention Denominator Value
PVC	40
Cast Iron	45
Ductile Iron	45

Survey Cycle

The survey cycle was completed in accordance with the two methodologies outlined in the section above. As previously mentioned, the survey areas comprised a variety of topographical, social and economic property types thus allowing a realistic productivity ratio to be ascertained. From the data obtained it can be reasonably concluded that: -

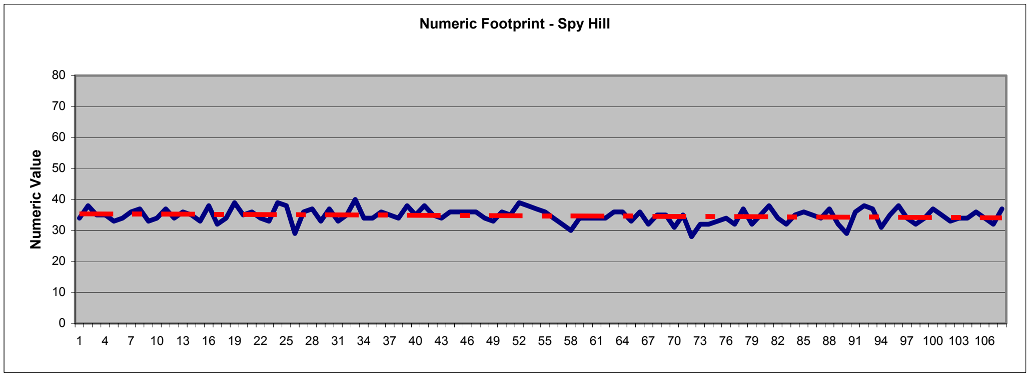
- In carrying out an all fitment survey, an average timescale between fitments of 5.51 minutes can be expected;
- In carrying out a fire hydrant & controlling valve survey only an average timescale between fitments of 5.0 minutes can be expected.
- Average daily distance listening on all fittings was 5.8km/ day
- Average daily distance listening on fire hydrants only was 14.2km/day
- It must be noted that coverage of mains length listening on fire hydrants only was 230% greater than doing that of all fittings.

(Timescales may vary dependent upon local climatic conditions)

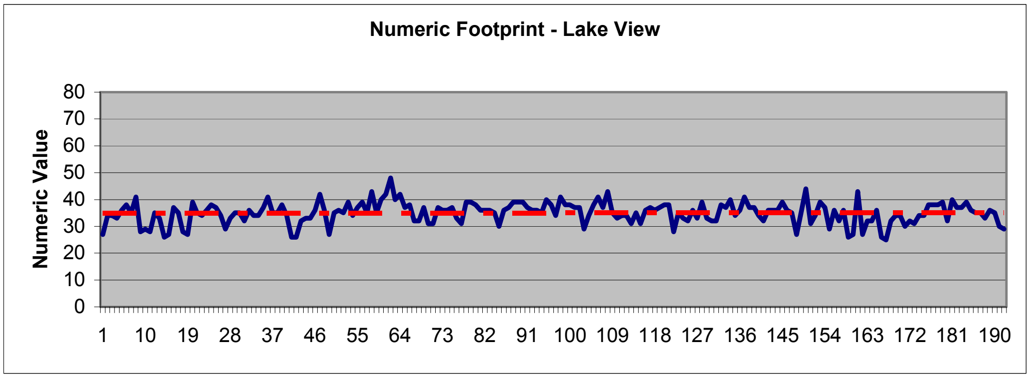
Numeric Footprints

In analysing the numeric data sets recorded during the survey it is possible to build a graphical representation of each district meter area. Each representation may then be considered as a numeric footprint of the water supply characteristics and utilised as a reference point when carrying out any future survey, especially if any significant time period has elapsed.

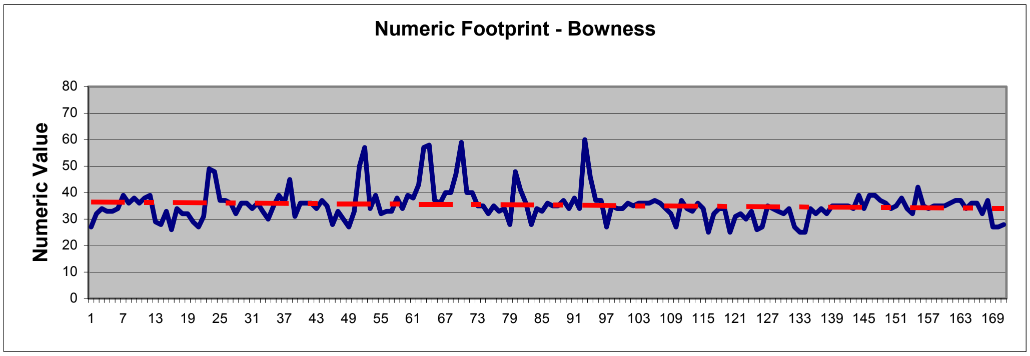
Graph 1: Numeric footprint – Spy Hill



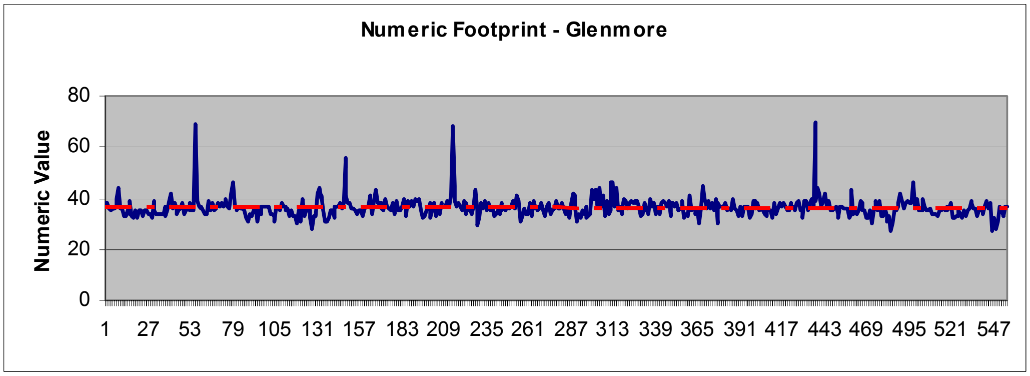
Graph 2: Numeric footprint – Lake View



Graph 3: Numeric footprint – Bowness

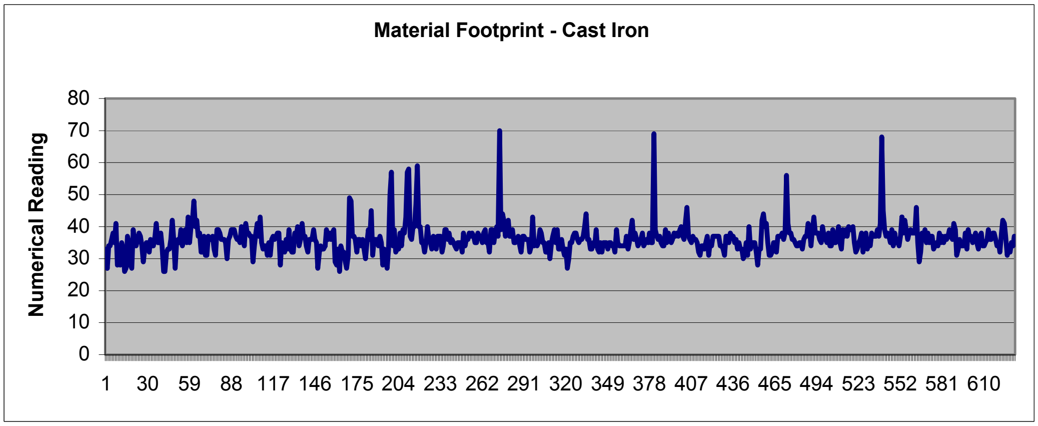


Graph 4: Numeric footprint – Glenmore

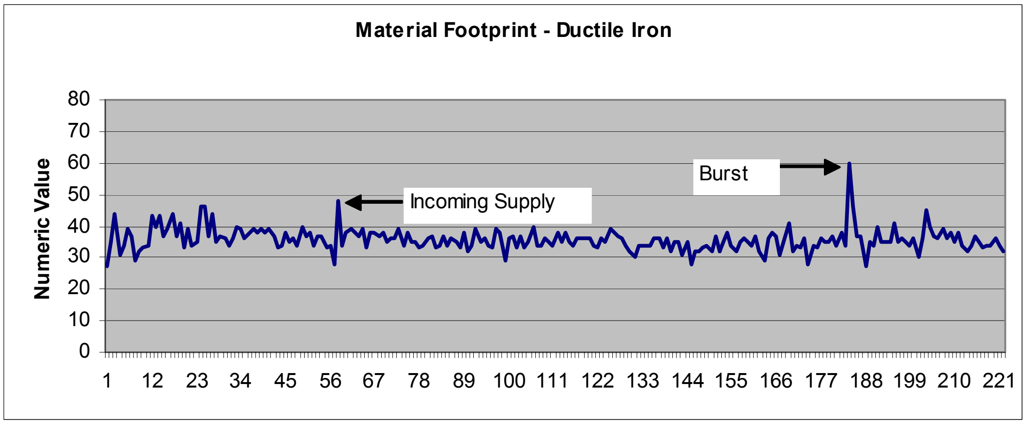


On the whole it was found that the footprint for most supply mains situated within the survey areas and constructed of a metallic composite such as Cast or Ductile Iron recorded a comparable numeric value. This was the case for both normal operating circumstances and where a leak noise was generated. However the footprint obtained for non-metallic composites such as PVC was recorded at a consistently lower numeric value; this can be anticipated due to the inherent low dispersal of sound through non-metallic composites.

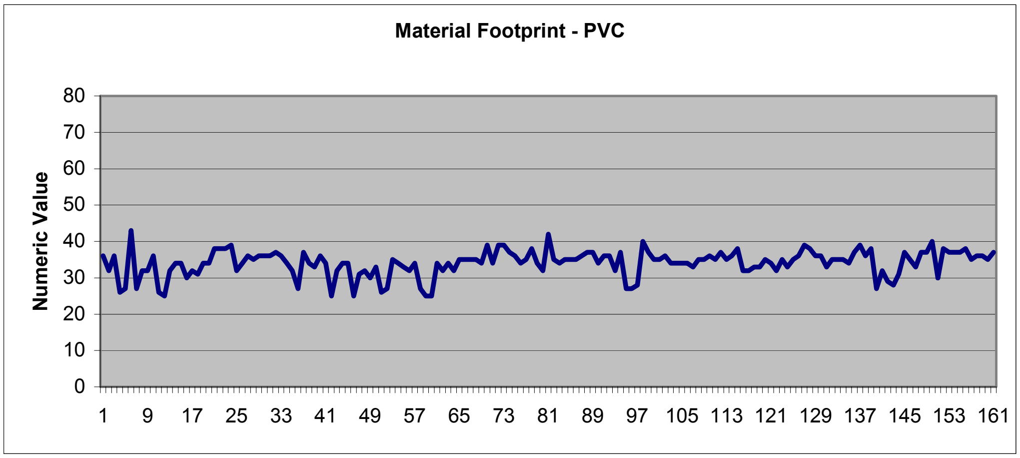
Graph 5: Numeric footprint – Cast Iron



Graph 6: Numeric footprint – Ductile Iron



Graph 7: Numeric footprint – Non Metallic



Conclusions

From the data sets obtained from completing the survey, the following conclusions have been made:

- There is a correlation between a numeric value, size of water leak and the pressure within the area.

- A calibration exercise should be carried out in each area to establish the acoustic noise transmittal properties and the intervention number.
- The propagation of sound values (using the AS3) on the non-metallic material mains and infrastructure appear to be much less to that of metallic mains. Therefore without the addition of supplementary data sets taken from confirmed leak types on this material type, the methodology established cannot be substantially supported.
- Although the secondary survey option may be useful in identifying a catastrophic failure within a survey cycle, the primary survey option to incorporate all fitments is suggested as perhaps a more robust method for inclusion into a routine pro-active control plan.
- Above ground fire hydrants were found to be generally susceptible to additional ambient and surrounding noise levels.
- Variation in local climatic conditions could in some instances impact on the numeric recordings obtained. In the case of high background noise, a numeric reading offset should be applied to compensate for the "uplift" in ambient conditions around the equipment.
- Fire hydrants, where water is replaced by air, gave a lower numeric value as the noise transfer was reliant totally on the wall material of the main. It is therefore considered that noise transferral is greatly enhanced when a main is pressurised with water and that sounding should only be carried out on fittings that are connected to a pressurised main.
- When carrying out an all fitments survey option, an average timescale between fitments of 5.51 minutes per fitting for each two-man team may be anticipated under average conditions.
- Average daily distance listening on all fittings was 5.8km/day
- Average daily distance listening on fire hydrants only was 14.2km/day
- This technique has proved successful and more investigation is required to obtain a generic footprint for non metallic pipe-work. This shows that some form of ALC can be performed with success from an unskilled operator with minimum training

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Innovations in step testing using sluice valve metering

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Keywords: step-test; metering: valve

Overview

One of the main challenges in leakage control is obtaining accurate information on the location of leaks across the pipeline network. Performing step tests has provided the best way to locate areas of high leakage of demonstrating the results following the repair of leaks. However, to date, this has required the setting up of monitoring areas and expensive installation of metering points across the distribution network.

This paper describes work that led to the development of a new portable valve flow meter that enables a conventional sluice valve to be converted into a temporary metering point. The Accuflow™ valve flow meter has opened up new step testing techniques which provide a cheaper and more effective way to pinpoint leaks. With a sluice valve on nearly every corner, metering flows across the network has at last become a realistic option for the Leakage Engineer. These new techniques will be of particular benefit in countries where high set-up costs have previous prevented the use of conventional step testing.

What improvements in step testing are needed?

The history of system monitoring for leakage control has long relied on meter installations at fixed locations across the water distribution system. These are set up so that all flow entering or leaving a defined District Meter Area (DMA), normally covering around 1,500 properties, is metered. This approach has the major advantage of quantifying the leakage, coupled with the ability to pinpoint the leak location using a step test procedure. The meters used are normally helical vane meters in fixed chamber installations. In some cases temporary insertion flow meter are used, which require a fixed insertion point and chamber to be installed on the water main.

However, the conventional approach to step testing using fixed metering points has a number of disadvantages. These include:

- The cost and effort required to installing meter points, and the lack of flexibility in changing these once established.
- The need for temporary shut off of customers' supplies for periods of an hour or more. This can cause problems for some customers and result in discoloured water.
- Where shut off steps are monitored by a meter (or meters) sited at the DMA boundary, it can often be hard to separate legitimate use from leakage.

In recent years, as an alternative approach, acoustic flow logging has been used. A number of loggers are normally deployed for a period across a defined area of the network. The analysis of the pattern of persistent noise generated by leaks provides an indication of the location of leakage. This approach has the advantage that it does not require any permanent installation, so is cheaper to operate, as the equipment can be moved from area to area. However, the level of noise recorded does not always have a strong relation to the size of the leak; for example, a large volume of leakage from a complete pipe break is likely to generate far less noise than small volume of leakage through a small crack. The level of noise detected is also dependent on the distance from the sensor and the type of pipe material.

The conclusions from the above considerations shows that the desirable approach would combine the flexibility and cost advantages of acoustic logging, with the kind of leak flow information provided by conventional step testing. Any such improvements would need to:

- Measure actual flows in the distribution network, rather than noise levels.
- Avoid expensive fixed installations.
- Eliminate the need for temporary shut off of customers' supplies, or at least reduce them to very short periods.
- Provide metering as close as possible to each step, reducing the impact of customer usage on leakage measurements.

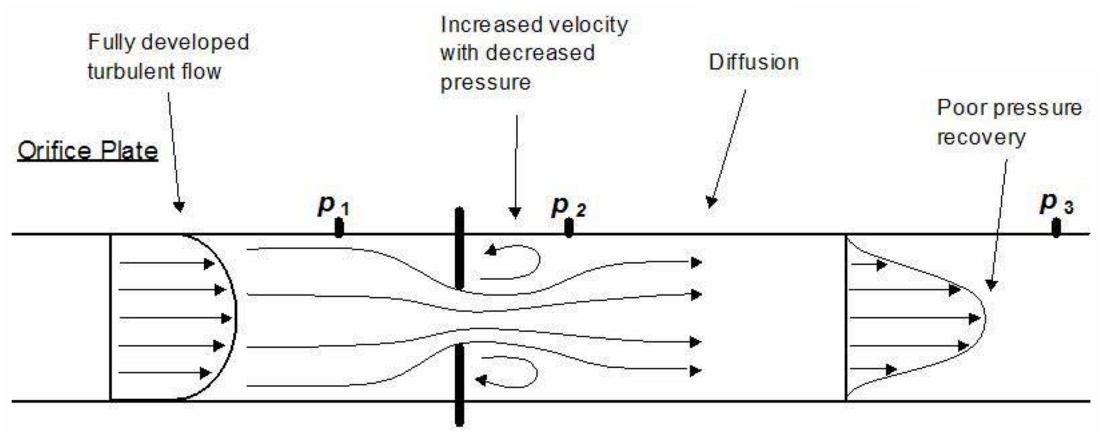
Developing new alternatives to conventional step testing

As described above, neither conventional step testing nor acoustic logging provides a completely satisfactory solution to leakage control, with each approach having its advantages and disadvantages. Extensive experience of using both techniques led RPS Water to review options for a completely new approach to leakage monitoring. Research carried out by Dŵr Cymru Welsh Water indicates that a third of repaired leaks had no noticeable impact on reducing measured leakage levels. This being the case, methods that provided a quantifiable assessment of leakage were seen as the most fruitful approach, so new options for system metering were investigated. The practical experience of the team at RPS made a major contribution to the development process by recalling that skilled leakage inspectors could estimate the size of a leak by closing down a stop valve and listening to the volume of the noise. Generally, the louder the noise, the greater the volume of leakage that was found. This empirical observation lead to questions as to whether this principle could be enhanced by the use of electronic acoustic measurement and the identification of a relationship between the flow rate and the acoustic profile.

These ideas started an initial investigation by RPS into possible links between flow rate and the acoustic signature. A variety of existing meter sites with sluice valves provided ideal locations for testing the theory. By logging the meter readings, and carrying out acoustic measurements as the sluice valve was slowly closed, RPS established that there is a positive correlation between the acoustic noise level and the full bore flow volume. The establishment of this principle justified the setting up of a Research and Development programme on valve flow metering. To help deliver this, RPS agreed a joint development project with Dŵr Cymru Welsh Water and Technolog. Dŵr Cymru Welsh Water contributed operational testing and support for the development of procedures which make effective use of the new technology. Technolog provided a wealth of experience in developing "state of the art" electronic devices for logging and measurement which are robust enough for field use.

Initial R&D – Defining the basic principles of valve flow metering

Valve flow metering is a new technique, although its principles are founded on traditional engineering hydraulics. The measurement method used is generally based on Bernoulli's equation of fluid flow, as applied to orifice flow meters. Here there is a relationship between the flow rate, the size of the pipe, the size of the orifice, the upstream pressure and the downstream pressure immediately after the constriction. In summary, by measuring the upstream and down stream pressures for a given pipe and orifice, the equation can be used to calculate the flow rate. A diagram illustrating the orifice flow meter is shown below



The normal sluice valve provides a means of varying the size of the orifice, although the orifice itself is at the bottom of the pipe, rather than in the centre, as in a traditional orifice plate. This tends to introduce additional factors in the measuring process, including difficulties in measuring the pressure immediately downstream of the constriction. However, as a sluice valve is closed, this also causes increased noise due to the turbulence generated as flow passes through the constriction. The greater the flow, for a given input pressure, pipe and valve combination, the larger the acoustic profile. This principle formed the basis for the development of the valve flow meter. In developing this technology, considerable research and testing was carried out to establish the relationships between flow rate and the resulting acoustic noise profile generated as a valve is closed, for different pipe size and pressure regimes.

The initial research started in 2004 and was based on over 300 individual field tests across Dŵr Cymru, rather than on laboratory based pipe configurations. This empirical approach means that the relationships developed apply to real life situations, not artificial ones. Flow measurement algorithms developed from the research activities opened up the way for development of an electronic control unit. Currently the valve flow meter has been calibrated for use on pipes of 200 mm and below, as this size range includes the majority of local distribution mains.

Developing the prototype valve flow meter

Once the principles, accuracy and algorithms had been established, project activities focussed on the development of the valve flow meter device itself. The challenges were provide a robust device which will fulfil the variety of functions required; operation as a conventional valve key, acoustic measurement, rotational measurement, input of data, output of operational instructions, together with processing, presentation and logging of results. This complex set of requirements necessitated an extended iterative approach, utilising practical field trials to ensure the device was robust and accurate. A number of prototypes were developed and extensively field tested; a picture of one of the early prototypes is given below.

The development of the prototype device and field trials was carried out during 2005 and patents were successfully obtained for the valve flow meter. The patent application process confirmed the unique nature of the device. More advanced field trials and the development of and manufacture of the first production model, called the Accuflow™, was completed during 2006. Development to date has involved over 1000 field tests which have has shown that the Accuflow delivers overall accuracy levels of +/- 10% at flow rates ranging from 0.3 litres/sec to 3 litres/sec. In relation to leakage control work, accuracy at low flows is an important characteristic of the Accuflow. By comparison, the minimum flow specified for a typical 100 mm helical vane meter used for DMA monitoring is 0.5 litres/sec.

The Accuflow device utilises leading edge technology for sensors, electronic control, graphical user interfaces and data logging and is pictured below. It delivers a unique combination of technologies integrated into one device, providing, for the first time, a way to measure flow using existing sluice valves. It incorporates the following functions:

- A traditional valve key, to open and close standard distribution sluice valves.

- A patented sensor system to log the rotation of the device when opening or closing a valve to take a flow measurement.
- An acoustic transducer, located in the bottom of the Accuflow, which rests on the valve head to accurately record changes in the acoustic profile.
- A LCD display unit and keypad for the user input of key data which provides visual and audible signals giving step-by-step instructions on how to close the valve in a controlled fashion to take the measurements.
- A microprocessor unit which records and processes the input and sensor derived data to provide an estimation of flow.

Development to date has identified the following requirement for the Valve Flow Meter to operate effectively:

- A pipeline diameter of between 50mm and 200mm, with a sluice valve the same diameter as the pipe
- A sluice valve in reasonably good condition, with leak proof closure
- A clean valve head to provide a good contact with the acoustic sensor
- The facility to take a pressure reading upstream of the valve (normally within 10 m)
- The ability to completely close the valve for a short period, although this is only normally required for 10 seconds or so.
- Flow rate and inlet pressure during the period of the measurement should be fairly constant.

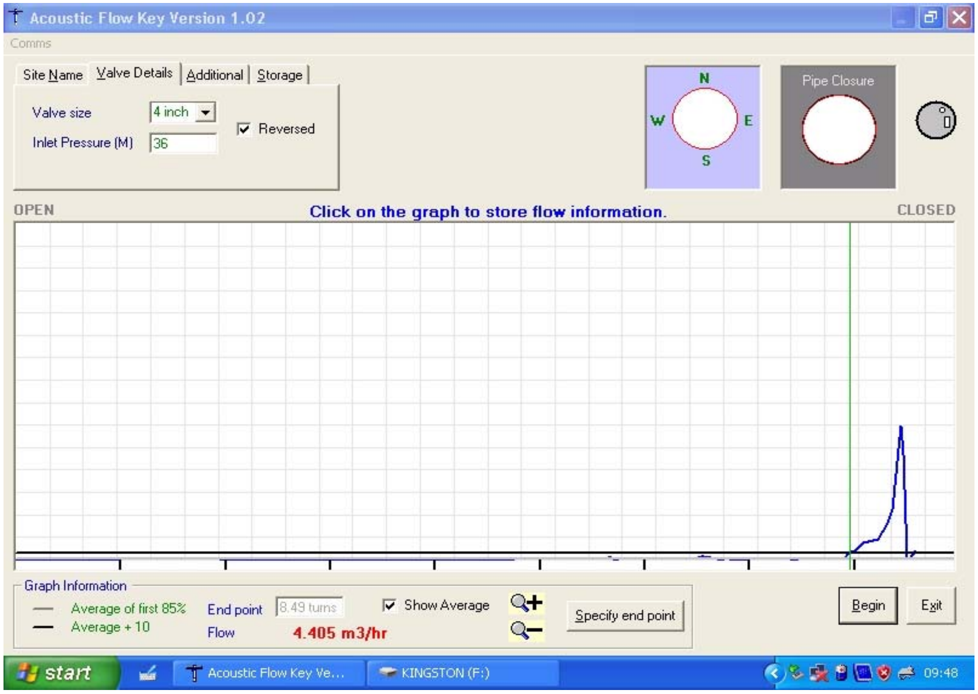
How to take a flow measurement using the Accuflow

The Accuflow valve flow meter can be easily operated by one technician, having been designed for ease of use. However, the situations where it is likely to be of most use, such as measuring night flow across the distribution system, will required staff with a high standard of training and experience to get the best results out of the device.

The basic steps in using the Accuflow valve flow meter are outlined below. They assume that the valve has been exercised and checked prior to arrival on site, as have any valves necessary to isolate the flow to the valve being used as the metering point.

1. Carry out any isolation needed to ensure all flow passes through the valve selected to act as the meter point.
2. Place the Accuflow on the sluice valve to be used as the metering point.
3. Check that the valve is fully open.
4. Measure the upstream pressure.
5. Switch on the Accuflow and input the site location, upstream pressure, pipe diameter and direction of close for the valve.
6. Press start and close the valve as instructed by the display and audible prompts, stopping and restarting closing of the valve as instructed.
7. Complete closure of the valve and wait the flow is displayed before staring to reopen the valve.
8. Reopen the valve in accordance with normal operational procedures (note that once the Accuflow has indicated that valve is ready to be reopened, the Accuflow can be switched off and removed for use at another location, and the valve can be reopened using a conventional valve key if desired).

The time required for the flow measuring operation will depend on the valve size. The valve can be closed in larger steps at first, but then in much smaller stages from 75% closed, as instructed by the Accuflow display. The total operation will take about ten minutes for a 100mm pipe. If an incorrect valve pipe size has been input the device will identify this and will indicate whether a retest is necessary. Any major fluctuation in flow during the measuring operation will also trigger an unsuccessful test indication on the display. A typical display is shown below.


Acoustic Flow Key Version 1.02

Valve size: 4 inch

Inlet Pressure (M): 36

☑ Reversed

Click on the graph to store flow information.

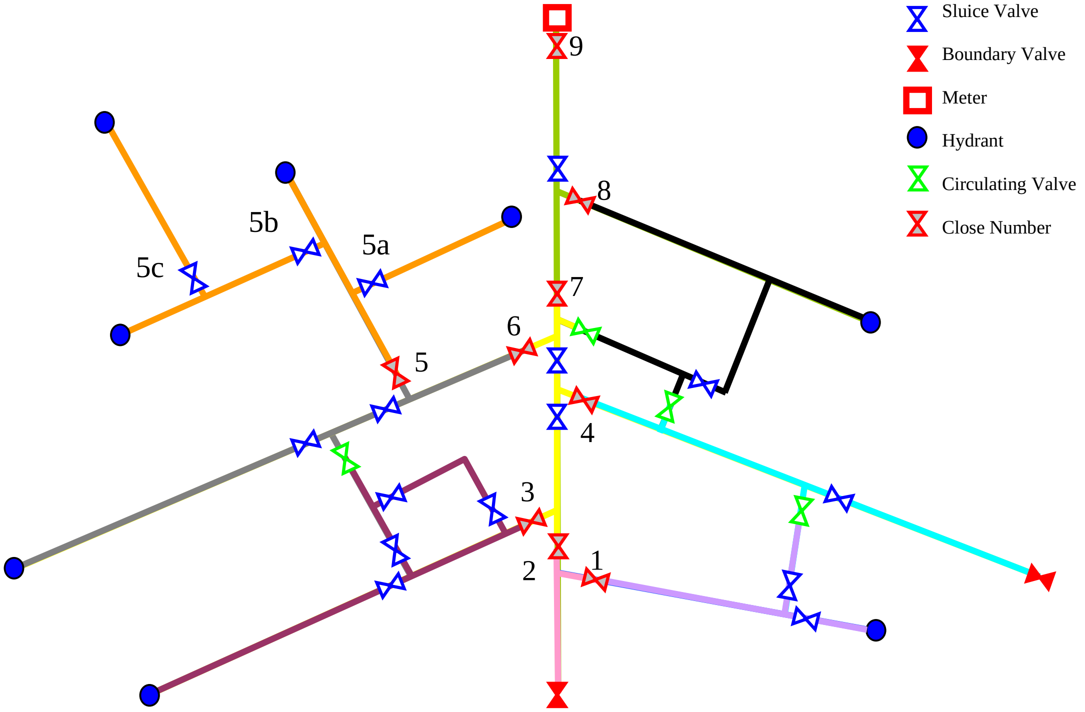
End point: 8.49 turns

☑ Show Average

Flow: 4.405 m3/hr

Developing new step testing methods using the Accuflow

Since November 2006 a number of RPS Water leakage control teams trained in using the Accuflow™ have been working with five water industry clients across the UK. This approach has allowed the device to be used in a wide range of operational situations. The step testing procedure is normally carried out at night and is similar to that conventionally used for step testing, but with some important differences. These can be explained by reference to the schematic diagram below.



The first step is to isolate the DMA by closing the boundary valves and to close all the circulating valves to ensure that each step has a single feed. Testing is then carried out by using the Accuflow to measure the flow at point 1, as explained under "How to take a flow measurement using the Accuflow". As soon as the Accuflow has calculated a flow value, the valve at point 1 can to be reopened with a conventional valve key, while the Accuflow is moved to point 2 to start taking the next flow reading. The process continues at the remaining step points, 3 through to 9. Where a DMA inlet meter is fitted, it is recommended that the flow is logged during the step testing exercise to provide an additional cross check of the flows into the DMA. Once the all steps 1 through 9 have been metered using the Accuflow, it is possible to review the results to see if any additional readings within the DMA are beneficial. For example, if there were an unexplained high flow at step 5, it would help pinpoint the location of leakage if further Accuflow readings were taken at the three sluice valves downstream of step point 5 (shown as 5A, 5B and 5C).

Cost and performance comparison of conventional step testing against the Accuflow

The varied conditions experienced across the water companies involved in the trials have providing valuable experience in refining the principles of step testing to maximise the advantages of using the Accuflow. The costs of carrying out step testing using the Accuflow in an existing DMA is similar to conventional step testing using radio linked equipment. Where a DMA is not already set-up, step testing can be implemented using the Accuflow without incurring the capital set-up costs associated with a DMA.

In addition to cost advantages, the Accuflow step testing procedure outlined above has major performance advantages over conventional step testing. Firstly, the ability to use the sluice valve at each shut off step as the actual metering point, rather than the remote meter at the inlet to the DMA. The effect of taking measurements at local street level eliminates the effect of any legitimate night use upstream of the step test point. Secondly, the approach is far more flexible, as extra measurements can easily be taken at any sluice valve to help better pinpoint leakage for further investigation. Any improvement in the measurement and pinpointing of leaks has significant potential to reduce the number of non-productive leaks that are identified for repair.

In relation to performance in quantifying and pinpointing leaks, all experience to date has indicated that step testing using the Accuflow produces significantly better results than conventional step testing. This is of particular importance given the high cost of leak repairs. Research carried out by Dŵr Cymru Welsh Water indicates that over a third of repaired leaks have no noticeable impact on reducing measured leakage levels. The cost of repair of these non-productive leaks to the UK Water Industry has been estimated to be in excess of £100 million per year. Trials so far have indicated that it is not unrealistic to expect that the number of non-productive leaks could be reduced by a third by using improved Accuflow based step testing. This would provide a potential saving of over £11 million each year across the UK.

Options for replacing DMAs with Accuflow monitored areas

The cost of setting-up, monitoring and maintaining a DMA has a high ongoing cost and typical values are given in Table 1 below. The Accuflow offers the possibility of carrying out area monitoring at far lower cost, which may prove to be a satisfactory and cost effective approach to leakage monitoring for many situations. The cost of using the Accuflow to valve in a discreet area at night four times a year to provide leakage monitoring information is given in Table 2 below

Table 1 - Cost of setting-up, monitoring and maintaining a DMA

Activity	Cost per year		Cost over 10 years	
	£	€	£	€
Initial DMA set-up costs (typically £5k to £50k, assume £15k)	1,500	2,175	15,000	21,500
Maintaining boundary integrity	400	580	4,000	5,800
Logging and data transfer	300	435	3,000	4,350
Flushing dead end mains	400	580	4,000	5,800
Replacing boundary meters	300	435	3,000	4,350
Total cost per DMA (average 1,500 properties)	2,900	4,205	29,000	42,050
Cost for a utility with 300,000 properties	580,000	841,000	5,800,000	8,410,000

Table 2 - Cost of monitoring an area of 1,500 properties using
Accuflow testing, assuming four monitoring visits per year

Activity	Cost per year		Cost over 10 years	
	£	€	£	€
Capital cost of equipment (five year life,	40	58	400	580

assuming 200 areas monitored each year)				
Labour costs for testing (4 visit per year for 3 hours at £50/hour)	600	870	6,000	8,700
Total cost per 1,500 properties	640	928	6400	9280
Cost for a utility with 300,000 properties	128,000	185,600	1,280,000	1,856,000

These cost figures indicate that using the Accuflow can be expected to cost well under three quarters of the cost of an approach using conventional DMA monitoring. Even if Accuflow monitoring was necessary every month, it would still work out cheaper than continuous DMA monitoring. For utilities without the regulatory requirement for continuous leakage monitoring, establishing leakage monitoring areas using the Accuflow provides a new cost effective way to identifying areas with greatest levels of leakage.

In summary …….

The innovative technology of the Accuflow and valve flow metering offers significant cost and performance benefits when compared with other established techniques such as acoustic logging, DMA monitoring and conventional step testing. The use of enhanced step testing using valve flow meters has been shown to improve identification of those leaks worth repairing and so significantly improve the efficiency of leakage detection and repair activities. The advantages include:

- It requires no fixed installations and so reduces capital costs.
- The flow into any zone can be seen as soon as the valve is closed, rather than relying on logging or additional staff at a remote input meter.
- It provides high quality quantitative data on the levels of leakage and enables better targeting of leaks worth repairing.
- Unlike acoustic logging, it does not relay on noise to indicate the size of a leak, which can sometime give misleading results leading to unnecessary repairs.
- The metering points can be close to the area being monitored, reducing the impact of customer usage on leakage measurements.
- The controlled nature of the valve operation reduces the likelihood of causing small bursts on weak mains or of discoloured supplies.
- The short period of complete valve closure required means minimal disruption to customers' supplies.
- It allows follow-up monitoring to determine the effect of leak repairs.

The “Holy Grail” of leakage control has always been the ability to accurately pinpoint the location and volume of leaks, without the cost of expensive fixed asset installations. The Accuflow™ provides a significant step forward towards this goal.

Further information on the techniques described in the technical paper can be obtained by contacting Arthur Arscott at RPS Water:
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Development of an Enterprise System of Integrated Water Leakage Management Applications (*i* WLMA) for Bangkok's Metropolitan Waterworks Authority

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Keywords: Water Leakage Management Applications (WLMA); SCADA System; Knowledge Repository

Abstract

Bangkok's Metropolitan Waterworks Authority (MWA)[1] supplies potable water to a population of approximately 12 million throughout the capital city of Thailand. In order to continually improve levels of service to the customers, the MWA recently embarked on a major programme to introduce the latest information and communication technology. The project is scheduled for completion during 2007 and is divided into four contracts – one for each of the four main regions. Each contract involves setting up appropriate District Metered Areas (DMA's) and Pressure Management Areas (PMA's), each of which is designed by the project team and includes meter installations, Remote Terminal Units (RTUs) and SCADA installations. The development of the software called "Integrated Water Leakage Management Applications (iWLMA)," used to manage the operation of the water reticulation system (referred in this paper as simply the "System Software") forms the basis of the management information system which creates all checks and balances required to maintain an efficient operation. Overall system integration of all regions will be coordinated at a central data repository which will involve the collection and processing of data from SCADA system on both DMA's and Trunk Main's RTUs, billing data from the Customer Information System (CIS) and financial information from the enterprise resources planning (ERP) system (SAP) to the System Software.

This paper focuses on the System Software design and development which was one of the key features of the $70 Million US project. Using capability maturity model and model driven architecture approaches in design and development to ensure proper standardization and quality control, the project team has created sophisticated software applications that can easily be upgraded and modified as required in future without requiring any major re-design. The development team involved over 50 personnel at any one time and the selection of J2EE and the Windows platform ensured that it was possible to coordinate the various activities in a clear and systematic manner with proven and low ownership costs. The paper describes some of the key features of the System Software architecture and modules including the water balance, economic level of leakage, night flow analysis, field service management, pressure management), infrastructure information management, management reporting and knowledge repository. Each element has been based on the accepted best practice world-wide where possible although certain modifications were required to accommodate some of the unique features of the MWA water supply system which will be discussed in the paper.

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[1] http://www.mwa.co.th

MWA in Retrospect

Established on July 13, 1910, the Metropolitan Waterworks Authority (MWA) supplies potable water to around 1.77 million customers serving approximately 12 million people in Bangkok Metropolitan Areas covering over 2,000 km^2. by utilizing her four treatment plants and 14 pumping stations with total production of 1,700 Mil.m^3/year through 22,000 km of pipe lines including transmission, trunk main, distribution and service pipes.

MWA's Program on Water Loss Management Technology

For the past 10 years, MWA has attempted to reduce the water loss by increasing budget and activities in active leakage control including sounding of all distribution mains twice a year. Figure 1. shows the results of %UFW reduction over the 10 years period against the cost of active leakage control.

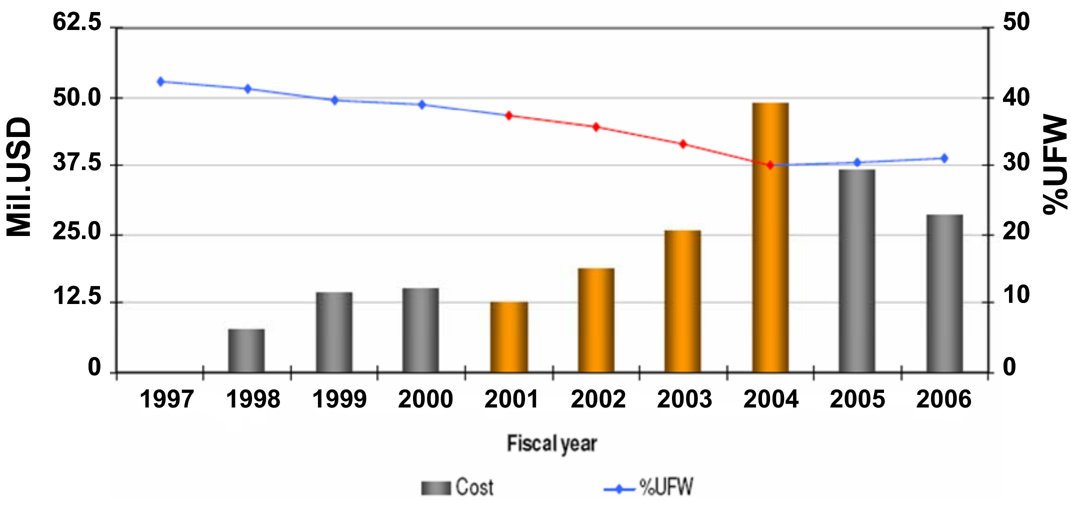

Figure 1. MWA Cost of Leakage Control Vs. Unaccountable for Water (UFW)

However, MWA forecasted that in the near future, they would encounter the lager water loss problems with the system high natural rate of rising of 8.18% (NESDB, 1999), due to the aging of the system together with the policy from the board to increase the service level by increasing average pressure of the entire system from approximately 5m to 6.00 m by the end of 2007. To cope with such a large operational endeavor and service level as well as high current water loss (30% UFW), MWA has turned to the latest information and communication technology (ICT) by developing an ICT solution for water loss management project with the total budget of $70 Million US. The project is scheduled for completion during 2007 and is divided into four contracts – one for each of the four main regions. Each contract involves setting up appropriate District Metered Areas (DMA's) and Pressure Management Areas (PMA's), each of which is designed by the project team and includes meter, Pressure Sensor, Pressure Reducing Valve (PRV) installations, Remote Terminal Units (RTUs) and SCADA installations, as well as 15 control center installations. Each control center is equipped with a System Software and computer system that integrated to the centralized databases and other legacy system at MWA headquarter.

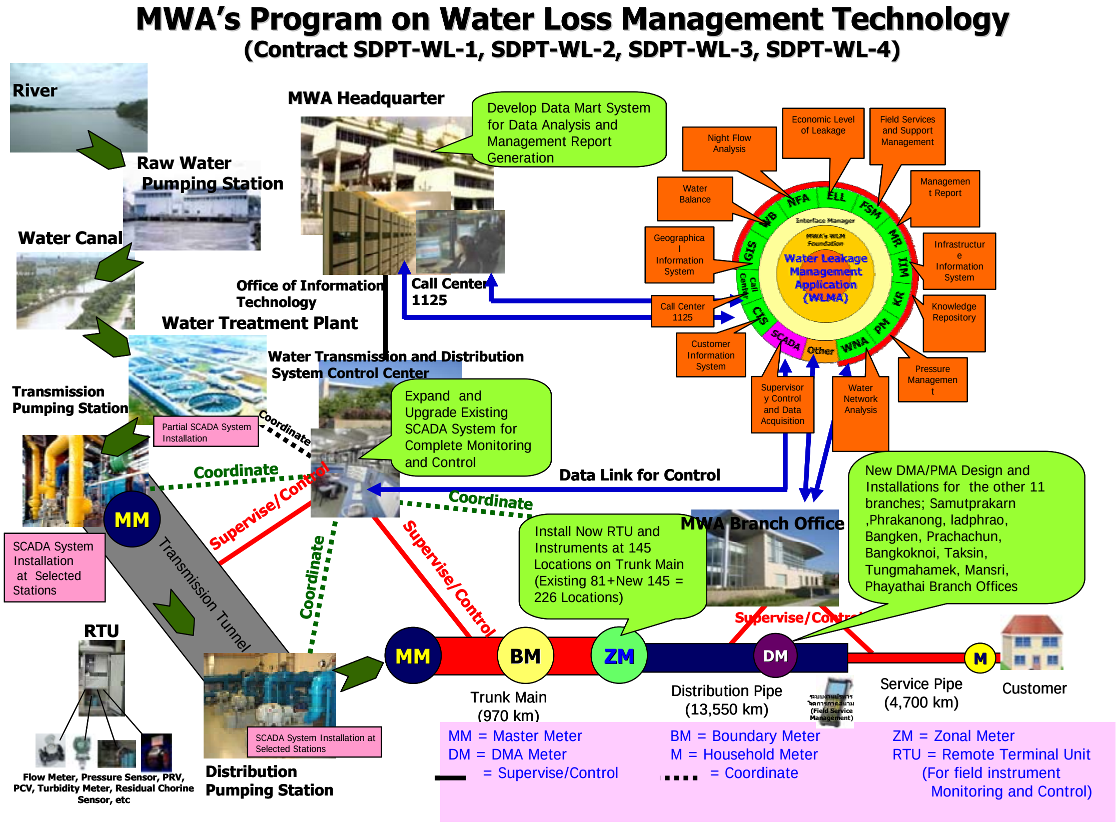

Figure 2. MWA Water Loss Management Program (dividing into four projects) using Information Communication Technology

The above figure illustrates the project scope and integration to a core technology call integrated water leakage management applications (i.e. "the System Software"). This forms the basis of the management information system which creates all checks and balances required to maintain an efficient operation. Using GIS based information, overall system integration of all regions will be coordinated at a central data repository of each branch which will involve the collection and processing of data from the DMA's and Trunk Main's as well as the billing data from the Customer Information System at the MWA's headquarters which is transferred at 10 Mbps.

"System Software" Design Concept and Methodology

The design of the "System Software" originated from the corporate direction and objectives of MWA which in turn defined the objectives of the water loss management program as outlined in the following sections.

Key Objectives of the Water Loss Management Program

- To establish the MWA water loss program for the next 10 years
- To develop a framework of best practices to minimize cost of MWA's overall water loss management program;
- To capacitate key MWA personnel in the field of water loss management

Overall Policy Based on Applied Total Demand Management and Non Revenue Water (NRW)

- ❖ Adapt Total Demand Management (Wide Bay Water, 2002) concept to water loss management to improve MWA's profit
- ❖ Improve water loss forecasts by applying NRW reduction practices and standard IWA water balance.
- ❖ Develop knowledge and skilled personnel in MWA for long term benefits to the Utility

Design Methodology

For the directive and policy provided by the top executives of MWA, the design of the "System Software" was based on a standard approach called RESPECT^{TM} (Requirement System and Project Engineering Collaborative Technique) as shown in Figure 3. This approach is very sophisticated and powerful and is capable of combining processes and requirements from many different aspects of the MWA's water business. The design and technology selected in this project is also highly adaptable to cope with the future changes particularly in MWA water loss management processes.

For its desirable capability, "System Software" was designed to have:

- automated alert and reporting systems for active leakage control activities (ACL) with SCADA systems,
- real time monitoring system for both trunk main and distribution pipes (DMA) levels,
- real time and on-line field management support system for leak detection and leak repair services,
- knowledge repository system and enterprise portals, and
- integration to GIS, CIS, 1125 Call Center, ERP/SAP, Water Network Analysis and to existing DMA data loggers.

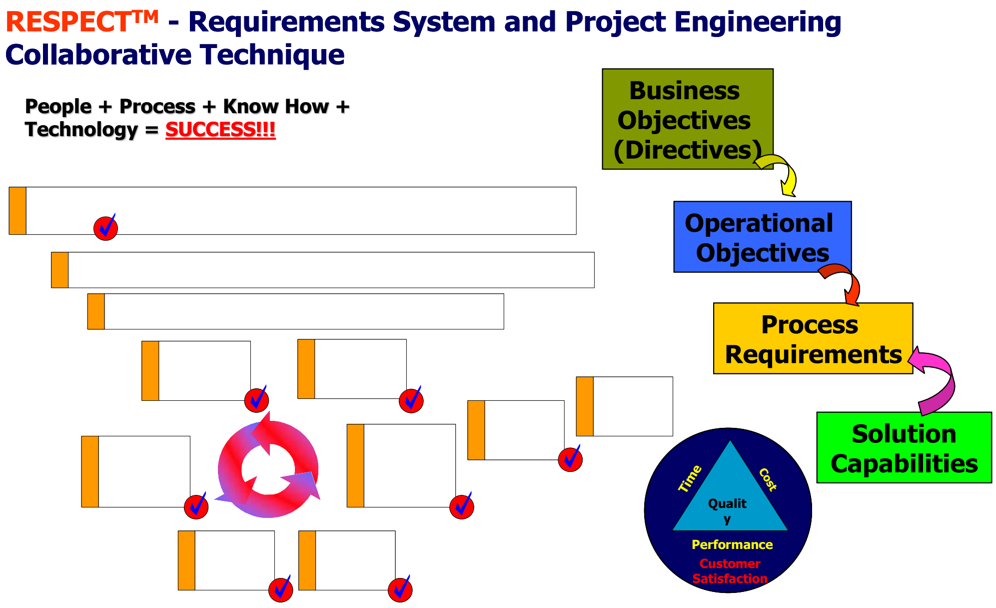

Figure 3. "System Software" Design Concept and Methodology used for development of "System Software"

"System Software" Architecture, Technology and Tools

Figure 4. illustrates the linkages between the various MWA departments directly involved in water loss management using the "System Software." The system is Web based and

assists the central distribution control center to monitor and control the production rates and distribution of water according to the demands from each branch well ahead of schedule., Moreover, with the integrated real time monitoring capabilities of the system, the branch managers and distribution control managers, can cooperate and resolve problems in specific areas quickly and effectively.

In day-to-day operation, the measuring flow and pressure data from DMA's and PMA's can be analyzed automatically and daily reports can be generated with alarm reports sent immediately to the appropriate persons via SMS and e-mails. When, a leakage alarm is generated, the software also generates the job cards for the leak surveys and repairs. Before and after each repair, the "System Software", with its links to the MWA Call Centre, also generates notices to the appropriate MWA's customers of possible interruptions to the supply.

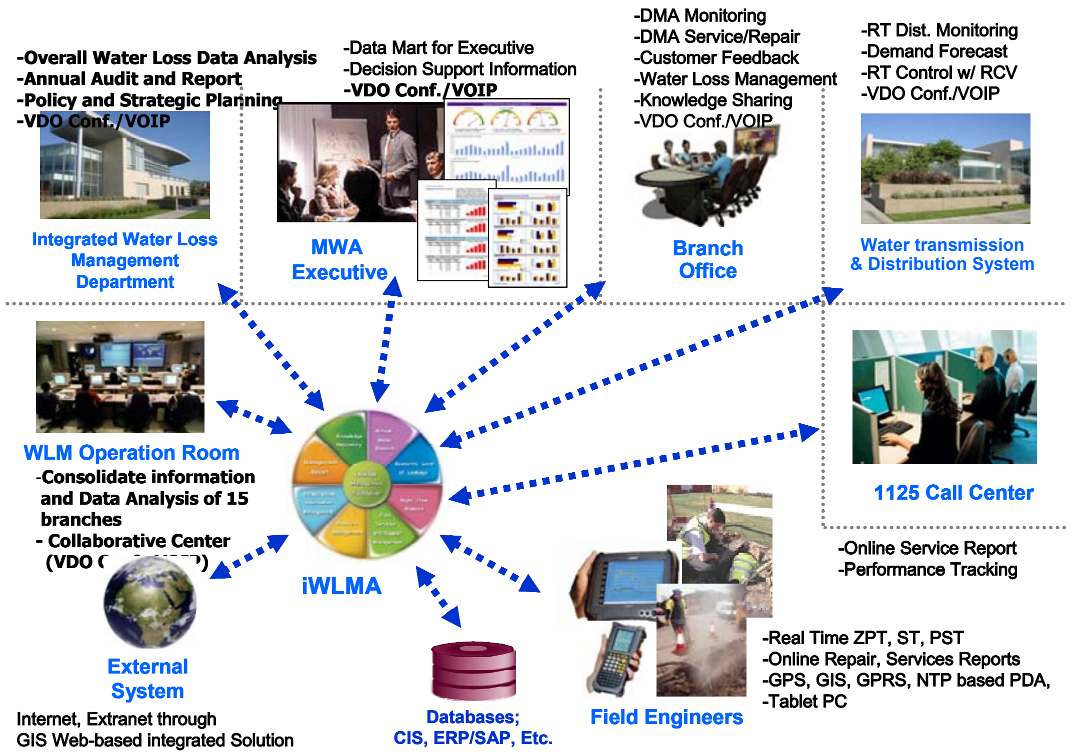

Figure 4. Overview of "System Software" Interoperation among MWA Departments Directly Involved in Water Loss Management

Conceptual Design of "System Software"

As mentioned previously, in the early phases of development, the, "System Software" was rationalized using the framework of requirement chains to capture the corporate requirements together with the international best practices, The resulting frameworks are then combined with knowledge based solution framework called "Enterprise Knowledge Portal or EKP" to form the basic framework of the "System Software". Figure 5. illustrates the "System Software" conceptual design combining the knowledge of TDM, BABE, and various technological disciplines such as instrumentation, SCADA, GIS, mobile computing and database system.

"System Software" Architecture

For the conceptual design, the software architect proposed the opened architecture based on web services of "System Software" as shown in Figure 6. The architecture elaborates on services provided to all applications and modules involved with "System Software". The major services are as follows:

Foundation Services – Serves as the core services for the information system. Its capabilities include management, control and customization of algorithms, parameters, variables, units, language, alerts and schedules in multiple dimensions both in time and space.

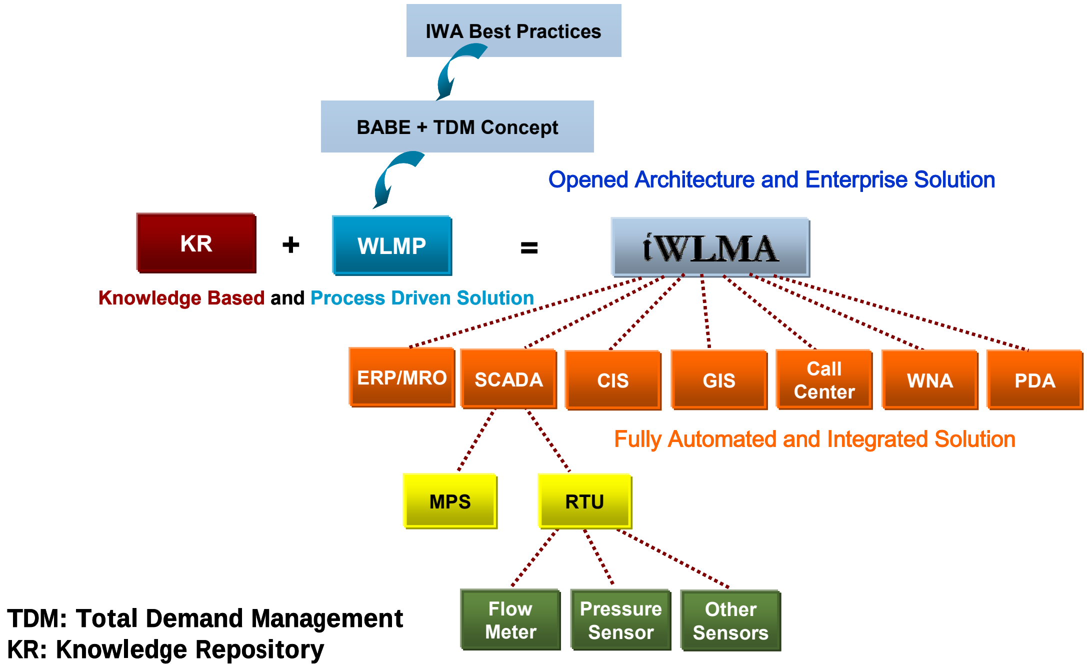

Figure 5. "System Software" Conceptual Design

External Interface Manager – Provides interface services to other systems for data transfer. The services were designed so that they provide standard protocols for data transfers such as OLE - Object Linking and Embedding, OPC - Object Linking and Embedding (OLE) for Process Control, Web Services, FTP – File Transfer Protocol/SFTP – Secure File Transfer Protocol and SQL - Structured Query Language.

Security Services – Serves as secure environment for users to access to the system and networking. The basic services involve i) Authentication; to verify the person to use the system (the system work with LDAP – Lightweight Directory Access Protocol, and SSO – Single Sign On), ii) Authorization; to give the right to the authenticated person to use the system (the system utilizes access control levels to control multiple user access to several services and modules at a given time), and iii) Accounting; to log events of system usages, user access, and data manipulation by all or particular users.

User Interface Services – Provides management and control functions for displays such as graphs, web-based GIS, multi-plotting functions, document and files conversions including .pdf format.

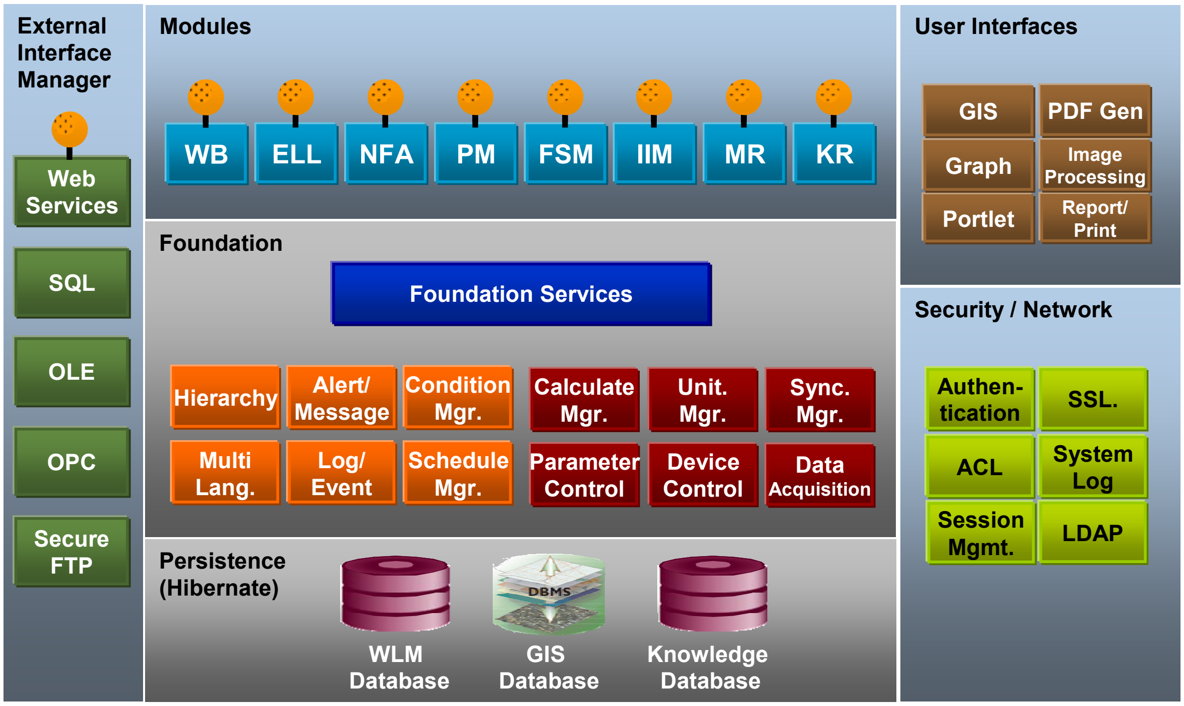

Figure 6. "System Software" Enterprise Service Architecture

“System Software” Development Technology and Tools

In development of an enterprise system, the longevity of the system is one of the most important aspects to ensure such a large investment protection for the long run. Today, a good enterprise system should last 10 – 20 years with minimal cost of ownership and maintenance. Using components based design oriented approach, the development team considered the opened architecture and design technology with independent framework called model driven architecture (MDA) from Object Management Group (OMG) (Dsouza, 2001) and (Soley, R. and the OMG staff 2000).

Furthermore, the team employed Java Enterprise Edition (J2EE) on the Windows platform and capability maturity model (CMM) framework to ensure proper standardization and quality control throughout the development lifecycle to build sophisticated applications that can easily be upgraded and modified as required in future without requiring any major re-design. The development team involved over 50 personnel at any one time and the selection of opened standard interfaces including OPC, XML, web-based GIS, and so on ensured that it was possible to incorporate the various subsystem and also the legacy in a clear and systematic manner with proven and low ownership costs.

For the algorithms, the development team has an honor to adapt formula, algorithms, and analysis schemes from various valuable sources including IWA conference papers and reports, as well as international public software developed based on Burst and Background Estimate (BABE) in recent years. Also, several key algorithms were introduced and clarified with tremendous help from the co-author, Dr. Ronnie Mckenzie, especially his guidance on water balance, pressure management, and economic level of leakage (ELL) throughout the development of this “System Software” integrated enterprise application.

Modules and Key Features of “System Software”

As component based technology, “System Software” was designed to allow several plug-in modules to its foundation services previously mentioned. For the water loss management purposes, “System Software” currently provides eight modules that seamlessly interact with each others at many levels, e.g., from object to services. The eight modules are Annual Water Balance (AWB), Economic Level of Leakage (ELL), Night Flow Analysis (NFA), Field Services and Support Management (FSM), Pressure Management (PM), Infrastructure Information Management (IIM), Management Report (MR), and Knowledge Repository (KR).

The Key Features of “System Software”

General features of “System Software” include but not limited to customization of formula, alert agents from Active Leakage Control (ALC) Algorithms, seamless integration with GIS vector operation and other characteristics such as upload/download as-built, zooming, piping details, color coding for pipe aging, type of pipes, etc. Monitoring features include leak, pressure, field activity, system infrastructure, performance indicators, target setting and so on.

Field Services and Support Management (FSM) module provides information services for team management, contract management, field operation for mobile device with GPS and GPRS and job management. The major functions of FSM include leak detection, leak repair, equipment installation and maintenance, valve auditing, zero pressure test, step test, pressure step test, and night consumption survey.

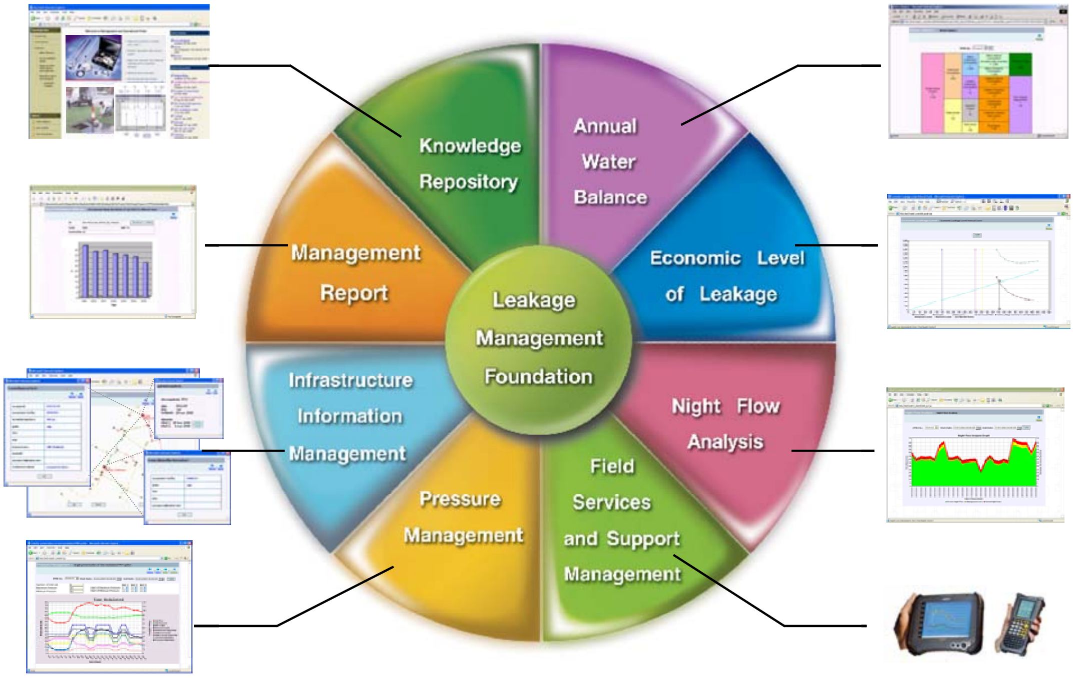

Figure 7. “System Software” Major Modules

BABE analysis package involves several modules and features such as Annual Water Balance (AWB) for water auditing and some performance measurement, Pressure Management (PM) for pressure analysis/simulation and PRV control, Night Flow Analysis (NFA) for unreported burst monitoring and minimum night flow and ESPB analysis, and Economic Level of Leakage (ELL) for intervention frequency.

Management Report (MR) provides information services for management staff with management information and performance indicators (PI) for decision making in forms of graphs, charts, summary reports, etc., whereas Knowledge Repository (KR) offers all levels of users with appropriate knowledge and contents that are suitable for individuals needs for their day-to-day operations as well as for subject matter searching through automatic achieving systems and networks.

Conclusions

This paper presents background of MWA water loss management program and their development of enterprise information system called “integrated Water Leakage Management Applications or “System Software”.” The paper also elaborates on design concept and methodology to develop the opened architecture for the enterprise software. Finally, a brief features and modules of the software are mentioned.

With this enterprise application is being commissioning in mid of 2007, MWA expect to gain benefits in their operational improvement and water loss reduction from both real time collaborative information and analytical ability from the software together with the automated features of event alerts in case of excessive leakage, low pressure, sudden system shut down, and so forth. The next phase of the project is to deploy “System Software” to all departments and improve software performance in reporting areas. The next phase of the project is to integrate the System Software to water quality control system and production while installing more pressure reducing valve (PRV) and remote control valve (RCV) to complete the master plan.

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⇪ NESDB – Office of the National Economic and Social Development Board

RELIABILITY AND AVAILABILITY ANALYSIS FOR WATER DISTRIBUTION NETWORKS

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Keywords: reliability; networks; cut-set method

Introduction

The most important consideration in the planning and operation of a water distribution system is to satisfy consumer demands. Thus, it is imperative to provide all users with good quality water in adequate amounts at reasonable pressure at all times to ensure a reliable water distribution system.

In general, reliability is defined as the probability that a system performs its mission within specified limits for a given period of time in a specified environment. Reliability of water distribution system is defined by Kaufmann et al. (1977) as the probability that the system will perform its specified tasks under specified conditions and during a specified time. Goulter (1995) and Cullinane et al. (1992) defined reliability of water distribution system as the ability of the system to meet the demands that are placed on it. The demands are specified in terms of the flow to be supplied and the range of pressure at which these flow rates must be provided

The range of combinations of ways in which a failure can occur in a water distribution system constitutes one, and perhaps the major, source of many theoretical and practical difficulties which have been encountered in establishing suitable (comprehensive and computationally tractable) measures of reliability which can be used in the practical design and operation of water distribution systems. If we define the system reliability as "the probability that the flow can reach all the demand-points in the network", among the reliability methods identified, the minimum cut-set method, defined in this paper, appears to be efficient, and is easily programmed on a computer.

Reliability of a System

Hydraulic availability is defined as the ability of the water distribution system to provide service with an acceptable level of interruption in spite of abnormal conditions (Cullinane et al., 1992). Availability is evaluated in terms of developing the required minimum pressure. Pressures between 137.9 kN/m^2 and 551.6 kN/m^2 (Shinstine et al., 2002) are considered to be desirable pressures under normal daily demands.

Hydraulic availability is defined as the ability of the water distribution system to provide service with an acceptable level of interruption in spite of abnormal conditions (Cullinane et al., 1992). Availability is evaluated in terms of developing the required minimum pressure. Pressures between 20 psi and 80 psi (Shinstine et al., 2002) are considered to be desirable pressures under normal daily demands.

Goulter and Coals (1986) proposed the use of discrete relationship between availability and pressure as shown in Figure 1. The availability during a time period t can be expressed by the following mathematical relationship:

$$HA_j = \begin{cases} 1, & for\ P_j \geq PR \\ 0, & for\ P_j < PR \end{cases} \tag{1}$$

Where $HA_j =$ hydraulic availability of node j;

$P_j =$ pressure at node j;

$PR =$ required minimum pressure.

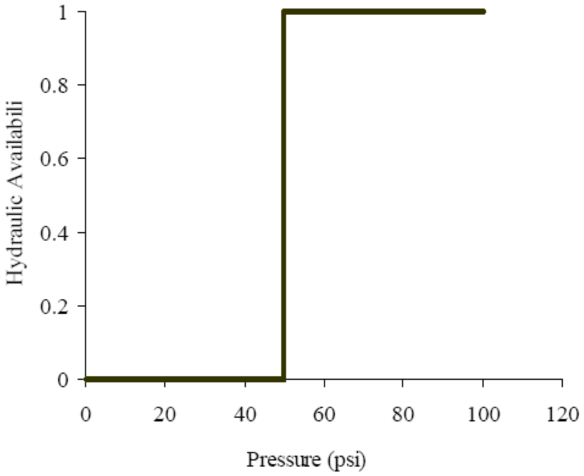

Figure 1: Hydraulic availability time step

Cullinane et al. (1992) formulated an approach that describes availability index as a continuous "fuzzy" function. Using this concept, a significant index value may be assigned to pressure values slightly less than the arbitrary assigned required minimum pressure value, PR. Accordingly, a curve similar to Figure 2 can be developed which resembles the curve of a normal distribution. Thus, the hydraulic availability function can be described mathematically as:

$$HA_j = P(PR \leq P_j) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{\frac{(H-\mu_H)}{\sigma_H}} e^{-\frac{t^5}{2}} dt = P\left(\frac{H-\mu_H}{\sigma_H}\right) \tag{2}$$

Where: $P_j =$ value of nodal pressure;

$\mu_H =$ mean nodal pressure;

$\sigma_H =$ standard deviation of pressure

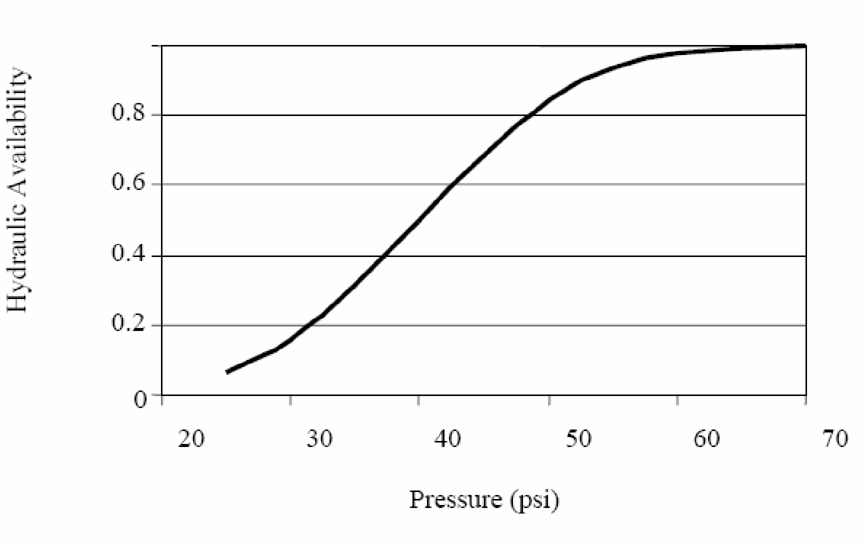

Figure 2: Continuous hydraulic availability function

Pipe Failure Probabilities

The probability of failure of pipe ***i***, $\boldsymbol{P_{i.}}$ is determined using the Poisson probability distribution:

$$P_i = 1 - e^{-\beta_i} \tag{3}$$

and

$$\beta_i = r_i L_i \tag{4}$$

Where: $\beta_j =$ expected number of failures per year for pipe ***i***;

$r_i =$ expected number of failures per year per unit length of pipe ***i***;

$\sigma_H =$ the length of pipe ***i***.

In order to apply the developed methodology in calculating the complete pipe failure probability, consider a hypothetical water distribution system as shown in Figure 3.

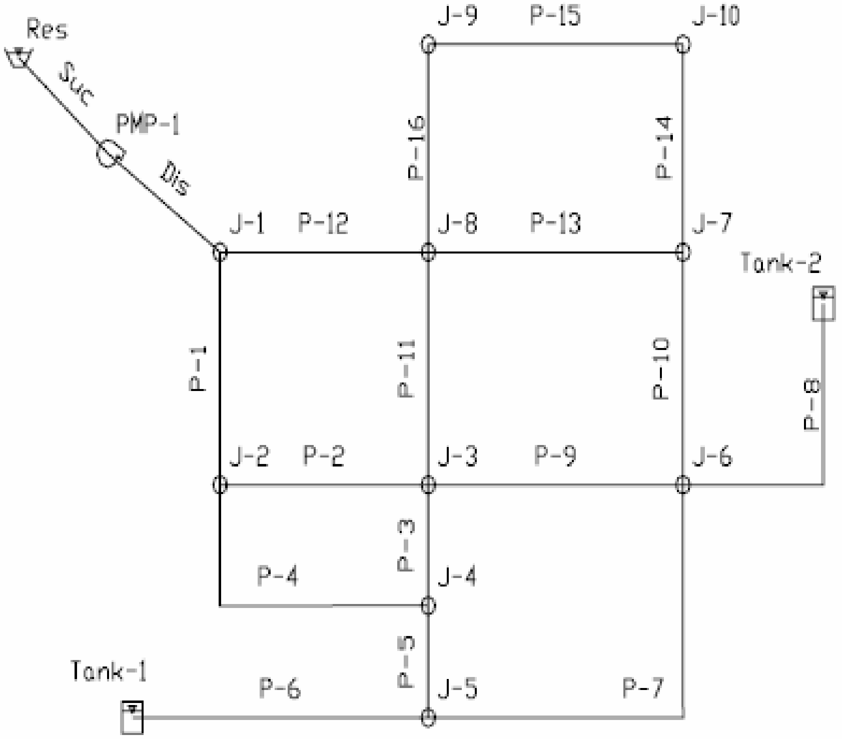

Figure 3: Example water distribution networks

Mathematically, the probability of occurrence of two independent events *A* and *B* is given by

$$P(A \cap B) = P(A) \cdot P(B) \tag{5}$$

Assume that a pipe in water distribution system is unable to satisfy the nodal demand. Then, failure is assumed to occur when the flow in the pipe exceeds the capacity of the pipe. According to Hazen-Williams equation, the flow rate in the pipe *Q_P,* in SI system, is given by (Viessman et al., 1998):

$$Q_p = 0.849 C_{HW} A R^{0.63} S^{0.54} \tag{6}$$

Where CHW = Hazen-Williams coefficient,

A = pipe cross-sectional area (m2),

R = hydraulic radius = area/wetted perimeter (m),

S = slope of hydraulic grade line.

If the pipe is considered flowing full, then the cross-sectional area $A = \frac{\pi}{4}d$ and the wetted perimeter $P = \pi d$, substituting the values of A and R in Equation (6)

$$Q_p = 0.27842 C_{HW} d^{2.63} S^{0.54} \tag{7}$$

The flow rate directed into the pipe will be equal to pipe distribution factor multiplied by the demand at the junction. Mathematically, it is given by

$$Q_D = D_P QJ_i \tag{8}$$

Where

D_p = distribution factor of the pipe,

QJ_i = water demand at junction i.

Therefore, the performance function, Z, of the pipe can be defined as

$$Z = Q_P - Q_D \tag{9}$$

Consider pipe P-1 and junction J-2, as shown in Figure 4. In order to calculate the failure probability [$P(A)$] of the pipe to fulfill the demand, the input parameters of Equations (7) and (8) are considered as random variables. The probability distributions are assumed for the input variables and their means and coefficient of variations are calculated as shown in Table 1.

Table 1: Statistics of input variables (Demand failure probability), Pipe P-1

Input variable	Mean	CV	Distribution
C_{HW}	100	0.1	Normal
D_P	2.0348	0.0692	Normal
QJ	0.16712475	0.0133	Normal

Assuming a normal distribution for Z, the probability of failure $P(A)$ is calculated using the following equation:

$$P(A) = P(Z < 0) = \int_{-\infty}^{0} P_Z(Z)dZ = P\left(z < \frac{X - \mu}{\sigma}\right) = P\left(z < \frac{0 - 0.0182}{0.0502}\right) = 0.3582 = 35.82\% \tag{10}$$

Similarly, the probabilities of other pipes can be calculated.

Assuming a normal distribution for Z, the pipe replacement probability $P(B)$ is calculated using Equation

$$Z = \frac{\ln(1+R)F_n}{C_{n+1}} - N(t_0)e^{A(t-t_0)} \tag{11}$$

Where R = discount rate,

F_n = replacement cost at time t_n,

C_{n+1} = repair cost of (n+1)th break.

t = time in years,

t_0 = base year for the analysis,

A = growth rate coefficient (1/year)

$N(t)$ = number of breaks

$$P(B) = P(Z < 0) = \int_{-\infty}^{0} P_Z(Z)dZ = P\left(z < \frac{X - \mu}{\sigma}\right) = P\left(z < \frac{0 - 2.3639}{2.533}\right) = 0.17537 = 17.53\% \quad (12)$$

Therefore, the complete failure probability, P_{com}, is given by

$$P_{com} = P(A) \cdot P(B) = 0.3582 * 0.17537 = 0.0628 \quad (13)$$

Nodal and System Reliability

The minimum cut-set approach is adopted to calculate the nodal and system reliability, R_{node} and R_s. According to Su et al. (1987), the minimum cut set can be defined as “a set of system components (e.g., pipes) which, when failed, causes failure of the system”. However, when any component of the set has not failed, it does not cause system failure (Billinton and Allan, 1983).

Assuming that a pipe break can be isolated from the rest of the system, the minimum cut sets are determined by closing a pipe or combination of pipes in the water distribution system and using a hydraulic simulation model to determine the values of pressure head at each demand node of the system. In this study, EPANET was used (Rossman, 2000). By comparing these pressure heads with the minimum pressure head requirements, the reliability model can determine whether or not this pipe or combination of pipes is a minimum cut set of the system or an individual demand node. A minimum cut set for a node is one that causes reduced hydraulic availability at that node, while a minimum cut set for the system is a cut set that reduces the hydraulic availability for any node in the system. To calculate the number of combinations for pipe closure for the cutest determination, it is observed that failure of two or three pipes is purely a “random” phenomenon. Therefore, in order to determine the pipe combinations for the cutest determination, subsets of pipe combinations should be determined by applying a random approach. For instance, if there are K numbers of pipes in the water distribution system, then out of those K pipes, T subsets should be randomly generated and each subset could have only one pipe or a combination of two or three pipes. A flow chart of the procedure is shown in Figure 4.

According to Shinstine et al. (2002), for **n** components (pipes) in the ***i***th minimum cut set of a water distribution system, the failure probability of the ***i***th minimum cut set (*MCi*) is

$$P(MC_i) = \prod_{i=1}^{n} P_i = P_1 \cdot P_2 \cdot ... \cdot P_n \quad (14)$$

Using the step function for hydraulic availability and assuming that the occurrence of the failure of the components within a minimum cut set is statistically independent, for a water distribution network with four minimum cut sets (MCi) with the system reliability, Rs, the failure probability of the system Ps is then defined (Billinton and Allan, 1983) as

$$P_s = P(MC_1) + P(MC_2) + P(MC_3) + P(MC_4) = \sum_{i=1}^{4} P(MC_i) \quad (15)$$

In general form

$$P_s = \sum_{i=1}^{M} P(MC_i) \quad (16)$$

The system reliability, R_s, is expressed as

$$R_S = 1 - P_S = 1 - \sum_{i=1}^{M} P(MC_i) \quad (17)$$

where M = number of minimum cut sets in the system.

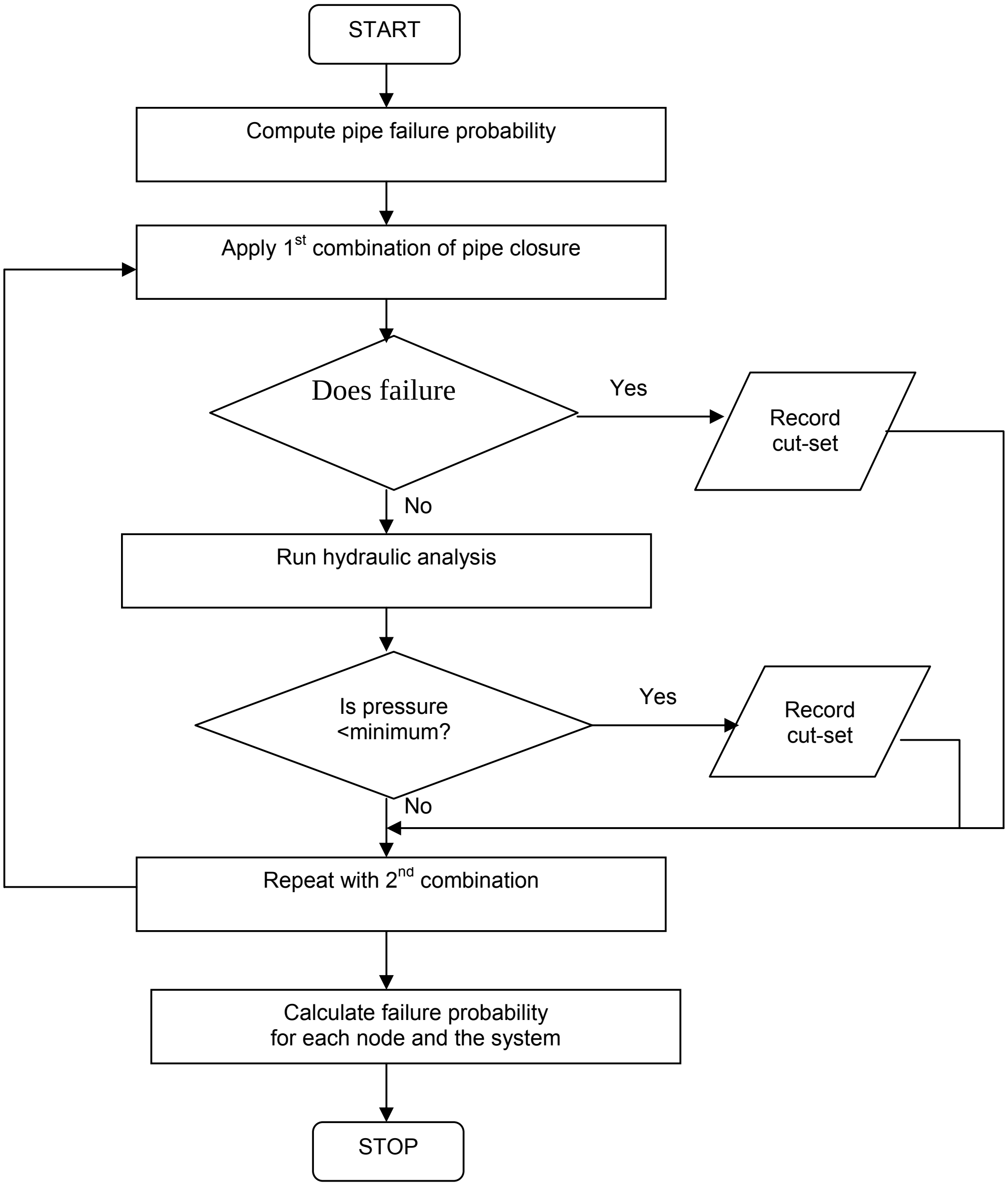

Figure 4: Minimum cut set reliability flow chart

It is possible to weigh the nodal terms as the function of the nodal demand. Nodal reliabilities can be computed with the same relationship including only failures that affect the individual node.

Using the continuous hydraulic availability concept, a true minimum cut set does not exist. The probability of a cut set occurring is consistent; however, reliability is defined as the pipe reliability and hydraulic unavailability (1–*HA*). The system reliability is then expressed as

$$R_S = 1 - P_S = 1 - \sum_{i=1}^{M} \left(1 - HA_{net}^{i}\right) P\left(MC_i\right) \tag{18}$$

where $HA_{net}^{i} =$ network hydraulic availability (Fujivara and DeSilva, 1990)

$$HA_{net} = \prod_{j=1}^{J} HA_j \tag{19}$$

where $HA_j =$ hydraulic availability of node *j*.

If *HA* equals one, the failure probability of the cut set is not included in Equation (18); thus, it is identical to Equation (17) for the step function hydraulic availability case. To compute the system reliability with continuous hydraulic availability, all cut sets are included.

In order to calculate the nodal and system reliability of water distribution system, nodal demands and Chezy's roughness coefficients for pipes are considered as random values.

The pipe failure combinations required for the cutest calculations are determined by assuming randomness in the simultaneous failure of two or three pipes. Then, steady state hydraulic analysis is performed using the hydraulic simulation software EPANET, and nodal pressures are calculated for different combinations of pipe closures. The nodal and system reliabilities are calculated using the minimum cut-set method. The flow chart of the methodology is shown in Figure 4 and the calculated nodal and system reliabilities are summarized in Table 2.

Table 2: Nodal and system reliability

Node ID	Nodal Reliability
J-1	1
J-2	0.99971
J-3	0.99923
J-4	0.99817
J-5	0.97075
J-6	0.98278
J-7	0.99972
J-8	0.99991
J-9	0.99971
J-10	0.99971
System Reliability	**0.956**

Conclusions

From the results of this study obtained by applying the developed methodology, the system reliability turned out to be 95.6% and the nodal reliability of all the nodes come out in the range from 97.0% to 100%. This means that the probability that modeled water distribution system will have a required minimum pressure of 35 psi at all the junctions is 95.6 %, and the probability that each junction will have a required minimum pressure of 35 psi varies from 97.0% to 100% depending upon the individual junction as summarized in Table 2.

Since the adopted minimum cut set approach for calculating nodal and system reliability requires the mean and standard deviations of the nodal pressures for hydraulic availability calculations, therefore mean and standard deviation of the nodal pressures significantly affect the nodal and system reliability. Therefore, higher values of mean and standard deviation of nodal pressures will result in reduced nodal and system reliability.

Also, the developed method is more feasible for large water distribution networks. In large networks, it is also possible to consider many random combinations of pipe closures. Therefore, it is recommended to use the developed methodology in large water distribution networks.

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Cost efficient leakage management in water supply systems

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Keywords: leak detection; failure statistic; pipe renewal

1. Introduction

Companies for public water supply must manage two basic tasks:

- fulfilment of the public mandate (customer satisfaction, availability, corporate image, and reducing and keeping the pipeline network losses low and having low risks from external influences)
- efficient business management (cost of supply, corporate success and a long-term cost and rate structure)

The extent to which the company succeeds in striking a balance between these two tasks and successfully manages them is determined by the quality of the management and the long-term value of the supply systems.

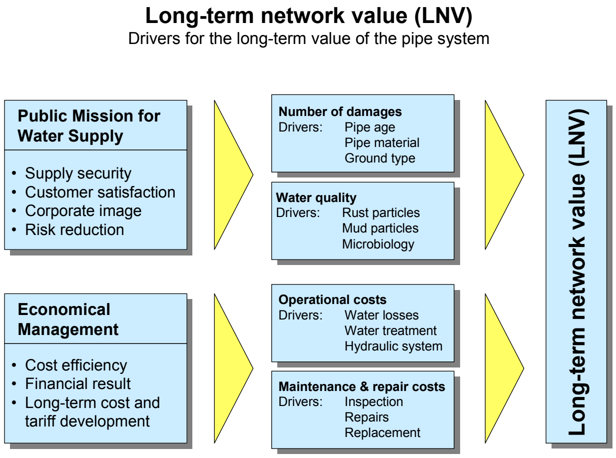


Figure 1: Factors influencing the management of the water supply system

The condition of the pipeline system is determined by the level of annual pipeline network losses and the amount of damage or alternatively repairs. For the definition of the amount of damage, it is important for leak testing to be performed continuously so that these figures refer to existing and not to (coincidentally) discovered damage.

2. Recording the Condition of the Pipeline Systems

Pipeline systems consist of pipelines (feed lines, main lines, supply lines, and connecting lines), internal parts (valves etc.), and fittings. Within the scope of modern company management, the pipeline system is managed in a GIS. For the individual objects, there are defined procedures for inspecting or alternatively defining the condition and the functional performance.

According to DIN 31501, the following terms apply as elements of maintenance:

- Inspection: in terms of scheduled monitoring of the operating condition and regular checking of the actual condition of system components and operating equipment.

- Maintenance: in terms of constant maintenance measures to maintain the desired condition, which can take place both on a regular basis as well as event-driven.
- Repair: as unforeseeable measures as a consequence of malfunctions (repairs) as well as foreseeable measures (improvement, renewal) to restore the desired condition

2.1. Goal-Orientated Maintenance

Within the scope of goal-orientated maintenance, the operating condition of drinking water pipeline networks must be monitored regularly and their internal parts must be monitored in addition due to special circumstances to make sure they can be found, that they have no leaks, and that they work.

Inspections of the system components must be documented in suitable lists and statistics containing the day, systems used, and the respective results. The results of the inspections must be managed in damage statistics. Furthermore, a variety of information regarding events, maintenance work, costs, and assessments of the inspected items must be documented.

2.2. Reason for Inspection Work

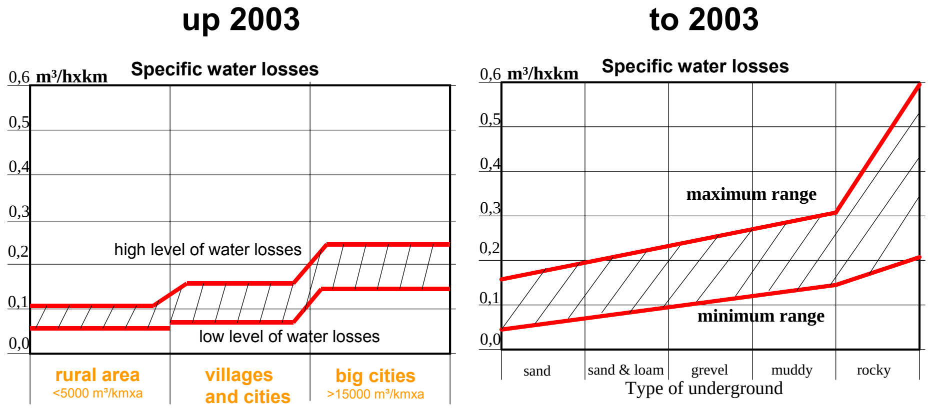
- high water losses as a result of the annual water balance
- high annual damage rates
- fluctuating changes to the feed quantity or damage rate
- changes of pressure or pressure surges
- hazardous construction measures close to the pipelines
- changes to the surface of the routes
- customer complaints

2.3. Inspection for Leaks in the Lines

The type, extent, and time intervals of line inspections is mainly determined by the level of water loss according to the creation of the annual balance, according to deviation between the registered feed quantity and comparative values, according to the frequency of damage, and according to the local conditions (subsoil, pipeline material, supply pressure, and so on).

The foundation for preparing the annual loss balance requires the maintenance and analysis of all feed and delivery quantities by means of suitable measuring equipment. So-called “internal consumption“ and other water deliveries that are not billed must be recorded exactly and documented.

DVGW has developed key figures that provide an approximate value for the level of pipeline losses. However, is has been established that key values for the level of water losses can only be related to local conditions, which are affected by many factors. Each company must form its own key values and derive conclusions from them for technical and economical measures.



up 2003

to 2003

Figure 2: Key values from DVGW (W 392) with different approaches

Both key values have their justifications, but it appears that the key value up to 2003 makes the affect of the subsoil more comprehensible for the practitioner

Range of water losses (in m³/kmxh)	big supply systems	urban supplier	rural supplyer	recommendes inspection periods
low water losses	< 0,10	< 0,07	< 0,05	inspection latest in 6 years
medium water losses	0,10 - 0,20	0,07 - 0,15	0,05 - 0,10	inspection all 3 years
high water losses	> 0,2	> 0,15	> 0,10	inspection all years

Table 1: Inspection time periods for the chart as of 2003 (DVGW W 392)

Besides statistical key values of an entire supply network, key values for individual supply zones or measurement zones should be determined. This data structure is easy to setup in a GIS in connection with the failure programme.

3. Water Losses in Drinking Water Pipeline Networks

The water losses are reduced for hygienic, supply-related, ecological, and economical reasons. Low water losses are an important indicator of good pipeline network condition and lead to availability and reduced costs for maintenance. The most accurate and comprehensive measurement possible for the water volumes fed into the pipeline network and discharged from it is an important element of determining water loss. Here, the model, installation, and size of the water meters must be selected according to the technical standard.

3.1. Calculation of Water Losses

The inflow quantity and the consumption quantity yield the difference that is sometimes referred to at the gross loss.

This difference includes
- real pipeline network losses and
- apparent losses such as:
  meter deviations
  creep losses
  water theft
  quantities provided but not billed
  quantities for the fire department
  internal or company consumption
  other unrecorded quantities

It is expedient to document all unmeasured consumption quantities, even estimated quantities, for future optimisation of the “shortages”.

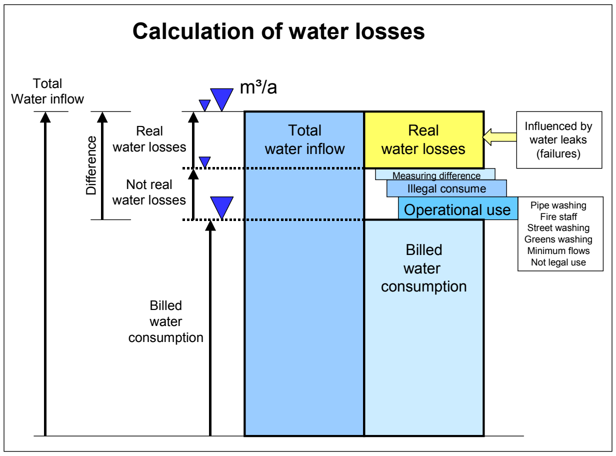


Figure 3: Quantitative calculation of water losses in a supply system

3.2. *Early Detection of Pipeline Network Losses*

Early detection of water losses involves the use of permanently installed water meters that delimit the entire supply zone or sub-zone (pressure or supply zones), as well as the feed lines. These quantity values must be documented carefully and can provide a clear indication of the development and existence of water losses based on their levels. On one hand, this could be weekly quantities, daily quantities, or night time minimum values, which must be processed based on the consumption structure. Here, it is practically impossible to derive any general key values. The consumption trend can also be read from the long-term comparison of inflow quantities.

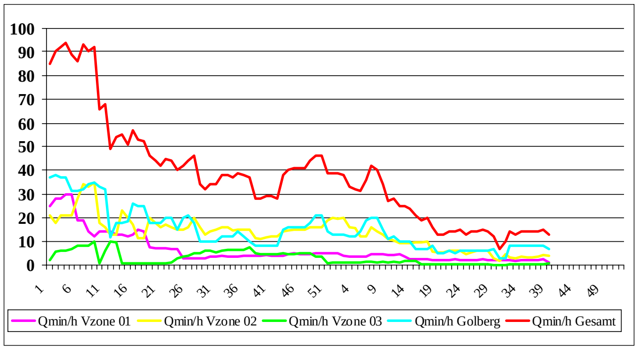


Figure 4: Example of inflow quantity monitoring with weekly minimum values over 2 years for three supply zones, a supply line, and in total

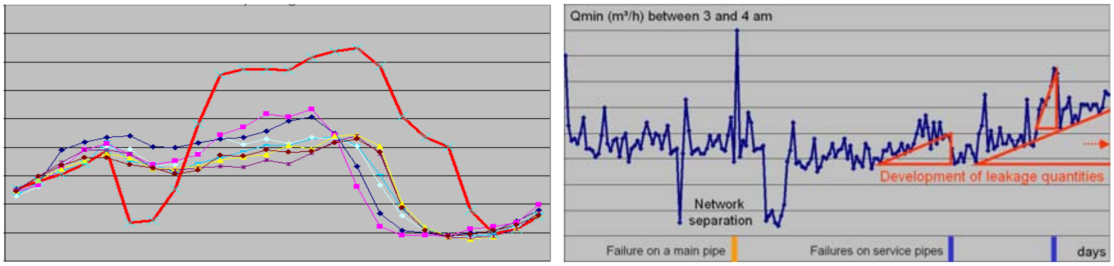


Figure 5: Examples of feed quantity monitoring with hourly values

Right picture: night time minimum consumption values from Jan. to June with repaired failures and the development of leakages quantities

Leftt picture: daily load graph with hourly values over 1 week, before and after the repair of a leakage point

3.3. Factors Influencing the Level of Water Losses

The level of water losses is influenced by many factors, which in part can't be influenced. Here, we are mainly referring to the installed pipeline system and its installation quality, which was selected and installed many years ago according to the standard at that time (pipeline materials, installed parts, connection systems, installation technology, etc.).

Therefore, it is particularly important to find out those factors that permit an economically and technically feasible procedure to effectively reduce the pipeline network losses. Extensive knowledge of the supply system on the whole is necessary for this decision, as well as specific knowledge of the pipeline system and all internal parts and their condition. The Geographic Information System (GIS) as graphical and alphanumerical pipeline documentation, the results of a GIS-conforming damages file, and the results of a GIS-conforming pipeline network analysis are instrumental for this.

Naturally the results of the damages analysis must be input from a systematic and regular pipeline network inspection so that influences on the pipeline components and weaknesses are not shown based on dominant events and situations.

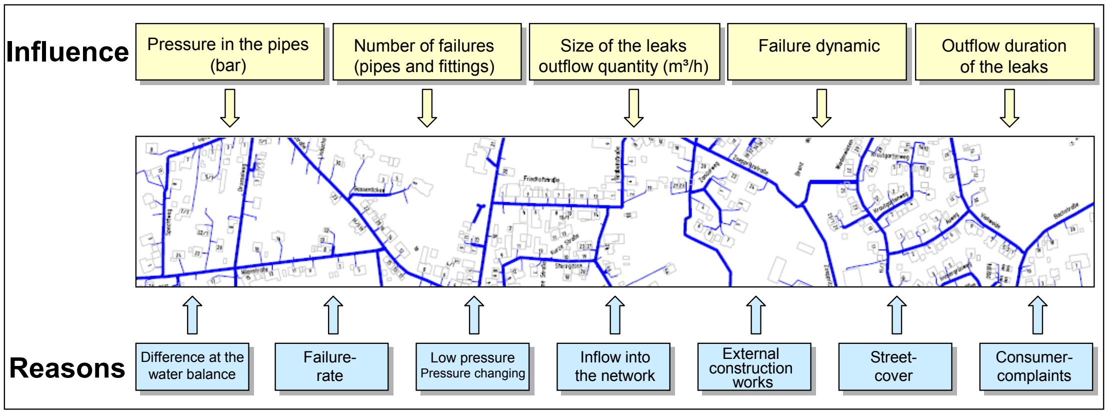
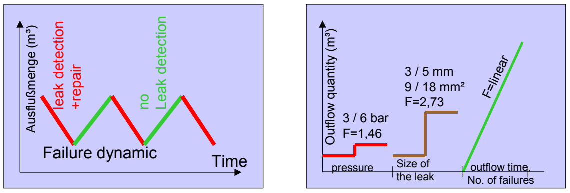


Figure 6: Influences affecting the level of water losses

Reason for the inspection and sources of the data

A differentiation of the influence factors on the level of pipeline network losses is required so that the local problems can be dealt with selectively and the desired success of lowering the pipeline network losses can be achieved. The individual influencing factors must be identified and evaluated from the existing, long-term analysis of the operating data.

Influence factors for the level of water losses



Influence factors	Influence		Measure
Size of the leaks	not	influencable	
Failure dynamic	not	influencable	
Number of failures	not	influencable	
Water pressure	partly	influencable	Pressure reducer
Running time of leaks	short	influencable	Selective leak detection
Pipe rehabilitationlong	long term	influencable	Rehabilitation works and pipe replacement

Figure 7: Selective influences affecting the level of pipeline network losses

Besides selective influences affecting the level of pipeline network losses, the causes of the damages must be dealt with, which are also influenced by local conditions. Here as well, one must take into consideration that we are dealing with existing situations, which we can no longer influence afterwards for 30, 50 and more years.

Therefore, identify and act!

3.4. Procedure to Record and Reduce Water Losses

Looking for leaks (method of determining and localising leakage points) is broken down into 2 procedural steps:

- Prelocation (procedure of narrowing down likely leakage points to the smallest possible area or network section with inflow measurement or acoustic system)
- Localisation (acoustic procedure to localise the leakage points down to the point as a basis for excavation and repair)

The reasons for initiating a search for leaks could be:

- routing or regular inspection of the pipeline system at the recommendation of rule groups or operational guidelines
- other causes according to Section 2.2

4. Damage Statistics

Damage statistics are entered in the PC program for all repairs made to the water supply system. The repairs are entered on a pre-made damage form with clearly defined names and terms so that all criteria are available to be analysed. In its recommendation W 395, DVGW gives information about the required use of damage data.

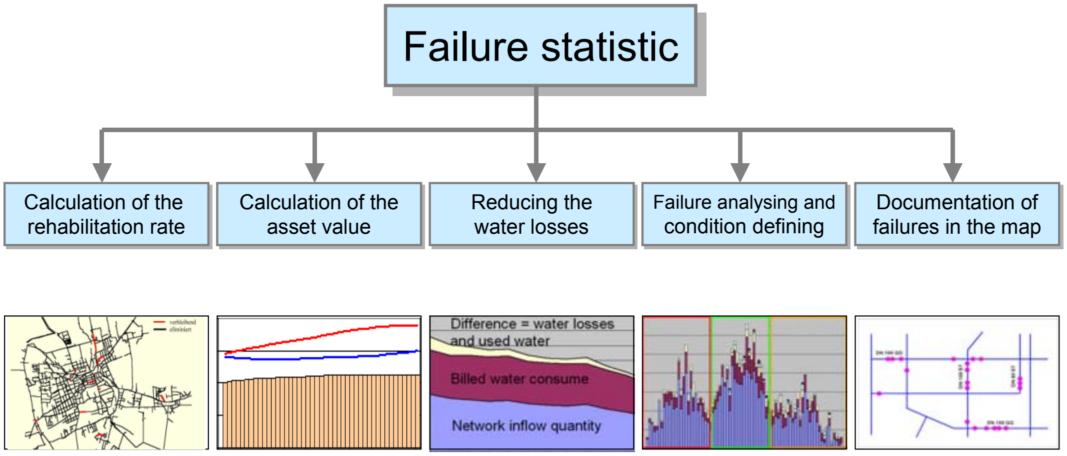


Figure 8: Influences affecting the results of damage data

It appears that to assess the condition of the supply system and to make other statements for future measures, damage data is necessary over an extended period of time so that damage trends can be recognised and evaluated.

The establishment of damage statistics is an indispensable requirement of operators of pipeline systems for the documentation and assessment of the condition of the system.

Following data are necessay for defining and analysing the failures:

Place of the failure	Defect on	Type of defect	Measure
Date of repair	Location	Information	Condition of the pipe

The content of the damages file covers all the built-in components of the supply system.

The analyses and evaluations require experience and knowledge of the assessment of weak points because besides generating the statistics, these results are used to assess future investments and strategies to reduce water losses. Data from more than 10 years is necessary for a careful assessment of the pipeline condition. Identical to the damage data, the pipeline inventory data should be managed synchronously to determine annual key values for changes to the damage dynamics. A modern GIS maintains an archive for the system inventory and the damage data. That way, the damage dynamics can be assigned to the respective current pipeline inventory of the past.

4.1. Analysis of the Damage Data

It is important to know where the weak points in the network are located:

- in what system components
- in what streets or zones
- type of damage and cause of damage
- reason for repair (leak localisation or self evident)
- when did the damage occur or alternatively when was it repaired
- additional information about the pipeline, bedding, and measures

With this information it is possible to conduct the necessary analysis to assess the condition of the system.

The results can be statistically analysed for the entire network or individual supply zones, but also selectively for individual streets or line sections. The analysis of the damages file is varied and is performed in accordance with the problems so that technical/economical decisions can be made.

Note: The failures in a supply system are not uniformly distributed in their position!

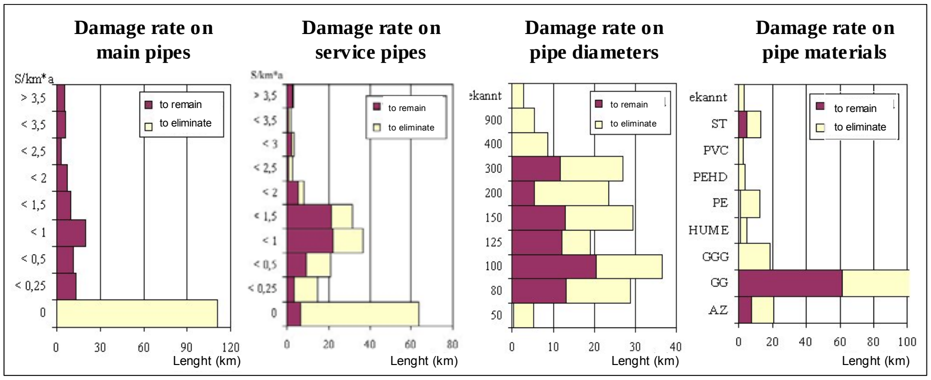


Figure 9: Damage assignment according type of line, dimensions, and material

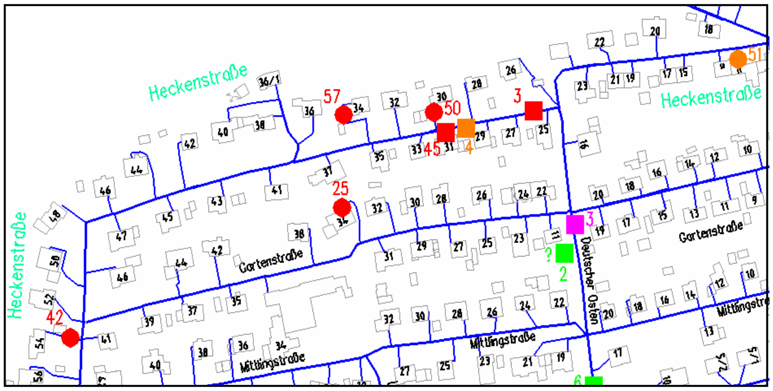


Figure 10: Local assignment of the different damage in a GIS

4.2. Key Values for Damage Rates in Supply Networks

For orientation purposes, DVGW reports guide values for damage rates. They are reported in worksheets and in the annual statistics as operating key values. The data in the following table are average values within one year.

Each supply company should maintain equivalent statistics and use them to establish a trend of a time period of at least 5 years to assess the condition of the pipeline system.

Every company must establish its own key values taking into consideration the local conditions and develop a strategy for operation management based on them.

Ranges for failure rates in the supply network	**Failure rates in the pipe network**	**Failure rates in the pipe network**	**Failure rates on fittings**	
	Mains- and supply-pipes	Service-pipes	Valves butterfly valves	Hydrants
	Failures/km*a	Failures/1000HS*a	Failures/1000pc*a	Failures/1000pc*a
Low failure rate	< 0,1	< 5	< 2	< 5
Medium failure rate	> 0,1 bis < 0,5	> 5 bis < 10	> 2 bis < 5	> 5 bis < 10
High failure rate	> 0,5	> 10	> 5	> 10

Table 2: Key values for the assessment of the pipeline network damage rate (DVGW)

4.3. Connection Between Loss Trends and Damage Trends

The loss trends and the damage trends aren't necessarily connected. The results of many analyses of pipeline networks have shown that a reduction of the amount of damage is essentially dependent on renewal of the pipeline components.

On the other hand, a reduction of the pipeline network losses is essentially dependent on a reduction of the elapsed time for the individual damage.

Therefore, identify losses as quickly as possible and then localise and repair them immediately.

This refers to the substance and the availability because an old pipeline network with very dynamic damage can't be kept functional in the long term through repairs. Therefore, systematic renewals are absolutely necessary.

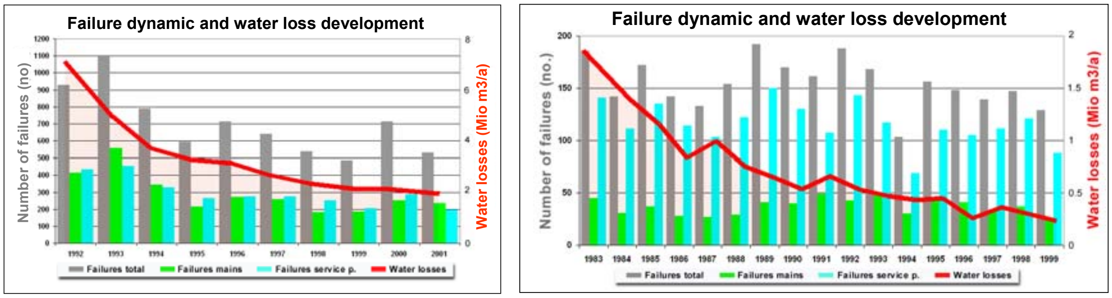
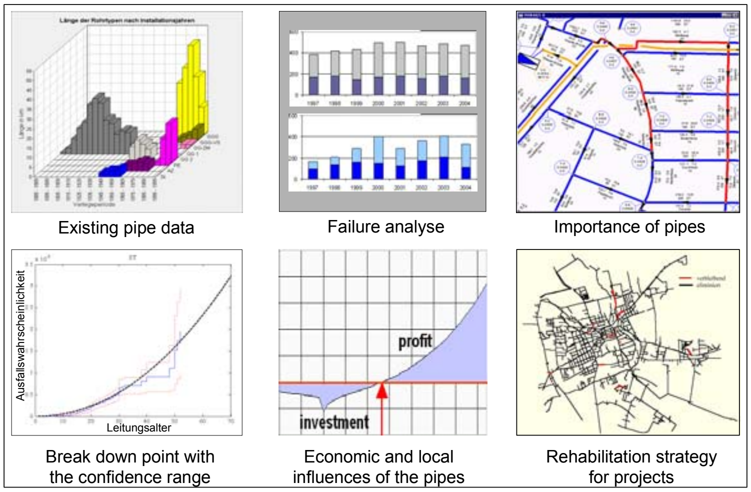


Figure 11: Two practical examples of the connection between lowering losses and damage trends

5. Maintaining the Substance of Pipeline Systems

The pipeline systems and facilities are constantly ageing and therefore are also more susceptible to damage and water losses. The availability become less certain and the costs for inspections and maintenance increase. As with all system components in our lives, which are in permanent use and subject to a great variety of loads, there is always wear and tear. Here, we are talking about the service life of the lines and facilities. This is the service life after which the pipelines and system components have to be renewed in order to ensure the reliability and efficiency of the supply.



Existing pipe data

Failure analyse

Importance of pipes

Break down point with the confidence range

Economic and local influences of the pipes

Rehabilitation strategy for projects

Figure 12: Steps for a renewal procedure for pipeline systems:

existing pipeline data – damage analysis – service life of the existing pipelines – necessary rate of renewal corresponding to forecast analysis – economic efficiency of renewal – renewal projects shown graphically in GIS

The substance of the pipe system is influenced by the level of water losses and the numbers of the failures on the pipelines. Both factors are to reduce in practise.

- Reduce of water losses by monitoring, leak detection and repairs
- Reduce of the numbers of failures by replacement of pipelines on basis the failure statistic and rehabilitation strategy

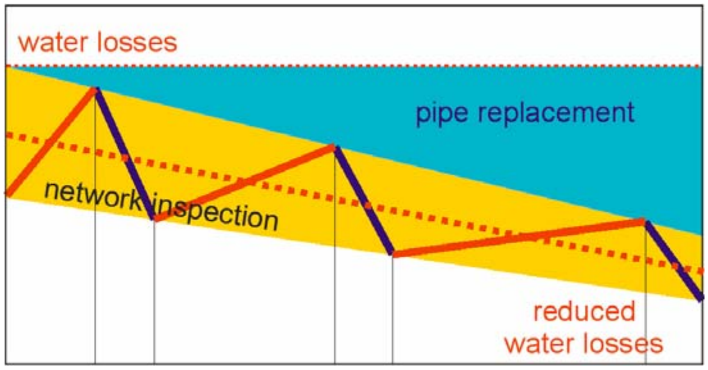


Figure 13: Influence factors for the substance of the pipelines and reducing water losses

6. In Conclusion

Inventory control and recording the condition of supply systems is indispensable. The guidelines for the scope of the inspections and the inspection cycles are recorded in the relevant guidelines of the trade associations.

Each company must establish its own key values so that local conditions are taken into consideration. Based on these key values, each operator of supply systems must develop its own strategy for maintaining the supply and above all for maintaining the substance of the asset value so that operation is guaranteed in the long term both technically and economically. With the described procedure, ways have been shown with which optimum operations management can be achieved for an efficient reduction of the pipeline losses in harmony with the philosophy of the company, the local influences, the current condition of the systems, and the economic possibilities.

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“Integrated Decision Support for Water Loss Management in Water Distribution Networks in Developing Countries by the example of Peru”

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Keywords: Water Loss Management; Decision Support; Multi Attribute Value Theory.

Abstract

This work contributes a decision Support for an Integrated evaluation of Water Loss Management projects for Developing Countries. The use of decision support helps to select Water Loss Management projects and adequate measures, improves transparency of the decision process among the players and meets better acceptability overall.

Potable water loss from distribution systems causes one of the major unsolved problems for a safe access to water and sustainable development and sanitation for people, particularly in developing countries. The design of water loss management often involves the task of identifying the optimal alternative out of a number of different concepts and measures meeting high benefits by consideration of Multi-Criteria-Analysis. Criteria of the analysis consider the techniques, socio-cultural and ecological aspects after adapting and supplementing decision models based on additive weighting methods and the outranking method PROMETHEE. The decision-making process has been tested by a case study in Peru and Germany whereas different general conditions and levels of state of the art have been considered.

Introduction

Water resources in developing countries are often not just scarce but in many cases also badly managed. On the one hand, this has serious consequences for the health of the population and, on the other hand, can considerably aggravate the limited availability further still. Often there are significant shortfalls in the organisational structure, in the technical supply and transport facilities and in the management of the supply companies.

All around the world, customers expect water supply companies to supply high-quality drinking water that is available any place any time at the correct pressure and in sufficient volumes. Supply should cover the customer’s individual needs and the price should be affordable for the customer. For ecological and economic reasons, supplying drinking water to the tap should preferably be effected without significant losses. In many regions of the world we are far from meeting these expectations. The commercial-political and eco-political need for low-loss supply of drinking water is, however, by no means confined to developing countries. For economic, ecological and demographic reasons (population growth), this requirement applies to all regions and nations worldwide without restriction.

The aim of the research project is to use the often Scarce Water Resources in Developing Countries more rationally and more efficiently. If no structural and technical improvements regarding water use in Peru are implemented within the next few years, the situation may become very serious. Peru is the only country in South America where a future water shortage is anticipated; based on the national average, only in Peru is there a strained hydrological relationship.

This is anticipated to be 980m³ per capita by 2025, meaning that Peru will suffer from a water shortage. Hence this project should include the provision of a well-defined evaluation and decision support model to facilitate the assessment of various alternative projects according to multi-criteria aspects whilst taking into account the various interests and objectives of the parties involved and therefore making an often subjective decision process transparent and largely verifiable. This is demonstrated using the example of water loss management in water distribution systems and is subjected to a practical test by comparing two case studies in Germany and Peru.

Materials and Methods

In order to achieve the aims of this project, the evaluation procedures and methods of multi-criteria decision support were applied to two case studies in Peru and Germany. the Model structure is shown in Figure 1.

In order to gain a better understanding of decision making in water losses minimisation projects, the different economic, social, technical and ecological constraints of supplying water in Germany and Peru were briefly compared. The most important differences in strategic and actual practices could be described as follows:

- Germany has a Preventative Strategy to combat water losses, which has essentially led to low levels of water loss. In contrast, Peru relies on a Failure Strategy to combat water losses.
- Apparent Water Losses in Germany have a low, almost negligible impact. Apparent water losses in Peru have a medium to big impact.
- There are no legally specified goals with respect to the high water losses in water distribution systems in Peru and in addition these losses are not allowed for in the tariffs. In Germany there are commercial, technical and economic incentives for minimising water losses.

The water balance provides the basis for analysing water losses in water distribution systems the various components of the water balance are distributed and defined differently in different countries. For this reason, the specifications of worksheet W392 of the DVGW (German Technical and Scientific Association for Gas and Water) were adopted for this work.This worksheet contains specifications for compiling the water balance. This predetermined model generally corresponds with the “Best Practice Terminology” of the IWA (International Water Association).

Economic sustainability of a water supply company was selected as the primary objective for the analysis of water losses. The following criteria were used to implement this objective: Economic Effectiveness, Economic Efficiency; Technical Efficiency; Ecological Efficiency Socio-Cultural Efficiency.

Various evaluation methods were checked for suitability in terms of implementing sustainability in water management systems. In this respect, a vector diagram was used to analyse the effects of water losses minimisation measures. This analysis involved the development of criteria that would facilitate prioritisation of these measures within a water loss minimisation project. The PROMETHEE procedure was considered the best possible option for performing the analysis.

The Decision Matrix and the preference function required for application of the PROMETHEE procedure in water loss minimisation projects were defined on the basis of the selected indicators and the Vectorial Analysis of water loss minimisation measures.

A method for the realistic and rational assignment of weighting factors for the selected indicators was suggested. A procedure for further evaluation of the round-table measures was suggested.

A method for analysing and evaluating the effects of water loss minimisation measures was established . In this regard, the following were calculated:

- The effects of each measure on the selected indicators.
- The costs and benefits of each measure and, therefore, their effect on the annual running costs of supplying water.

The following components of the decision support model were automated in an Excel document:

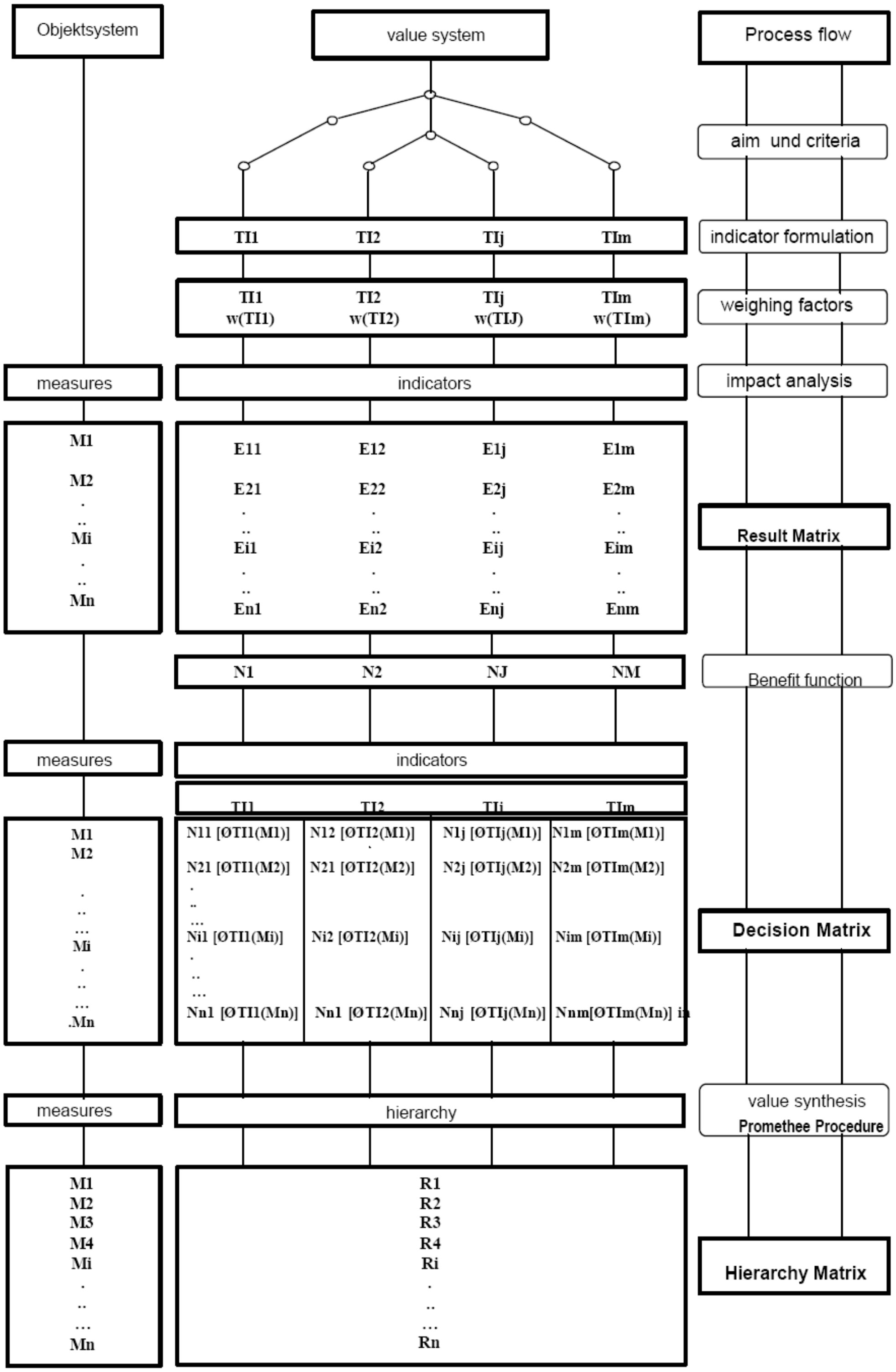
- The PROMETHEE procedure with the result matrix and the preference function for water loss minimisation projects.
- Assignment of weighting factors for the various indicators.
- Analysis, evaluation and ranking of the water loss minimisation measures,
- Round-table evaluation and ranking of the minimisation measures by means of a suitable voting procedure.

A sensitivity analysis was carried out in the case of Epsel S.A. (water supply company in Chiclayo-Peru) to check the stability of the decision when constraints and weighting factors are changed within certain limits.

The methodology established to analyse and evaluate the effects of water loss minimisation measures provides a foundation for the future analysis of other measures. This methodology was graphically illustrated in the examples specified.

The process is fully automated and delivers digital numeric results, such as charts, that clearly illustrate the decision process and, in particular, the results. The process is controlled by the parties involved, who can weight their various interests as desired (negotiation process) and makes the effect on the achievement of objectives, or extent thereof, completely transparent.

Compiling and programming the Excel tool makes it easier for the user to use the decision model and analyse the results. Although not all possible Water Loss Minimisation Measures and modalities of water loss minimisation are included therein, with the methodology presented, extending and adapting the tool does not require much effort. The same applies for taking into account other indicators and new measures.



Own Presentation

Figure 1: Model structure of Decision Support for selection of measures to Minimize Water Loss

Results and Discussion

On the whole, the use of evaluation procedures and Decision Support Methods in selecting projects to minimise water losses in water distribution systems proved to be beneficial and largely objectified compared with a more or less intuitive or subjective appraisal of the strengths and weaknesses of specific alternative projects. The value of applying such methods lies, in particular, in the fact that they break complex decision problems down into steps. This way, the decision-maker is no longer required to solve a complex, often difficult-to-understand overall problem, but can instead proceed on a step-by-step basis that is easy to follow. Furthermore, the effects of individual objective (monetary indicators) and subjective parameters (non-monetary indicators, such as "Illegal Water Connections") can be revealed within a decision support model. This makes the decision process more transparent and more efficient, whereby, even for developing countries with considerable political influence on decision-makers at water supply companies, this work can, on the whole, contribute to objectification, to augmenting good (rational) water practices and therefore to a new water culture (shift in values).

As a result of the failure strategy used up until now to combat water losses, the water supply company: EPSEL S.A- (Lambayeque -Peru) project region exhibits very high water losses. Hence it was relatively easy to identify the potential for reducing water losses using the known measures. Similarly, the amount of apparent water losses was shown to be significant. It is possible that further endeavours in the field of pressure management may result in the attainment of other economic benefits.

As a result of the preventative strategy used up until now to combat water losses, Langenselbold-Germany exhibits only low levels of water loss. In the German case, in which the level of water loss is low, other endeavours to reduce water losses should be carefully considered. Determining the effects of water loss minimisation measures is a difficult task because their effects are becoming increasingly negligible and are below the measurement error limits. Optimisation of the measures already implemented, such as, for example, optimisation of dynamic pressure management, may still represent an economic alternative for achieving a further reduction in water losses.

Reducing water losses in water distribution systems is beneficial from a technical and economic point of view only under certain circumstances. In the case analysed involving a system in a developing country, it was confirmed that minimising water losses has a positive effect on the economic efficiency of the water supply system. In the case of Peru, this potential for improving efficiency is not exploited due to the lack of regulations regarding water losses. Hence, policy would have to be amended in order to create an incentive for reducing water losses. Definition of the indicators and objectives regarding water losses should be adapted to suit each individual case.

Conclusions and Recommendations

a) An evaluation and decision support for water loss minimisation measures can be generated by applying various methods of multi-criteria evaluation and decision support.

The following steps should be taken:

- Using the indicators specified the objectives (Economic, Ecological, Technical and Socio-Cultural objectives) of the water loss minimisation measures or projects must be defined.
- The decision-maker must identify the relevance of the various objectives by means of weighting factors.

▪ A Water Balance must be compiled (for one year) for the water networks in question, which must take into account the various components of Authorised Consumption and water losses.

▪ Potential measures for reducing water losses must be identified. The various measures should be conceived by an expert who is able to gather the required information, recognise the options for water loss minimisation and suggest various measures accordingly. A water loss minimisation project must, therefore, be comprised of measures that are adapted to the respective reality.

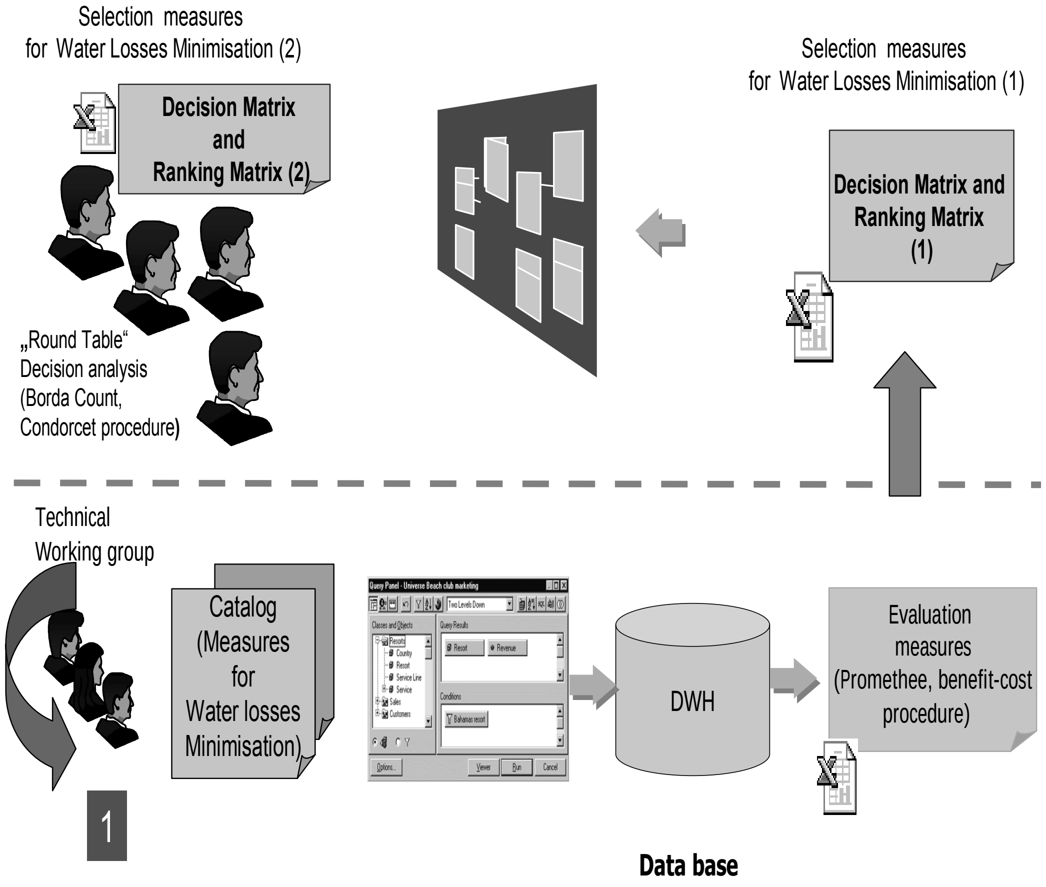
▪ The effects of the projects or measures on the corporate indicators as well as their effect on the annual costs must be assessed.

▪ It is important to check operational profitability on the basis of orientation towards the economic levels of leakage. Measures to reduce network leakages are, from a purely operational point of view, only sensible up to a certain point.

▪ The various alternative projects for minimising water losses are compiled and analysed on the basis of the evaluation methods studied (Benefit-Cost and PROMETHEE).

The PROMETHEE procedure and the preference function specified are used to establish the ranking of the measures. This ranking is forwarded to the parties involved "Round Table" (e.g. public authorities, consumers, legislative bodies, health authorities); the target group selects a new list of priorities (using a group decision procedure with specific voting rules) according to their decision interests. A very simplified flow chart of the process is shown in Figure 2. During the search for a suitable voting procedure that would facilitate as fair, democratic and transparent a decision as possible in a vote several voting methods were compared:

- Condorcet Method
- Borda Count
- Majority Vote
- Single Transferable Vote



Figur 2: Course of decision process in practice

The Condorcet method and The Borda method are the most well-known voting methods. They are conceived in such a way that one compensates for the weaknesses of the other. The Condorcet Method has the advantage that changes to one's own order of preference do not influence the voting result. However, the method has the disadvantage that different preference weightings are not taken into account and that it can quickly amount to inconsistent results, or "cyclical majorities" as they are called. So that both methods do not produce inconsistencies, even small groups need large majorities. These requirements give the individual participants in the voting procedure lots of power because they can act as veto players and cause the voting process to collapse. Since conflicting interests are to be expected in a vote, it is not likely that a clear majority will be forthcoming in the first ballot. Voting in several stages is therefore recommended, since subsequent negotiations regarding the interests are inevitable. In this respect, complete orders of preference for the alternatives available in the vote should be submitted by the voters in each ballot. That way, any change in personal interests can be tracked during the voting process. Thus, hopefully, decisions based on personal interests can be recognised should a voter drastically change his/her position with respect to their interests.

- At the end of the process, a validity check is carried out in order to scrutinise the purely logic-mathematical analysis, to check for faults in the input data of the data structure or indicators and to check whether the results obtained are consistent with the expert knowledge.

b) Additionally benefits of decision support methods lie in the more or less objective decision process comparable to the intuitive and/or subjective weightings of the strengths and weaknesses of individual project alternatives for the improvement of the decision making process.

The advantages of PROMETHEE are the separate analysis of strengths and weaknesses as well as the possibility to allow generally and define different preference levels. However, those properties may lead to rank reversals and depends on the assertiveness of substantial arguments and severe discipline in leading serious discussions. For the weighting step a pragmatic scoring procedure has been developed. Further, analysis sensitivity is proposed that reveals the influence of the partly subjective weighting factors on the results. Within each case study a catalogue of several evaluation criteria has been developed with respect to fundamental and international accepted goals of water loss. Therefore a democratic choice at a „Round Table" will be examined in this work. By quantification the singular benefits the whole or maximum extent of goal achievements can be derived under current circumstances and time. So it constitutes a custom tool for various decision alternatives and points out different impacts clearly. The stakeholders at any time are able to present their contribution by scale in their organisations.

c) Efforts to reduce water losses will, however, only be successful in the long term if all socially relevant groups appreciate the Ecological, Economic and Social dimensions of the problem.

d) The reduction of losses from drinking water supply systems is a Political, Financial, Ecological and Socio-cultural problem. Within the water supply company the causes of losses from the water supply system are more often found in the organisational, administrative and operational areas than in the technical areas.

e) Intensification of international cooperation (e.g. Capacity Building, Good Governance, Appropriate Technologies) with a view to reaching a low-loss supply with high-quality drinking water should be pursued under the given general conditions (e.g. climate, politics, Water Culture, corporate culture, government culture, Water Availability, quantity, quality). It is important that all available expertise is systematically distributed to the employees of water supply companies with respect to the management, organisation, operation and maintenance of water supply systems. It would also be useful to actively continue the international standardisation of terminology and practices, methods and equipment in respect of supplying drinking water. The administrative and technical procedures for detecting, locating and repairing leakages should be further developed on an international scale.

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Monitoring System for the close meshed Water-pipe network to the City of CRAILSHEIM - Germany

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Abstract

Occurring water losses are an increasing challenge for supply companies. In order to reduce these losses efficiently detailed knowledge of the areas with major deficiencies is required. The below presented water loss management-system (WLM) enables the suppliers to observe the activity within the pipe network and to identify leakages early.

Measuring probes, scattered all over the pipe network, and a central unit to analyse the collected data build up the system. The probes act autonomously. The parameter flow, pressure and noise are recorded. The data is stored temporarily and via GSM (mobile phone network) transmitted to the central unit.

The precision of metering is very high. Therefore even very slow movement of water down to
1 cm per second can be detected. In this way, the supplier can improve the installed district metering system – or build up one. Right in the moment, when a new leakage occurs, both a first prediction of the localisation of the burst and a dim quantification is possible. Through the rough localisation the expenses for further leak detection (correlator…) can be reduced substantially. The water supplier can classify the case according to its economic urgency.

Key words: EnBW, monitoring of entire network, resolution of 1mm/sec., WLM-System, AQUALYS, Sluice-Gate, quantifying the amount of Leakage, flow, pressure, noise, leakage duration

Introduction

The Crailsheim Utilities Water Supply

Crailsheim is a central point in the northeast part of the state of Baden-Württemberg. It lies within the Schwäbisch Hall county. The catchment area contains approximately 80,000 people, while Crailsheim itself has about 17,000 residents.

With 20,000 jobs, many in the industrial sector, the city has a good economic structure, enhanced by convenient access to the German highway system (A6 and A7).

The city of Crailsheim is the sole participant in Crailsheim Utilities Ltd. (STW). This utility provides power, gas and water to the city. Other services are available, such as the district heating supply. The STW had 114 employees in 2004 and produced € 34 million in revenue.

The water supply is carried by 121 kilometres of pipeline network. In 2003 this network supplied 2.34 million cubic metres of drinking water. At least 70% of that amount is made available from neighbouring springs through the Jagst Group Water Supply Cooperative. The remainder, over 700,000 cubic metres in 2006, was drawn from the Northeast Württemberg Water Supply Cooperative.

The STW service area stretches over the expanse of 401 metres above sea level up to 466 metres above sea level. The average elevation measures approximately 420 metres above sea level. In this service area normal water pressure ranges between 6.47 bar and 1.67 bar.

The greater portion of the network built with large nominal diameter conduit. The size of conduit used fall between DN40 and DN300. The preferred material is PVC (approx. ⅔ of all piping). PE pipe is also extensively used (approx. ¼ of all piping). The rest is covered by metal pipe. The average age of the conduit network is minimal.

Water loss grew steadily in the years up to 2003. The relative loss rate of 7 to 8% is considered negligible. This, however, is in part do to consumption-intensive industrial operations. If one considered the specific water losses, one will find a loss of 130 litres per kilometre, an amount that lies in the area between average and high loss. Specific water loss rates present a far clearer picture of the condition of the conduit network than relative loss rates.

The service area structure is clearly divided. The area is organized into a total of five zones, two of which exist due to geographical reasons: two districts are structured as their own service zones. The remainder of Crailsheim is divided among the other three zones. Zones 1 (industrial area) and 3 (city centre) form the main consumption focal points.

The zones are separated from one another by dampers and pressure relief valves.

Water metres are installed on the elevated tank discharges as well as some of the zone crossing points. Remote data transfer is not available. Only the data on the elevated tanks are transmitted to the control room.

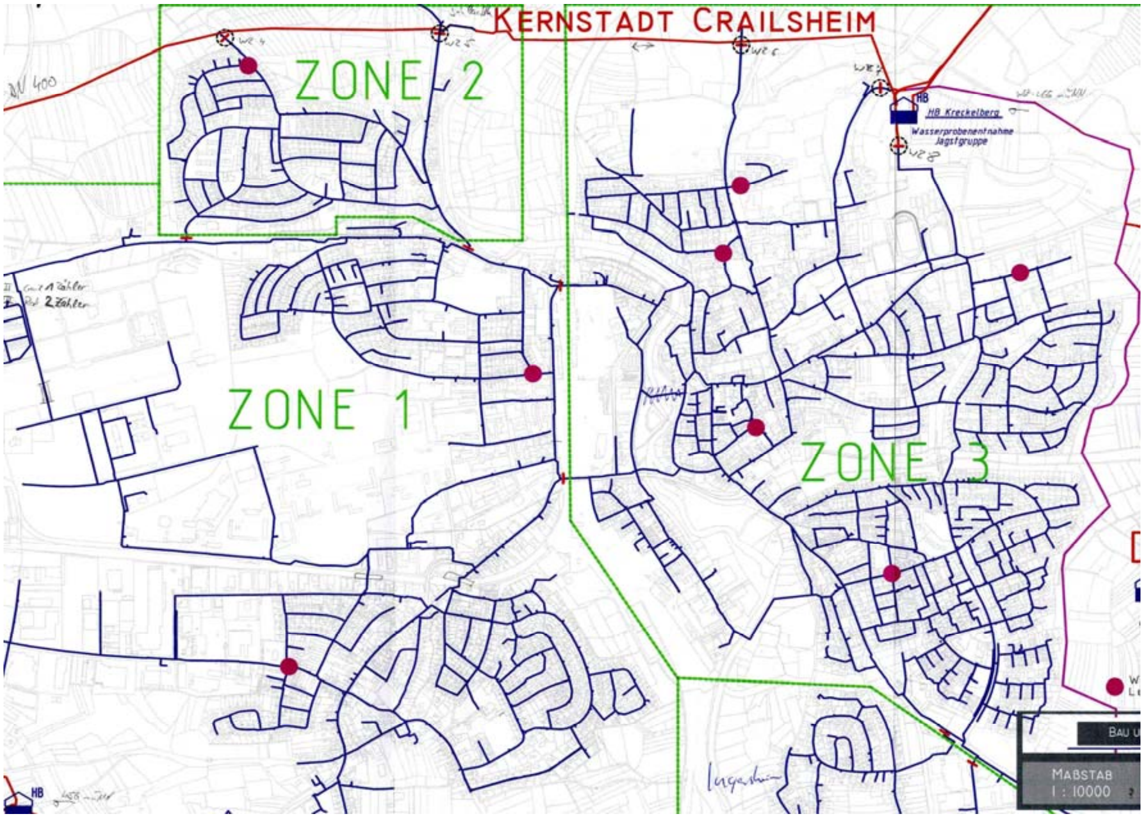

Figure 1: Conduit network overview

Motivation for Monitoring Water Loss

The focal point of the service area is divided between the two largest zones in the network. These already consist today largely of plastic conduits. If a leak is detected and is identified by zone metering, comprehensive and complicated locating work is necessary in order to find the leakage point and repair it.

The WLM sensor technology, sold in Germany by RBS wave Ltd, a subsidiary of EnBW (Energy Baden-Württemberg AG), promised to be of some remedy here. The sensor technology is based on measuring probes, that are installed directly into the conduit network and monitor flowthrough, pressure, and noise levels during non-peak usage times (overnight hours).

The sensor technology detects leakages very early. Even minimal flow velocities below 1 cm/s can be reliably recorded. The technology enables high-resolution monitoring of the entire supply network without physically disconnecting the zones, a process that is hydraulically disadvantageous. This greatly reduces the effort involved in locating leaks.

Furthermore, the sensor technology enables quantifying the amount of leakage upon identifying the leak, so that a reliable estimate of the economic urgency can be assessed.

With the pervasive installation of WLM sensors, the STW desired on one hand to be informed early about the occurrence of leakages and, on the other hand, to sustainably lower the cost of locating leaks.

Presentation of WLM Sensor Technology

The Measuring Probes and their Function

The sensors are brought into direct contact with running drinking water through a borehole in the pip. They have a stainless steel housing that can withstand pressure up to 35 bar, and they are IP68 protected. The sensor housing contains all measuring devices as well as a data logger for 38,000 lines of data.

The number of sensors used and their position in the network depends on the network structure and the desired accuracy of the resulting data. This layout should be based on a virtual pipeline network model, which enables an exact precalculation of the hydraulic factors to be expected at each desired installation point.

Each sensor uses a magnetic-inductive measuring electronic to measure the speed occurring at the installation point. The minimum measurement limit is approximately 1 cm/s. The flow speed is transmitted with a resolution of 1 mm/s.

Further, the effective flow pressure at the reading point can be precisely defined to 0.01 bar. In addition, the sensors capture the flow noise of the water using hydrophones. Measuring the noise inside the pipes reduces the dependency on piping material, since the water itself is a better conductor of sound than the plastic used in manufactured pipe conduits. In this way leakage noise, distinguished by physical transmission characteristics, can be registered and easily recognized in contrast to flow noise.

A microprocessor controls the measurement process and can produce immediate analyses of reading data on the spot. It also regulates the transmission of data over a mobile telecommunications network. In addition, either a time- or event-controlled operating mode can be selected.

The transmitted data are received, archived and processed at the main office by means of a specially developed software environment called AQUALYS. The reduction of accrued reading data to relevant daily values and the threshold value calculation based on them are the primary functions. The program also displays the data graphically and enables simple comparison of various reading times of individual sensors or the comparison of different sensors with one another. In addition to data import, AQUALYS also provides an export function, with which the data can be automatically used by other programs. It is also possible to connect via port to an existing control system (e.g. SCADA).

Through real-time monitoring and refined reading data processing, it is possible to very quickly identify small changes in flow characteristics at a measuring point. This makes it possible to recognize the emergence of leakages in a timely manner and be able to estimate the amount of loss as well as immediately determine the close area where the leak point is located.

The duration of leaking time is thereby significantly shortened, and the expense of locating the leak is also markedly reduced.

Installing the Measuring Probes

The measuring devices are placed directly into the drinking water conduit through a 40-mm borehole. The sensor head is inserted into about 11% of the open pipe diameter. At this depth the least amount hydraulic dependence of the speed profile on the flow conditions (laminar or turbulent) is expected. From the transmitted speed the total flowthrough at the reading point is determined.

In the past, the best experience was made with universal shut-off pipe saddle from the manufacturer Hawle Fittings Ltd. in Freilassing, Germany. Operation is very simple, and the durability of the seals is guaranteed by the manufacturer. The pipe saddles are delivered predesigned to fit the pipe material and the outer diameter. With their help it is possible to drill a hole while the water system is under service. A stainless steel plate make it possible to shut-off the borehole at any time.

The sensor can be installed in existing or newly constructed ducts. To do this, only a simple universal pipe saddle is needed. The sensor is screwed through the clamp outflow and aligned with the pipe length.

If there are no ducts available and no new ones will be constructed, a special sluice solution offers a simple and cost-effective alternative. A built-in sluice gate, adjusted for the pipe covering at the installation point, is screwed onto the pipe saddle, allowing the sensor – even during service – to be placed in the desired position and removed again at any time.

Figure 2: Built-in sluice gate

The modem box, also an IP68-protected housing, is now connected to the measuring device. This box contains a backup battery as well as the modem. In duct installations the box can be connected directly to the measuring device; with DIN-system installations a cable connection with a maximum length of 7 metres is permitted.

The modem must be outfitted with a GSM antenna ideally mounted at an exposed location. The precise requirements for the antenna position are determined by the mobile network coverage.

Next the power supply must be connected. There are several alternatives available here that can be suited to the local conditions. Connection to the power grid should be considered the optimal solution. This option, however, is often not realistic because of cost considerations. An inexpensive alternative that has arisen is a connection to the street lighting power supply. The rechargeable battery is charged via a power supply during the street light operating phases.

Alternatively, the energy supply can be secured via photovoltaic cells or a high-capacity gel battery that when discharged must be removed and recharged.

Installing the AQUALYS Software Environment

The AQUALYS software is equipped with a set-up programme file to simplify installation. The AQUALYS software for the WLM system organises the individual sensor data request from the CPU on one side and the graphical and numerical analysis of these data on the other side. The user-friendly software gives the authorized operator all information regarding the current leakage status for the sector of the conduit network being monitored.

The start screen displays a structural overview showing if and where leaks have occurred. A detailed view of the leak zone then shows the numerical values and a timing diagram for the individual values—flow(bidirectional), pressure and noise.

Field Experience from Crailsheim Utilities

Installation of the Sensors

A total of 20 sensors in two deliveries were installed in the STW service area. The installation experience was very good. Despite the difficulty that the pipe covering at many installation points was unknown at the time of ordering, the installations could still be undertaken quickly. The specially customized variable built-in sluice gate made this possible.

The real highlight of the solution developed is the system of power transmission. The typically used winding, which could only be made at the plant and hindered subsequent shortening of the sluice components, was rejected. Instead, a clamp system was used that Hawle Fittings had already employed in other applications, with good results.

Through this clamping the supplied sluice gate could be shortened according to the local conditions that were found when the conduit was uncovered. Putting in the universal pipe saddle and then drilling the hole to the pipe wall could be executed parallel to lengthening the built-in the sluice gate. If the necessary empty pipe is laid, the ditch can be refilled and closed. A typical access cover (measuring 40 x 40 cm) shows where the closure is and enables access at any time to the sluice gate, the sensor and the modem box.

Starting Up the Entire System

Before starting up the system, the AQUALYS software environment must be installed. AQUALYS is built on server-client architecture and thus enables trouble-free access for multiple network users. The AQUALYS server controls the connected database and gathers all communication from the probes. It processes the data received and makes the basis for visualisation available for the clients.

The programme is customizable for local conditions. The sensors are configured and the operating parameters are established. After several days of data collection, the first relevant graphics can be produced.

Test Run

After delivery and start-up of the first 10 sensors, the system was subjected to a thorough field examination. Defined flow characteristics were set at diverse positions via relocations and selective samples from standpipes. The samples were measured using a mobile flowthrough metre. These values serve as a benchmark for the sensor reading data. It should be noted that these values do not constitute absolute reference values, since the measuring device was not calibrated.

The results were positive throughout. The reading from Bergwerkstrasse is used below as an example.

Constraints:

A total of four flow velocity levels were measured for several minutes. Then a noise-producing cover was placed on the outflow of the standpipe. The sensor position was about 40 metres away from the standpipe. The sensor was installed in a DN200 cast iron pipe.

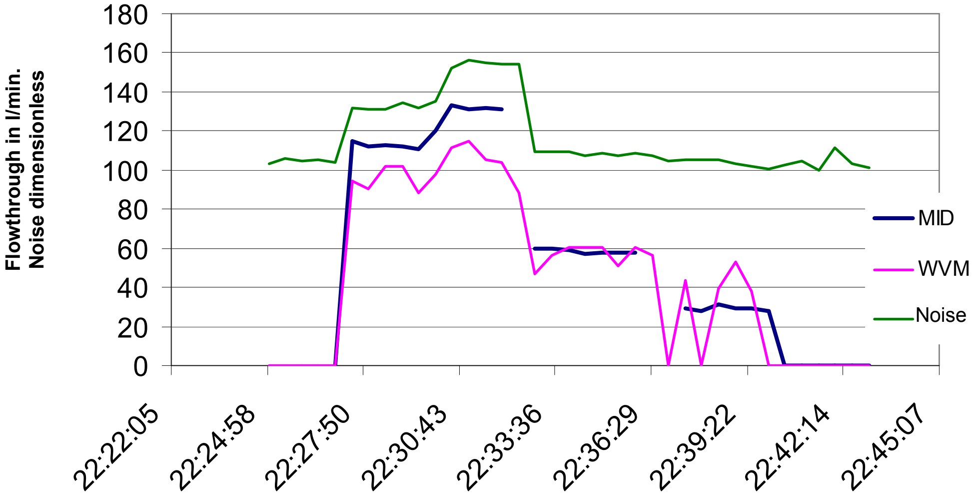

Figure 3: Data analysis from the Bergwerkstrasse measuring point

Evaluating the Flowthrough Record:

The flow levels of 110 l/min, 135 l/min, 60 l/min and 30 l/min were conducted one after the other (this corresponds to an average flow speed of 5.8 cm/s, 7.2 cm/s, 3.2 cm/s and 1.6 cm/s).

In the area of accuracy and replicability of the results, the sensor shows good alignment with the MID reference values. The deviations, however, are below 30% of the MID values and are almost completely aligned in the average flowthrough range.

The lower flowthrough range corresponds to a current speed of about 1.6 cm/s. The leak flow volume suppression was set at 2.0 cm/s during the reading. For this reason several zero values are included in the average value calculation. This minimizes the validity and leads to greater display fluctuations.

The deviation increases in this area accordingly, and the replicability of the reading results falls. The absolute deviations at 1.3 cm/s (corresponds roughly to 50 l/min) are within an acceptable range.

Conclusion:

In the current range of >3.0 cm/s the sensor read the flow speed very precisely. Higher flowthroughs tended to be underreported.

In the lower reading range the sensor still reads the flow velocity, but it cannot display the size so precisely with the selected settings in this range.

Evaluating the Noise Level Record:

The noise recording shows the flow noise directly on the sensor head very clearly. The flow noise increases or decreases direct in proportion to the flow speed. The noises that the noise cover produces are not shown in the reading curve.

Conclusion:

The sensor indicates very clearly noises from flowing water. After the insert of the noise cover it could not create a typical leak noise in the water column, the sensor could not record it.

Continuous Operation

The entire system was started up in April 2007, after the delivery of the second set of 10 sensors. The first 10 sensors were installed and in use already a year prior.

Since then there have been no problems attributable to the technology itself. The sensors function reliably and require no maintenance.

Several minor difficulties during start-up were caused by the number of technical connections outside of the AQUALYS software environment. The system was connected to the control room' s extensive alarm system. This required special attention and led to an increased amount of work.

By means of the sensor readings, the knowledge of flow activity in the conduit network could be considerably expanded. Because of the **increased number of measuring devices in the network**, it is now possible to make more accurate forecasts concerning important strands of the network. This increases the operational security of the water supply.

Already after a few weeks of use, the STW could identify irregularities and indications of leakage.

Outlook

The leakage monitoring already delivered indications of unusual network activity during non-peak hours, leading to a suspicion of leakage. By employing selected valve tunings to the intermeshed network, the damage locations can be more accurately localized. Then leak location can be conducted using traditional methods (noise correlation...).
By greatly reducing the risk, the required amount of labour can also be lowered.
At the same time, markedly shorter response times also reduce the average leakage duration. This results in less water loss.

Quickly addressing the located loss points in order of priority, according to the loss amounts displayed by AQUALYS, are aligned to.

Trunk Mains Leakage – The missing part of the jigsaw

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Keywords: trunk ; leakage; Sahara

Abstract

Advances in technology mean there are now techniques available to detect, quantify and precisely locate leakage from large diameter mains. Trunk mains leakage has tended to be forgotten in the drive to reduce leakage. The lack of suitable tools to detect and locate leaks has resulted in the focus remaining on distribution zone leakage.

WRc's database of results from surveys on trunk mains using the newer techniques show that there is significant leakage in these pipes. The large volumes carried by these mains means that even a small percentage of the total volume - too small to be detected by metering - could be a significant volume of leakage.

A comparison of the costs of carrying out active leakage detection on distribution networks with the costs of targeting and repairing leaks on trunk mains shows that reducing leakage in trunk mains is a cost effective solution to leakage reduction.

Background

Huge effort and expenditure is made on tackling leakage in water networks throughout the world. The primary focus has been on leakage from distribution networks where there are well developed strategies for monitoring leakage and targeting leakage reductions. Meters can be installed easily and the tools required to physically locate leaks on these mains have been available for many years. These tools range from the simple listening stick through to leak noise correlators and noise loggers. Network modelling packages are well developed to assist in targeting areas suspected to have high levels of leakage.

It appears that a lack of detection and location technology and the difficulties in accurately metering flows in large diameter mains has resulted in leakage from trunk mains and transmission mains taking second place in strategies to reduce overall leakage.

The Sahara® Leak Location System – developed by WRc plc in the United Kingdom – is specifically designed to detect and locate leaks in larger diameter mains, typically over 250mm in diameter. The system has been used for 10 years in the UK and globally with over 2500 surveys completed. The results of the surveys give a much better understanding of the levels of leakage in trunk mains from several.

The Sahara® Leak Location System

The Sahara system allows inspection of a pipeline while under pressure and in service. The system is used to detect and locate the precise position of leaks on water mains.

The Sahara system will pick up signs of leakage allowing maintenance to be carried out before water leaking from the pipe becomes a problem. Previous surveys and test have shown that the system is capable of finding leaks as small as 1litre/hour under normal operating conditions. The system can be used for identifying specific leaks whose existence is known from other signs, such a new pipe failing a pressure test, as well as for detecting and prioritising leaks in a network as part of a large-scale leakage reduction programme.

The Sahara system operates by deploying a hydrophone into the pipeline to be inspected. The hydrophone is connected to a signal processing and display unit via an umbilical cable. The sensor travels along inside the pipe pulled by the flow of water acting on a drogue attached to the front of the sensor. As the sensor passes any leak on the pipeline it will detect the noise being generated by the water escaping through the leak. At this point the operator is able to stop deployment of the sensor (by stopping deployment of the umbilical) and then position the sensor at the leak position by withdrawing or deploying the umbilical as necessary.

An approximate quantification of the leak size is made during the survey, allowing the pipeline operator to prioritise any repairs and to assess the overall level of leakage in that pipeline.

Once the sensor is sited at the leak, the position of the sensor can be determined using a locating system mounted in the sensor head. A second operator can track the position of the sensor head during deployment using this locating device giving an accurate indication of the sensor location and pipe track. Having pin-pointed the position of the sensor, the exact location of any leak can be marked on the ground over the pipe.

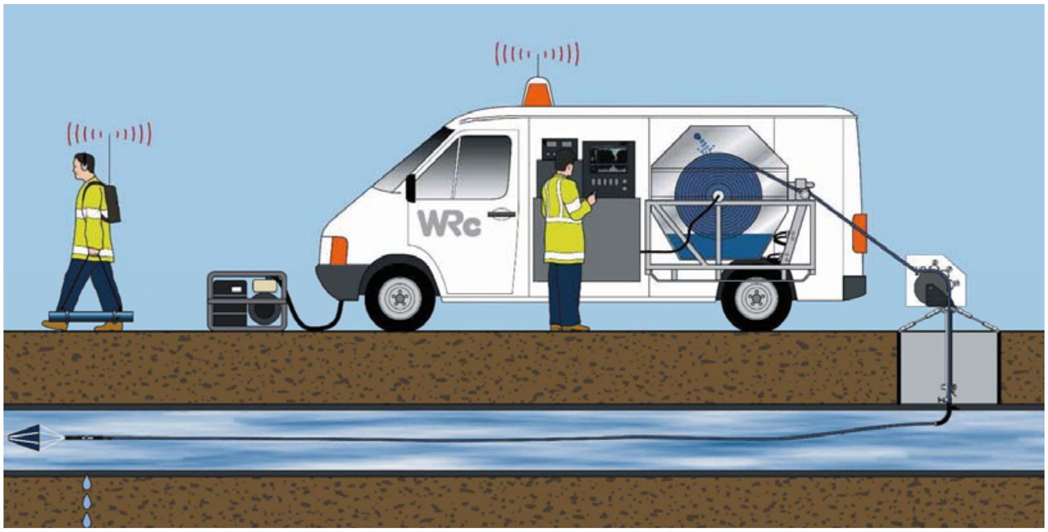

Figure 26 - The Sahara Leak Location System

The Sahara system is best suited to work in relatively straight pipelines where deployments of up to 2 km from a single insertion point are possible; other constraints may restrict the length that can be surveyed from each insertion point. Careful planning of the work will maximise the distance that can be surveyed.

The Sahara system is deployed through a tapping made into the pipe. Typically these insertion points may be air-valve connections, insertion flow meter tappings or specially installed points.

Great care is taken over sterilisation of the equipment and, in particular, the cable to prevent any contamination. The system can be used on live water mains with no disruption to customers.

Applications

Although originally developed as a method for detecting leaks in larger diameter water mains for the purposes of leakage reduction, the sensitivity of the system has been found to make it a reliable method for proving pipes are not leaking – if no leak is found during the survey the inspected section can be considered to be leak free. This has extended the range of applications which is now:

- Leakage reduction through large scale survey programmes.
- Locating of specific leaks initially identified by other means
- Proving the integrity of critical mains – demonstrating no leaks are present
- Assessing the condition of mains in areas where failure would have a high consequence
- Locating leaks in new pipelines that are failing pressure test
- Locating leaks smaller diameter, non metallic mains where other techniques have failed.

The majority of surveys carried out have been reduce leakage in the trunk mains networks for a number of UK, European and North African water utilities. More recently the system has been used extensively in North America.

Data Collection

During the survey various parameters are recorded including:

- Pipe diameter,
- Pipe material,
- Operating pressure at the entry point,
- Flow velocity (at the start of the survey),
- Distance surveyed, distance of any leaks from the insertion point,
- Estimate of the leak size.

Analysis of survey results

Summary

Over 2500 surveys have been carried out. The results of these surveys are summarised in Table 15.

Table 15 - Total number of surveys and leaks detected

Surveys Completed	Total Length Surveyed (metres)	Number of Leaks Found	Average distance between leaks (metres)	Leaks per km
2 510	1 265 080	1 702	743	1.35

Analysis by leak size

During the surveys the size of the leak is estimated. The technique uses five standard categories based on the acoustic signature of the leak noise detected. Several variables are recorded for leak size estimation. These include:

- the absolute level of the noise at the position of the leak,
- the distance from the leak position where it is first detected,
- the relative magnitude of the frequencies in the leak noise spectrum.

It is often very difficult to get an accurate assessment of the actual leak size. However, when the main is repaired efforts have been made to correlate the estimated size with the actual leakage rate. Thames Water have carried out an assessment based on the surveys carried out for them using this technique with the results shown in Table 16

Table 16 - Correlation between estimated and measured leak size

Sahara description	Sahara size category	Approximate measured size			
		min m^3/hr	max m^3/hr	median m^3/hr	median Mld
Barely audible	1	0	0.4	0	0.005
Very small	2	0.4	4	2	0.06
Small	3	4	17	10	0.25
Medium	4	17	29	23	0.55
Large	5	29	42	35	0.85

Using this method 647 leaks have been assessed to estimate their size. Table 17 shows the number falling into each category.

Table 17 - Number of leaks detected by size

Leak size category	Number of leaks
Barely audible	198
Very small	229
Small	125
Medium	62
Large	33

Combining Table 16 & Table 17, the weighted average size of leak found is 6.9m^3/hr (0.16Mld). For the UK the average distribution mains leak size is estimated to be in the range 3 to 5 m^3/hr.

Leak number and size by material

The figures above give an overview of the general level of leakage found in all mains and can be used to indicate the average distance between leaks. However a more useful technique would be to consider the number and size of leaks found in different pipe materials and pipe diameters. In general water companies will be able to provide information on the size and material of their mains. Correlating the data provided here with their mains should indicate those mains with a higher probability of leakage. The data collected to date has been analysed by material to give the results below. It should be noted that there are 157 leaks for which a size was estimated but for which the pipe material was not recorded – the results for these have been shown as "Unknown"

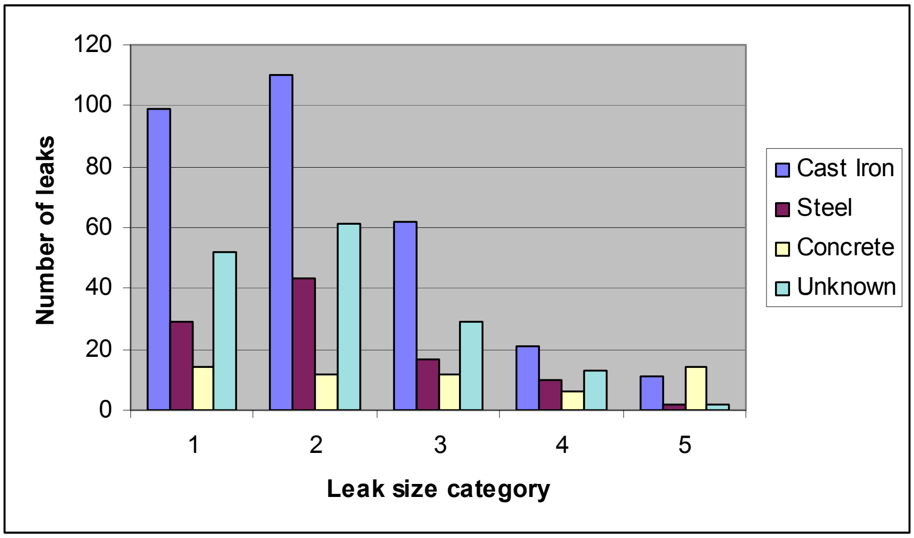

Figure 27 - Numbers of leaks classified by estimated size and material

If the figures displayed above are normalised to show the proportion of each size of leak found (rather than the absolute number) then the results are as shown in Figure 28

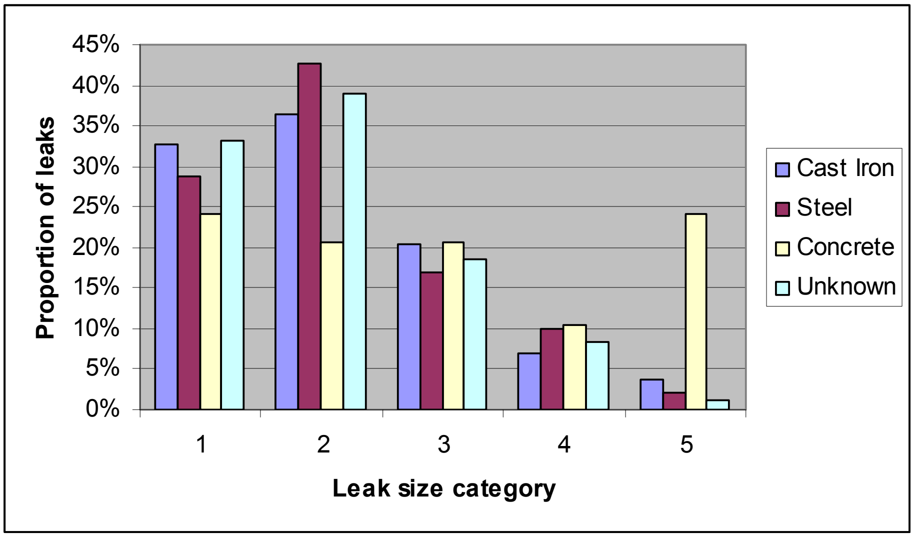

Figure 28 - Relative numbers of leaks by estimated size and material

Average distance between leaks

To give a guide of the rate of occurrence of leaks the average distance between leaks found has been analysed.

Analysing surveys where the material of the pipeline has been recorded and where there is a significant length the results are as shown in Table 18

Table 18 - Average distance between leaks (by material)

Material	Length surveyed (metres)	No of leaks found	Average distance between leaks (metres)	Leaks per km
Cast Iron	167734	443	379	2.6
Ductile Iron	11110	12	926	1.1
Steel	105485	136	776	1.3
Concrete	305827	224	1365	0.7

The data can also be analysed by region as shown in Table 19.

Table 19 - Average distance between leak (by region)

Region	Length surveyed (metres)	No of leaks found	Average distance between leaks (metres)	Leaks per km
UK	167734	443	373	2.7
Morocco	11110	12	1336	0.75
Portugal	105485	136	767	1.3
Australia	305827	224	817	1.2

Cost calculation

The cost of using the Sahara system will vary from location to location and will be dependent on variables such as:

- mobilisation costs,
- cost of enabling works,
- rural/urban location,
- local taxes or insurances.

In order to assess the cost effectiveness of Sahara, the following scenario has been developed which uses the average number of leaks per km and the average leak size from the data discussed earlier. The scenario allows the long term costs and benefits to be evaluated.

For this scenario, it is assumed that 10 km of trunk main are surveyed each year for 3 years. All the leaks found are subsequently repaired and leaks do not re-occur within 10 years. This means that the water saved per leak will accumulate as all the leaks are repaired and then be held at this level for at least 10 years. Further assumptions are shown in below:

Average size of leak detected:	0.141	Ml/d
Average length of survey	500	m
Length of trunk main surveyed per year:	10	km
Number of leaks found per km:	1.35	
Total length of trunk main surveyed in 3 years	30	km
Installation and inspection cost per insertion chamber:	11000	£
Assume a new insertion chamber is needed for all surveys.		
Average repair cost per leak:	7500	£
Discount rate	5.75	%

The long run marginal cost (LRMC) of water saved can then be calculated. The LMRC is calculated as the net present value of the implementation costs divided by the net present value of the water saved, presented as a cost per cubic metre.

Table 20 shows the LRMC values.

Table 20 - LRMC Calculation (UK)

Sahara leak location		NPV	Years			
	Units		1	2	3	4 to 10
Capex	£	0	0			
Installation & survey costs	£	590,793	220,000	220,000	220,000	0
Repair Costs	£	271,072	100,942	100,942	100,942	0
Water saved through repairing leaks	Ml (Megalitres)	15,034	413	1,238	2,063	2,476
Long run marginal cost	**(LRMC)**	**5.7**	p/m3	(over 10 years)		

Under this scenario the LRMC for using Sahara is 5.7 pence/m^3 (discounting the costs and benefits over a period of 30 years gives a LRMC of 2.7p/m^3), which can be compared with the long run marginal cost of water.

Case Studies

Thames Water – London, UK

Thames Water supply water to 13 million customers across London and the south east of England. Thames Water have used the Sahara system to inspect their trunk mains network in London for 10 years. Many of the pipes in London are over 100 years old and Thames Water has an active programme to replace many of it Victorian cast iron distribution mains.

Thames Water are required to reduce leakage by the Water Industry Regulator and also have resource restrictions that put further pressure on the company to reduce water loss wherever possible.

Since embarking on the inspection programme nearly 1500 surveys have been completed and over 1250 leaks found. The majority of these survey have been carried out in cast iron mains with lead run joints. The remaining surveys have been carried out in steel mains. The average distance between leaks (for all the surveys carried out) is 505m. If only the first survey at each site is considered the average distance is 373m.

During the first two years of the inspection programme it was reported that Thames water had been able to reduce its water loss from its trunk mains network by 65Mld through detecting leaks with the Sahara system. The leaks detected ranged in size from over 0.5 Mld to those assessed to be of the order of 10 litres per hour.

During the early stages of the survey programme Thames water established that the sensitivity of the system was such that it could be used to demonstrate that pipes were not leaking (if no leaks were found).

Lydec – Casablanca, Morocco

Lyonnaise des Eaux de Casablanca (LYDEC), is a subsidiary of Suez Environnement (Suez group of companies) and the concessionaire of water distribution, wastewater treatment and electricity distribution in Casablanca, Morocco. It has 710 000 customers for water and supplies 170Mm^3 of water per year. Lydec must buy all the water it supplies to customers from bulk water supply companies.

The transmission network inspected is generally constructed from concrete pipes and was laid approximately 50 years ago.

Since 2000 over 400 surveys have been carried out for Lydec in Casablanca with over 200 leaks found. The total length of mains surveyed is nearly 300km which includes re-surveying a majority of the mains in the city. The average distance between each leak is 1336m. It should be noted that in general each section of pipe has been surveyed twice with repairs made between the surveys the average distance would be significantly less if only the first survey at each site were considered.

In the first phase of surveys, the reduction in water loss attributed to leaks located using the Sahara system and subsequently repaired equated to an overall reduction in water into supply of 3%. This reduction can be shown to have an immediate and direct effect on the financial position of the company.

EPAL - Lisbon , Portugal

EPAL (Empresa Portuguesa de Águas Livres) looks after the water supply to over 3 million consumers both in and around the city of Lisbon, Portugal. In 2007 EPAL commissioned surveys of 22km of transmission mains in Lisbon and the surrounding area. The mains surveyed were mostly (20km) pre-stressed concrete pipe.

The surveys were carried out from access points created at pre-existing chambers on the pipelines. The chambers were either air valve or valve chambers.

The survey programme found 29 leaks on the inspected sections - an average of 1 leak every 767m. Although work to assess the actual quantity of water saved through repair of the leaks is continuing, early indications are that the volume saved is approximately 5000m^3/day.

Sydney, Melbourne, City West and Yarra Valley Water – Australia

During 2006 a survey programme was arranged for a group of Australian water companies – Sydney Water and Melbourne Water, City West Water and Yarra Valley Water from Melbourne.

40 surveys were carried out on a variety of mains with materials including cast iron and wrought iron. The total length surveyed was over 26 km and 32 leaks were found. This gives and average of one leak for every 817 m.

Conclusions

The results documented here have been built up from over 2500 surveys which have been carried out on trunk mains of a variety of materials and in a number of regions around the world.

The average distance between leaks is of the order of 750m across the range of materials, with the figure for cast iron mains less than 400m between leaks.

The average leak size has been estimated and verified to be 6.9m^3/hour (0.16Mld).

Calculating a long run marginal cost for leak detection and repair in trunk mains yields a figure of 5.7pence (Sterling)/m^3.

The figures presented show that not only is leakage from trunk mains a significant issue in the overall picture of water loss but it can be tackled in a cost effective manner.

Decision Support System (DSS) for Water Loss Reduction: Approach Based on Simulation Models

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Keywords: decision support system; water loss reduction; water network simulation.

Introduction

Decision Support Systems (DSS) for the reduction of water losses have the purpose to provide indications on the most suitable strategy to be followed either from the technical or economic point of view. The topic is strictly connected to the rehabilitation of the entire infrastructure of water distribution networks (WDNs), under the structural and operational profile. A preliminary analysis is needed by means of DSS as initial phase and, subsequently, via the checking of possible choices, in terms of effectiveness and cost and in the search of the best integration of available technologies. This is also evident because of the WDN complexity and of the limited economic resources. Its final purpose should therefore be to help stakeholders optimise the use of available information.

The paper provides indications about the application of two types of DSSs, underlining how much the selection of a DSS, either based on synthetic models or based on numerical simulation models, has to be consistent with the knowledge of the WDN and with the expected timing to complete water losses reduction projects.

Finally, some results achieved with the application of Sanflow, HDF, Econoleak and Presmac, developed by WRC (South Africa Water Research Commission), and with physically based models, as HyNet® developed by EHS S.r.l., are presented. The case study is a portion of the Modena water supply network.

Decision support systems and water losses reduction

The level of detail applied in the description of the WDN and the type of approach selected for the hydraulic analysis are the primary criteria to distinguish between different kinds of DSSs used in water loss reduction models. In all DSSs the knowledge of hydraulic state, essentially pressure and flow in some points of the WDN, can be derived from direct measure of the hydraulic variables (*lumped-parameter* DSSs), or from numerical analysis through a simulation model (*distributed-parameter* DSSs). In the latter measured data are used for calibration, allowing a numerically based spatial knowledge of the hydraulic condition, which is missing in the former.

Lumped-parameter DSSs are based on a simplified mathematical approach simplified with respect to the physical system configuration, in fact the spatially distributed variables are represented as a single scalar instead. This simplification appears reasonable when the system is homogeneous enough

Distributed-parameter DSSs have the remarkable potentiality of catching the spatial distribution of water losses, at least qualitatively. In this way, particular attention must be given to spatial distribution of physical components of water losses, because this leads to a numerical evaluation of the hydraulic consequences of different actions, like water leakages detection and repair campaigns, pressure active control and pipe and components rehabilitation or renewal, combined into possible technical scenarios.

Case Study: the Ganaceto DMA

A DMA (District Metered Area) of the Modena water supply system was selected, in order to carry out, on a real case study, a comparison between the two kinds of DSSs for water losses reduction. The selected system is managed by HERA S.p.A.

The DMA includes the villages of Ganaceto, Lesignana and Villanova, and it has, as natural border, the Secchia river. The selected case study has an area of 2.4 km^2 and it serves 2925 inhabitants. The Ganaceto sub-system is situated in a flat area, with an average level of 32.86 m over the sea level.

The distribution system covers a total pipe length of 35.4 km with 540 connections. Pipes are mainly in asbestos cement and polyethylene. The DMA has only one entry point, located at 41 meters over the sea level on the bridge of Secchia river.

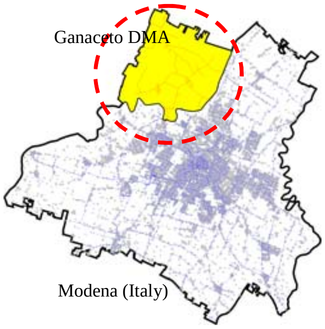

Figure 1 DMA of Ganaceto (Modena, Italy).

The Utility, HERA, monitored the DMA with a monitoring time interval of 3 minutes, using a flow meter (at the entry point) and two pressure meters (one located at the entry point, one inside the DMA). The data used in this paper were collected right after the DMA realization.

The study was performed applying the following steps: (a) collection and analysis of data provided by HERA: data on asset, on water demand and consumption, pressure and flow data measured; (b) estimation of the DMA daily leakage flow, with the minimum night flow method; (c) definition of possible leak reduction scenarios, with pressure management, Active Leakage Control (ALC) and Rehabilitation of some pipes.

Daily Real losses evaluation through Minimum Night Flow method using SANFLOW and HDF

The Minimum Night Flow (MNF) allows the calculation of water losses in the hour of minimum consumption Q_{MNF}, by subtraction of the Customer Night Use (CNU) from the measured minimum night flow.

Since the night flow may be attributed to residential and non residential use, the former was computed with the SANFLOW model by considering the number of served customers and it was estimated equal to 2.64 m^3/hour. The night consumption of the 107 not residential customers was estimated with a survey made in November 2004. Detailed data on consumption, referred to night period between 1.00 to 5.00 hour, were collected for each customer during that survey.

Work activities of the area include six cow farming's and one industrial mill. For the latter it was possible to estimate, from data provided by the customer, a night consumption of 0.9 m^3/h. The night consumption for the farming is based on the assumed consumption of each cow (4l/h) and on the total number of cows.

In conclusion, a non residential night consumption of 4.86 m^3/h was estimated; summing the residential and non residential consumptions, an overall night consumption of 7.50 m^3/h was estimated.

Table 1 Total Real Loss evaluation by Minimum Night Flow (MNF) method.

Date	MNF Time	Measured MNF (m^3/h)	CNU (m^3/h)	*Total Real Loss* Q_{MNF} (m^3/h)
23/09/04	2:40 AM	33.48	7.50	25.98

Using the results obtained by the MNF method, the total volume was computed of water lost in the DMA during the day September 23th 2004, because of burst and background losses (Table 1).

The value of total daily losses Q_d was computed by taking Q_{MNF} times the Hour Day Factor (HDF). This factor was calculated using the pressure profile at Average Zone Pressure (AZP) point, a specific network point whose pressure is taken as representative of the whole DMA. HDF concerning the MNF hour (2:40 AM) is equal to 21.63 hour/day and leads to a Q_d value of 562 m^3/day, then WR1 is 43% (Alegre et al. 2006).

Lumped-parameter DSS on a case study

Definition of possible scenarios for water losses reduction by using ECONOLEAK and PRESMAC

The network aging process produces, as a direct consequence, the increase of the real water losses phenomenon, which can be reduced and managed by integrated application of different strategies that can be summarised in: Active Leakage Control, to discover the leakages points. it includes different possible techniques to be applied separately or together. Pressure Management, one of the most efficient methods towards water loss reduction especially if applied together with active leakage control; high-quality results were archived by pressure reduction valves installation at inlet points. Speed and Quality of Repairs have effect on the activity duration of the leakage and on further pipe repairs frequency . The assets management allows to reduce leakages by replacing, rehabilitating or renewing the assets.

Real losses management with Active Leakage Control in the Ganaceto DMA

The ECONOLEAK model allowed to define the optimal frequency for leakage control. The result is based on the estimation of real annual losses made using the Burst and Background Estimate –BABE- method linked to an economic analysis (Lambert,1994).

Computation of the real yearly leakage using BABE

For each system component, the BABE methodology computes real losses on the basis of their nature (Background, Reported and Unreported Bursts Losses) for the different WDN components.

For every system component Background Losses (Table 2) were computed as the product of the flow rate, referred with Pressure Correction Factor ($PCF=(AZP/50)^{N1}$) to the average pressure (AZP=33.5 m) of the DMA, times a parameter characteristic per system components. This parameter is the length for the main system, and the number of connections for connections and service pipes.

Table 2 Evaluation of Background losses in the DMA of Ganaceto.

Background losses	Flow rate@50m	Units	N1	Background Losses (m^3/day)
Distribution mains	51.3	l/km/h	1.5	23.83
Connections	3.2	l/conn/h	1.5	22.78
Supply Pipes	1.3	l/conn/h	1.5	9.11
Total				56.72

Reported Bursts losses (Table 3) were estimated per system components as the product of the flow rate (corrected with PCF), times the burst frequency (evaluated from the number of repairs done by the Utility after customers complains) and the leakage duration.

Table 3 Evaluation of Daily Reported Bursts Losses in the DMA of Ganaceto.

Reported bursts losses	Activity (day)	Flow rate@50m (m^3/h)	Frequency	Units	N° of Burst	Reported Losses (m^3/day)
Distribution mains	4	12	0.310	per km/year	10.9	34.0
Connections	50	1.6	30.359	per 1000 conn/year	16.4	67.8
Supply Pipes	60	1.6	20.386	per 1000 conn/year	11.0	54.6
Reported Losses						156.4

The sum of Background and Reported Burst Losses is the theoretic lower limit (Base Level), achievable only with very frequent leakage detection activities able to remove (or minimize) Unreported Burst losses.

Unreported Burst losses (Table 4) was estimated as the product of the flow rate (corrected with PCF) times the burst frequency and the leakage activity duration, sum of the semi-period between two consecutive ALC survey and the repair time.

In this case the effect of 5 different ALC frequencies was analysed. The estimation of unreported bursts is based on observed data regarding bursts discovered during the last leakage detection campaign on the DMA in 2002.

Table 4 Evaluation of unreported daily burst losses for different ALC frequency (* per km mains/year, **per 1000conn/year).

Unreported bursts losses	Flow Rate @50m	Frequency	Bursts	Unreported Losses with ALC				
	(m^3/h)		N°/year	every 6 months	every 1 year	every 2 years	every 4 years	every 6 years
Distribution mains	6	0.0078*	0.274	8.1	15.9	31.4	62.4	93.5
Connections	1.6	4.0681**	2.197	24.6	41.2	74.3	140.7	207.0
Supply Pipes	1.6	3.0375**	1.640	19.7	32.0	56.9	106.4	155.9
Total (m^3/day)				52.4	89.1	162.6	309.5	456.4

For the five ALC frequencies the annual real loss was calculated by summing the base level of leakage with the Unreported burst leakage, as shown in figure 3.

Economic Analysis in order to derive the optimal ALC frequency for the DMA

An economic analysis was carried out by comparing the costs due to water losses to the annual operational costs for the active control with planned monitoring actions. The cost of water losses is computed by multiplying the lost volume, times an unit cost of 0.06 €/m^3 for energy demand for well's pumping and disinfection processes.

For the different active control techniques, the assumed costs are listed on the following table (Table 5) together with the operational costs necessary for the number of operators requests for interventions and the costs of interventions themselves.

Table 5: ALC and pipe reparation costs.

Costs to detect and repair leaks		
Cost for regular sounding per km	480	€/km
Cost for Correlation per km	480	€/km
Administrative Set Up Costs	3000	€/year
Cost to repair Mains Leak	1600	€
Cost to repair Connection Leak	900	€
Cost to repair Service Leak	900	€

Figure 2 shows the costs of active leakage control and monitoring for different leakage detection frequencies and the corresponding lost water costs. The total annual costs curve is then obtained by interpolation of the costs of water production and the costs of leakage detection for the 5 frequency classes.

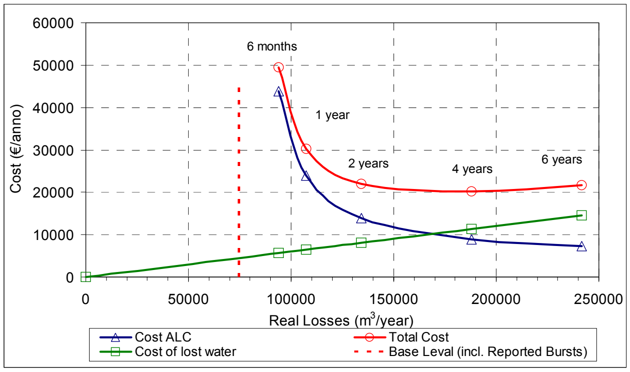

Figure 2 Costs of ALC and lost water for different survey frequencies.

The minimum of the total annual costs curve identifies the optimal frequency for leakage detection. The result for our case study is close to 4 years between two consecutive leakage detection campaigns (for a water cost of 0.1 €/m^3 the optimal time between two ALC surveys becomes close to 2 years).

Real Losses reduction with pressure management for the DMA

It is confirmed by practice that pressure level of a drinking water system needs to be constant in time and close to the level required to guarantee service to customers. The recoverable water volume through pressure reduction was evaluated by PRESMAC.

The model input is the flow entering the DMA and the pressure at the AZP point and at the critical point wherein the pressure experiences the lowest value.

Based on the estimation of water losses during the night and on the pressure profiles, PRESMAC allows to separate the inflow into two components, one pressure dependent and one pressure independent (as shown in Figure 3).

Two hypothesis of pressure control were suggested for the case study. The first scenario introduces a pressure reduction valve (PRV) at the entry point of the DMA.

The second scenario introduces a PRV combined with controller that regulates the valve in time (figure 4 – right): the higher pressure value is fixed at 28 meters between 5.00 am and 10.00 p.m. and at 25 meters during the night. As shown in table 8, the daily flow decreases as effect of the reduction of its pressure dependent components. The PRV fixed outlet was equal to 28 m as shown in figure 4 (left).

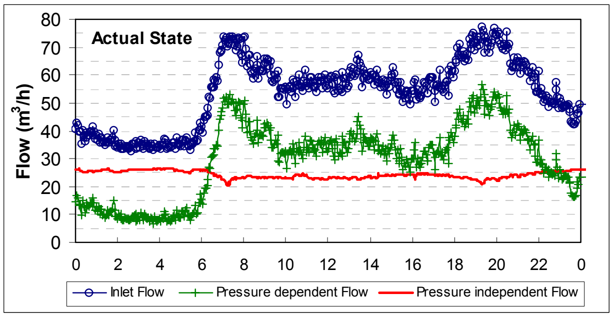

Figure 3: Actual inlet flow separated in pressure dependent and pressure independent components.

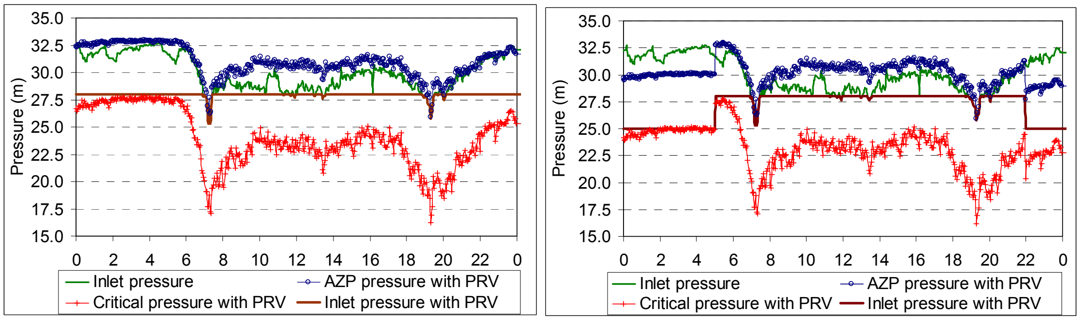

Figure 4: Main pressure profiles for the first scenario (left) and for the second scenario (right)**.**

The DMA object of study already presents a pressure level close to the minimum required to guarantee the service; due to the daily flow pattern, pressure tends to decrease considerably in some hours during the day, hence it is not possible to produce higher pressure reductions.

Table 6: Retrieved daily water volume with fixed outlet PRV and time modulated PRV.

Daily input volume at current situation (m³/day)		Scenario 1	Scenario 2
Indipendent from pressure	711.6	711.6	711.6
Dipendent from pressure	583.4	550.1	535.7
Total	1295.0	1261.6	1247.3
Recovered volume (m³/day)		33.3	47.7

However, it was possible to save more or less 9405 m³/year as consequence of pressure reduction in the former scenario and 13545 m³/year in the latter scenario (Table 6).

Distributed-parameter DSS on a case study

The numerical simulation models

The previously described models are able to analyse the water balance at a system scale. The level of detail of these models is constrained by the lack of information on the network physical characteristics. The network physical asset is indeed synthesised by few parameters, therefore losing the real picture of the interaction between network and plants of the WDN when it becomes too complex..

As a matter of fact, the evaluation of system parameters becomes more and more complicated as the number of system components, entry points or plants increases. Highly variable altimetry profiles may be an additional obstacle to the use of Lumped DSSs. In similar conditions such tools should rather be applied that, even if more complex, are able to consider the real system dimension and simulate its real performances. In addition, by applying these tools, it is possible to observe the system behaviour in the current state and in possible conditions resulting from the application of different intervention strategies, as pressure management and asset management.

The network model for the DMA of Ganaceto

The system model for the DMA of Ganaceto was built using available data on pipes and customers consumption. Leakages were considered to be concentrated at nodes as described in (1). The evaluation of the coefficient α for each network node leads to a leakage allocation within DMA.

$$q_i = \alpha_i P_i^n \tag{1}$$

where P_i is pressure at i-th node. The evaluation of the α coefficients was made by the application of a genetic algorithm coupled with a hydraulic solver able to find the configuration of the α coefficients of leaking points by minimising a specific fitness function. This function (2) is based on the deviation between the measured pressure and flow values and simulated ones: the variables are the α coefficients themselves. The exponent *n* was assumed uniform inside the DMA and estimated equal to 0.75 by try and error procedure.

A general expression of the used fitness function is (Artina et al., 2006):

$$F = \min\left[\sum_{i\in IN} K_1\left(\frac{P_{is} - P_{im}}{P_{im}}\right)^2 + \sum_{j\in IT} K_2\left(\frac{Q_{js} - Q_{jm}}{Q_{jm}}\right)^2\right] \tag{2}$$

where P_{is} and P_{im} are simulated and measured pressure values at the i-th node and Q_{is} and Q_{im} are simulated and measured flow at the j-th pipe. K_1 and K_2 are two weighting parameters selected as function of the measurements equipment uncertainties. The calibration was made with data available at MNF time.

In order to guide the genetic algorithm during the calibration phase, a procedure to define the susceptibility to leakage of each pipe was developed. This procedure takes into account: pipes attributes (material, diameter,..), pipes bedding conditions (bedding material, presence of groundwater,..), pipes operational conditions (average pressure compared to the nominal one), pipes loading conditions (dynamic and static load) and all available information about each pipe history (failures, installation year,..). As a final result, a label is attributed to each pipe indicating its leakage susceptibility level.

The second step was to drive the algorithm to explore, firstly, the configurations predicting the higher leakage level for the most leakage-prone pipes. Picture 5 shows the network sensitivity to leakage and a map of the α coefficients whose value is related to global leakage amount for each node.

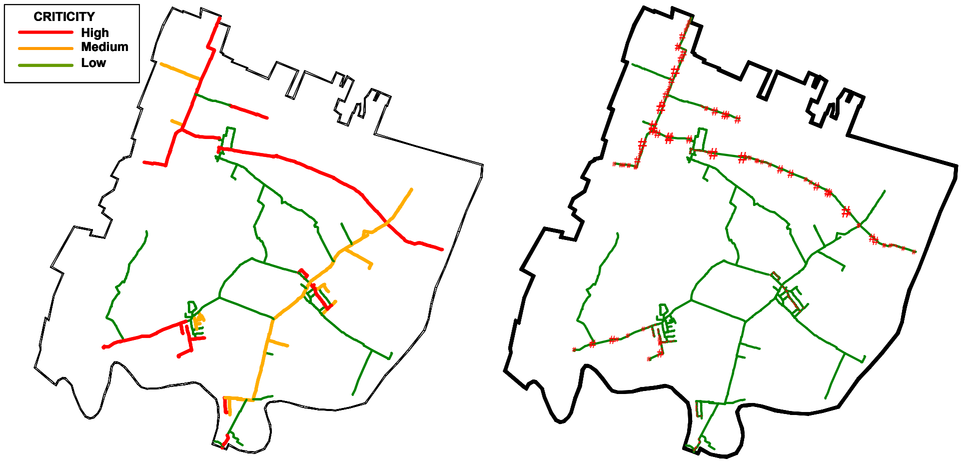

Figure 5: Pipes susceptibly to leakage (left) and estimated α coefficients distribution (right)

Figure 6 shows the measured flow entering the system and the measured pressure in a point inside the DMA and the simulated ones. It should be noticed that a higher weight was assigned to the water balance component than to the pressure component. In fact the entering flow profile is very close to the measured one, while the pressure profile is rather under-estimated. The deviation between measured and simulated pressure is higher during the hours of maximum demand, probably as a consequence of the use of roughness coefficients taken from literature, or, to put it better, not calibrated.

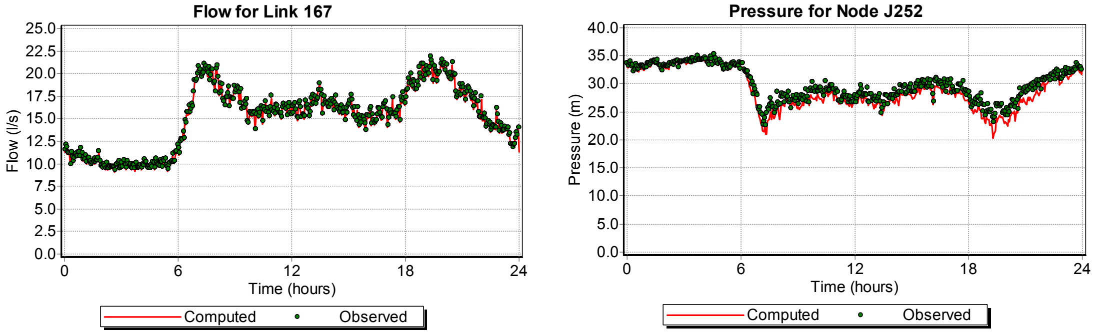

Figure 6: Flow entering the system (left) and pressure at j252 node (pressure gauge point) metered in DMA and computed ~~ones~~ values as results of calibration activity.

After the calibration described, the model~~,~~ allows to simulate the behaviour of the network for different intervention scenarios. The calibration balances the physical leakages computed with the water balance and the hydraulic characteristics of the system and the measured data. That means it is not to be read as a tool to pinpoint the leakage position but only an α configuration coherent with pressure and flow measures, with physical DMA characteristics and with real losses estimate. This result may be a suitable starting point for further activities as field campaigns.

The calibrated model, which describes very well the system performance with regard to leakages and consumptions, is able to simulate different intervention activities effects. Assuming September 23rd 2004 as a reference, two intervention scenarios were proposed.

Scenario A

Under the same hypothesis considered in the analysis with PRESMAC, a pressure reduction scenario obtained using a simulation tool was suggested.

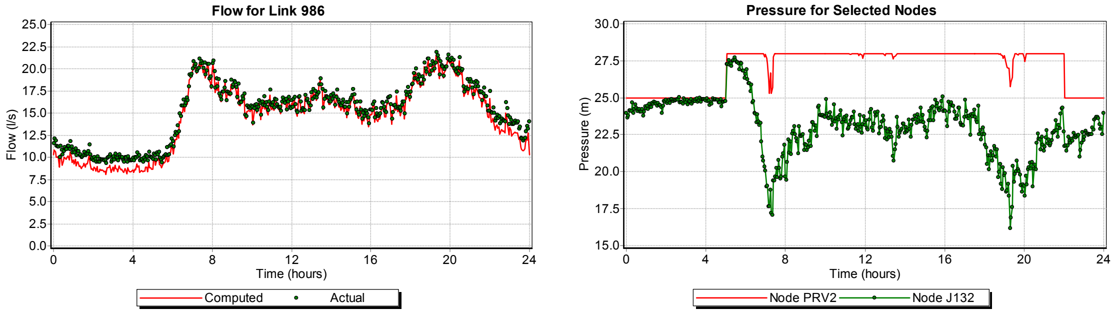

Figure 7: Flow at the inlet point of DMA for actual state (measured) and for simulated scenario (left). Pressure profile at the inlet and at the critical point (right) with time modulated PRV**.**

Although the calculated pressure profile at the critical point (Figure 7 – right) confirms that only a limited pressure reduction is realistic, yet it is possible to save a water volume of 8800 m^3/year (with fixed outlet PRV) and 12750 m^3/year (with time modulated PRV).

Scenario B

For the pipes assumed to be more sensitive to leakage, a rehabilitation plan, joined with the same pressure reduction hypothesis considered in Scenario A, was considered in scenario B. In order to represent the rehabilitation effect, a reduction of 60% of coefficient α was assumed (due to uncertainties in α determination). The two pressure configurations of scenario A were maintained in scenario B. Some graphical results are shown in figure 8.

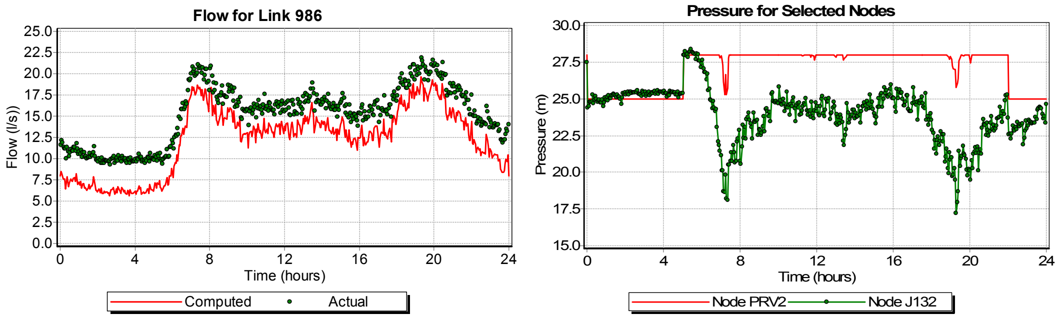

Figure 8: Scenario B with time modulated PRV: Flow at the inlet point of DMA for actual state (measured) and simulated (left). Simulated Pressure profile at the inlet and at the critical point (right)**.**

A distribute-parameter model may simulate ALC in the same way of rehabilitation, but with a different evaluation of the volume that can be saved. In fact the model is able to evaluate, the volume saved during the simulated period of time excluding, in the first case, the volume that is not possible to revenue (considering lower α abatement).

The volume saved in scenario B, for the two proposed pressure configurations, was evaluated in 30425 and 31325 m^3/year respectively.

The calibrated model of the network is able to estimate the water volume saved with the specific intervention even on a system's portion, under the uncertainty limit of the allocation method of leakage points. If the result predicted by the model will be confirmed by the planned leak detection campaigns, it will be possible to suggest different monitoring frequencies for different network portions.

Conclusions

The DMA of Ganaceto is a particular case study that allows the application and comparison of lumped-parameters DSSs and distributed-parameters ones. Both kinds of DSSs require the estimation of the night consumption in order to evaluate the real loss component of water leakage. In order to reduce the daily water leakage rate of the DMA , estimated in about 40-50%, possible alternative strategies were analysed.

Lumped-parameter DSSs shows some application difficulties for districts with more than one inlet point; moreover pressure profile measures are required for particular network points (AZP and critical points), which are not so simple to identify before district realization or without a network model. In addition, the evaluation of recovered volume, for example by pressure reduction, is highly affected by AZP position and by the choice of the N1 exponent value. Information about burst events (reported and unreported) in conjunction with some flow rates are also requested. In particular, burst flow rate are not usually collected; these limitation made, in some cases, the application of this DSS kind not easy to carry out.

Some obstacles are removed in distributed – parameters DSSs. No preliminary identification of particular points inside the WDN are indeed requested and no limitation on the number of the DMA entry points is needed. Further on, the exponent N1 may be estimated as the one that allows the best fitting between simulated and measured data, so that neither it is assumed *a priori* nor it is estimated by means of complicated measure procedures. Distributed – parameters DSSs are far more suitable for leakage prediction even before DMAs are defined. Finally, such models are recommended in case of lack of a sufficient number of campaign measurements.

The study presented in this paper shows that the two typologies of DSSs provide comparable and robust results if the requested data are available and correctly introduced. The selection of the more suitable tool for WDN analysis may therefore be done on the basis of data availability and of the aim of the analysis.

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Reliability assessment & data classification using discriminant functions & factor analysis

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Keywords: Pipe networks; Factor Analysis; Reliability assessment

Abstract

The paper deals with the reliability analysis of the offshore pipe networks in the North Sea (NS) and in the Gulf of Mexico (GoM), using the discriminant analysis/classification (DAC) method based on pipe characteristics. The pipes are divided into two groups, the “failures” (failed at least once) and “successes” (never failed). Several scenarios, based on different combinations of variables, are analyzed. A sensitivity analysis, regarding the available data, takes place to examine the stability of the results. The goals of the study are to develop a model that can correctly classify the network pipes to “successes” and “failures”; to define the pipe characteristics having a major impact on a pipe’s behavior; to assess the pipe reliability; and to check whether the method can be also widely applied.

Introduction

The paper compares the results of the discriminant analysis/classification (DAC) method (Tatsuoka et al., 1954), used for pipe reliability assessment, in two offshore pipe networks (in the North Sea –NS- and in the Gulf of Mexico –GoM-) supplying oil and gas. As Discriminant Analysis (DA) separates distinct groups of *units* (objects/observations), the network pipes are divided in “failures” (if failed at least once) or “successes” (if never failed), using failure data records. DAC method uses pipe characteristics, such as length, diameter, wall thickness, operational pressure, product (type of fluid supplied, e.g. water, oil, gas), grade and lifetime, as variables. In order the analysis to be based mainly on dimensionless variables, joint variables, resulting from simple ones, are introduced. Multivariate techniques (e.g. DAC) have proven to be very effective on pipe reliability assessment (Bakouros, 1988). DAC method reveals the correlations among pipe characteristics that influence failure rates. Also, joint variables are very important as they “carry” more information than simple variables do. As DAC method highly depends on the quantity/quality of pipe break records, suitability problems for pipes supplying various fluids might arise, if their failure data records do not meet predefined standards (usually the case in Water Utilities). Classifying water pipes into groups is trigging, as the meaning of "failure" must be defined. Failing and surviving pipes can be distinguished based on the water losses rate (pipes experiencing leaks or breaks respectively) or on the total water volume being lost (pipes experiencing breaks or leaks respectively).

Methodology

Discriminant analysis

Researchers are thoroughly studying pipe failures for many decades now, trying to figure out the way that failures occur and thus be able to suggest cost effective reliability improvements (De la Mare et al., 1980). Usually these studies classify failures based on their *average rate of occurrence* in terms of *incidents per kilometre per year*. Studies revealed that failure rates must be used as decision making criteria with scepticism otherwise may lead to foggy suggestions and very restrictive models for pipe failure probability estimation within a defined period of time, as they do not clear out which pipe characteristic is the most significant one (Kanakoudis, 1998). DA has been widely used since its introduction, some fifty years ago (Bakouros, 1992). Sayles has used Discrimination Function Techniques (DFT) to classify reliability information in a meaningful form (Sayles, 1980). DA is a multivariate technique used to separate distinct groups of objects/observations and investigate observed differences when casual relationships are not well understood. DA's goal is to graphically or numerically describe, the differential features of objects/observations coming from several known populations. DA tries to establish "discriminants" whose values assist to separate the populations as much as possible.

If n is the given population of objects/observations (with several of their characteristics already observed), the idea of classification is to *split* them into two groups on the basis of their internal similarities. The basic idea of discrimination however is fundamentally different: given the existence of two complete and mutually exclusive populations P_1 and P_2; and two random samples of individuals originated from P_1 and P_2 respectively; the goal is to establish a rule to allocate individuals, of uncertain origin, to the correct population. This may be achieved under some defined condition of optimality, e.g. by either making as few mistakes as possible or more realistically on a cost basis by minimizing the total misclassification cost. There are several types of "discriminant" functions (linear, quadratic, exponential, logistic), integrating the weighted characteristics of the sample. The present study uses the linear type, as it interprets well the contribution of each variable to the discriminant power:

$$Z_m = U_0 + U_1 X_{1m} + U_2 X_{2m} + ... + U_i X_{im} \quad (1)$$

where: Z_m is the value (score) on the canonical discriminant function for case m; X_{im} is the value of the sample's "ith" characteristic (e.g. length, diameter,); and U_i is the best discriminant coefficient or "weight" to attach to that value to get the best discrimination.

DA's goal is to determine the specific variables "X_i" and their coefficients "U_i" that efficiently separate the samples into distinctly different sets, through the following steps:

1) Determine the representative samples from the distinct populations;

2) Determine their characteristic values (X_1, X_2,…,X_n);

3) Determine the characteristics supposed to give the best discrimination;

4) Calculate the corresponding Z and U_i values that provide the best separation based on the selection of the characteristics;

5) Repeat steps 3, 4 using other combinations of characteristics until an optimum separation of the samples into their distinct sets is achieved.

There are certain basic assumptions underpinning the application of DA: a) At least two groups and two cases per group are needed; b) The number of discriminant variables is unlimited, providing that the number of cases exceeds this number by more than two; c) If a joint variable resulting from the linear combination of others doesn't contain any additional information, should be avoided; d) Each group is a sample of a population following a multivariate normal distribution of the discriminating variables, as DA's accuracy of is not so sensitive to minor distortions on this normality assumption (Lachenbruch, 1975); e) A linear discrimination procedure is optimal if the variances of independent variables in one group are the same as in others and if the correlations among independent variables in one group are the same as in others (covariance matrices are equal) (Krwanowski, 1977; Madria et al., 1979); f) Data must be correctly classified into two or more populations, as failing to do this would result in the wrong probabilities of group memberships and therefore any use of the discriminant function based on such populations would be misleading (Fisher, 1936).

A discriminant variable X_i is considered stable if the sign of its non-standardised coefficient U_i agrees with the sign of the corresponding difference between the mean values M_{ig} of variable i for cases in group g (e.g. for two groups $sign(U_i)*sign\ (M_{i1}-M_{i2})>0$). Unstable discriminant variables are excluded from the analysis otherwise they are thought to be a result of very unequal variance-covariance matrices (Bakouros, 1988). Unfortunately the effect of instability seems to have received very little attention to date. Standardized coefficients S's can be computed from the non-standardized coefficients U's using Equation.2:

$$S_i = U_i \sqrt{\frac{w_{ii}}{(n-g)}} \tag{2}$$

where: w_{ii} is the sum of squares for variable i; n the total number of cases; g the number of groups. By examining the absolute magnitude of the standardized coefficients (the larger the magnitude, the grater the variable's contribution is), the most important variables are defined. Effectively DA attempts to maximize the distance between the Z-score centroids and it discriminates between new samples using a criterion given by:

$$Z(critical) = \frac{Z_{mf} + Z_{ms}}{2} \tag{3}$$

where: Z_{mf} and Z_{ms} are the average Z-scores for the pipe "failed" and "survival" group respectively. Using the "Bayes Theorem" it is possible to calculate the probability that any pipe with known Z-score to belong to either group (failure probability prediction) (Bakouros, 1992).

Checking the success of discriminant functions

Any discrimination procedure can be evaluated using two different statistical techniques:

a) The first measures the difference between groups over the discriminant variables by testing the null hypothesis of the equality of the centroids of each group and was formulated by Wilks (Bakouros, 1988) as a ratio of determinants, denoted as Lambda (Λ):

$$\Lambda = \frac{|W|}{|T|} \tag{4}$$

where: W, T are the determinants of Sums of Squares and Cross Products matrix (SSCP) and total sample SSCP matrices respectively. As (Λ) value gets closer to 0, the group centroids are separated better and discrimination is higher. The use of Λ is more like a step for testing the statistical significance of the distance between group centroids than a final product providing a discrimination measure.

b) The second measures the correlation between groups and discriminant functions or the amount of discrimination each function carries. As the actual eigenvalues cannot be interpreted directly, they can be used to compare the relative magnitudes of each function to see how much of the total discriminating power each one has. There is another value that reveals the relationship between groups and discriminant function, called CCC (canonical correlation coefficient) (Bakouros, 1988), related to the eigenvalue by the formula (the closer the CCC value gets to 1, the stronger this relationship becomes):

$$R_i = \sqrt{\frac{\lambda_\iota}{1+\lambda_\iota}} \tag{5}$$

Classification

A common way to interpret DA results after developing a suitable discriminant function is through the classification matrix (the number of correct and incorrect classifications made by the relative rules based on the particular discriminant function). If N_{ij} denotes the number of units actually belonging in group i but classified in group j, then the classification matrix is defined to be a 2X2 matrix:

$$\begin{array}{ll|c|c|l} & & \text{Predicted} & \text{Group} & \\ & & 1 & 2 & \\ \cline{3-4} \text{Actual} & \text{Group 1} & N_{11} & N_{12} & N_{1j} \\ \cline{3-4} & \text{Group 2} & N_{21} & N_{22} & N_{2j} \\ \cline{3-4} & & N_{i1} & N_{i2} & N \end{array}$$

The elements of the main diagonal denote the number of correct classifications and the off-diagonal elements the number of incorrect classifications (errors). The above matrix can be the source for the following classification indicators:

- Effectiveness 1 (EF_1) $= N_{11}/N_{i1} * 100\%$
- Effectiveness 2 (EF_2) $= N_{22}/N_{i2} * 100\%$
- Effectiveness Total (Ef_t) $= (N_{11}+N_{22})/N * 100\%$

EF_1 and EF_2 give the discriminatory ability related to one of the two populations, while Ef_t indicates this ability over both populations (is the appropriate indicator when the goal is to maximize the total percentage of population correctly classified). In pipe networks where the misclassification cost is very high, it is more important to maximize EF_1 or $EF_{2.}$

The NS and the GoM pipe networks

Comparing methodologies

Failure records for the NS pipe network showed that the overall annual failure rate is 1,3 incidents/1000Km. Most of the pipes failed during their installation and most of the failures occurred in the open sea, revealing spatial clustering. Anchors and third party were the main causes of failure and oil pipelines were the most vulnerable. Failure records for the GoM pipe network showed that the overall annual failure rate is 0,67 incidents/1000Km. Almost all pipes failed during their operating period revealing also spatial clustering (near the platform). Corrosion was the main cause of failure and oil pipelines were the most vulnerable.

“Average failure rates” proved that failure probability does not increase with age (due to corrosion, erosion and fatigue). Additionally, pipe failure probability estimation was based on failure rates, which were calculated separately for parameters such as product, length or diameter. Therefore the intercorrelation among all these parameters was totally ignored. More sophisticated statistical analysis techniques (such as Correlation and Regression based techniques) must be applied in order all these parameters to be thoroughly examined. Correlation and Regression techniques are particularly useful in quantifying relationships between a single dependent variable and one or more independent variables. But in order to analyze pipe failure rates, all variables should be considered simultaneously. Therefore, Factor Analysis was used which is a multivariate statistical technique that addresses the interrelationships among a total set of observed variables. Factor Analysis gave the first clues that the use of statistical techniques, based on the intercorrelation of all the variables, can result in better understanding of how a pipe behaves. As Factor Analysis wasn’t able to provide a pipeline reliability prediction “model” based on all the pipe characteristics affecting the failure occurrence, the discriminant analysis was the next method to try.

Case study networks

Based on whether a pipe has faced at least one failure either before or after its operation, this pipe is considered as a success (did not fail) or a failure (did fail), forming two groups of pipes. The inventory file of the case study network in the NS consisted of 138 oil and gas pipes from which only 29 failed noting 59 failure incidents while in the GoM consisted of 133 oil and gas pipes from which 42 failed noting 54 failure incidents [Tsitsifli et al., 2006]. Inventory and incident files were then created for the NS and GoM pipe networks containing pipe code numbers, product in the pipe, length, diameter, events and causes of failures, failure dates etc. In Table 1 mean values, standard deviations, minimum and maximum values of the two case studies data are shown.

Table 1. Mean value, standard deviation, MIN and MAX values of the variables used.

Variable		Mean	Standard Deviation	MIN	MAX		Mean	Standard Deviation	MIN	MAX
Length (km)	**NS network**	50,430	65,585	1	452	**GoM network**	12,05	19,91	0,1	114
Diameter(inches)		22,365	8,807	3	36		8,00	4,57	2	30
Thickness(inches)		0,610	0,165	0,079	1					
Pressure(bar)		112,556	32,645	18	255		89,55	41,13	12	330
Grade(N/mm^2)		411,411	41,52	253	480					
Lifetime(months)		64,72	52.36	0	221		109,93	82,13	1	386

The selection of the discriminant variables

The best set of independent variables can be defined directly or in a stepwise way. According to the first, all independent variables enter the analysis concurrently, creating the discriminant functions, regardless of their discriminant power. In the stepwise method independent variables are selected according to their discriminant power. Sometimes the full set may contain some excess information about the group difference or some of the variables may not be useful in discriminating the groups. By selecting the "next-best" discriminator at each step, a reduced set of variables is reached which is better than the full set, due to the possible presence of unstable discriminators in it. As variables are selected for inclusion on an incremental basis, other previously selected variables may lose their discriminant power. This occurs, as the "carried" information regarding group differences is now available in some combination of the other variables included. Such variables are redundant and should be eliminated. In the present study the direct way is followed in the beginning, creating all possible combinations of the available variables and then based on the stepwise method 22 possible scenarios about the NS network and 11 scenarios about the GoM network are being analyzed, all of them having low Wilk's Λ values.

The importance of using joint variables in reliability prediction.

In DAC method the choice of the variables is too important since the characteristics of each unit carry its information. The present study introduced joint variables that proved to be very important and multilateral, carrying significant information for the units as they combined simple variables that alone did not constitute efficient discriminants (Stourm, 1997). Experience showed that a large (in terms of length) pipe with a small aging rate has a different attitude than a pipe of small length with the same aging rate. By combining the pipe's length and lifetime a new characteristic is created describing its attitude. The presence of a characteristic into joint variables may enforce or weaken the function's discriminant power, depending on the emphasis that is being given on its power or on the fact that the carried information is redundant. The fact that the joint variables can express the physical meaning of a phenomenon is too important as it affects the pipe's behaviour and reliability. Dimensionless joint variables prevent the analysis from depending on the dimensional differences of the characteristics. Finally, these variables have indirect relation with the meaning of various phenomena, which can take place in the study field. Table 2 presents both simple and joint variables for the NS and the GoM networks respectively, where the combinations of the variables forming the different scenarios for both networks are presented in tables 3 and 4.

Table 2. The "simple" and "joint" variables for NS & GoM networks used in the present study

Simple	Units	NS	GoM	Joint	Stands for	NS	GoM
Length-L	Km	x	x	DIM1	[Grade/0,1(Pressure)]	x	
Diameter-D	inches	x	x	DIM2	[100(Length)] / [2,54(Diameter)]	x	x
Wall thickness-TH	inches	x		DIM3	[1000(Thickness)/(Diameter)]	x	
Operating pressure-OP	bar	x	x	DIM4	$[100(\sigma_L)(\sigma_E)(0,1^2)/(\text{Grade})^2]$	x	
Grade-GR	N/mm^2	x		DIM5	$2{,}54\ \pi\ 10^{-5}$ (Diameter)(Length)(Lifetime)	x	x
Product-PR		x	x				
Lifetime-LT	Months	x	x				

Table 3. The 22 scenarios of the NS network

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
L	x	x	x	x	x	x	x	x	x	x	x	x		x		x						x
D	x	x	x	x	x	x	x	x	x	x	x	x			x		x					
TH	x	x	x	x	x	x	x	x	x	x	x	x		x				x			x	
OP	x	x	x	x	x	x	x	x	x	x	x	x			x				x			x
LT	x	x	x	x	x	x	x	x	x	x	x	x		x						x	x	x
GR	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
PR	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
DIM1	x		x						x	x	x	x	x		x	x	x	x	x	x	x	
DIM2	x			x				x		x	x	x	x	x		x	x	x	x	x	x	
DIM3	x				x			x	x		x	x	x		x	x	x	x	x	x		x
DIM4	x					x		x	x	x		x	x		x	x	x	x	x	x		
DIM5	x						x	x	x	x	x		x	x		x	x	x	x	x		

Table 4. Scenarios (11) of the GoM network

	1	2	3	4	5	6	7	8	9	10	11
L	x	x	x	x		x	x	x	x	x	
D	x	x	x	x	x		x	x	x		x
OP	x	x	x	x	x	x		x	x	x	
PR	x	x	x	x	x	x	x		x		x
LT	x	x	x	x	x	x	x	x		x	
DIM2	x		x		x	x	x	x	x		x
DIM5	x			x	x	x	x	x	x	x	

Results and Discussion

The NS network

For the NS network the correct classification percentages were satisfactory (71.2%-98.3% for failures; 63.3%-90.8% for successes; 67.3%-92.9% in total). Wilk's Λ values ranged form 0.38 and 0.83, while CCC values ranged form 0.409 to 0.788, revealing a good discrimination for most cases (Figure 1). Results revealed that pipe's lifetime is the most crucial characteristic (15/22 scenarios), while the less important one is the supplying fluid (product) (16/22 scenarios). Using the 'critical Z-score' the pipes can be pre-classified as failures or successes, without knowing whether they had failed in the past or not. Any pipe with Z-score greater (or less) than the respective 'critical Z-score' is expected to survive (or fail). The correct prediction probability ranged from 79.6% to 96.6% (failing group) and from 57.8% to 88.1% (surviving group) (Fig. 1). Sensitivity analysis of the data available took place, to check the stability of the results, proving that the importance of the characteristics and the correct classification proportions are "fixed". Scenario No.1 is the best one regarding Wilk's Λ, CCC, EF_f, while No.15 is the worst one regarding Wilk's Λ and CCC. Scenario No.16 is the worst one regarding EF_f . Considering EF_t the 'ranking' is different. Nevertheless it was considered more important to correctly classify the pipes expected to fail instead of the ones expected to survive, as to misclassify a pipe into group F when it actually belongs to group S has less economic and safety implications than to misclassify a pipe into group S when it actually belongs to group F. Coefficients U_i were used to calculate the Z-scores for the two groups of pipes, but as they belong to variables with different dimensions they are not comparable until they become standardized. If the value of the U_i coefficient of a variable is positive, e.g. lifetime (or negative, e.g. DIM4) then any increase of this variable increases (or decreases) its Z-score and thus the pipe's chance to 'survive' (Fig. 2).

The GoM network.

For the GoM network the correct classification percentages were not that good (51.9%-61.1% for failures; 76.9%-86.8% for successes; 70.3% to 75.2% in total). Wilk's Λ values ranged form 0.780 to 0.804, while CCC values ranged form 0.443 to 0.469, showing a low discrimination potential (Fig.1). DIM2 is the most crucial variable (7/11 scenarios), while operating pressure is the less important one (8/11 scenarios). ‘Critical Z-scores’ were also calculated for the GoM network. The correct prediction probability ranged from 51.85% to 70.37% (failing group) and from 52.74% to 69.23% (surviving group) (Fig.1). A sensitivity analysis similar to the one performed in the NS network also proved that the characteristics importance and correct classification proportions are ‘fixed’. Scenario No.1 is the best one regarding Wilk’s Λ, CCC, EF_f, while scenario No.11 is the worst one regarding Wilk’s Λ, CCC, EF_t. Scenario No.6 is the worst one regarding EF_f. Regarding the U_i for the GoM network, a negative non-standardized coefficient (Ui) means that any increase in the dimension of the pipe’s corresponding variable decreases its Z-score and the chance of its belonging to the “failed” group (e.g. pipe length, diameter, product, lifetime, DIM2 and DIM5) (Fig.2). This is because in the NS network the “survived” pipes’ Z-scores are bigger in value than the “failed” ones, while in the GoM network, the “survived” pipes’ Z-scores are smaller in value than the “failed” ones (Fig.3).

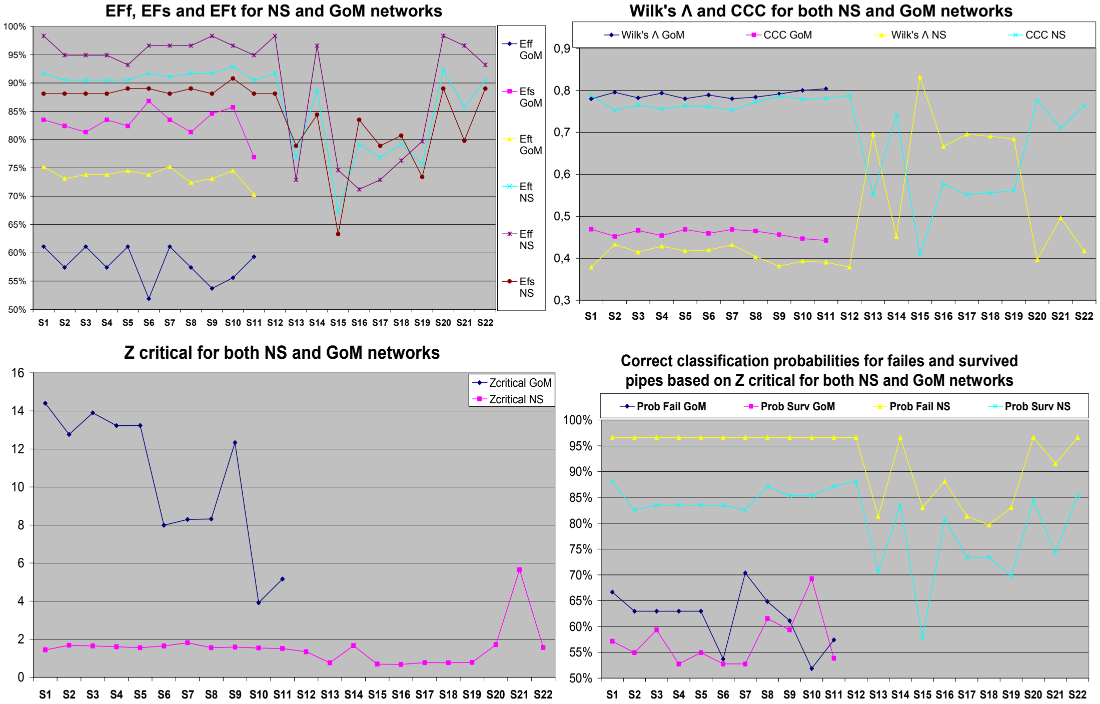

Figure 1. Λ, CCC, EF (failures, successes, total), Z-critical & correct classification probabilities

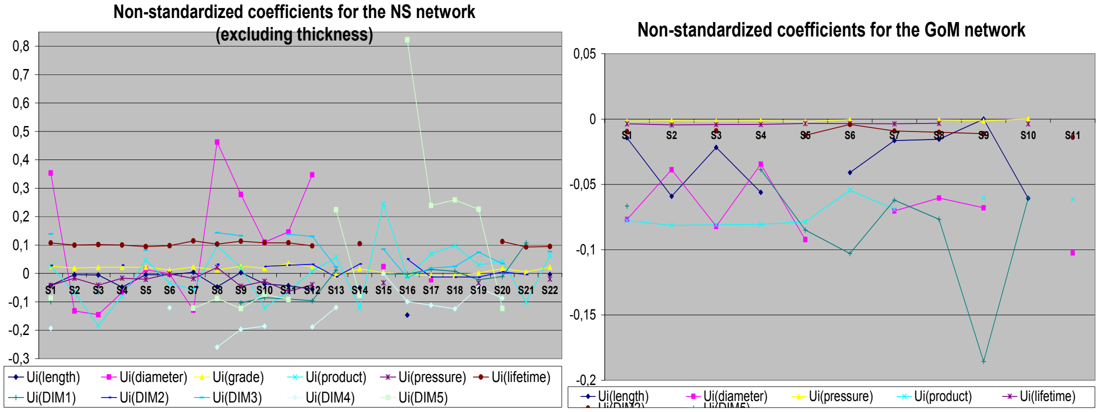

Figure 2. Non-standardized coefficients (Ui) for both NS (excluding thickness) and GoM networks

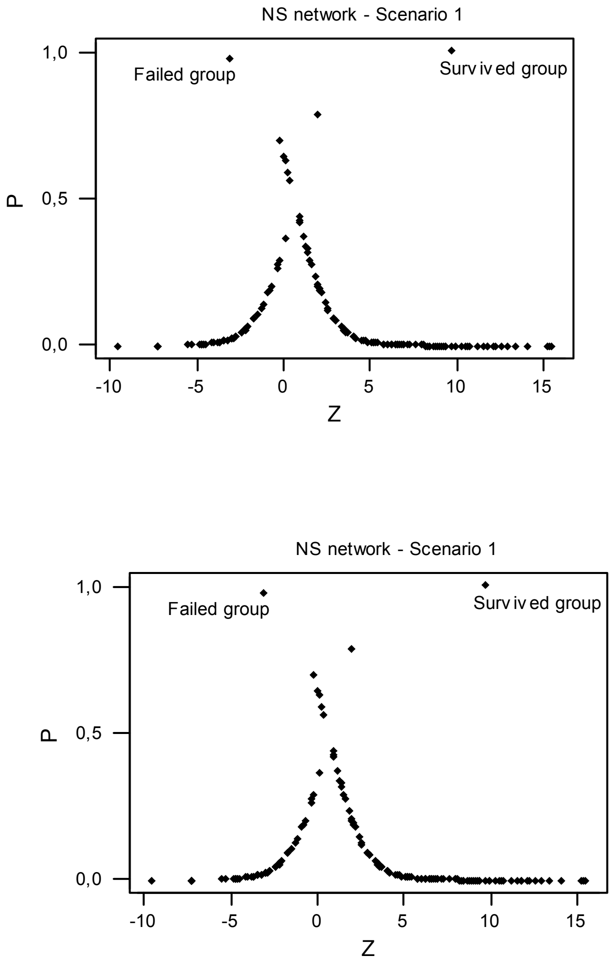

Figure 3. Z-scores for scenario no.1 for both networks

Water Pipe networks

Attempting a SWOT (Strengths/Weaknesses/Opportunities/Threats) Analysis of the DAC method (Figure 4), the water distribution network of Thessaloniki was taken as a case study (Kanakoudis & Tolikas, 2001). DAC method proved to be practicable and quick in predicting the reliability of pipe networks from its current application. DAC method is used in order to define the pipe characteristics affecting the most the behavior of the pipe and thus classify the pipes into “successes” or “failures”. It is also used to develop a linear model that when it will be used the pipes will be classified to “failed” ones or “survived” ones with high probabilities. The results taken from the DAC method’s application are pretty good and can be much better depending on the data available. In order the DAC method to be successful in predicting the network’s reliability, sufficient and reliable data records should be available. Such records exist for oil and gas pipe networks because the companies responsible for them well know the immediate compensative character of the “product” and have applied procedures of full detection and observation of the occurred failures. Regarding the water networks the present data and their up-to-now analysis has not given the expected results mainly due to the fact that only the last few years the actual “value” (regarding its environmental prospect), of the water being lost has been acknowledged. Data records sometimes may be unreliable due to the ineffective way of data collection (application of the available pipe breaks/leaks data records of the water pipe network of Thessaloniki) (Kanakoudis & Tolikas, 2001; US Army Corps of Eng., 1980).

Moreover the DAC method demands two groups of pipes, failed and survived ones. For water pipe networks there is no data for survived pipes. Therefore, the distinction has to be based either on water loss rate (surviving pipes are those experiencing leaks while failing ones are those facing breaks); or on water loss volume (surviving pipes are those experiencing breaks while failing ones are those facing leaks). The water-loss-volume based distinction is better since leaks are responsible for greater water losses in (5:1 compared to breaks). The analysis has been limited by classifying pipes in only two groups, based on whether a pipe has experienced a break or not. It thus failed to capture the evolution of failure patterns on individual mains, which could start from very few infrequent breaks and advance into a multiple failure stage, with frequent breaks. Thus, the break data records are not used to predict future failure incidents, based on the pipe's performance. No useful quantitative descriptions regarding the impact of pipe aging were possible to be made, and thus the predictive power of the derived models would be limited to longer time periods.

The research team is putting efforts: a) to develop the methodology in order to get more qualitative and reliable results based on the existing failure data records; b) to come up to "solid" proposals for the way that failure data records should be kept by Water Utilities and the information they should provide in order the DAC method to be successfully applied in water networks.

Figure 4: SWOT analysis of the DAC method

Conclusions

Regarding pipe reliability analysis, previous studies examined the average failure rates or used other statistical techniques (Bakouros, 1988), without being able to correlate the parameters or produce a "model" that estimates pipe reliability based on pipe characteristics. The study's results proved that the DAC method can be successfully used to predict the reliability of oil and gas pipe networks as it considers a large number of different and complex pipe characteristics in order to simultaneously study the differences of the two pipe groups (successes/failures), using the Z-score index. DAC method is able to examine pipe failure behavior aspects and produce a pipe reliability prediction model based on pipe characteristics (Bakouros, 1988). Z-scores resulted from a canonical discriminant function (a linear combination of discriminant variables). The introduction of joint variables, introduced other parameters that may have indirect impact on a pipe's reliability, increasing the analysis accuracy. Pipe lifetime was the characteristic that provided better discrimination in the NS network while the variable DIM2 provided better discrimination in the GoM network. For the NS network pipe lifetime had positive coefficients (as the specific variable's value increased, the failing probability decreased). For the GoM network pipe's length, diameter, product, and lifetime had negative coefficients (as the specific variable's value increased, the failing probability decreased). Therefore, for the GoM network long pipes seemed to be safer, as revealed by the failure rates results and large diameter pipes were safer than small diameter ones against the odds. This can be explained as larger diameter pipes have thicker walls compared to the small diameter ones. Although unexpected, for both networks the failing probability of a pipe could decrease with time as current modes of stress are lower in intensity. The discrimination accomplished in the NS network comparing to the GoM network was much better, due to the larger number of pipe characteristics available in the NS network in comparison with the GoM network. By increasing this number, the pipe reliability estimation model becomes better and better (Stourm, 1997) (in terms of discrimination and correct classification percentages). Regarding the effective use of DAC method for water pipe reliability assessment, following the first unsuccessful attempts, further study is being done at the time being, regarding a thorough SWOT analysis that will help the research team to focus on the strong points and eliminate the weak ones.

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