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Advances in Water Supply Management

Edited by

Čedo Maksimović, David Butler & Fayyaz Ali Memon

Department of Civil & Environmental Engineering

Imperial College London, United Kingdom

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Preface

This volume contains papers selected for presentation at the International Conference on Advances in Water Supply Management held at Imperial College London on 15–17th September 2003. The conference is the latest in the CCWI (Computers and Control in Water Industry) series that has been organised in rotation previously by the universities of Exeter, De Montfort and Brunel. These conferences were conceived as a forum for presentations of developments in and implementation of informatics and control technologies in the water industry. Originally focussed on the UK water scene, they have gradually developed into an important international event bringing together specialists from all over the world. We continue the tradition of welcoming papers from academic experts and practitioners.

This time the conference has been jointly organised by the Urban Water Research Group of the Department of Civil and Environmental Engineering at Imperial College London, Centre for Urban Water-UK (CUW-UK) and the WATERSAVE network on water recycling and conservation.

While deciding on the programme and detailed topics for this event, the organisers had in mind several important developments that have taken place in the years since the last CCWI conference held at De Montfort University in 2001. These include:

a. Raised awareness of the importance of urban water systems in general and water and sanitation in particular (in both the developed and developing world) as pointed out at both the Johannesburg and Kyoto summits.
b. Realisation that water consumption (demand) management is a vital component of any water resources strategy, especially related to reducing water losses during delivery and mitigating unnecessary burdens on the environment.
c. Increased willingness of research funding bodies around the world to fund well-argued projects in the field, even in competition with other worthy topics. Results from many of these projects are presented here.
d. Development of innovative diagnostic, modelling and management tools based on ever-developing informatics technologies.
e. Acceptance of the cross-cutting nature of computing and control to other facets of the water industry.
f. Importance of wider social and economic issues.

The papers are grouped into ten chapters covering the full spectrum of interest in the field. Chapter 1 highlights the importance of data management and information that can be extracted from asset databases, for example. In Chapter 2 we raise the important topic of leakage detection and the management of leakage throughout water distribution networks.

A good number of papers were received that can be classified as concerning analysis, design or rehabilitation of distribution networks and these are drawn together in Chapter 3. Chapter 4 includes papers with a particular emphasis on network optimisation, wheras Chapter 5 focusses on network operation. Of increasing importance is the issue of water quality management in distribution systems, which is covered in Chapter 6. We then move to new topics as compared to previous conferences. Chapter 7 highlights work done on the effectiveness of water conservation options and in Chapter 8, several studies in the area of water economics are discribed. Water consumption trends and demand forecasts are dealt with in Chapter 9 and the volume concluded with Chapter 10, where specific country experiences with demand management are laid down.

It is hoped that the papers presented will serve both the research and professional communities dealing with water supply systems, in particular within the context of the integrated urban water system. Prudent implementation of the innovative techniques presented here such as diagnostics, demand management, uncertainty analysis and transients models as well as more "traditional" ones such as optimisation and network analysis, should lead to more sustainable and cost-effective solutions.

We most kindly thank the members of the International Scientific Committee for their assistance in screening the abstracts and assessment of selected papers.

Čedo Maksimović
David Butler
Fayyaz Ali Memon

Chapter 1 – Data management

Statistical methods for water data analysis and processing

H.V. Lobanova
State Hydrological Institute, St.Petersburg, Russia

ABSTRACT: Water supply in many cases deals with river runoff, which has a stochastic nature. In modern changing conditions homogeneous and stationarity of river runoff is broken by a human influence in a form of direct impact on river channels and watersheds and in a form of anthropogenic climate change. By this reason classical statistical tools and models have to extend for new conditions. Inhomogeneous – nonstationary model has been developed as a basis for a description of river runoff. For practical realization of this general model some methods have been developed such as: methods for extraction of runoff components connected with anthropogenic impacts and long-term modern climate change, statistical tests for outliers and stationarity of parameters of a probable distribution function extended on peculiarities of hydrological data (autocorrelation and nonsymmetric), an assessment of empirical probability for non-regular data, an assessment of random errors of quantiles and other. Applications of developed methods and models are given in the some examples for different hydrological characteristics in different climate conditions.

1 INTRODUCTION

Water supply is formed from the following main sources: river runoff, underground runoff and water from lakes, ponds, etc. Design of intakes for water supply as well as other engineering structures and their management depends on the natural fluctuations of hydrological characteristics: low flow, floods, water resources (annual and monthly runoff). In period of many decades of investigations and analysis of observed data it has been established that hydrological characteristics have a stochastic nature of long-term fluctuations (SNIP, 1984). Therefore a probable distribution function (PDF) is the main model of such process (Kendall, Stuart, 1970). Today there are two main groups of factors which give a deterministic sense in natural stochastic changes. They are: modern climate change and anthropogenic influence on watersheds and in river channels. Classical statistical methods are based on the distribution function theory and they are suitable for homogeneous and stationary conditions. The application of this approach in changing conditions is limited by two main reasons:

- inhomogeneity of time series of hydrological characteristics (annual, monthly and extreme runoff) connected with extraordinary outlying observations, different factors of anthropogenic influence and climatic processes of different time scales, including long-term modern climate change;
- non-stationarity of parameters of time series of hydrological characteristics connected with non-stationarity of anthropogenic factors and climate change.

The third main peculiarity, which is suitable for flood time series in general, is their non-regularity of appearance, that varies from several flood events during a year to some events during a century.

As a result, observed time series of hydrological characteristics are a complex process or a composition of different components: some of them are stochastic (natural fluctuations) and some are deterministic-stochastic (components of man's impacts and anthropogenic climate change). Therefore the first step is an extraction of the homogeneous (or quasi-homogeneous) components from the complex process and after that different strategies of processing and simulation are applied.

2 MODEL AND METHODOLOGY

General model of inhomogeneous and non-stationary process can be suggested as a super-position or a sum of the quasi-homogeneous components (Lobanov, 1998). Main equation of such process detected of the inhomogeneous part is as follows for hydrological characteristics:

$$Y_{com} = Y_{int.} + Y2_{dec.} + Y_{cl.} + \Sigma Y_{i\,ant.} \tag{1}$$

where: Y_{com} is an observed hydrological characteristic as a non-homogeneous composition process, $Y1_{int.}$, $Y2_{dec.}$ are "natural" homogeneous components of inter-annual and decadal time scales (CLIVAR, 1995); $Y_{cl.}$ is joint component connected with "natural" centural and anthropogenic climate change, $\Sigma Y_{i\,ant.}$ is a sum of homogeneous components connected with different factors of direct man's activity (hydropower regulation, irrigation, etc.).

Joint climate change component can be divided into two parts: natural ($Y_{cl.nat}$) and anthropogenic ($Y_{cl.ant}$).

$$Y_{cl.} = Y_{cl.\,nat} + Y_{cl.\,ant} \tag{2}$$

For this aim physical models can be applied as well as palaeo reconstructions of some thousand years for a comparison of data in two different periods: today and before industrial time for a deviation of modern increasing into two parts: "natural" and anthropogenic.

Part of non-stationarity can be expressed in terms of non-stationarity of the basic parameters of time series: mathematical expectation (m) and variance (D) for some homogeneous components, such as components connected with anthropogenic factors:

$$\boxed{\begin{array}{l} m_{Y1\,ant..}(t) \neq const, \\ D_{Y1\,ant.}(t) \neq const, \\ m_{Y2\,ant.}(t) \neq const, \\ D_{Y2\,ant.}(t) \neq const, \\ \ldots\ldots\ldots\ldots\ldots \\ m_{Ym\,ant.}(t) \neq const, \\ D_{Ym\,ant.}(t) \neq const, \\ m_{Y\,cl}(t) \neq const, \\ D_{Y\,cl}(t) \neq const \end{array}} \tag{3}$$

Equations (1)–(3) can be applied for a description of each year statistical events. For description of dynamic properties other characteristic need to use.

The next problem is the description of dynamic properties of every homogeneous component. Today the autocorrelation coefficient (or autocorrelation function) is used as usual for presentation of dynamics of time series structure. In changing conditions it is not enough and direct characteristics of cycles could be suggested for a description, such as: coefficients of cycle function or some general cyclic characteristics, for example: period (T) and amplitude (A) of cycle, duration of rising (T_{up}) and reducing (T_d) of cycle, mean speed of rising (V_{up}) and reducing (V_d) and cycle volume (W). Under dynamic point of view the equation (3) can be re-wrote for each cyclic characteristic of each homogeneous component.

According to a structure of general model the suggested methodology for simulation and determination of design runoff values includes the realisation of the following positions:

- assessment of extraordinary outlying observations which are proper to a non-symmetric distribution and autocorrelating time series of hydrological characteristics;
- extraction of runoff components connected with anthropogenic impact and restoration of "natural" runoff time series for different input informational conditions;

– extraction of homogeneous different time scale components in "natural" conditions including a long-term component of modern climate change;
– determination of the model's kind for each extracted homogeneous component (stochastic or deterministic-stochastic kind of the model);
– determination of the parameters of stochastic models and their quantiles;
– determination of the parameters of deterministic-stochastic models and their values for future period of water project's operation on the basis of regular properties, scenarios and expert analysis;
– determination of design hydrological characteristics in changing conditions as a sum of all extrapolated values obtained on the basis of stochastic and deterministic-stochastic models with their random errors:

$$Y_p=\Sigma(Y_{p\,nat.} +\Delta Y_{p\,nat.})+ \Sigma(Y_{ant.} + \Delta Y_{ant.}) + (Y_{clim.} + \Delta Y_{clim.}) \tag{4}$$

where Y_p is a design hydrological value of *p-th* probability in changing conditions, $\Sigma Y_{p\,nat}$ is a sum of design characteristics for "natural" components with a stochastic kind of model, $\Sigma Y_{ant.}$ is a sum of extrapolated components with deterministic-stochastic kind of model connected with factors of man's activity, $Y_{clim.}$ is an extrapolated component connected with the joint effect of centural climate change ("natural" + anthropogenic) and $\Delta Y_{p\,nat}$, ΔY_{ant}, ΔY_{clim} are random errors of computed values;

– monitoring and re-computation of the design hydrological characteristics with receiving of new information.

3 METHODS (TOOLS)

3.1 *Criteria for outlying data*

Empirical distributions of hydrological characteristics, especially extreme events (floods and droughts) often include one or some more extreme events, which are far away from the main group of data – outliers. There are two main reasons of outliers: their empirical probability is more rare than it was obtained for short time series and outliers connect with big errors of observations. There are many recommendations for identification and treatment of outliers (Ferguson, 1961; Barrett and Lewis, 1984; FEH, 1999) The statistical tests are used for an assessment of statistical significance of such outlying data. The most popular are Dixon's and Smirnov-Grubbs criterions. Main peculiarities of these and many other criteria are a normal distribution and a random behavior of the sample, which is used for the testing. From other side, the time series of hydrological characteristics have a non-symmetric distribution and sometimes – a significant auto-correlation. Therefore the classical tests Dixon's and Smirnov-Grubbs have been extended for the sample with auto-correlation and non-symmetric distribution by Monte-Carlo stochastic modeling. The results are given for example in: Lobanov & Lobanova (1983), Lobanov (1983). The main conclusions of this research are: a skewness increases the statistics of the criteria for maximums and reduces them for minimums of empirical distributions and an auto-correlation have less impact and increasing of the sample size.

3.2 *Extraction of man's impact components*

The main factor of man's influence on maximum and minimum runoff is a seasonal re-distribution of runoff by reservoirs. It leads to a reduction of maximum flood discharges and increasing of low flow. Other man's influence factors, such as water intake and outtake, wood cutting, etc are not significant for maximum discharges and impact on low flow as usual. The common man's impact component (for example of dam regulation) could be obtained as a difference:

$$\Delta R = R_{real} - R_{nat}, \tag{5}$$

where ΔR is the quantitative assessment of dam influence; R_{nat} is the runoff in dam site in natural conditions; R_{real} is the runoff in dam site in real conditions of operation.

For the synthesis of natural runoff in dam site several methods could be suggested:

– the method of inflow and water balance of reservoir:

$$R_{nat.} = R_{real} - (P - E - \Delta R_1 - \Delta R_2 + \Delta R_3 \pm \ldots) \tag{6}$$

where P is precipitation onto the surface of reservoir, E is evaporation from free water surface, ΔR_1, ΔR_2, ΔR_3, … – are runoff changes accordingly by the filtration to shores of the reservoir, by the filtration to the bottom of the reservoir, additional underground inflow to the reservoir and other factors;

– the relationships between the runoff in dam site during the natural period and its analogues to do temporary interpolation of these relationships during the hydropower station operation;
– the relationships between the runoff in dam site and in downstream sites, which are under the natural conditions, but only if link degree in the equations characterises the natural runoff;
– the using of the runoff formation models computed for the natural period of runoff formation. These models can be used for the runoff synthesis during the hydropower station operation period under the condition, that the runoff factors fluctuation regime has been remained as a natural regime;
– difference methods, which determine the natural runoff through a difference between the upper stream site runoff and the dam site runoff in common during the natural period and the station operation period;
– the determination of reservoir influence (effect) using the analysis of common time-series characteristics (mean value, variance, etc.) during two periods, they are: periods before and after the hydropower station construction.

The using one or another of six suggested methods is determined by such parameters as the availability of necessary information or a necessary computation accuracy. Application of the suggested methods for extraction and analysis of different hydrological characteristics (floods, low flow, annual runoff) connected with large dam's regulation for Russian rivers is given in the work (Lobanova, 1999).

3.3 *Extraction of climate change component*

There are two ways for determination and extraction of a component connected with modern climate change: use of climate scenarios (Jones, 1999) and extraction of long-term climate change component from observed flood time series with its future extrapolation in the nearest future (1–2 decades). The first way has a great uncertainty connected with different results obtained by different scenarios and is applied for average (annual, seasonal) meteorological characteristics at least. Therefore the empiric-statistic way has been used together with the scenario assessment. For extraction of long-term climate change components three groups of statistical methods have been used:

– consecutive averaging with periods 10–12 years, that is as a filter of high and middle frequencies of time scale of climate variability;
– method of smoothing of amplitudes of cycles (Lobanov & Lobanova, 1999);
– truncation method of decomposition (Lobanov, 1995).

Two main characteristics of extracted climate change component are obtained: statistical significance (as contribution in the common time series variation) and direction of the tendency in the nearest period. The results obtained by different methods are compared and combined under the same conclusion about significance and direction of climate tendency. Extrapolation of climate component in the nearest period is based on the autoregression model.

3.4 *Assessment of empirical frequency of non-regular flood events*

Well-known formula $i/(n+1)$ is suitable for regular random events, where i – rank, n – common number of events. Flood or droughts events can be several times in a year and extremes of extremes – several times in century. For calculation of empirical probability (frequency) of distribution of such non-regular events a new formula has been developed:

$$\boxed{P_i(X_i) = \frac{1}{n+1}[1 + \frac{n-1}{n-T_m}(n - \sum_{j=1}^{m-i+1} T_j)]} \tag{7}$$

when $X_1\ (T_1) > X_2\ (T_2) > \cdots > X_m\ (T_m)$ and where $T_1,\ T_2, \ldots$ is a duration of hydrological events or time intervals between them, m is a common number of random events in the time series, i is a rank number, n is a size of the sample.

3.5 *Assessment of design values on the basis of stochastic model*

There are some problems connected with determination of a design value with non-exceeding probability (or return period). The first of them is an effective fitting of empirical distribution of hydrological characteristic to chosen analytic approximation (Pirson III, Kritsky-Menkel distribution and other). The most adequate criterion includes two kinds of errors: standard error of fitting for whole distribution ($\sigma\varepsilon$) and maximum error in the zone of small or high probabilities which is used for extrapolation ($max\ \varepsilon$). This way the joint criterion will be:

$$\sigma\varepsilon + max\ \varepsilon = B\varepsilon \rightarrow \min \tag{8}$$

The second problem is connected with setting of designed probability. This probability or frequency is linked with the time interval, that includes the period of observation (T_{ob}) and period of future operation of water project (T_{op}). Therefore the designed frequency (P_d) has to include both of two these periods and the formula of the designed probability will be as follows (for case of floods or upper part of distribution):

$$P_d = [1/(T_{ob} + T_{op})] * 100\% \tag{9}$$

The last problem is connected with determination of confidence interval of design hydrological characteristics or random errors of quantiles. Today the theoretical approach is used but two other empirical methods could be suggested too:

- period of observation divides into two parts: hypothetic period of observation (T'_{ob}) and hypothetic period of operation (T'_{op}) and their ratio has to be equal to the ratio of the real observation and operation periods;
- observation periods of the longest time series or palaeo-reconstruction over the world are divided into two parts: observation sample ($T'_{ob} = T_{ob}$) and operation sample ($T'_{op} = T_{op}$) for testing and estimation of quantile errors that are taken the same for the future too.

3.6 *Monitoring and re-computation of design hydrological characteristics*

Main concept in modern changing conditions that the design flood is not the constant value. There are the following general reasons of such non-stability and necessity for re-computation of design floods for existing water projects:

- short size of observed data in the past and as a result – big random errors of design values;
- development of new effective methods of computations;
- modern changing conditions of direct man's impact and modern climate change.

The general scheme of re-computation includes: determination of "natural" runoff without impact of man's activity, restoration of long-term "natural" time series, assessment of modern climate change, computation of design flood and comparison with the design value which has been obtained before and as the last step – development of recommendations for change of operation rules including possible reconstruction of existing water project.

4 CASE STUDY

4.1 *Assessment of outlying data by classical and extended criteria*

Two examples of assessment of outliers in empirical distributions of floods have been chosen. Example 1 of application of extended criteria in a comparison with the classical ones is given for the longest records of annual maximum runoff in UK (25 time series). As a result, statistically significant outliers have been obtained in 13 records when classical criteria are used with autocorrelation coefficient $r(1) = 0$ and coefficient of skewness $C_s = 0$ and for 7 time series only when the extended tests have been applied with $r(1)$ and C_s obtained on the basis of long-term records. Example 2 deals with time series of maximum discharges of rainfall floods and depth of flood runoff for 80 gauged sites in a region of the Carpathian Mountains (Ukraine). When classical criteria have been used an inhomogeneity of empirical distributions connected with outliers has been obtained for 37 time series of maximum discharges and 40 time series of depth of floods. Time series of observations in this regions have different period and often it does not exceed 50 years. Therefore for an increasing of accuracy of sample parameters $r(1)$ and C_s their generalization over the area has been fulfilled. In the result average value of $r(1) = 0$ for maximum discharges as well as for depth of floods and regional value of $C_s = 1.74$ for maximum discharges and $C_s = 1.48$ for depth of floods have been obtained. Using these values and extended criteria it was established that a hypothesis of homogeneity is declined for 4 sites of observations only under given level of significance in 5%. Analysis of outliers for these sites showed that all inhomogeneous maximums took place in 1980 and periods of observations for these sites less than 50 years. Therefore it is possible to conclude that the empirical probability of this flood is less that it can be obtained on the basis of such sizes of records. This way, the classical criteria give outliers more often, than they can be for non-symmetric distribution which used practically always for maximum floods. Also it is interesting to mark if $r(1)$ and C_s have been obtained on the basis of each individual record the hypothesis of homogeneous is accepted in all cases but it is not correct, because outliers increase the skewness considerably.

4.2 *Extraction and analysis of the components connected with human activity*

The largest reservoirs of Russia have been chosen for extraction of component of operation and assessment of sustainable management of floods, low flow and water resources. In Russia the intensive reservoirs construction has begun in 1930s on the European part and in 1950s in the territory of Siberia. Therefore now the experience of some decades has been accumulated of the conditions of hydropower stations operation and the experience of their real influence on the natural river runoff regime. Two main problems are decided:

- to give the quantitative assessment of hydropower station influence on the natural fluctuations in dam site by the experience of station operation;
- to determine the reservoir effect propagation in the downstream.

Methods dedicated in 3.2 this paper have been applied for the restoration of "natural" runoff and extracted of runoff component connected with operation. The common results for different reservoirs are as follows:

- the given studies and estimations have shown for hydroelectric power stations in the territory of Russia that depending of the kind of hydrological characteristic and the conditions of hydropower station operation their influence varies from 5% to 70%;

- as a rule, less changes take place in the annual runoff connecting with the runoff reduction by the evaporation from the reservoir surface, the infiltration to shores and the bottom of the reservoir, etc. and they are equal from 8–9% in arid zone and to 4–5% in a wet zone;
- the maximum runoff and low flow are mostly subject of changes and magnitude of this reducing for floods and increasing for low flow depends first of all on the reservoir purpose (power engineering, water-supply, flood regulation, etc.) and is equal from 30% to 70%;
- a propagation of reservoir control influence to the downstream from dam site is depending first of all of the correlation between water releases through the dam and a place and a volume of side inflows in downstream;
- the analysis of conditions of control has shown that the maximum water discharges reduction depends in the most cases on the natural inflow to the reservoir and their relationship has the correlation coefficients on average 0.6–0.8 that is necessary to take into account in water projects of dam to downstream in the zone of reservoir influence;
- the reservoir filling effect has length in time from 1 to 6 years, after that the value of anthropogenic component dispersion is sharply decreasing and became practically constant, which characterises the stable work and sustainable management of floods, low flow and water resources during period of hydropower operation for the most reservoir in Russia.

Example of assessment of impact of regulation for 4 reservoirs is given in Table 1.

4.3 *Extraction and analysis of long-term climate change*

Statistical methods given in 3.3 have been applied for an extraction of long-term climate change component in time series of floods in two regions: UK area and in a region of the Carpathian Mountains. For Central England 14 records with long-term time series have been chosen and contributions of climate trends are given in Table 2, where "10 yr." is a method of consecutive 10-years averaging, "Ampl." is a methods of smoothing of amplitudes of cycles and "Trunc." is a truncation method, A is a watershed area in sq. km and n is a period of observation in years.

As it is seen from the Table 2, contributions of climate changes are different in some cases when different methods of extraction have been used. The most similar results take place for the most long-term time series. For these time series the climate change leads to a decreasing of maximum runoff with the exception the Trent River – site Trent Bridge, but this time series has the end of observations in 1969. As a result, the assessments of climate changes are reliable for records of observations more than 70 years. For less size of records huge interannual and decadal fluctuations do not allow to detect centural tendencies.

In the second example time series of maximum flood discharges and depth of flood runoff have been restored for a long-term period in the Carpathian region and an average period of

Table 1. Results of runoff regulation by main reservoirs of Russia in different regions.

Impact on runoff		Name of reservoir			
Kind of runoff	Kind of impact	Tsymlian-skoje	Krasnoyar-skoje	Novosibir-skoje	Volgograd-skoje
Summer low flow	change in dam site (%)	+24	+15	+40	+10
	distance of impact (km)	>500	200	200	50
Winter low flow	change in dam site (%)	+10	+300	+30	+110
	distance of impact (km)	170	>1000	150	>300
Maximum runoff	change in dam site (%)	−65	−57	−10	−15
	distance of impact (km)	>500	920	100	>300
Annual runoff	change in dam site (%)	−20	−5	<5	<5
	distance of impact (km)	>500	40	–	–

Table 2. Contributions of long-term climate change component in time series of annual maximum runoff in Central England region.

No.	River-Site	A sq. km	n, year	Contribution, % by method 10 yr.	Ampl.	Trunc.
1	Don – Doncaster	1256	110	−12.4	−8.4	−9.5
2	Burbage Brook – Burbage	9.1	56	−2.5	11.2	−0.1
3	Trent – Trent Bridge	7490	82	6.6	8.3	11.0
4	Willow Brook – Fotheringhay	89.6	53	2.2	1.2	−0.2
5	Harpers Brook – Old Middle Bridge	74.3	50	51.9	31.7	0.2
6	Ise Brook – Harrowden Old Mill	194	50	−4.7	−17.9	−1.1
7	Nene/Kislingbury – Dodford	223	53	4.4	5.3	3.5
8	Nene Brampton – st. Andrews	232.8	53	20.1	11.9	1.5
9	Nene/Kislingbury – Dodford	107.0	47	2.5	−15.6	2.4
10	Ash – Mardock	78.7	53	−0.8	−5.3	0.3
11	Thames – Kingston	9948	112	−6.7	−9.1	−12.3
12	Thames – Days Weir	3444.7	57	−1.3	−6.8	−10.8
13	Severn – Bewdley	4325	71	−4.5	−7.1	−6.5
14	Avon – Evesham	2210	54	−0.4	−0.5	6.7

observation became 70–75 years. Long-term component of climate changes for each time series has been extracted by 3 methods too and the results have been generalized. Number of statistical significant climate tendencies were 34 cases (or 42%) for maximum discharges and 27 cases (or 34%) for depth of flood runoff. Among them negative tendencies take place for 22 time series of discharges and for 22 records of depth of runoff. Positive climate tendencies are not big and observed in different parts of the region. Negative tendencies are joint in the one Eastern High Carpathian homogeneous region. Contributions of climate tendencies in common variation of observations are changed from 10–12% up to 30–35%.

4.4 *Assessment of empirical probability for non-regular flood events*

In this example on the basis of the general formula (7) the hydrological computations for POT (Peak Over Threshold) in chosen 15 time series in UK with the longest records have been fulfilled. For comparison the results of computations of annual maximums for the same sites are given too. Difference in the errors of fitting for AM (Annual Maximums) and POT is not significant, but parameters of empirical distributions are differ. The relationships between parameters are represented by the following empirical equations:

$$POT(m)=0.796AM(m)-8.478, \quad \mathrm{R}=0.991 \tag{10}$$

$$POT(Cs/Cv)=0.993AM(Cs/Cv)+3.907, \mathrm{R}=0.875 \tag{11}$$

where m, Cs/Cv – are mean and ratio of skewness and variation coefficients for empirical distributions of POT and AM, R is a correlation coefficient.

As it follows from (10)–(11), the mean of POT-distributions is equal 0.8 from a mean of AM-distributions, variation coefficients practically the same and the ratio Cs/Cv on 9.91 more for POT empirical distributions than for AM-distributions. As a result, differences in quantiles of two kind of distributions have positive as well as negative signs and are not significant in some cases.

4.5 *Assessment of the random errors of design values*

In this example palaeo-hydrological time series of annual runoff for the Dnieper River – site Lotsmanskaya Kamenka (4136 years) is used, which has been reconstructed by G.Shvetz (1967).

Table 3. Standard random errors of parameters and quantiles (design values) obtained by palaeodata.

Parameters

			Average		Cv		Cs/Cv	
n_1	n_2	ns	ε, m^3/s	ε, %	ε, m^3/s	ε, %	ε, m^3/s	ε, %
100	100	40	827	15.5	0.094	17.4	1.76	41.8
	200	39	930	17.2	0.094	17.4	1.62	39.5
50	100	80	842	16.7	0.128	23.1	1.54	35.3
	200	78	859	16.4	0.127	22.0	1.35	29.9

Quantiles of P% probability of non-exceeding

			P = 0.5%		P = 1%		P = 10%	
n_1	n_2	ns	ε, m^3/s	ε, %	ε, m^3/s	ε, %	ε, m^3/s	ε, %
100	100	40	4480	27.9	3610	25.3	1590	18.4
	200	39	4470	25.9	3410	23.0	1570	17.4
50	100	80	4990	30.1	3940	27.3	1640	19.8
	200	78	4980	29.6	3980	26.9	1700	19.6

The relationship has been obtained between maximum and annual runoff with a correlation coefficient R = 0.83. On the basis of this time series as a quazi-general totality, the standard random errors of parameters and quantiles have been obtained for samples with "present" periods 50 and 100 years and for the periods of extrapolation in 100 and 200 years which correspond to period of operation of water projects in the future. These errors have been calculated as differences between parameters and quantiles in consecutive samples and characterise the main idea of modern computations: transfer today conditions to the future. The results are given in the Table 3, where n_1 is a size of "present" time series, n_2 is a size of "future" time series (period of extrapolation), ns is a number of samples from "general totality", ε are random errors (in m^3/s and in %).

As it is seen in Table 3, standard errors in quantiles 0.5%, 1% and 10% were 26–30%, 23–27% and 19–18% accordingly. Existing theoretical assessment of standard random errors obtained by stochastic modelling give their values on 30–40% less than the palaeo information.

4.6 *Re-computation of design hydrological characteristics*

Two examples of re-computations of design hydrological characteristics in changing conditions have been chosen. In the first example, the application of the offered approach of re-computation has been realised for the cascade of the Dnieper hydropower stations, which has more than 50 years period of work. It is established that the recomputed design floods were on 30–50% above than computed earlier for water projects, that causes a necessity of reconsideration of the operation rules of dams' regulation and in some cases – reconstruction of dams. Increasing of design maximum discharges connects, in general, with increasing of ratio of skewness coefficient (Cs) to coefficient of variation (Cv). In past, when short-term time series took place this ratio had been established as 2, in case of re-computation $Cs/Cv = 3.5$.

The second example connects with the well-known catastrophic flood in Lensk city – the Lena River (Eastern part of Russia) in 2001, when 18 thou. of inhabitants became homeless and were about 40 thou. victims. Lensk city was flooded in whole and destroyed during this inundation and today problem connects with a restoration of the city and with a project of flood safety dam. Main discussion takes place about a design maximum water level with return period of 100 years.

Former design maximum level was 170,07 m without flood 2001. For re-computation of design maximum the following methods have been developed and applied:

– using of historical maximums inside (2001) and outside (1878) of time series of observations;
– application of a composition of free water levels and ice jam events for a development of joint frequency curve of maximum water levels by PC stochastic simulation (Monte-Carlo approach);
– combined analysis of meteorological and hydrological long-term records of characteristics of snowmelt floods in Eastern Siberia area for an assessment of modern climate change impact as well as using of scenarios of climate change for this region.

In the result it has been obtained that existing design level in 170.07 m is not suitable for a height of flood safety dam, because it is very low and more realistic design water level taking into account observed catastrophic maximum 2001 and historical level 1878 is 171.87 m that is about 2 m higher than the existing one.

5 CONCLUSION

Methodology and system of methods are suggested for water data analysis, processing and computations in modern changing conditions. Application of methods has been given in some examples connected with assessment of outlying data by classical and extended criteria, extraction and analysis of the components connected with human activity, extraction and analysis of long-term climate change, assessment of empirical probability for non-regular flood events and random errors of design values as well as re-computation of design values.

REFERENCES

Barnett, V. & Lewis, T. 1984. *Outliers in statistical data.* Second edition. John Wiley.

CLIVAR 1995. A study of climate variability and predictability. Science Plan, WMO/TD, No. 690.

Ferguson, T.S. 1961. On the rejection of outliers. *Proceedings of the 4th* Berkely Symp. on Mathematical Statistics and Probability, 1, pp.253–287.

FEH (*Flood Estimation Handbook*). 1999. Institute of Hydrology, UK.

Jones, J.A.A. 1999. Climate change and sustainable water resources: placing the threat of global warming in perspective. Special Issue. *Hydrological Sciences Journal*, 44 (4), 541–557.

Kendall M.G. & Stuart, A. 1970. *The advanced theory of statistics*. Vol. 2 Inference and relationship, Charles Griffin & Company limited, London.

Lobanov, V.A. 1983. Assessment of efficiency of the statistical tests for outlying data in the case of the Pirson III distribution. *Ref. Journal Geophysics,* No. 8, pp.130–142.

Lobanov, V.A. & Lobanova, H.V. 1983. Adaptation of Smirnov-Grubbs test for non-symmetric and autocorrelative consequences. *Ref. Journal Geophysics,* No.8, pp. 14–28.

Lobanov, V.A. 1995. Statistical Decision in Changing Natural Conditions. *Proc. Int.Conf. on Statistical and Bayesian Methods in Hydrological Sciences*, Paris.

Lobanov, V.A. 1998. Modelling of dynamic properties of hydrological processes. In: *Hydrology in a Changing Environment* (ed. by H.S.Wheater & C.Kirby), Vol. 1, 153–162. John Wiley/British Hydrological Society, Chichester, UK.

Lobanov, V.A. & Lobanova, H.V. 1999. Trends in Cold Climate Characteristics in *Urban drainage in specific climates. Cold climate*. UNESCO, pp.171–199.

1 Lobanova, H.V. 1999. Assessment of Sustainable Floods and Water Resources Management by reservoirs in Russia. *Proc. Int. Conf. Lakes99*, Copenhagen, Denmark.

SNIP 2.01-14-83. 1984. *Norm and Rules in Construction*, State Standard. Moscow, Ministry of Construction of Russia.

Shvets, G.I. 1967. Restoration of annual runoff of the Dnieper River for the historical time. In book, *Long-term runoff fluctuations and stochastic methods of their computations.* Moscow State University.

Auto-validation of district meter data

D. Burnell
Management Science Department, Lancaster University

ABSTRACT: Metered Districts in urban areas generate huge volumes of data which require automatic screening. Districts with legacy networks can accidentally be breached, distorting the data's integrity. This paper outlines a way of harnessing 15-minute data to pick out odd-shaped days and to classify "jumps" in this data. The method splits demand into final usage and leakage by comparing each day's observed diurnal pattern with that expected. Usage and leakage trends are then monitored for changes in level, using forward and backward exponential smoothing. Bursts and repairs will affect leakage but not final usage, whereas a breach will in effect alter District property count and hence usage. The method allows jumps not only to be spotted but also classified. The algorithm is illustrated for several urban District usage data sets.

1 INTRODUCTION

1.1 *District metering*

In the last 15 years the UK water industry has invested heavily in "District Metering" to help control leakage in water distribution networks. By closing off the relevant boundary valves, the network is partitioned into distinct "Districts" of at most a few thousand properties. Flows in and out of the district are metered continuously, and reported typically every 15 minutes. Data is collected via telemetry into corporate systems and used to help prioritise network maintenance and renewal activity. District Metering is seen by the UK Water Industry Regulator as a key tool in leakage control.

In a large metropolitan area, with a legacy distribution network and many interconnections, it is hard to maintain complete district integrity. The network is subject to ongoing maintenance, some of which requires valve operations to isolate sections of main. Large commercial customers which straddle districts may vary their mix of supply. Low-pressure complaints may prompt urgent re-valving. A single valve operation (even inside the District if it connects to a through trunk main), may be enough to lose hydraulic integrity. The flows are as measured, but the district supplied is now different. The net effect is to re-define the District on the ground and so distort the *meaning* of the flows measured.

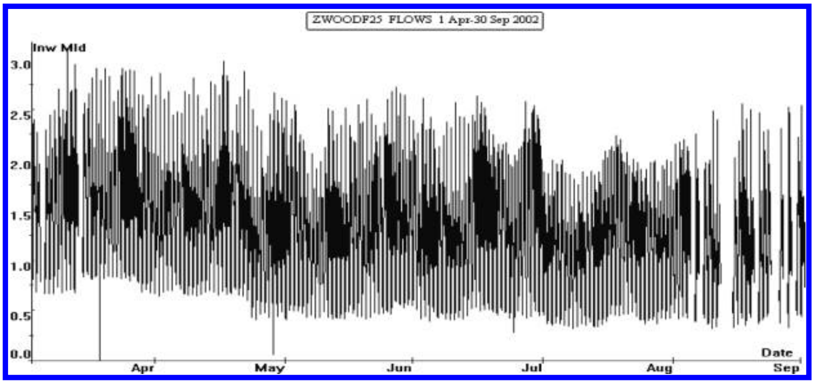
1.2 *Analytical challenge*

Network managers will aim to minimise these disturbances and track the field activity which causes them. A desirable supporting activity is *to develop analytical ways of spotting and classifying changes in districts from the flow data itself*. This paper, which is part of long-term research to fully exploit District Meter data, explores the problem from a statistical perspective; proposes one way by which it can be tackled by using diurnal patterns; and illustrates how the algorithm performs on a sample of urban metered districts.

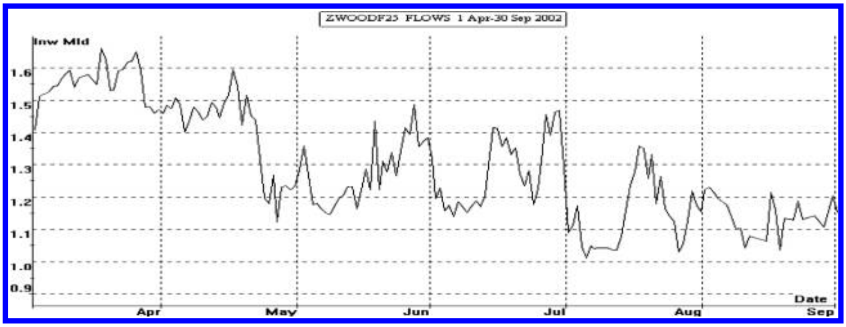
2 STATISTICAL PERSPECTIVE ON "DATA JUMPS"

2.1 *Analysing non-stationary timeseries*

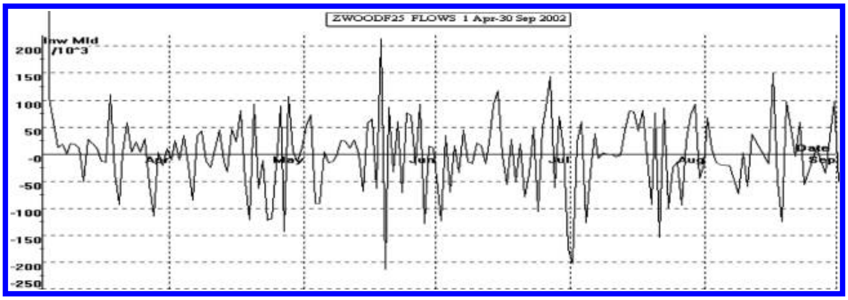
Most timeseries analysis assumes "stationarity", i.e. steady usage and variability over the period being studied. This research seeks automatic ways of picking out non-stationary behaviour. To illustrate the problem, one timeseries of urban District usage over the summer of 2002 is shown, (with its diurnal pattern heavily compressed): how can jumps in this series be identified?



The usual way of removing stationarity is by taking first differences. If there is seasonality, differences should be taken between values in successive cycles. Here there is both a diurnal and weekly seasonality. Taking average daily flow and removing weekly seasonality gives:

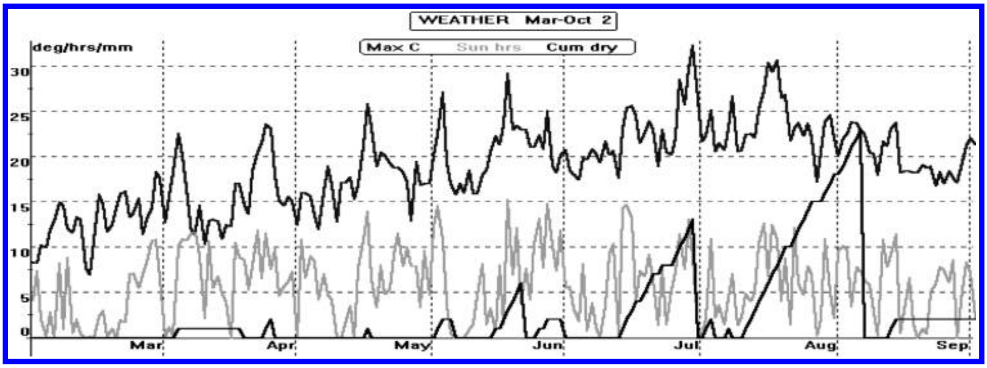


This clarifies some differences in level e.g. towards the end of May. The difference graph, subtracting successive de-seasonalised values, is as shown below:

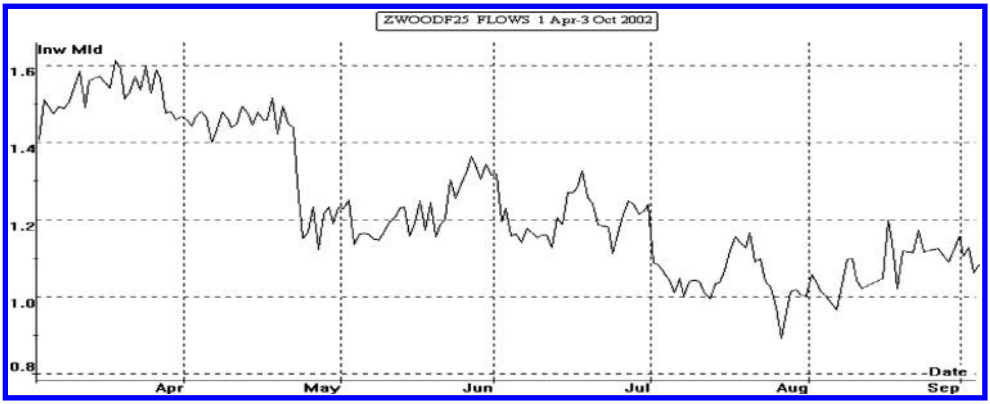


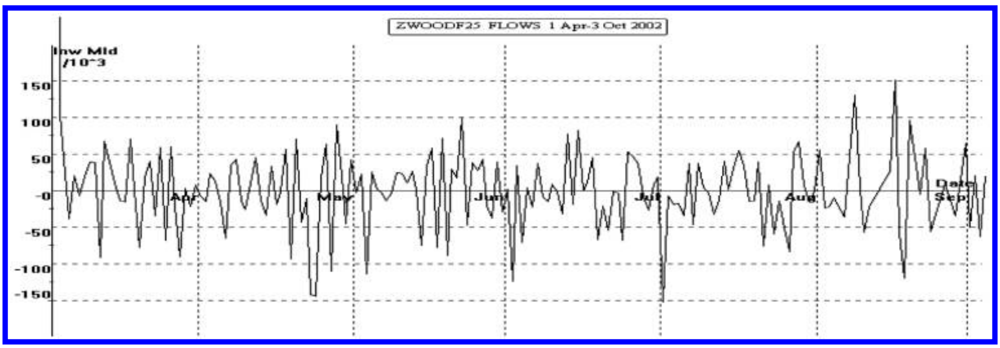
If there are known external factors causing a timeseries to change level which can be removed, it is sensible to do so. One such factor for UK summer water use is spells of hot, dry weather.

The links with weather can be seen by comparing the previous graph with a graph of maximum temperature, sunshine hours and cumulative hot days since rain. E.g. the extra usage at end-July and mid-August exactly match the hot dry weather peaks.



This impact can be modelled by using linear regression to find the mix of weather factors which best accounts for variation in district usage, over a set of Districts. This gives a way of removing the worst of the hot-weather impacts from the original timeseries. The de-weathered (and de-seasonalised) timeseries and the associated difference timeseries are as shown:





Spikes signifying change in level arise in late May, early July, early August and mid September. It is still not clear, however, how to "decode" the spikes or interpret what they mean.

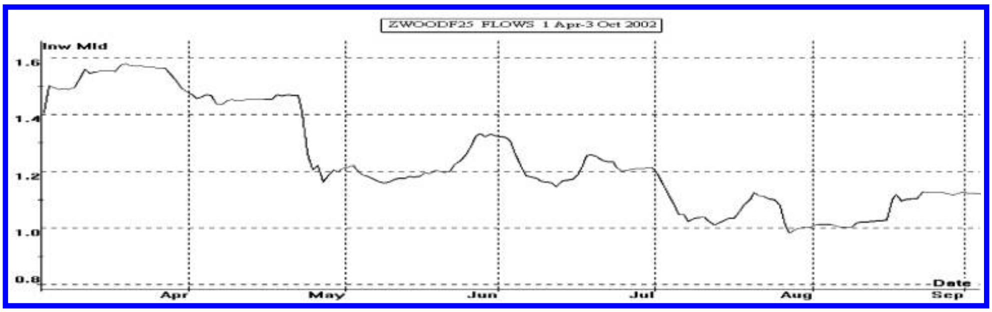
2.2 *Tracking signals and adaptive forecasting*

An alternative way of spotting jumps in timeseries is to consider the use made by forecasters of "tracking signals". The most commonly-used forecasting method is exponential smoothing, applying a discount factor to past values. The "best" discount factor will vary over time. In stable times a low discount rate is appropriate, giving little weight to the new information. In turbulent times only recent data is relevant and the discount rate is high.

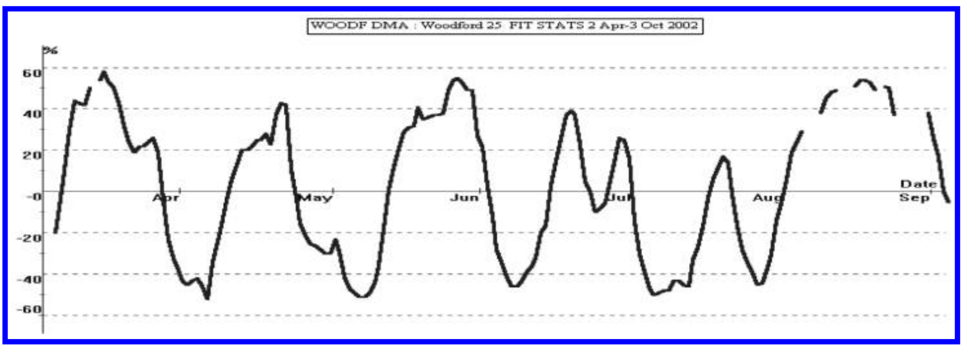
There are established ways to automate this process and give an *adaptive forecast*, i.e. one in which the forecast parameters change with the data. Trigg and Leach (1968) use "tracking signals" to make the forecast more responsive in turbulent times.

- The error is tracked by finding the ratio of the exponentially-smoothed error to the smoothed Mean Absolute Deviation of error;
- In stable times the errors tend to cancel so the smoothed error is relatively small;
- If the level of the timeseries changes then all the forecasting errors are in the same direction, the smoothed error rises and the ratio rises towards one.

The signal shows when there has been a change in the level of the series. When this happens the smoothing constant (the discount rate) is changed to give more weight to recent data so the forecast moves to the new level more quickly. Results applying exponential smoothing plus the Trigg and Leach algorithm (to de-weathered, de-seasonalised daily usage) are as shown.



The tracking signal is shown in the lower graph: does this offer a clue to data-jumps?



There is an even clearer end-of-May drop. However it is hard to pick out other level-changes from the tracking signal approaching plus or minus one. And with this approach there is still no information on the likely cause of the usage-drop.

3 ALGORITHM TO SPLIT OUT USAGE AND LEAKAGE

3.1 *Outline of approach*

Tracking signals can "see" no future data. It may be over-ambitious to validate District Meter data by these methods on the day it arrives. However, auto-validation, say, working a week in arrears would still be helpful for validating a large set of District Meter data.

One can move beyond standard statistics by making better use of the diurnal data. The core idea behind the proposed algorithm, to identify and classify jumps in District data, is to split each day's metered consumption into final domestic and commercial usage residual leakage by exploiting the fact that

- final usage has a strong and predictable pattern over each day of the week, whereas
- leakage (apart from major burst or repair days) is largely constant within the day.

Once these two working variables have been extracted, they are each tracked over the data-period, using exponential smoothing to identify potential jumps. Decomposing the data into usage and leakage at the outset maximises the chance of being able to assess whether any jumps observed are primarily in leakage or in final usage. Unexplained jumps in final usage should not arise from changes in leakage alone, so point strongly to District re-definition.

3.2 *Decomposing total consumption on a day into final use and leakage*

The strong diurnal patterns of water use in DMAs are discussed elsewhere (Race & Burnell, 2001). The following steps are applied to decompose 15-minute data on District consumption for each day into final-usage and leakage elements:

(a) build the expected commercial usage diurnal profile for the District, given the average metered use of each commercial site from the billing system and the typical profile for sites of that category and size taken from panel data;
(b) build the expected domestic profile. This will reflect the mix of houses and flats in the District and their standard profiles for that weekday. (If these counts come from Post Office address-location data, flats can be distinguished by two or more properties at the same grid-reference). It will also include any adjustment needed for a "slump" in the morning peak usage during school holidays;
(c) using the weather model for the District and the day's weather, estimate the weather impact. Make any adjustments needed for a rise in afternoon and evening use during hot weather. Hot weather affects diurnal shape of demand as well as its overall level;
(d) aggregating (a)–(c) gives the expected final use profile, assuming the District is as defined by its polygon and sites have on average standard usage profiles;
(e) apply linear regression to account for the year 96, 15-minute district usage values for the day in terms of a constant plus a "usage-scale-factor" times the expected final usage. The constant term will correspond to leakage;
(f) comparing predicted and actual usage values over the day, identify the outlyer high and low points (these may be data-spikes which could distort the regression);
(g) repeat the linear regression (e) but excluding the outlyer points and allowing for pressure-related variation in leakage over the day.

If the regression R^2 is below 70%, or the constant is negative, the day is excluded from further analysis – it has a suspect shape. If the R^2 exceeds 70% the constant indicates leakage, hence the day's metered consumption has been split into final-usage and leakage.

For a tight district with standard domestic and commercial usage the usage scale factor should be around 1.0. Values below 0.8 or above 1.2 may imply a breached district and this data is also suspect. Crucially for present purposes, changes in the usage scale factor over time suggest that the number of properties supplied by the boundary meters has changed i.e. that the district boundary has altered and the District has been re-defined.

3.3 *Detecting a shift in usage or leakage over time*

To find a shift in level on usage, leakage or both over the study period, the following algorithm is applied:

- work forward through the daily district data, applying exponential smoothing to the usage and leakage values derived, and storing the smoothed values for each;
- similarly work backwards through the data, applying exponential smoothing;
- on each day, compute the gap between the forward and backward smoothed "usage plus leakage" values;
- the gap is largest where the data-level looking backwards is furthest from the data-level looking forwards, i.e. there is a jump in the data;
- find the mean and standard deviation of the daily gaps computed over the data period. Days on which the gap exceeds two standard deviations are identified as potential "jump days" for subsequent manual inspection.

3.4 *Refining the algorithm*

The challenge with such an algorithm is to ensure sensitivity to real changes but indifference to short-term noise. The algorithm as outlined works with two derived values of the daily data, usage and leakage. Any errors on splitting the data on a given day could lead to spurious jumps being generated. This suggests applying exponential smoothing to the raw regression data rather than to the derived usage and leakage values. To do this

- store the count and summations used to compute the day's regression;
- apply exponential smoothing to each summation before applying the regression formula;
- use the day's regression R^2 to control the smoothing factor (as a kind of tracking signal).

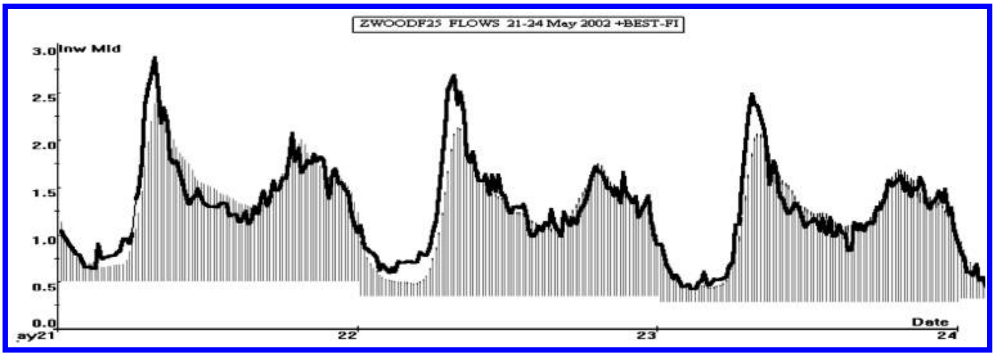
This makes usage and leakage derived for each day reflect recent days as well as the latest day.

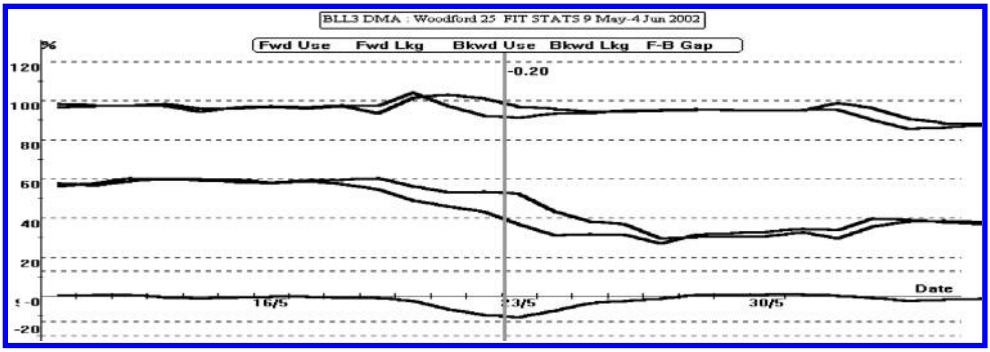
Better data on the shape and size of final domestic use for the District, e.g. using Census data to characterise the District's population, could further enhance the algorithm.

4 JUMP-CLASSIFICATION IN PRACTICE

4.1 *Identifying a jump due to the repair of a major leak*

The first illustration is the District shown earlier. The graph shows a close-up for May 21st–23rd. For each day the "backcloth" is the expected diurnal usage profile for that day, using the methods described in Section 3.2. A good fit has been obtained. As can be seen, there is a sudden drop in usage on May 22nd. The following graph shows the algorithm at work from May 9th to June 4th.



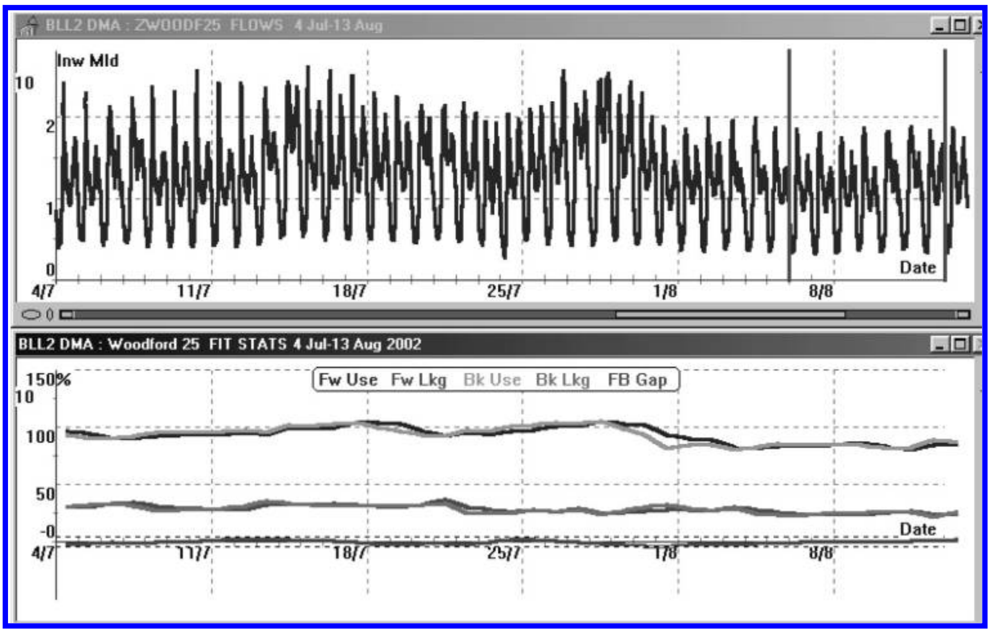


(a) The top pair of lines show forward and backward exponential smoothing on the inferred final usage for each day. Both values are around 97%, so the diurnal profile is close to that expected from the property count. There is little difference between the two trends, suggesting there is no shift in final usage around May 23rd.
(b) The middle pair of lines similarly track forward- and backward-smoothed leakage. There is a big difference around May 23rd.
(c) The bottom line is the gap between forward and backward trends and is used to pick out jumps. Its amplitude is largest on May 23rd and a jump on that day has been auto-detected.

Because the main contribution to the gap arises from leakage, this implies this jump is the result of a repair which has brought down leakage. Operational records in fact show that a repair gang made a significant repair within the District on that day.

4.2 *Identifying a drop in implied usage*

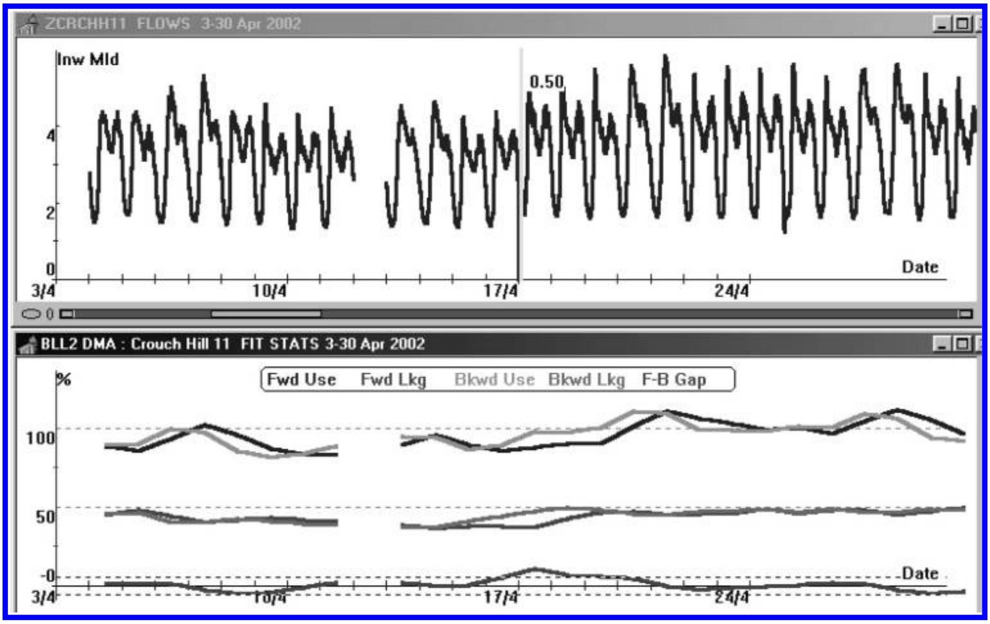
Illustration two is the same District at the end of July. Though there is a rise in overall usage, the weather-correction has been applied. The inferred usage is relatively steady until July 31st.



Examining the usage and leakage trends, the August drop is due mainly to a reduction in final usage. However, some drop is to be expected, given this is the start of the summer holidays. A quarter of UK households have children. If each go away for two weeks out of a six-week summer holidays, 8% of households will be missing each week. More investigation is needed of the effect of holidays on domestic usage, as discussed elsewhere (Burnell, 2003) .

4.3 *Data jump due to a breach*

Illustration three, on a different urban District, shows the algorithm picking up what is almost certainly the effect of a boundary valve-change on April 17th . The inferred usage is 82% of that expected before this date and 100% afterwards, so the District's integrity was more likely restored rather than lost at this date. The change in daily amplitude before and after the jump-day is characteristic of a valve-change rather than a leak; the algorithm has detected this.



5 CONCLUSIONS AND FURTHER QUESTIONS

District Metering produces large sets of automatically-collected data which will inevitably include missing data and other errors. Ways of automatically and routinely validating this data are needed, including recognition of data-jumps caused by District re-definition. An algorithm for picking these out, by examining diurnal patterns, is specified and illustrated.

Water use differs from most statistical variables in that analysts have first-hand knowledge of its behaviour. More advanced "black box methods" such as neural networks may come into their own, but probably only after well-understood aspects of water use have been fully exploited.

The precision with which jumps can be categorised will depend on the consistency of final usage and the frequency with which network events overlap. Further research is needed:

- to quantify the variability of final domestic usage in water-use monitoring panels, including the impact of summer holidays;
- to apply ward-level Census data to Metered Districts so as to refine understanding of how domestic use and diurnal patterns vary across communities;

- to quantify variability in commercial usage over time;
- to understand more about the diurnal behaviour of district breaches, by using district and multi-district hydraulic models, calibrated on the meter flow and pressure data;
- to look across districts and find matching events in adjacent districts which can augment the data-jump interpretation being offered.

REFERENCES

Trigg, D.W. and Leach, A.G. 1968. *Exponential Smoothing with an adaptive response rate*, Operational Research Quarterly, Vol 18.

Race, J.J. and Burnell, D.A. 2001. *Making the most of District Metering: Modelling patterns in water use*, Water software systems: Theory and applications.

Burnell, D.A. 2003. *Lifestyle and its effects on domestic water use*, Advances in water supply management.

Implementation of a confidence grading system for clean water asset data

B. Jankovic-Nisic, I. Ledrappier & A. Bradshaw
Halcrow Water Services, Snodland, Kent, UK

ABSTRACT: Increasing economic and regulatory pressures have been placed on Clean Water Providers to show that they have a complete understanding of all their assets. A significant amount of money and effort has been spent on developing Integrated Management Systems. These systems are designed to collate all information from all sectors within the water company in a central database. Consequently, a control of the quality of the data input into the asset database is crucial, especially when dealing with large water distribution systems. The task of control of data quality involves collating and reviewing data thus ensuring that the best quality information is kept for future reference and updates. Halcrow Water Services (HWS) are currently aiding Northern Ireland Water Services (NIWS) in the process of investigating and planning rehabilitation strategies for a large proportion of their water distribution system. This is just one part of the Water Services Northern Ireland Asset Management Plan where efforts will be made to increase confidence in, and reliability of information review. The Detailed Zonal Studies (DZS), currently being carried out, involve an extensive collation of information from many sectors to undertake a comprehensive analysis of the condition and performance of the clean water assets. In order to provide an assessment of the confidence placed on this information, HWS has developed and applied an automated process that assigns a confidence grade to each set of data. This information is then fed back into the relevant Water Service Asset Database, which provides NIWS, with a record of assessment of the quality and reliability of their data. The methodology and the experience gained from the implementation of the confidence grading system are outlined in this paper.

Keywords: confidence grading, asset management

1 INTRODUCTION

Effective data management tools have become a prerequisite for the development of Asset Management Plans within a water company. The largest proportion of time spent during the development of Asset Management Plans is on collecting and managing raw data in order to make it useable for analysis. Therefore, in order to maximise efficiency both now and in the future, it is necessary to establish a data capture methodology and to develop procedures that will allow for systematic data updates. This will minimise the time needed for data review and collation.

Moreover, the main task of the data management tool does not only involve data storage but its maintenance and the use by a final user – the technical and management staff of the water company. Data verification plays a major role in the process, especially with raw data often coming from multiple sources with varying degrees of confidence. The complexity of data management and verification is increased when dealing with large water distribution systems, where asset databases contain more than one characteristic for each of the assets.

This shows the DZS to comprise of two stages:

Needs Stage
All data relating to the zone is collated and reviewed. New data is generated where appropriate and shortfalls identified with regards to the condition and performance of the network.

To complete the stage a number of Work Modules are carried out. The first three modules provide system familiarisation, data collation and data review. The fourth and fifth modules analyse the structural and hydraulic condition of the mains in the study area. The sixth and seventh modules look at the performance of Communication Pipes and the Quality of Water via sampling.

The output of the stage, module eight, shows perceived failures of the network for given reasons.

Options Stage
This stage provides potential solutions to the perceived problems detailed within the Needs Stage. Preferred solutions are identified and recorded as part of a Draft Rehabilitation Plan.

It has been recognised that some of the problems associated with the complexity of data management could be overcome by the development of a confidence grading system. Such a system should be capable of assigning a level of confidence to all information entered into the asset database. Ideally, the criteria for determination of confidence grades should be agreed before any information is entered into the database. However, it is often the case that the database contains a large proportion of unverified data, usually due to the absence of effective data management procedures.

This paper reports on the methodology and the experience gained from the implementation of the asset confidence grading system for one of the Water Mains Rehabilitation programmes in Northern Ireland. The first stage of the programme involved a comprehensive data collection, review and collation exercise in order to identify shortfalls in the condition and performance of the water distribution system. The second stage dealt with the solutions to the shortfalls and needs identified during the data collation.

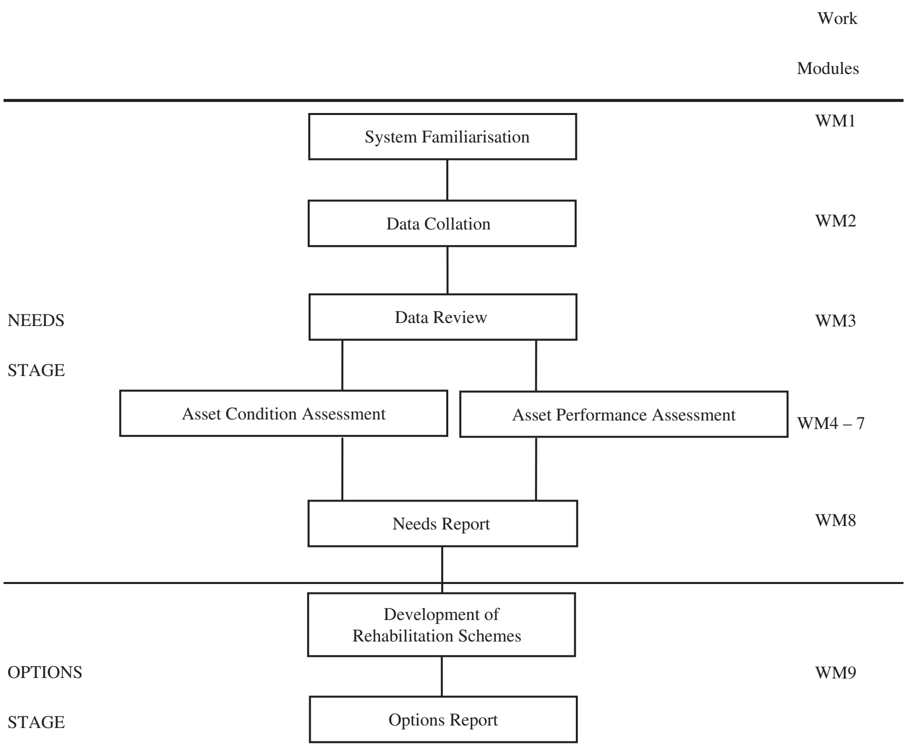


Figure 1. Overall flow chart.

2 CONFIDENCE GRADING METHODOLOGY

The confidence grading system in the UK has been developed and published by OFWAT in order to set standards for all water companies to improve the quality of information stored in their asset databases. The confidence grading matrix is shown in Table 1, as identified by WRc (1994) in Managing Leakage Reports. Confidence grades consist of an alphanumeric code coupling reliability letter and accuracy band.

The reliability bands are defined as follows:

A – Highly reliable
Actual – Data based on sound records, procedures, investigations or analysis which is properly documented and recognised as the best method of assessment.

Forecasts – Based on extrapolations of high quality records covering or applicable to 100% of the Company's area, kept and updated for a minimum of five years.

B – Reliable
Actual – Generally as A but with some minor shortcomings, for example the assessment is old, or some documentation is missing, or some reliance on unconfirmed reports or some extrapolation.

Forecasts – Based on extrapolations of records covering or applicable to more than 50% of the Company's area, kept and updated for a minimum of five years.

C – Unreliable
Actual – Data based on extrapolation from limited sample for which Grade A or B are available.

Forecasts – Based on extrapolation of records covering more than 30% of the Company's area.

D – Highly unreliable
Actual – Data based on unconfirmed verbal reports and/or cursory inspections or analysis.

Forecasts – Based on forecasts not complying with bands A, B or C.

The accuracy bands 1–6 indicate level of uncertainty in the data from level 1 – 1% to level 6 – 50–100%. Blank fields in the table indicate that the confidence grades are considered to be incompatible.

Although initially built and applied for assessment of the quality of leakage and unmeasured consumption estimates, the confidence grading system is widely used to determine reliability of all clean water asset data. Its use has been therefore extended to wider purposes such as General Asset Data, Asset Performance Data, Consumption Data, Customer Complaints, Water Quality and Historical Interruptions to Supply etc.

However, little or no guidance is available on how the confidence grading should be applied to data verification of water assets. Recent experience shows that the reliability bands are not easily applied uniformly for all clean water asset information. For any given data set, it can be difficult to distinguish between reliability and accuracy bands, or confidence grades with any certainty. There are various examples that show how the assigned grades could be misleading. For example, a confirmed record of pipe diameter might fall under any of the A or B grades, as the actual

Table 1. Confidence grading matrix (WRc, 1994).

	Reliability bands			
Accuracy bands	A	B	C	D
<1%	A1			
1–5%	A2	B2	C2	
5–10%	A3	B3	C3	D3
10–25%	A4	B4	C4	D4
25–50%			C5	D5
50–100%				D6

reliability band is not always easy to assess. Similarly, it is uncertain what confidence grade should be placed on the age of pipe if it is known that the pipe has been laid, for example, in the 70's. In addition, verbal reports from operations staff may be more reliable than the data extrapolated from confirmed records, which would not be the case if the confidence grades were applied according to Table 1. This implies that the effective confidence grading system has to be developed in such way to suit specific asset database and therefore specific needs of a project.

Thus, in everyday practice, some adjustment to the originally established confidence grading structure by OFWAT is unavoidable.

3 NORTHERN IRELAND WATER SERVICE WATER MAINS REHABILITATION PROGRAMME – SILENT VALLEY/FOFANNY/CAMLOUGH ZONAL STUDY

3.1 *Background*

The Department of Regional Development (Northern Ireland) Water Service (NIWS) has commenced a programme of work to investigate and plan the rehabilitation of the water main network of Northern Ireland.

As part of the programme, Halcrow Water Service (HWS) has undertaken a Distribution Zonal Study (DZS) designated as "Silent Valley/Fofanny/Camlough Zonal Study". This study, and others like it being carried out throughout the province, provide a comprehensive analysis of the condition and performance of NIWS's assets within the zones.

The general approach to the study is shown Figure 1.

3.2 *The study area*

This study covered an area of approximately 600 square kilometres, supplying approximately 22,350 properties with potable water. It incorporated the western side of Newry Town, to Castlewellan in the North East and Attical in the South East. The area also encompassed Rathfriland, Ballydrumman

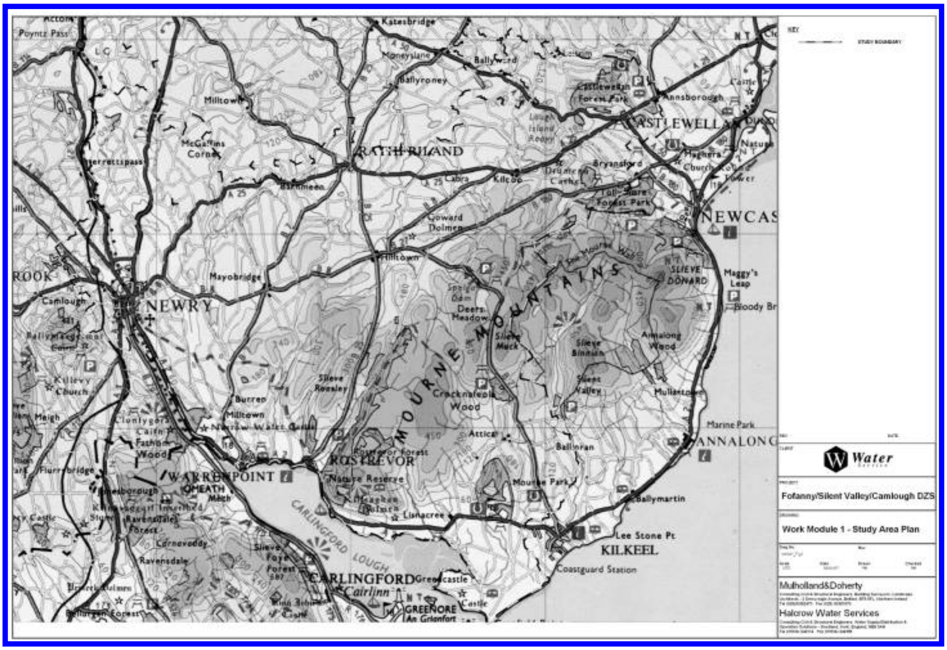


Figure 2. Area of the zonal study.

in the North down to the South Coast districts of Warrenpoint, Rostrevor and Cranfield. It comprises approximately 1051 km of potable, live mains. There are approximately 869 km of Distribution Mains, 41 km of Distribution Trunk Mains and 109 km of Trunk Mains. Private, Raw and Scour Mains constitute the remaining 32 km.

4 PROJECT CONFIDENCE GRADING

To assist with the quoting of realistic margins of error, HWS developed a confidence grading system and applied reliability and accuracy bands to all data collated in line with in Table 1 above.

Step 1
In the first instance all data collated was entered into a "data register" and an overall confidence grade assigned to the data sets as a whole for accuracy and reliability. For example, a confidence grade was given to the MapInfo "Burst" table as a whole rather than to the individual bursts within that table.

The assignment of these initial grades proved to be difficult due to individual data within some tables being of differing reliability and accuracy. The confidence grades assigned were reassigned once all individual grades were calculated. The most common grade was then used as the overall confidence grade for that data set.

Step 2
Following Step 1, grades where then assigned to all individual assets within the data sets. Confidence grades were, for example, given to geographical position, diameter, material, analytical scoring, year laid and general confidence.

The assignment of smaller sets of data were carried out manually, larger sets by computer analysis as detailed below.

5 POPULATION OF CONFIDENCE GRADING CRITERIA

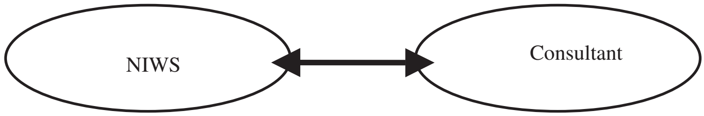
During the first stage of the project, which involved extensive data collation and review, HWS collected data from all the various sources that were identified. At the same time, NIWS were updating their respective data on a daily basis. Updated information from HWS was periodically returned to NIWS for subsequent updates of the central databases. In order to ensure a smooth data flow, this cross updating of the tables could only be made possible by:

- Always assigning unique reference to each asset
- Ensuring that tables used by both sides have the same structure and using the same software to store the information (MapInfo 6.0)
- Providing an audit name and time when all amendments were made
- Entering data source into the table, where possible
- Assigning a confidence grade to each asset to allow best quality information is kept for future updates
- Entering additional information in the comment line for each asset and standardising comments, detailing as much information regarding "what", "from where" and "from whom" updates were made

The data sources used during the project are outlined in Figure 3.

The first step was to design a confidence grading system that could be used by NIWS and HWS simultaneously. Table 2 shows the confidence grading system adopted for this project. For example, the mains data that was graded in this process was: diameter, material, year laid and geographic position.

The Categories indicate the source of the data and NIWS audit dates and suitable confidence grades have been assigned to each category.



- Mapinfo digitised data
- Paper drawings
- Operations staff verbal reports
- MS Access and Excel data

- Data collected during field tests
- Burst data and mains cutouts
- Operations staff verbal reports
- Data obtained by extrapolation
- Network model data

Figure 3. Data sources available during the project.

Table 2. Confidence grading criteria used in the project.

Category	Data source	Audit date	Confidence grade
1	Not existent	None	D6
2	None (poorly documented data from NIWS digitised and not confirmed)	None	C5
3		2001 and later	A3
4	NIWS Audit data	1999 and 2000	B3
5		1998 and earlier	C3
6	Consultant (unconfirmed field test data, verbal reports or data obtained by extrapolation or engineering judgment)	None	D3
7	Burst data and mains cut outs	None	A2

The confidence grades were chosen to, where possible, comply with the grades established by OFWAT, as shown in Table 1. and to indicate the level of confidence in the data sources as shown in Figure 3.

Category 1 represents all data whose location is known but no other data characteristics are available. These data have been assigned with the worst D6 confidence grade in agreement with Table 1. For a large proportion of the data that had been previously digitised, there was no information that would indicate if the information had been confirmed. This data has been considered unreliable and has been assigned very low confidence grade (C5). Pipe data digitised by NIWS that has been confirmed by entering the name and the date of an audit, was considered to be well documented. The actual confidence grade was further assigned according to the date when the information was entered. Recent records collected during the time of the programme by NIWS were assigned grade A3 (Category 3) with slightly lower accuracy than the information obtained from the burst records. Records confirmed before 1998 were collected from older paper drawings and it was very probable that this information needs to be updated with more recent data. Old records were assigned confidence grade C3. Unconfirmed verbal reports from operations staff passed to the consultant were considered to be of D3 grade. The data obtained from burst information and mains cut outs is of good quality and it has been assigned confidence grade A2. This data would be only updated when the new pipes are laid.

6 AUTOMATION OF CONFIDENCE GRADING

Due to the size of some of the asset tables, the process of manual confidence grade updates would be a very time consuming task. For example, the mains table contained around 10,000 records, and it was necessary to develop a process that would automatically update the confidence grades according to the criteria agreed in advance. Using the criteria established in Table 2, an automated

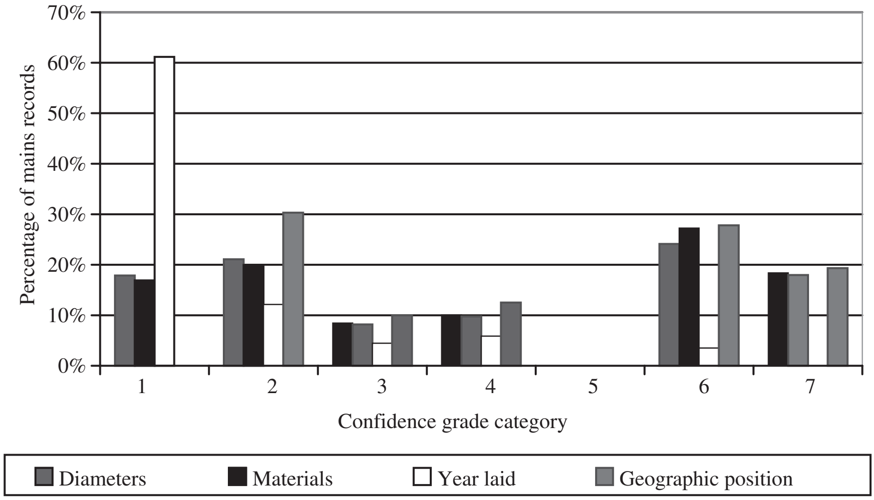


Figure 4. Summary of grades assigned for the mains table.

process was designed in MS Access to aid the update of the asset database. A number of queries were developed in order to examine each asset characteristic (for example, pipe material, diameter, year laid) across each category. Each query filters the data source, audit date and the information that needs to be graded and the assigns the confidence grade according to Table 2. The queries for each characteristic were then combined into one entity using macros. This allowed the automated process to be carried out by simply running a single macro for each characteristic, thus significantly minimising the time needed for an update of large tables.

7 SUMMARY OF GRADES ASSIGNED

The percentage of asset data falling into each of the confidence grades defined in Table 2 is shown in Figure 4 for the mains records. The figure also shows the extent the quality of data improved during the water mains rehabilitation programme. Before the commencement of the DZS programme, all mains data belonged to categories 1 and 2. After the DZS programme has been accomplished, about 40% of data has a confidence grade belonging to reliability grades A or B (categories 3, 4 and 7). These data have been verified during the time of the DZS. Further improvements can be expected once the data provided by HWS is confirmed by NIWS (category 6).

Overall, the reliability of about 80% of mains data was improved during the water mains rehabilitation programme. These are all mains that belong to categories 2–7. For the remaining 20% of mains data (category 1), data source or data characteristics was not available and improvements were not possible during the programme. This is considered a great improvement to data quality and reliability. Moreover, the confidence grading system indicated where further data improvements are needed.

8 CONCLUSIONS

The experience gained during the development of the confidence grading system can be summarised as follows:

– Implementation of a confidence grading system for clean water asset data is feasible.
– Automation of grading of large quantities of data is possible using simplified confidence bands using MS Access database software.

– Smooth data flow between the participants of the project can be ensured by assigning unique references and establishing a confidence grading system. During the simultaneous updates of the asset database the information with higher confidence grade is kept for future use. This also ensures that no duplicate data is stored.
– The confidence grading criteria must be developed in collaboration with all participants.
– The confidence grades are not easily applied for all data types uniformly.
– It is more important to grade the confidence in data relative to each other rather than trying to assign a uniform confidence grading system for all data types.
– Accuracy bands (1 to 100%) are difficult to assign particularly with assets that are buried.
– It is required that a minimum of data source, audit date and name are recorded for any information amended in the asset database in order to make automatic updates possible.
– It is important to keep track of what information has been changed. In order to achieve this, records should be included of what has been changed, by whom and when for each characteristic.
– Contractual review of the grading criteria should be undertaken. Current structure shown in Table 1 might not be appropriate to such a varied and large data set.
– The confidence grading system can show where lie any further data improvement needs.

REFERENCES

WRc, 1994. UKWIR Managing Leakage reports, Engineering and Operations Committee, UK.
Northern Ireland Water Services, 2000. Tender Documents for Watermain Rehabilitation Programme C.

Approach for restoration of missing data, long-term time series and generalization of results

H.V. Lobanova
State Hydrological Institute, St. Petersburg, Russia

G.V. Lobanova
Russian State Hydrometeorological University, St. Petersburg, Russia

ABSTRACT: Hydrological data, which are used in water supply management, as usual, have missing observations, different sizes of observations for different sites and very often – a short-term period of observation. For decision of many problems of hydrological computations an assessment of modern climate change long-term time series are necessary. Methodology and system methods for restoration of long-term time series and filling of missing have been developed. This system includes three main directions: use of relationships with the particular analogues for historical period, use of all analogies inside of homogeneous regions and development of regional relationships between different years and average historical conditions and use of relationships between different information in the particular site when possibilities of the regional analogues are exhausted. For generalization of restored data obtained by different methods two main approaches have been developed and suggested. Assessment of the efficiency of restored information has fulfilled on dependent and independent data. For this aim some criteria and indexes have been suggested. Application of developed methods of restoration are given for examples of maximum rainfall floods in the Carpathians region (Ukraine) and for assessment of water supply for Kalininskaya nuclear power station (Central region of Russia).

1 INTRODUCTION

Water supply management is based on the results of frequency analysis of hydrological characteristics. The necessity of reception of long-term time series of the hydrological characteristics for such analysis is caused at least by following two basic reasons:

– the more is a size of sample, the more precisely parameters of a distribution and design hydrological characteristics are calculated, since their random errors are proportional to a size of a sample (Kendall & Stuart, 1970);
– in modern changing conditions connected to an influence of man's activity and climate the significant importance has the estimation of homogeneity and stationarity, which can be executed only on the basis of long-term time series of observation;
– time series on the different gauged sites have to be practically the same size of observed data for an effective comparison of the results over the space;
– time series have to include the last years of observations and continually to be added by new data for a monitoring and checking of a stability of obtained climatic changes and their parameters.

Besides now there is a problem of reduction of hydrological network of observations and closing of a number of gauge sites. In many cases in the closed gauged sites it is necessary to have a continuation of observations and last years with absence of the data also can be considered as a situation of missing data. If these the last years with missing can be effectively restored for the considered hydrological characteristic, the closing of the given site still can be justified, if the restoration is not effective – a closing of site is inexpedient.

Hydrological data, as usual, have missing observations, different sizes of observations for different sites and very often – a short-term period of observation. Therefore the only one way is a restoration of missing data and synthesis of data before the beginning of observation period (or after closing of measurements) at the particular site.

2 EXISTING APPROACH

Existing approach for a restoration of missing data and long-term time series of hydrological characteristics is based on construction of linear regression equations with one or several analogues (Draper & Smith, 1970). There are special Recommendations (1979) for restoration of long-term time series of hydrological characteristics and their parameters. The sequence of restoration consists in the following:

- all equations satisfying to conditions of efficiency settle down in a decreasing order of correlation coefficients;
- at the first the restoration is fulfilled on the basis of the equation with the biggest correlation coefficient;
- if all possibilities of the best equation are realized the next equation with a correlation coefficient less than previous, but more than all others is used;
- this step by step restoration is continued until all equation answering to given conditions of efficiency will be used.

The equation of a multiple linear regression, on which the restoration is carried out, has the following kind:

$$Q = k_0 + k_1 Q_1 + k_2 Q_2 + \ldots + k_j Q_j + \ldots + k_l Q_l \tag{1}$$

where Q is a meaning of the hydrological characteristic in the site of restoration (short-term time series), $Q_1, \ldots, Q_l$ are hydrological characteristics in each site-analogue (common number is l), k_j – coefficients of regression equation calculated by the Least Square Method (LSM).

In the Recommendations (1979) the following four conditions of efficiency are given which should take place for each constructed equation:

$$n' \geq 6\text{-}10;\ R \geq R_{min};\ \ R/\sigma_R \geq A_{thr};\ \ k/\sigma_k \geq B_{thr} \tag{2}$$

where n' is a minimum number of observations for joint period, R_{min} – correlation coefficient of the built equation; σ_R, σ_k are standard random errors of a correlation coefficient and coefficient of regression equation; R_{min} is a minimum threshold of a given correlation coefficient ($R_{min} \geqslant 0.7$); A_{thr}, B_{thr} are threshold values of ratios: R/σ_R and k/σ_k accordingly (is given $\geqslant 2.0$ as usual).

If even one of conditions (2) is not carried out and even one of the coefficient of the regression equation does not satisfy to the fourth condition ($k/\sigma_k \geqslant B_{thr}$), this equation is not used for a restoration. The restored data calculated by the equation (1) have systematically less variance. The exception of systematic bias in variance is possible by a correction of calculated values on the basis of formula:

$$Q_i' = (Q_i - \overline{Q}_n)/R + \overline{Q}_n \tag{3}$$

where Q_i' is a correct calculated value, Q_i is a calculated value with bias of variance, $\overline{Q}_n$ is an average value for short-term time series for period in n-years.

3 SUGGESTED METHODOLOGY AND METHODS

3.1 *Methodology*

Methodology of restoration depends on the kind of data. As usual, water supply management deals with data of intra-annual and annual scales. The following methodology of data restoration can be suggested:

- use of the particular analogues and simple and multi-regression models for a long-term period;
- use of all numbers of analogues inside of homogeneous regions and development of regional relationships between data of two different years or between the particular year and averaged historical data (simple regression);
- use of different information in the particular site and development of relationships (simple and multi) between long-term time series of different information: between one month and other month data, between monthly data of the particular year and averaged monthly data for a long-term period, between runoff data and meteorological data, etc.

In such methodology the principle of step by step restoration is applied as well as a transition from using of spatial regular properties of synchronous of fluctuation to relationships between different information for the particular site or river basin. The consecutive application of all three approaches allows to achieve the most complete and effective restoration of the data. For each approach the particular methods have been developed.

3.2 *Method of the particular analogues*

Method of the particular analogues is based on the equation of multi-regression (1). Critical analysis of conditions (2) has shown that the condition $R/\sigma_R \geqslant A_{thr}$ is superfluous and can be excluded, since a confidential interval for R functionally depends on $n'$and R and for $n' \geqslant 6$ and $R_{min} \geqslant 0.7$ the correlation coefficient R always will be have a statistically significance (Sachs, 1972). The condition $n' \geqslant 6$–10 should be replaced on more strong, at least $n' \geqslant 10$. Condition $R \geqslant R_{min}$ where $R_{min} \geqslant 0.7$ is better to transfer in a condition of assessment of standard random errors of calculation on the basis of regression equation. It is known (Kendall & Stuart, 1970), that:

$$\sigma_\varepsilon = \sigma_Y \sqrt{1-R^2} \tag{4}$$

where σ_ε is a standard deviation of remainders, σ_Y is a standard deviation of short-term time series.

Then the ratio $\sigma_\varepsilon/\sigma_Y$ characterizes an average relative error of restored value. This is not enough and the following ratio it is necessary to introduce additionally:

$$\sigma_\varepsilon/Y_c \leq \Delta_{thr} \tag{5}$$

where Δ_{thr} is a given minimum threshold of a given relative error (in %), Y_c is a restored value.

On the basis (5) it is possible to set Δ_{thr} in 50%, that can give an average relative error for symmetric distribution about 25%. Condition $k/\sigma_k \geqslant B_{thr}$ for $B_{thr} = 2$ characterizes a statistical significance of equation coefficient for 95% confidence interval. Thus, the conditions (2) can be corrected in the following system:

$$n' \geq 10;\ R \geq R_{\kappa p'};\ \sigma_\varepsilon/Y_c \leq \Delta_{thr};\ k/\sigma_k \geq 2 \tag{6}$$

where $R_{\kappa p'} = 0.7$ and $\Delta_{thr} = 50\%$.

3.3 *Spatial models*

The spatial models are based both on a gradient and on synchronism of the considered hydrological characteristic in homogeneous area. Thus the construction of models connecting data of one year or years is possible, for which there is the information at considered (restored) site, with the data of other year in which the information is absent and it is necessary to restore this information (Rozhdestvensky & Lobanova, 1991). The system of one-factor equations for restoration will be as follows:

$$\boxed{\begin{array}{l} Y_{ij} = A1_{ik} Y_{k,j} + A0_{ik} \\ Y_{ij} = A1_{i,k+1} Y_{k+1,j} + A0_{i,k+1} \\ \ldots\ldots\ldots\ldots\ldots\ldots \\ Y_{ij} = A1_{i,m} Y_{m,j} + A0_{i,m}, \end{array}} \tag{7}$$

where: $Y_{k,j}$, $Y_{k+1,j}$, …, $Y_{m,j}$ are hydrological characteristics in j-th site-analogue in each k-th, $k + 1$th, …, m-th years, in which observations take place in the considered site; Y_{ij} are hydrological characteristics in j-th site-analogue in i-th year, in which observations in considered site are absent; $A1$, $A0$ are coefficients of one-factor equations.

One more kind of spatial model is the relationships between average hydrological characteristics for long-term period in each gauge site ($Y_{mean,\,j}$) and the characteristics in the same sites but in the particular years when observations in considered site are absent (Y_{kj}):

$$Y_{kj} = A1_k Y_{mean,j} + A0_k \tag{8}$$

3.4 *Relationships in gauge site*

Restoration at the particular site is carried out, if the possibilities of site-analogues and spatial models are exhausted. The equation for restoration on the basis of seasonal function (Lobanov & Lobanova, 1999) is similar to the equation (8) and is applied, if it is required to restore data for some months (monthly runoff). This equation is not usually apply for the restoration of runoff in months with extreme values (summer and winter months) for not to apply extrapolation, as less reliable procedure. The equation is as follows:

$$Y_{il} = B1_i Y_{mean,l} + B0_i \tag{9}$$

where Y_{il} is a monthly runoff in lth month of ith year, $Y_{mean,l}$ is a average runoff for long-term period in lth month.

Another kind of equation can be between monthly runoff of the particular months or between monthly runoff or other hydrological characteristic and its meteorological factors in the form of empirical relationships or models, such as "precipitation – runoff".

4 METHODS FOR GENERALIZATION OF THE RESULTS

In application of different methods and models of data restoration the situation is possible, when a concrete hydrological characteristic is obtained by different methods and as a result several meanings of the same characteristic are calculated. In this case there is a problem, which of them to prefer. The most simple variant of the decision – to choose the value, which is obtained with the smallest error. At the same time there is the theoretical proof that the random error of calculated value will be less in the case of several methods than they use only one, but the best with the minimum error (Kendall, Stuart, 1970; Kazakevich, 1977):

$$\boxed{\sigma^2_{\varepsilon joint} = \frac{m}{\sum_{j=1}^{m} \frac{1}{\sigma^2_{\varepsilon i}}} \le \sigma^2_{\varepsilon \min}} \tag{10}$$

where $\sigma_{\varepsilon joint}$ is a standard error of calculated value obtained by m methods, $\sigma_{\varepsilon j}$ is a standard error of calculated value obtained by *j-th* method, $\sigma_{\varepsilon \min}$ is a standard error of calculated value by the best method.

Equation (10) is suitable for the case of independent methods, where the random errors of calculated value have no any significant correlation. For generalisation of calculated values obtained by different methods two main approaches could be applied:

– averaging of calculated values with weight coefficients inversely proportional to the standards or variances of random errors of their calculation (Wall, 1987):

$$Y_{joint} = \frac{\sum_{j=1}^{m} 1/\sigma_{\varepsilon j} * Y_j}{\sum_{j=1}^{m} 1/\sigma_{\varepsilon j}} \qquad Y_{joint} = \frac{\sum_{j=1}^{m} 1/\sigma^2_{\varepsilon j} * Y_j}{\sum_{j=1}^{m} 1/\sigma^2_{\varepsilon j}} \tag{11}$$

where: Y_{joint} is a generalised calculated value on the basis of m methods, Y_j is a calculated value by j-th method, $\sigma_{\varepsilon j}$ is its standard error;

– probabilistic approach, which takes into account the normal distribution of random errors for each calculated value and their generalisation in the form of maximum likelihood or composition of distributions of errors (Beshelev, Gurevich, 1980):

$$P(Y_i')_{joint} = \prod_{j=1}^{m} \frac{1}{\sqrt{2\pi\sigma_{\varepsilon j}}} e^{\frac{-(Y'_{i,j} - \bar{Y}'_i)^2}{2\sigma^2_{\varepsilon j}}} \qquad P(Y_i')_{joint} = \sum_{j=1}^{m} \frac{1}{\sqrt{2\pi\sigma_{\varepsilon j}}} e^{\frac{-(Y'_{i,j} - \bar{Y}'_i)^2}{2\sigma^2_{\varepsilon j}}} \tag{12}$$

where $Y_i'{}_{joint}$ is calculated value by m methods.

5 ASSESSMENT OF EFFICIENCY OF RESTORATION

It is necessary and important to receive a restored hydrological characteristic, but not enough. The second important aspect of restoration is the estimation of its efficiency. Thus for each site and each characteristic it is possible to estimate: whether effectively to carry out data restoration or not. For an estimation of efficiency two methodical approaches are applied: an estimation of efficiency of restoration on the dependent and independent information. In the first case an error of restoration of each value is defined by the formula (4). At an estimation of an error on the independent information from time series of observation, for example, one value is consecutively excluded, which then is restored and compared with an observed value. Thus the real error of restoration can be determined. The most informative indexes for assessment of efficiency of restoration on the dependent information are accepted:

– a number of restored years (Δn in years and $\Delta n'(\%) = (N - n)/n * 100\%$ in %);
– a standard errors (individual $\sigma_{\varepsilon i}$ and average $\sigma_{\varepsilon_{mean}}$) in measured units ($m^3/s$, mm, etc.) or in %: $\sigma'_\varepsilon(\%) = \Sigma(\sigma_{\varepsilon i}/Y_r)/\Delta n$;
– a ratio of standard (or average standard) error of restoration to standard deviation of observed time series $\Delta\sigma_\varepsilon(\%) = \sigma_\varepsilon/\sigma_Y$ or the same ratio but of variances, that means the well-known statistical Fisher test: $F = \sigma_Y^2/\sigma_\varepsilon^2$.

Indexes of an estimation of efficiency on the independent data will be:

– independent error (individual $\varepsilon_{i\,\mathrm{In}}$ and average $\varepsilon_{mean\,\mathrm{In}}$) determined as a difference between observed and calculated values;
– standard error (individual $\sigma_{\varepsilon_{Ob}}$ and $\sigma_{\varepsilon_{mean\,Ob}}$) obtained from the equation by formula (4);

– ratio of errors obtained on independent data and on the basis of formula (4) for joint period of observations: $K_1 = \sigma_{\varepsilon_{In}}/\sigma_{\varepsilon_{mean\ Ob}}$;
– ratio of errors obtained on independent data and on the basis of formula (4) for dependent data for their period of restoration: $K_2 = \sigma_{\varepsilon_{In}}/\sigma_{\varepsilon_{mean}}$.

The offered ratios K_1 and K_2 characterize as far as an error determined on the independent data (a real error) is more than an error determined on the formula (a theoretical error). Thus, the ratio K_1 is determined for the same period of time as observed data and the ratio K_2 – for the different periods. On the basis of the chosen indexes it is possible to make a general conclusion about efficiency of restoration, taking into account the following conditions:

– K_1 = min, K_2 = min, i.e. the ratio of error obtained on an independent and dependent data should be minimum and aspire to 1;
– $K_1 \approx K_2$, i.e. the ratio should not essentially differ, that characterizes their stability in time;
– $\Delta\sigma_\varepsilon \cdot K_1 \leqslant$ 60–80%, i.e. the expected error determined on an independent data should not be significance and much more differ from the given error (Δ_{thr}).

6 APPLICATION

6.1 *Example 1*

For analysis the observed time series of floods (maximum discharge and depth of flood runoff) have been chosen in territory of Carpathians (Ukraine). The whole number of gauge sites were 80 and they are located in upper mountain basins of three main rivers: the Dnestr River, the Prut River and the Tisa River. The watershed areas (A) varied from 25.4 km^2 to 1000–2000 km^2, i.e. the small river basins were examined, on which the maximum rainfall discharged are the same as the maximum annual discharges, as a rule. The average altitude of watersheds (H) varied in a wide range from 300 m up to 1200 m. Slopes of rivers and watersheds also changed more than on the order: from 39‰ up to 446‰ for watersheds and from 2.8‰ up to 66.4‰ for rivers. Also forestness of watersheds are changed from 19% up to 95%.

The average size of time series was 43 years both for time series of maximum discharges as well as for time series of depth of runoff. The greatest size of long term time series is 89 years took place for the Prut River – site Chernovtzy, also its watershed area a little bit more than areas of other basins (A = 6890 km^2). The size of next the longest time series is 60 years for the Tismenitza River – site Drogobich (A = 250 km^2). The smallest period of observations in 12 and 16 years take place for the time series: the Malyi Siret River – site Verkhnie Petrovtsi (A = 488 km^2) and the Rika River – site Nizhniy Bistriy (A = 787 km^2). For 48 time series data available from the beginning of observations to 1999–2000, for other time series the end of observations was 1987–1988.

Restoration has been fulfilled for all time series by the approach which has been described in 3.2. Main results of restoration for effective sites are given in Tables 1 and 2 accordingly for the maximum discharges and depth of runoff. Effective sites have been chosen on the basis of conditions, given in 5, i.e. $K_1 \approx K_2$ and K_1 or $K_2 \leqslant 2$. The attempt of the further restoration on the basis of relationships between maximum discharges and depth of runoff had not been successful.

From the analysis of restoration of the maximal discharges follows, that the effective restoration take place for about half of common number of time series. Average relative error of restoration varied in limits 20–30% and its average value for all records is 28%. The average size of observed records was 43 years before restoration and became 63.2 years after restoration, i.e. the size of records was increased in 1.5 times or about 60% in terms of average $\Delta n'$. Average ratio of random error of restoration to standard variation of observed records ($\Delta\sigma_\varepsilon$) is equal 34.9%. The average ratio K_1 = 2.4 and K_2 = 2.6 and it means the standard errors of restored data can be in 2 times more for independent checking than it obtained on the basis of dependent data.

The analysis of results of restoration of runoff's depth (Table 2) has shown, that size of time series has been increased on 61% in average and their average period after restoration has become 62.6

Table 1. Indexes of efficiency of restoration for effective time series of maximum rainfall discharges in Carpathians.

No.	River – site	A, km^2	H, m	n	N	$\Delta n'$ (%)	$\sigma_{\varepsilon_{mean}}$ (%)	K_1	K_2
1	Dnestr – Strelky	384	620	53	82	54.7	30.2	1.8	1.7
2	Striy – Matkov	106	860	46	55	19.6	23.3	2.3	1.8
3	Slavska – Slavskoe	76.3	860	47	53	12.8	30.3	2.4	1.8
4	Svich – Mislovka	201	1000	46	73	58.7	31.8	2.1	1.7
5	Luzdanka – Goshev	146	660	50	57	14.0	33.1	2.0	1.9
6	Chechva – Spas	269	820	45	77	71.1	33.0	2.1	1.7
7	Lukva – Bondarov	185	480	47	57	21.3	30.0	2.3	1.9
8	Striy – Zavadovka	740	630	39	62	59.0	25.0	2.0	1.9
9	Bistritsa Nadvor. – Pasechna	482	1000	44	81	84.1	24.6	1.5	1.0
10	Ribnik – Ribnik	159	830	36	76	111	32.0	2.0	2.1
11	Cheremosh – Usteriki	1500	1100	43	83	93.0	25.5	1.8	1.4
12	Bely Cheremosh- Yablonitsa	552	1200	43	53	23.3	30.4	2.0	1.8
13	Pinie – Poliany	166	530	36	59	63.9	36.6	1.5	1.5
14	Stera – Zniatzevo	224	300	49	55	12.2	32.8	1.8	2.3
15	Udz – Zarechevo	1280	560	54	64	18.5	26.2	2.3	1.8
16	Liuta – Chernogolova	169	700	33	55	66.7	22.7	1.7	1.8
17	Goliatinka – Maydan	86.0	790	41	51	24.4	23.6	1.8	1.5
18	Repinka – Repino	203	780	49	54	10.2	29.7	1.7	1.8
19	Tisa – Delovoe	1190	1000	53	77	45.3	24.2	1.7	1.3
20	Chernaia Tisa – Belin	540	1000	43	90	109	22.9	1.9	1.4
21	Shopurka – Kobiletska Polya	240	1000	34	63	85.3	27.1	1.7	1.4
22	Teresva – Ustjgorna	572	1100	35	79	126	31.5	2.1	2.0
23	Teresva – Dubovoe	757	1000	51	55	7.8	19.7	2.3	1.8
24	Monranka – Russkaya Monr.	214	1100	38	63	65.8	28.1	1.9	1.8
25	Rika – Nizhniy Bistry	781	780	43	55	27.9	22.9	2.0	2.2
26	Pilipetz – Pilipetz	44.2	820	54	87	61.1	23.9	1.8	1.0
27	Bordgava – Shalanki	150	770	37	55	48.6	27.2	1.8	2.8
28	Prut – Tatativ	366	1000	41	81	97.6	25.3	1.9	1.2
29	Tisa – Vilok	3140	760	46	63	37.0	30.9	2.1	1.9
30	Latoritza – Svaliava	680	700	44	52	18.2	32.6	1.5	1.4
31	Kosovskaya – Poliana	464	540	42	52	23.8	27.0	1.6	1.6

years instead of 43 years. An average error of restoration is 28.3% and the contribution of an error of restoration to natural fluctuations was about 30%. The average ratio of errors obtained on the independent and dependent data is equal 2 as for maximum discharges. In the same time the number of effective restored time series is a little bit more: 36 instead 31 for maximum discharges.

The efficiency of restoration has been analyzed too for gauge sites, in which the data of observations were finished in 1987–1988. These sites were considered, as closed sites and it was estimated as far as better or worse data in these sites can be restored in comparison with others. Such time series were 24, but from them only 6 completely answered to conditions of "closed sites", since for them the information was restored only from 1988–87 to 2000. In Table 3 are given the averaged main indexes of efficiency for all time series, for 24 sites and separately for 6 sites, which were considered as "closed sites". As it is seen in table there are no differences between the results obtained for all time series and for "closed site".

6.2 *Example 2*

The second example of application is connected with restoration of total inflow in the lakes Pesivo and Udomlia which are used for water supply and as a cooling reservoir of outflow from Kalininskaya nuclear power station (the Central part of Russia). The inflow in the lakes is measured

Table 2. Indexes of efficiency of restoration for effective time series of depth of rainfall floods in Carpathians.

No.	River – site	A, km^2	H, m	n	N	$\Delta n'$ (%)	$\sigma_{\varepsilon_{mean}}$ (%)	K_1	K_2
1	Dnestr – Strelky	384	620	53	64	20.8	29.8	1.9	1.1
2	Dnestr – Sambor	850	570	55	58	5.5	22.6	1.9	1.0
3	Tismennitsa – Dorogobich	250	390	60	61	1.7	23.6	1.6	0.8
4	Striy – Matkov	106	860	46	57	23.9	17.7	1.8	2.2
5	Slavska – Slavskoe	76.3	860	47	54	14.9	20.2	1.9	1.9
6	Golovchanka – Tukhly	130	810	46	54	17.4	15.8	1.9	2.3
7	Luzdanka – Goshev	146	660	50	54	8.0	27.0	1.9	1.0
8	Lukva – Bondarov	185	480	47	54	14.9	24.7	1.5	1.9
9	Bistrisa – Solovinskay Guta	112	1100	52	57	9.6	22.7	2.0	0.8
10	Striy – Zavadovka	740	630	39	54	38.5	30.2	1.9	1.0
11	Striy – Turka	897	780	45	84	86.7	21.9	2.0	1.3
12	Bistritsa Nadvor. – Pasechna	482	1000	44	79	79.5	28.5	1.7	0.9
13	Yablonika – Turka	136	690	35	59	68.6	25.3	2.0	1.1
14	Seret – Starodginetz	672	590	48	79	64.6	24.3	1.8	0.8
15	Prut – Yaremcha	597	990	51	79	54.9	25.3	1.8	0.6
16	Prut – Chernovtzy	6890	450	89	89	–	–	1.8	–
17	Kamenka – Dora	18.1	870	55	77	40.0	30.8	1.7	0.7
18	Cheremosh – Usteriki	1500	1100	43	78	81.4	29.5	1.8	0.9
19	Cherny Cheremosh – Verhov	657	1200	43	76	76.7	30.3	1.8	0.9
20	Pinie – Poliany	166	530	36	54	50.0	32.7	1.8	1.0
21	Udz – Zhornava	286	670	49	55	12.2	24.5	2.0	1.0
22	Udz – Veliky Berezniy	653	620	34	54	58.8	17.0	2.0	2.8
23	Liuta – Chernogolova	169	700	33	55	66.7	14.7	1.9	1.5
24	Luzdganka – Neresnitza	149	770	33	55	66.7	29.2	1.7	1.1
25	Goliatinka – Maydan	86.0	790	41	54	31.7	31.0	1.8	1.2
26	Rika – Verhniy Bistry	165	920	43	55	27.9	22.6	1.5	1.0
27	Rika – Nizhniy Bistry	781	780	16	55	244	14.6	2.0	1.3
28	Rika – Khust	1130	680	48	55	14.6	21.5	1.4	1.1
29	Pilipetz – Pilipetz	44.2	820	44	54	22.7	39.2	1.7	1.0
30	Latoritsa – Mukachev	1360	570	54	55	1.9	46.0	1.9	0.9
31	Bordgava – Shalanki	150	770	37	55	48.6	29.6	1.8	1.0
32	Prut – Tatativ	366	1000	41	83	102	25.6	1.7	0.8
33	Studeny – Nighny Studeny	25.4	800	41	81	97.6	23.8	1.5	1.5
34	Latoritza – Svaliava	680	700	44	54	22.7	29.3	1.4	1.0
35	Kosovskaya – Poliana	464	540	18	67	272	23.0	2.0	1.0
36	Tereblia – Bovtzary	435	970	15	61	307	25.1	1.8	1.1

Table 3. Comparison of efficiency indexes for all gauge sites and for "closed sites".

	Maximum flood discharges				Depth of floods			
Number of sites	$\sigma_{\varepsilon_{mean}}$ (%)	K_1	K_2	% of effective sites	$\sigma_{\varepsilon_{mean}}$ (%)	K_1	K_2	% of effective sites
77	28.0	2.4	2.6	50	28.3	2.3	2.5	60
24	26.5	2.5	2.6	50	29.0	2.3	2.4	70
6	25.0	2.5	3.0	50	29.6	2.3	2.4	83

in the forth gauge sites with period of observation from 5 to 30 years. This period is not allows to calculate of design annual and monthly discharges with a high accuracy therefore the restoration of long term time series has been fulfilled. For this aim all three approaches described in 3.2–3.4 have been applied and long-term time series in 18 site-analogues have been chosen. Time series of annual and monthly inflow have been extended and their joint period became from 1882 to 2000. At the first spatial models have been applied which connected data in years of observations with data in other years. Two sites of inflow have been used with period of observations in 5 years: the River Khomutovka – site Satina Gorka and the River Sjucha – site Kamenka. Simple regression equations which have applied for restoration have a correlation coefficient about 0.80 – 0.98. On the basis of these relationships the modulus of flow have been restored in January–June for the River Sjucha – site Kamenka and in January–March for the River Khomutovka – site Satina Gorka and for period 1931–2000. Further, using the restored data of monthly water discharges the equations of multiply regression for long-term period has been developed between time series of inflow and historical time series in sites-analogues. Coefficients of correlation varied from 0.70 to 0.97 in general and only in same cases have been equal about 0.7 for the period of restoration 1881–1890. Thus way the monthly runoff has been restored for period 1931–2000 with higher correlation coefficients and smaller standard errors than for period 1881–1930. It is explained that for period 1881–1930 only two analogues could been used: the River Tihvinka – site Goreluha and the River Volga – site Staritsa.

The other way of restoration has been applied because an efficiency of restoration for period 1881–1930 not always satisfied to conditions (6). In this way the restored data have been calculated on the basis of the relationships for whole period of restoration with two analogues only: the River Tihvinka – site Goreluha and the River Volga – site Staritsa. Coefficients of multiply correlation changed also from 0.70 up to 0.92 and only for three months (April, July and September) were a little bit below.

To chose, which restored data to use in the further computations of inflow the time series of outflow from the lakes (the River Siezha – site Stan with $A = 400\,km^2$) has been restored too for the period 1881–2000. Correlation coefficients of the equations which have used for restoration of monthly runoff are varied from 0.7 (September) to 0.87 (March). Two ways of restoration as for inflow have been applied too: different site-analogues for periods 1881–1930 and 1931–2000 and the same site-analogues (the River Tihvinka – site Goreluha and the River Volga – site Staritsa) for whole period. For a final choice of inflow the correlation coefficients between restored by different methods inflow and outflow have been calculated. Taking into account the correlation coefficients, the restored data by the second way for January–May, July, September, December have been accepted and the data restored by the first way have been accepted for other months. Monthly values for January 1882, September–December 1882 and August 1881 have been obtained on the basis of equation (9) with correlation coefficients 0.71–0.79. Annual inflow in 1881 has been obtained as a sum of the restored monthly volumes of inflow.

7 CONCLUSION

System of the methods has been developed for restoration of long-term time series of different hydrological characteristics, which are used in water supply management. The system used a three main ways of restoration, which is based on the main regular properties of hydrological processes: relationships with the particular analogues during long-term period (time synchronous), regional relationships between data in different years (time-space synchronous) and the relationships between an average hydrograph and a hydrograph of the particular year as well as between different hydrological characteristics at the same site of observations. Some indexes for assessment of efficiency of restoration for dependent as well as for independent data have been developed. Two examples illustrate an application of the suggested methods for different hydrological characteristics (maximum, monthly and annual runoff) and for different regions (the Carpathians mountain region and plane region of the Central Russia).

REFERENCES

Beshelev, S.D. & Gurevich, F.G. 1980. *Mathematical-statistical methods of expert estimating*, Moscow, Statistics.

Draper, N.R. & Smith, H. 1970. *Applied regression analysis*. John Wiley and Sons, Inc., New York, London, Sydney.

Kazakevich, D.I. 1977. *Principles of theory of random functions and its application in hydrometeorology*. Leningrad, Gidrometeoizdat.

Kendall, M.G. & Stuart, A. 1970. *The advanced theory of statistics*. Vol. 2. Inference and relationship, Charles Griffin & Company limited, London.

Lobanov, V.A. & Lobanova, H.V. 1999. Trends in Cold Climate Characteristics in *Urban drainage in specific climates. Cold climate*. UNESCO, pp. 171–199.

Recommendations on restoration of river runoff time series and their parameters for long-term period. 1979. Leningrad, Gidrometeoizdat.

Rozhdestvensky, A.V. & Lobanova, A.G. 1991. Using of data of short-term hydrological surveys in river runoff computations. *Meteoroly and Hydrology* (in Russian), No. 12, pp. 84–91.

Sachs, L. 1972. *Statistische auswertungsmethoden*. Springer-Verlag, Berlin, Heidelberg, New York.

Wall, D.J. 1987. Flood peak estimates from limited at-site historical data. *J. Hydraul. Eng.*, Vol. 113, No. 9, pp. 1159–1174.

Extracting information from urban water network malfunction records with fuzzy inference rules

L.S. Vamvakeridou-Lyroudia
Department of Water Resources, Hydraulic & Maritime Engineering, National Technical University of Athens, Greece

ABSTRACT: In this paper malfunction records of a water supply and distribution network have been used to extract information for future optimal partial renewal and expansion. Instead of statistical analysis, which could not be carried out, due to the records' short time length, fuzzy set theory has been applied, to deduce fuzzy inference rules, characterizing "replacement desirability" for each pipe, such as IF [pipe condition] IS [a] THEN [replacement desirability] IS [HIGH], both the threshold [a] and definition [HIGH] derived by membership functions. The object was to include the results in the definition of augmented (or reduced) nominal cost of each pipe alternative (sub-string) at the genetic algorithm applied to the network. Real "raw" malfunction data have been collected from the city of Halkis, in Greece. Out of the various types of malfunction reports that existed, pipe bursts and hydrometer failure and replacement have been the more suggestive ones, related to pipe coverage depth, pressure, pipe material, street traffic, diameter, branched parts and multiple pipes coexisting in parallel in the same trench.

1 INTRODUCTION

Partial renewal and expansion of any existing urban water supply and distribution network is generally a complex design optimization problem, usually faced using genetic algorithms (GAs), multiobjective optimization or various other similar intricate mathematical methods. The problem becomes harder if limited funds are involved. In this case, the city water company should decide about priorities, in order to benefit the most from any pipe replacements and/or to avoid the worst problems that have occurred in the past or are likely to occur in the near future.

For many years, reliability assessment and optimal scheduling of pipe replacements is a much discussed technical problem, traditionally approached by statistical methods (Mays, 2000).

Network malfunction records may be used in order to help decide about priorities and/or high-risk network elements, which have to be replaced prior to others. Extracting this kind of information out of malfunction records involves various data mining techniques, apart from the classical statistical approach (Savic et al., 1999). Classification and decision trees have been previously applied to predict e.g. the risk of bursts for pipe mains (Babovic et al., 2002).

A usual problem concerning malfunction records is the short period of observation records. Although statistical analysis can be applied to such cases (Mailhot et al., 2000), the problem of only short, incomplete malfunction data being available remains.

Another problem concerning analysis of malfunction data is locality, namely that every analysis approach tends to focus on local data and conditions, which differ greatly from one country to another, so that development of generic models is hard and most approaches focus on case studies only. For instance analysis of breaks for the Copenhagen network using Bayesian networks (Babovic et al., 2002), sets the depth of the pipes to 1.4 m, whereas from the author's own collection of break records in Greece, only 5% of the pipes satisfy this depth condition. On the other hand, pipe age, a critical replacement factor in many European and North American water networks, is non-assuming in most Greek water networks, where the average pipe age seldom exceeds 20–30 years.

Finally, malfunction data analysis in most cases is limited only to pipe breaks. Other kinds of events, that cause the water company technicians to intervene in order to perform repairs, such as hydrometer failure, need for the pipe to be horizontally displaced, new connections and extensions etc, are usually ignored. It is interesting to analyze all these events, so as to detect any existing patterns. For instance hydrometer failure tends to appear more frequently in some streets or areas, so it could be related to pressure zones or traffic conditions.

In this paper, case study are real "raw" malfunction and intervention data have been collected from the water supply and distribution network of the city of Halkis, a city of about 80000 inhabitants, 90 km north of Athens, in Greece for two years. The time period is so short, that no reliable classical statistical analysis can be performed. However, by closely screening and studying the data, patterns of malfunction and/or pipe failure tend to appear.

Due to limited information, the approach chosen is a soft one. No prediction models are attempted. Fuzziness is introduced as a technique leading to pattern classification. Therefore, based on the malfunction data available, the objective is to classify as many network pipes as possible (for some pipes there are no relevant data available, so that no classification is possible) into three groups, according to "replacement desirability", linguistically described as [HIGH], [MEDIUM] and [LOW] respectively.

This linguistic tag, if added to every pipe of the network, can be included in the design optimization process, as shown in the next chapter.

2 DESIGN OPTIMISATION WITH GA INCLUDING PIPE REPLACEMENT

Mathematically, any water supply and distribution network is assumed to be a system, consisting of elements (pipes, pumps) whose diameters and curves respectively are yet unknown and nodes, simulating consumption points, source points (e.g. reservoirs, tanks) and junctions. Junctions and pipes may also form loops. For a water distribution network to operate successfully, both pressure (or energy heads) at every node and velocities at each pipe must not exceed preset upper and/or lower bounds, for any water demand scenario (network loading) imposed to the system. Those requirements consist the system constraints.

Given the desired topographical layout of a water distribution network, design optimisation aims at defining the "best" or optimal solution out of all existing alternatives, defining proper diameters for each pipe, so as to minimize the overall system cost, taking into consideration the system constraints.

For more than a decade genetic algorithms (GAs) have been applied for fluid (water and gas) distribution network optimisation. These include standard GAs (Dandy et al., 1996) and many variations, such as structured messy genetic algorithms (Walters et al., 1999), where GA has been used for multiobjective optimisation. GAs have also been combined with fuzzy reasoning, the latter being used both for the evaluation, ranking and selection procedure of the GA and for constraint handling (Vamvakeridou, 2001; Vamvakeridou, 2002).

Design optimisation becomes harder for an existing network in need of improvement and renovation, combined with expansion. In this case, more constraints are added to the system: some pipes should be retained (even if it is hydraulically awkward to do so), some may or may not be replaced (the link cost should be modified accordingly to include this possibility) or perhaps a second pipe may be added in parallel with the old one. Moreover, there are frequently more than one loading conditions to consider, each quite different from the other. In such cases, it is often difficult to find even one fully feasible solution, let alone the optimal one.

Besides, in such cases, replacement of existing problematic pipes is not the primary goal, as it happens with pure maintenance models, where for instance the objective is to replace a predefined percentage of the network pipes every year, meaning that the worst risky pipes have to be somehow selected and replaced. Within the partial expansion and renewal optimization problem, adding and modifying pipes aims primarily to improve the overall hydraulic performance of the network, because usually there are serious shortages, as far as pressure deficiencies, or tank supply are concerned. Therefore any optimisation algorithm would tend to retain existing pipes, especially large ones, if hydraulic performance is not affected, because replacement cost is high, while retaining them adds no cost to the objective function.

Any GA produces a population of random solutions (strings), which gradually evolves to better ones, as generations proceed. Each solution is assigned a cost, referring both to the actual cost of the pipes and to penalties for constraint violation. Probability of selection and survival for the next generation for every string is based on cost evaluation and ranking. Using fuzzy reasoning with GA, every computational element (pipe) is assigned a fuzzy cost evaluation (Vamvakeridou, 2001; Vamvakeridou, 2002), which can be modified accordingly, using the fuzzy linguistic tag of replacement desirability, previously computed for each pipe, according to conditions.

Therefore, the main scope of this paper, which is extracting information from malfunction data using fuzzy inference rules, aims not at prediction but at evaluation of each solution occurring during optimization by GA, using fuzzy reasoning. Thus, during the optimisation process, if a certain string alternative replaces a higher risk pipe, instead of a lower risk one, it is assigned an improved fuzzy fitness value, increasing probability for selection for the next generation.

3 FUZZY INFERENCE RULES

3.1 *Fuzzy definition of replacement desirability*

A fuzzy set is a set with not clear boundaries. Let X be the universal set. A fuzzy set A in X is defined by its membership function $\mu_{x,A}$ denoting the membership of each $x \in X$ to the fuzzy set A. Usually the range of values for membership functions is between 0 and 1, inclusive. So, the nearer the value of the membership function to 1, the higher the grade of membership of x to A, and vice versa. The set of all $\{x \in X, \mu_{x,A} \neq 0\}$ forms the support of fuzzy set A. Membership functions may be either linear or non-linear. In case of a membership function being a linear symmetric triangle, the relative fuzzy set is called a fuzzy number (Cox, 1994).

Suppose that a human expert, with a limited budget in hand, assesses the condition of the pipes of a water supply and distribution network. Depending on the general condition of a pipe, on malfunction records and/or similarities to other pipes for which it is known that a malfunction "history" exists, the expert might express that a specific pipe "had better be replaced by a new one". For other pipes the overall opinion might be "it could be replaced, if there are enough money, but not necessarily as priority", while for others the verdict would be "good condition, should not be tampered with yet". In fuzzy linguistic terminology, these three opinions form the distinction base for three fuzzy sets describing "replacement desirability": [HIGH], [MEDIUM] and [LOW], respectively. Instead of "replacement desirability", the expert might distinguish the pipes in three fuzzy sets as [HIGH_RISK], [MEDIUM_RISK] and [LOW_RISK] respectively. In this case consider risk being defined as a composition (fuzzy or crisp) of "consequence of a hazard" and "possibility of a hazard" (instead of the conventional "probability of a hazard", which refers directly to statistical analysis alone) (Ross, 1995).

Let suppose, now, that the expert grades the condition of each pipe by a variable $x \in X$, $X = [0,1]$, the lower and upper limit of the universal se arbitrarily defined as 0 and 1 respectively for convenience. The three fuzzy linguistic sets [HIGH], [MEDIUM] and [LOW] replacement desirability, are defined as shown in Figure 1. In fuzzy sets terms the [MEDIUM] fuzzy set is the fuzzy number [0.5], the [LOW] fuzzy set is the fuzzy inequality $x > [0.8]$, and the [HIGH] fuzzy set is the fuzzy inequality $x < [0.2]$, the brackets ([ ]) differentiating between actual numbers (crisp) and fuzzy ones.

An aggregated membership function is estimated for any grade x, $x \in X$ by the fuzzy union of the three sets, as follows:

$$\forall\, x \in X\,,\ \mu_x = \mu_{LOW \cup MEDIUM \cup HIGH} = \max\{\mu_{x,\,LOW}, \mu_{x,\,MEDIUM}, \mu_{x,\,HIGH}\} \tag{1}$$

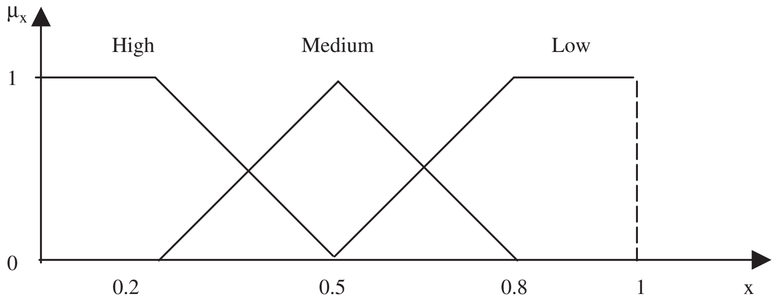


Figure 1. Fuzzy set supports for linguistic denomination of replacement desirability.

Thus the replacement desirability is [LOW] if $x > 0.65$, [MEDIUM] if $0.35 < x < 0.65$ and [HIGH] if $x < 0.35$. Taking into account that x represents the overall grade or condition of the pipe, discussed in the next chapter, the final inference rule applied for any pipe is:

If <condition> is [0.5] THEN replacement desirability is [MEDIUM]

If <condition> is less than [0.2] THEN replacement desirability is [HIGH]

If <condition> is greater than [0.8] THEN replacement desirability is [LOW]

In case of missing data, so that no grading can take place, the overall condition is considered to be [MEDIUM].

3.2 *Fuzzy definition of overall pipe condition*

It now remains to define the overall pipe condition, so as to grade it. Defining the overall condition of a pipe involves a series of factors. Suppose that the expert knows beforehand, that there are only two relevant factors for "pipe replacement desirability", e.g. small coverage depth and high (or fluctuating) pressure. For instance if coverage depth is more than 1.0 m, the pipe may be considered "safe" (membership function value $\mu_{DEPTH} = 1$ to the fuzzy set of "safe coverage depth"), whereas for smaller depths, μ_{DEPTH} gradually decreases. The same stands for pressure. If pressure is less than, say, 60 m, the pipe is assigned a membership function value $\mu_{PRESSURE} = 1$, to the set of "safe pressures", decreasing rapidly ($\mu_{PRESSURE} < 1$) for higher pressure.

Overall pipe grading is carried out by aggregating the two fuzzy sets, by a proper operator (AND), estimating the overall membership function value for both safe depth and pressure. This usually means the fuzzy intersection of the two fuzzy sets (the concept may accordingly be extended to more than two fuzzy sets, or factors):

$$\forall\, x \in X\,,\ \mu_x = \mu_{DEPTH \cap PRESSURE} = \min\{\mu_{x,\,DEPTH},\, \mu_{x,\,PRESSURE}\} \tag{2}$$

The combined membership function value will be used for grading the pipe and defining the [HIGH], [MEDIUM] or [LOW] replacement desirability, as shown in the previous chapter.

Fine-tuning and sophisticated simulation of the fuzzy sets is achieved by introducing non-linear membership functions for each factor and hedges (Cox, 1994) for intensifying or diluting them.

3.3 *Decision trees and fuzzy relations*

Next comes determination of the relevant factors involved in pipe grading. Many techniques can be used for this purpose, one of them decision trees. Decision trees, manually driven or automatically formed, Bayesian or deterministic, are a well-known approach to data mining and general knowledge discovery (Savic & Walters, 1999). They can be used either for descriptive or for predictive purpose. Usually part of the existing data is used for forming the tree, and the rest is used for testing.

In this paper the structure of a decision tree, including fuzziness is shown in Figure 2. Starting from a root, that is a basic condition that involves a primary decision, the tree proceeds by gradual

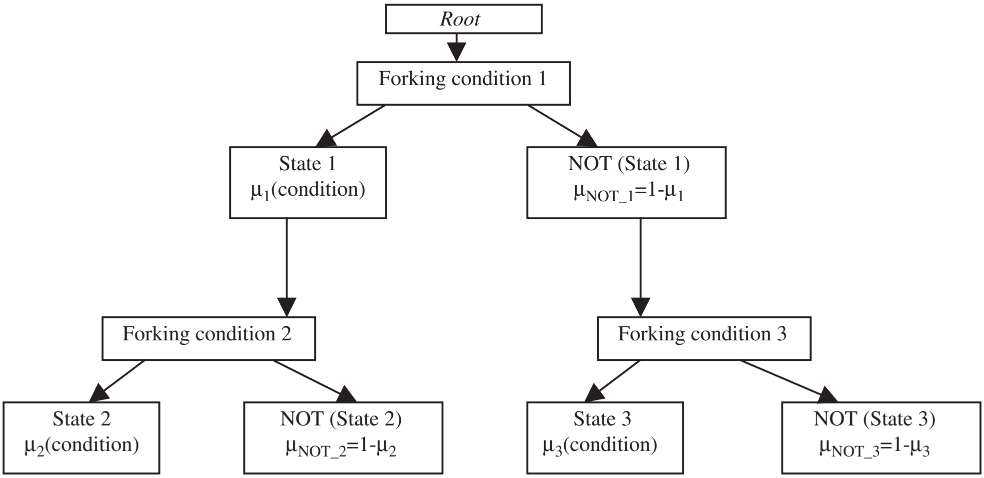


Figure 2. Schematic layout of a fuzzy decision tree.

forks to end foliage. Suppose that the root and the first forking condition is an essential pipe breakage cause (e.g. small pipe coverage depth, less than [1.0] m). According to records, the pipe, because of coverage depth, belongs to the set of fuzzy "safe" state 1 with a membership function value μ_1 ($0 \leq \mu_1 \leq 1$). Accordingly, it belongs to the not safe state, with the complement of the previous fuzzy set, namely $1 - \mu_1$. The smaller the coverage depth is, the smaller μ_1 is, so that the pipe is assigned a small grade in the universe of "safety". Next forking condition (2) involves another parameter, e.g. pressure, and respectively membership function values μ_2 ($0 \leq \mu_2 \leq 1$) for "safe" state and $1 - \mu_2$ for "not safe" state. Similar approach stands for forking condition 3.

According to the tree structure, there are 4 end foliages, and therefore 4 possible routes, connecting the root to them. According to fuzzy relations (Ross, 1995), one common way of estimating the final connection is the max-min operator:

$$\forall\, x \in X\,,\ \mu_x = \max\{\min(\mu_1, \mu_2), \min(\mu_1, \mu_{NOT\_2}), \min(\mu_{NOT\_1}, \mu_3), \min(\mu_{NOT\_1}, \mu_{NOT\_3})\} \qquad (3)$$

Instead of the max-min operator, the max-product operator might be used. It is also worth remarking, that each fuzzy set and its complement are not mutually exclusive, so that all four routes must be included at the computations.

Once the decision tree has been formed it can be used for inference rules, such as: IF the pipe depth IS [a] AND the pressure is [b] then replacement desirability is [HIGH]. Accordingly it can be used in the reverse sense in fuzzy logic for determining the critical condition of a pipe (e.g. IF (the user wishes) the replacement desirability to be [LOW] AND the pressure is [b] THEN pipe depth IS (or should be, at least) [a]. Discussion on the use on these inverse inference rules is beyond the scope of this paper (Mendel, 2000).

4 CASE STUDY

4.1 *The city of Halkis water network and the "raw" malfunction records*

The city of Halkis (about 80000 inhabitants – 90 km north of Athens in Greece) is built partly on mainland Greece and partly on the island of Evia, divided by the straight of Evripos, extending both on hills and flat areas near the sea. Water comes to the city from two points far apart, one in mainland Greece and the other on the island. The water supply and distribution network is presently divided in 20 zones or sub-networks. Some of them operate independently, supplied by one ore more tanks, while others are interconnected. This is due partly to topography (surrounding hills) and partly to the gradual way the network has been constructed, the water network (and the city) continuously expanding to new neighbourhoods.

As it happens in most small and medium sized towns and cities in Greece, the older water pipes are less than 60 years old (less than 7% – mostly within the old city), the average pipe age being 25–30 years, while a considerable part of the network (more than 25%) has been on operation for less than 10–15 years. The city and network expansion is continuous, new pipes and zones being constructed all the time. Only in 2002 works have started in 3 small new zones and another zone is due to start construction shortly.

By EU standards, the water network cannot be considered "old" or ageing. Nevertheless, it is a complex and problematic one, as far as the network smooth operation is concerned. This is due mainly to the fact that the city water network has never been studied as a whole. Even older studies that exist are out-of-date because of the rapid changes and expansions. On the other hand the Halkis water company has always been obliged to operate "on the edge", taking care of growing needs, under chronic time pressure, with limited funds.

In 1996 the city of Halkis Water Company-DEYAX Arethusa, decided to improve the city water supply and distribution network. At the time not even full precise maps of the network existed, let alone a mathematical model of it. In the course of the following years, fully digitised maps of the network and the relative mathematical simulation model have been painstakingly step-by-step completed, by the engineers and technicians of DEYAX, with the help and assistance of the author.

Although malfunctions leakage and breaks occurred all the time, no records of any kind existed. So, at the time, it was agreed that any intervention or event concerning the water network, be it pipe burst, pipe replacement, hydrometer replacement, malfunction report or complaint, would be reported by the technicians in special forms and entered in a database.

In this way, all "intervention events" at the water network have been reported from 1996 to 1998. During this period event records are assumed to be (hopefully) complete, that is no major events have been left out. Naturally, event description sometimes was vague or ambiguous, but conclusions could be made out of them in most cases. After 1998, "event" records are not complete. Some events have been reported, but not all of them. Overall, there exist 931 "raw" intervention events. Out of them only 47 events referred to pipe bursts, while 248 were related to hydrometer, or small diameter pipe replacements due to leakage. The remaining records referred to valve replacements, pipe trench problems, new junctions for expansion, various maintenance works etc.

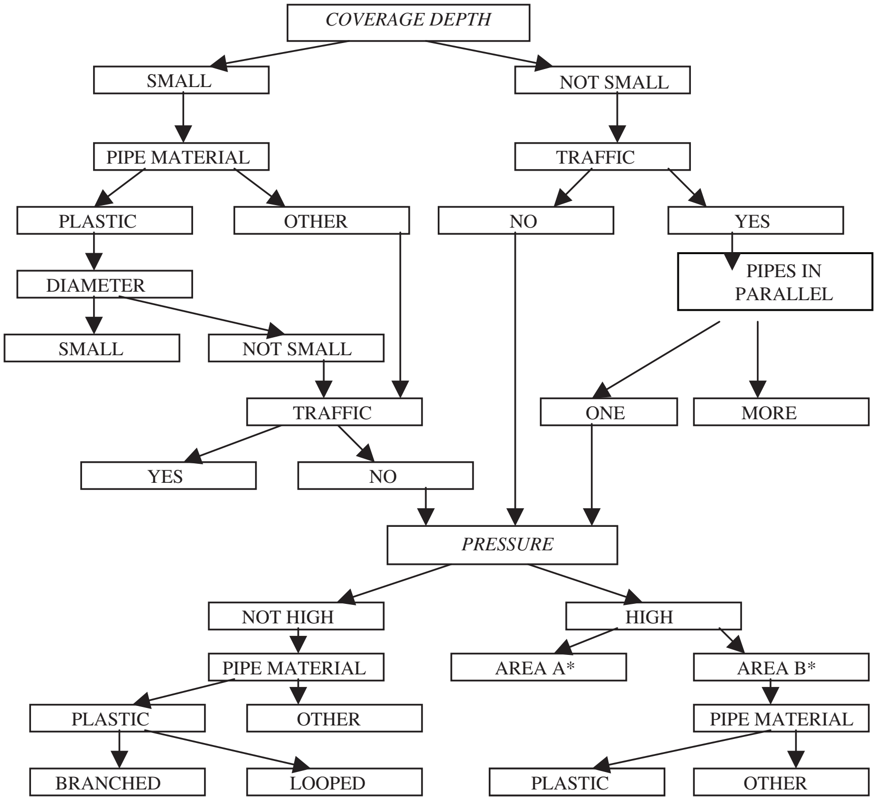
In the meantime digitised maps and the mathematical model of the network have been completed and in 2001, the project of optimal partial renewal of the city water supply and distribution network was assigned as research project at the National Technical University of Athens.

The existence of the mathematical simulation model of the network has been helpful, because it was possible to relate certain (not all) malfunction types and reports with hydraulic parameters, such as pressure and branched (instead of looped) network structure.

Information fields provided by the malfunction records are pipe coverage depth, diameter, material and remarks as to the trench (multiple pipes in parallel) and the position of the pipe (e.g. in the middle of the street or near the pavement) and roughly pipe age (in decades). Traffic information has been added afterwards, by locating the pipes on the map. In the same way, information about pressure and zoning (areas) has been added using the simulation model of the network.

4.2 *Data screening and processing*

By carefully studying the "raw" data, it was assumed that only pipe breaks and hydrometer failure was substantial enough for further analysis. Pipe age seemed to be non relevant, because practically all of the cases involved pipes aged less than 25 years. Concentrating on the rest, it was decided that pipe coverage depth (which was very small, less than 0.5 m in some cases), pressure and traffic seemed to be the most relevant factors for failure. Pipe breaks were divided in two groups, randomly: 30 events were used for calibration and the rest for testing. Random division has been repeated many times, using a different sample each time. Hydrometer failures (248 events) were divided randomly in half, for calibration and testing respectively.



* Areas A and B refer to specific parts of the network, B type areas representing problematic zones

Figure 3. Decision tree for pipe breaks.

Because the number of breaks was small, additional another 30 virtual non-break events have been added for testing, retrieving data from existing non-failure records (extensions, junctions etc). The calibration was considered successful if all (or most) of the testing real breaks were assigned a [HIGH] "replacement desirability" linguistic tag and all (or most) of the others [MEDIUM] or [LOW] tags.

Various roots and structures of the decision tree have been tried. From the first attempts, pipe coverage depth proved to be the most consistent of all roots, seconded only by pressure. The final decision tree for breaks is shown in Figure 3. The tree has actually two roots: starting from coverage depth, many foliages end to pressure check, which actually becomes the root for a second tree. The tree for hydrometer replacement (not shown here because of the limited space of the paper) is similar to the "pressure" part of the tree presented in Figure 3.

After several trials, fuzzy definition for [NOT_SMALL] coverage depth has been achieved by using a S-shaped membership function:

$$\mu_{NOT\_SMALL}(d) = \left\{ \begin{array}{ccc} 0 & for & d \le a \\ 2\left(\dfrac{d-a}{c-a}\right)^2 & for & a \le d \le b \\ 1-\left(\dfrac{d-c}{c-a}\right)^2 & for & b \le d \le c \\ 1 & for & d \ge c \end{array} \right\} \tag{4}$$

where a = 0.30, b = 0.65 and c = 1.00 are parameters and d the pipe coverage depth in m. Equation (4) has also been used for defining [HIGH] pressure, by setting a = 20, b = 40, c = 60 and substituting d for the pipe pressure in m. Accordingly the support and membership function of the small diameter fuzzy set is defined, by points, as follows:

$$SMALL\_D = \left\{ \frac{1}{63,P} + \frac{0.70}{75,P} + \frac{0.30}{80,A} + \frac{0.50}{90,P} + \frac{0.20}{100,A} + \frac{0,30}{125,P} \right\} \tag{5}$$

where P stands for plastic (PVC) pipes, A for asbestos cement pipes and 63, 75, 80, 90, 100 and 125 are the nominal diameters. Definition of the membership function values is based on analysis of the percentage of each pipe diameter to the total network length, as well the number of cases that preented failure.

5 SUMMARY AND CONCLUSION

In this paper malfunction records of a water supply and distribution network have been used to extract information for future optimal partial renewal and expansion. Instead of statistical analysis, which could not be carried out, due to the records' short time length, fuzzy set theory has been applied, to deduce fuzzy inference rules, characterizing "replacement desirability" for each pipe, such as IF [pipe condition] IS [a] THEN [replacement desirability] IS [HIGH], both the threshold [a] and definition [HIGH] derived by membership functions. The object was to include the results in the definition of augmented (or reduced) nominal cost of each pipe alternative (sub-string) at the genetic algorithm applied to the network. Real "raw" malfunction data have been collected from the city of Halkis, in Greece. Out of the various types of malfunction reports that existed, pipe bursts and hydrometer failure and replacement have been the more suggestive ones, related to pipe coverage depth, pressure, pipe material, street traffic, diameter, branched parts and multiple pipes coexisting in parallel in the same trench. Out of all the factors involved, pipe coverage depth was the most critical, whereas pipe age, in the specific case study was irrelevant, because of the small age of the network.

ACKNOWLEDGMENTS

The author acknowledges the support of the Halkis Water Company (DEYAX–Arethusa) who assigned the improvement of the city water supply network as Research Project to the National Technical University of Athens (Feb. 2001–Nov. 2002) and helped with the collection of the network malfunction data. The author acknowledges also the contribution of Ioanna Skorda, former student of NTUA, whose diploma thesis concerned the city of Halkis malfunction data and their relative initial analysis.

REFERENCES

Babovic, V., Drecourt, J-P., Keijzer, M. & Hansen, P.F. 2002. A data mining approach to modelling of water assets. *Urban Water* 4: 401–414.

Cox, E. 1994. *The Fuzzy Systems Handbook*. Cambridge MA, Academic Press Inc.

Dandy, G.C., Simpson, A.R. & Murphy, L.J. 1996. An improved genetic algorithm for pipe network optimisation. *Water Resources Research* 32(2): 449–458.

Mailhot, A., Pelletier, G., Noel, J-F. & Villeneuve, J-P. 2000. Modeling the evolution of the structural state of water pipe networks with brief recorded pipe break histories: Methodology and application. *Water Resources Research* 36(10): 3053–3062.

Mays, L.W. 2000. *Water distribution systems handbook*. McGraw-Hill International Editions.

Mendel, J.M. 2001. *Uncertain rule-based fuzzy logic systems*. USA Prentice-Hall Inc.

Ross, T.J. 1995. *Fuzzy logic with engineering applications*. McGraw-Hill International Editions.

Savic, D.A., Davidson, J.W. & Davis, R.B. 1999. Data mining and knowledge discovery for the water industry. In D.A. Savic & G.A. Walters (eds) *Water Industry Systems*. Vol. 2: 155–163, UK Research Studies Press Ltd.

Savic, D.A. & Walters G.A. 1999. Hydroinformatics, data mining and maintenance of UK water networks, *Anti-Corrosion Methods and Materials* 46(6): 415–425.

Vamvakeridou-Lyroudia, L.S. 2001. Water supply network design optimization using fuzzy reasoning and genetic algorithms. In B. Ulanicki, B. Coulbeck & J. Rance (eds) *Water Software Systems*. Vol. 1: 3–13. UK Research Studies Press Ltd.

Vamvakeridou-Lyroudia, L.S. 2002. Fuzzy reasoning combined with genetic algorithms for water supply network design optimisation. In K.C. Giannakoglou, D.T. Tsahalis, J. Periaux, K.D. Papailiou, T. Fogarty (eds) *Evolutionary methods for design optimization and control*, 442–447, CIMNE Barcelona, Spain.

Walters, G.A., Halhal, D., Savic, D. & Ouazar, D. 1999. Improved Design of "Anytown" Distribution Network Using Structured Messy Genetic Algorithms. *Urban Water* 1(1): 23–38.

Data mining and relationship analysis of water distribution system databases for improved understanding of operations performance

D.M. Unwin, J.B. Boxall & A.J. Saul
Department of Civil and Structural Eng., University of Sheffield, Sheffield, UK

ABSTRACT: Water utility companies in the UK maintain asset databases as a fundamental requirement for system management and understanding. In addition water companies maintain customer service records providing information relating to bursts leakage and water quality complaints. Typically these recorded data sets have been utilised independently as the rigorous integration of such information sources was not feasible due primarily to the method of recording data. With advances in data manipulation, and analysis systems, in particular the integration of geographical information systems (GIS), it is now possible to explore relationships between data. This paper presents details of the techniques that may be employed to rigorously filter data and establish data base associations. Using such associations it is possible to perform advanced analysis to examine relationships between the data. These associations can be used to enhance cost based and performance models associated with sustainability and life cycle analysis. Derivation of these types of relationship and the confidence in the methodologies used in defining them are essential to the water industry, in particular for the development of efficient and effective operation and maintenance strategies. Examples of these enhanced relationships are presented and the implications of the resulting trends discussed.

KEY WORDS: GIS, data associations, performance modelling, water distribution

1 INTRODUCTION

Water utility companies within the UK maintain asset, incident and customer service record databases as a fundamental requirement for system management. Much of this information is stored in independent unrelated databases. With improvements in data handling hardware, software and the development of techniques to interrogate and relate these databases, relationship analysis between these individual data sets can now be undertaken with improved confidence. Providing valuable insight and new knowledge regarding system performance, sustainability, and cost based performance modelling, Schindler and Garrard (1999). Improved confidence in relationship analysis of this kind and the methodologies applied to derive such associations are increasingly attractive to water service providers with respect to the development of holistic strategies regarding distribution system operation and management.

2 BACKGROUND

The recording of asset and incident data sets have typically (historically) been constructed and utilised independently. Incidents such as bursts, leakage, and various customer service complaints

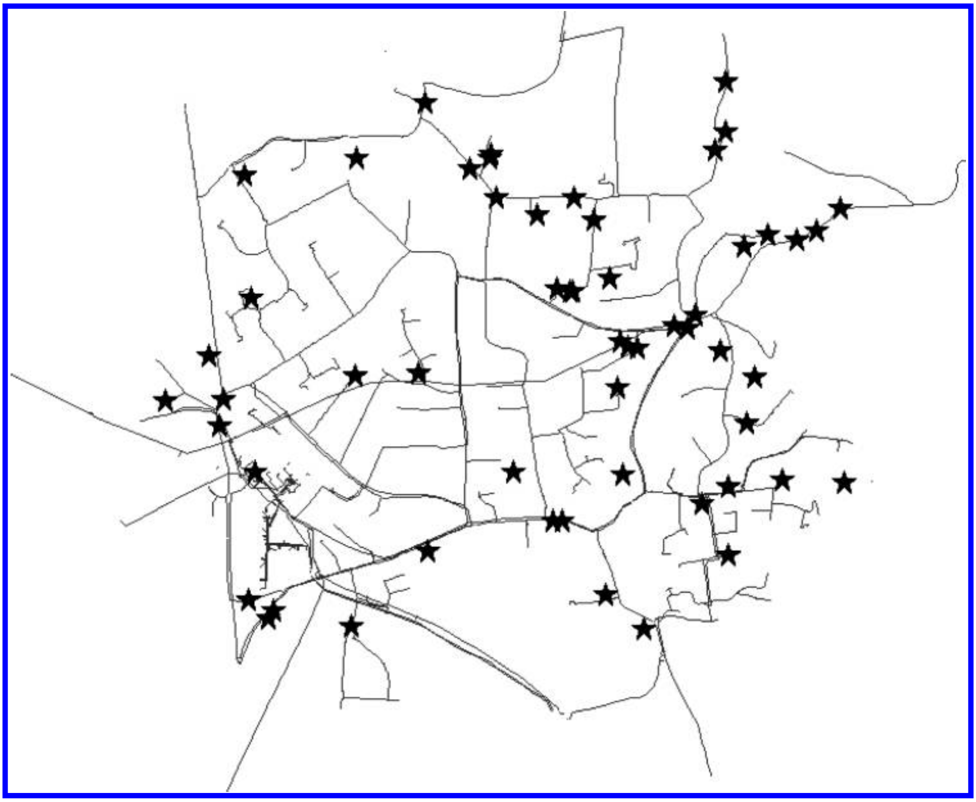


Figure 1. Map section built from four separate databases: 1 assets (lines), 2 bursts, 3 leakage, 4 customer service complaints (2, 3 and 4 displayed as stars).

are not recorded in association with particular assets, but are recorded by means of a geographical location, and are stored in separate, specific databases, Savic *et al* (1999).

Figure 1 shows a part map from a UK water service provider that details burst, leakage and customer service data and covers a seven-year data-recording period from January 1995 to December 2001. The map was built using *four* separate databases. As can be seen only two object types are visible. These are the assets, lines, and the burst, leakage and discolouration incidents grouped together and indicated by stars. This map highlights how asset and event records are typically recorded and stored. There is no distinction between individual asset properties or between bursts, leakage and customer service complaints. Interrogation of such data is very limited by the lack of explicit association between these data sets. Methodologies and software functionality capable of making direct associations are not currently available. However using layering capabilities of GIS software and through the development of proximity search techniques it is now possible to convert the separate incident databases from ones of geographical location only to ones that are linked and provide distinct unique associations between events and individual assets. This allows confident analysis between the previously unrelated databases to be completed to an industry acceptable standard.

3 METHODOLOGY

To obtain valid and useful information and knowledge from data of the form presented in figure 1 it is essential to distinguish between different event types and to accurately associate events with assets, creating pipe level incident data. Advances in data manipulation and interrogation techniques, and the visual display capabilities of GIS, mean it is now feasible to develop relationships between such unrelated data sets.

Confidence in data analysis can be gained by employing a practice of splitting asset and incident data into individual layers and sub-layers that can be presented and searched within GIS software. However incidents (bursts, leaks and customer complaints) are typically recording by geographical location only. Hence events are often at some distance from the asset that they are actually associated with. Proximity based searches may be developed and employed to associate an incident to the closest asset. However in isolation this may be subject to numerous errors because incidents may be recorded as being closer to the nearest asset rather than the asset involved.

In this work the association between asset and incident data sets is established by using information contained within the asset and incident databases, where matching criteria is present. A specifically developed code then matches the incident to an individual asset by proximity analysis based on the matching criteria. Such thematic analysis removes much of the un-certainty due to individual layers and sub-layers of data being considered for analysis individually rather than as a whole. Once associations have been established between the individual groups, suspect data can be removed. Such suspect data is classified as fields containing company specific default values, or un-populated data fields. The number of data fields to be removed depends upon the analysis to be carried out. Experience has shown that figures such as fifty percent removal of data are at times necessary. It is then possible to re-assemble the associated data to form a new complete data set. This can be confidently analysed, subject to final standard quality checking.

Much of the original data may be removed due to suspect data. However such removal is by its nature generally random, therefore the remaining associated data should behave as a representative random sample of the initial whole data set.

This methodology of associating data and then removing suspect data is slower than that of removing suspect data first then establishing associations. However this approach considers all assets and incidents for analysis, before any screening of the data. Therefore uncertainty introduced from any subsequent analysis is removed by the creation of a true representative sample.

4 CASE STUDY

Figure 2 shows the same part map as that in figure 1, but with the map rebuilt after the creation of sub layers corresponding to individual asset data (in this example material types), and to the three individual incident groups, i.e. bursts (circles), leakage (rhomboids) and customer service complaints (triangles). With asset and incident data now split into individual layers, thematic map querying was performed and associations identified. Subject to data quality checks, further analysis by statistical, methods was then undertaken. The results of this analysis are presented below.

To investigate burst occurrence and frequency of specific asset materials and asset ages the asset and incident data mapped in figures 1 and 2 was split thematically according to asset material type and age. This assessment data was then linked to local soil, rainfall and temperature data so as to investigate possible associations between asset performance over the data recording period and external physical conditions. Table 1 provides an overall summary of the main material types and the associated percentage of bursts obtained. Material types labelled “others” consist of materials such as copper, spun concrete etc. For brevity it was decided to group these into a single set as they formed a small percentage of the whole data-set.

From table 1 it can be seen that using the new data association techniques, cast iron constitutes 43.5 percent of known asset material by length, but it accounts for 72.5 percent of all known burst incidents. Prior to the rigorous data associations of this analysis methodology this was an unquantified trend. Hence this performance relationship is of direct relevance and benefit to the deployment of leakage detection and repair strategies.

On the basis that the resulted cast iron constituted the dominant known material type within this data set and accounted for the majority of reactive maintenance expenditure, the cast iron sub set was selected for further relationship analysis. Initially the burst frequency of these cast iron assets

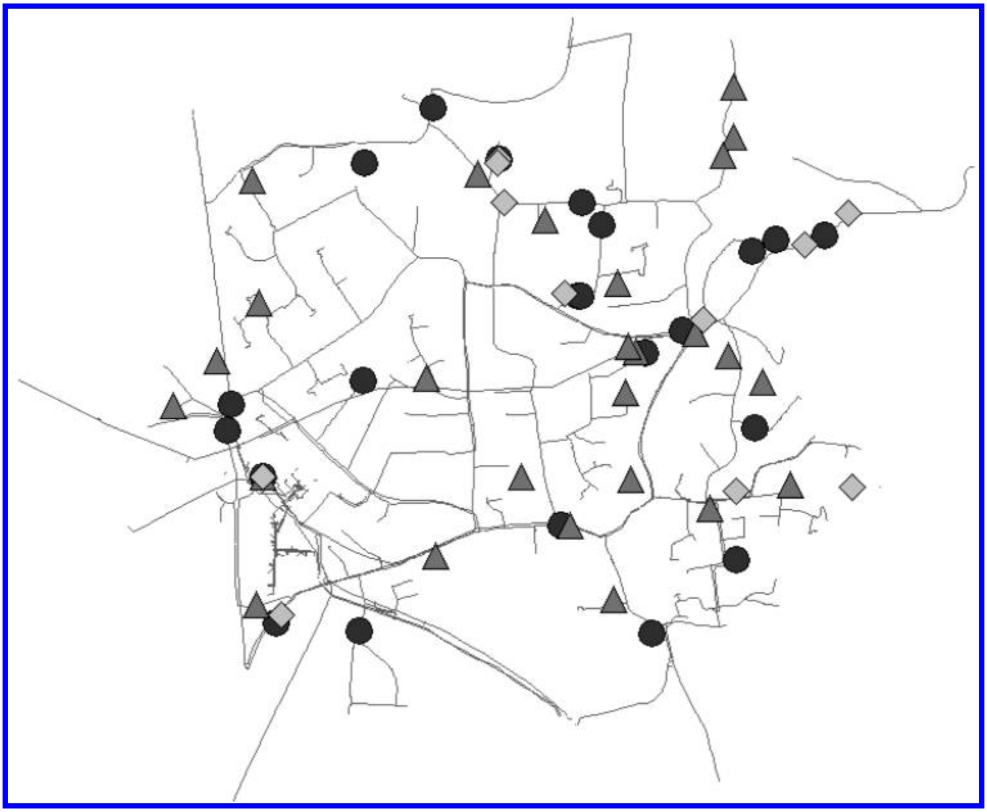


Figure 2. Pipe asset data split into individual material types and incident data displayed as different data layers.

Table 1. Material groups contained within this data set and the percentages of bursts affecting each material group.

Material type	% Of asset database	% Of burst incidents
Cast iron	43.5	72.5
Asbestos cement	30.3	11.4
Plastics	11.3	8.3
Unknown	11.5	6.7
Others	3.4	1.1

was investigated against the age of the asset. This has previously been shown to provide a useful indication of serviceability. For example Skipworth *et al* (2001) showed an increase in burst rate with pipe age, with the form of this relationship dependent on material types.

Using the associated data prepared using the new methodology it has been possible to confidently ascertain the age of the assets that correspond to each burst incident, as presented in figure 3. The figure does not appear to suggest an increase in burst rate with age. It can be seen that the bulk of the cast iron assets that burst during the record period were laid in the period 1900 to 1960. A peak burst frequency may be observed in the late 1800's and this could be associated with casting production techniques that were not as reliable as modern day methods. Similarly subsequent peaks may coincide with instances of post war rehabilitation as well as the mid century housing boom.

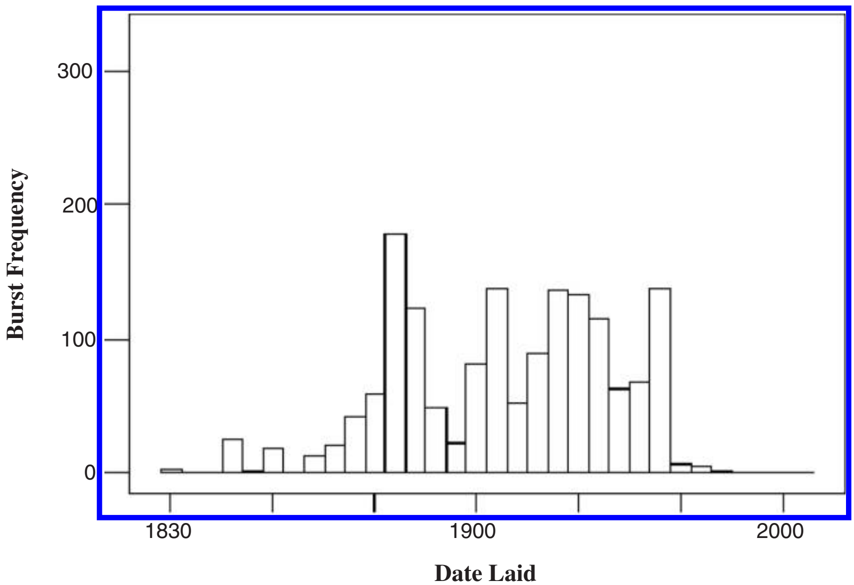


Figure 3. Histogram of date laid for cast iron assets affected by a burst incident.

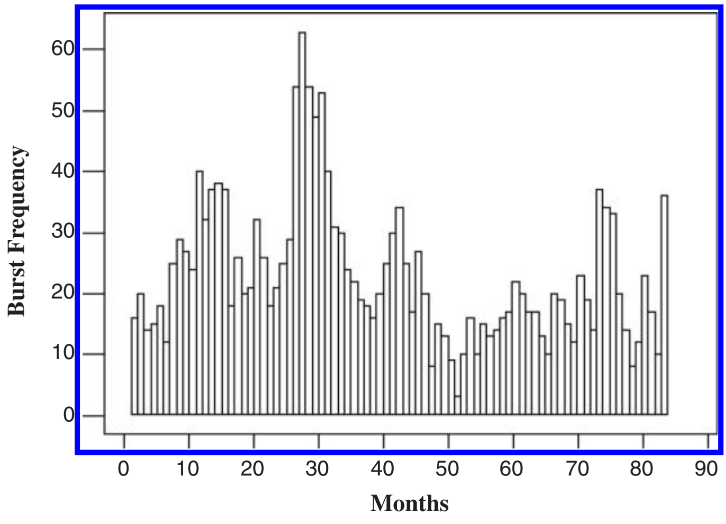


Figure 4. Histogram of monthly burst frequencies for the data recording period (01/95–12/01) for cast iron assets.

This type of association was also shown by Rostum (2000) and it is concluded therefore that manufacturing techniques, consumption and method of construction are all important variables in the prediction of burst frequency.

Figure 4 shows the results of data that was thematically mapped and queried to establish burst incidents that occurred on cast iron mains over the seven-year data-recording period. As can be seen from the figure, although the data has several peaks and troughs, there are some identifiable modes of performance within the data. These modes of data occur at approximately 12–15, 28–32, 42–46 and 73–77 months respectively. When related to monthly rainfall and temperature data

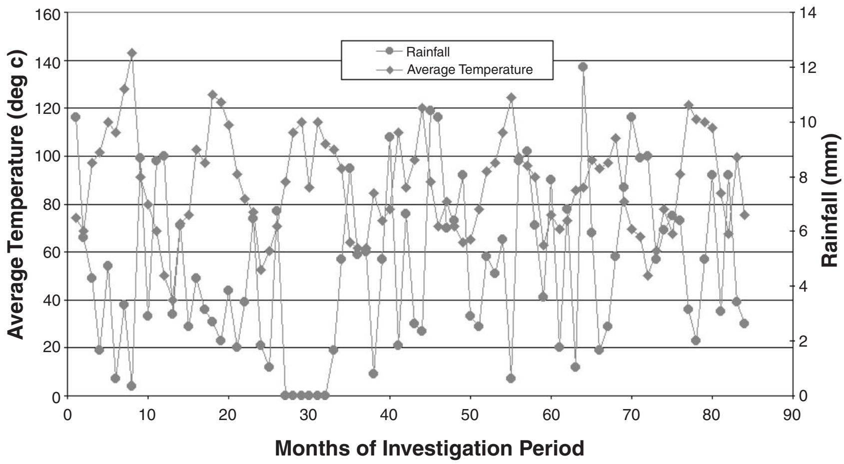


Figure 5. Monthly rainfall and temperature data for the data-recording period (01/95–12/01).

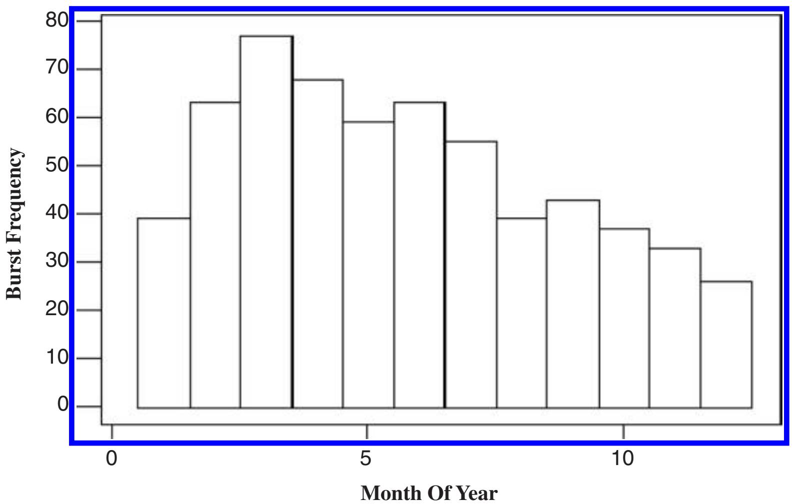


Figure 6. Histogram of burst frequencies on cast iron distribution assets during 1997.

presented in figure 5. These modes show some general correlation in that peak burst frequencies generally occurred at times of high or low rainfall and the extremities of the observed temperature range. Hence in any model to predict burst frequency it is hypothesised that environmental factors need also to be taken into account.

This suggests an apparent relationship between burst rate and environmental conditions, together with the local soil type, (in this case estuarine deposits and softer clays). It may be hypothesised that there is a link (amongst may be others, such as asset age, demand increase etc.) between geological effects (soil heave) due to local weather conditions and burst frequencies of cast iron assets.

As a consequence the relationship between the frequency distribution of burst events throughout a year, in this case 1997, the year that contained the highest burst frequencies, was studied and the results of the analysis are shown in figure 6. It can be see that the burst frequency rises to a

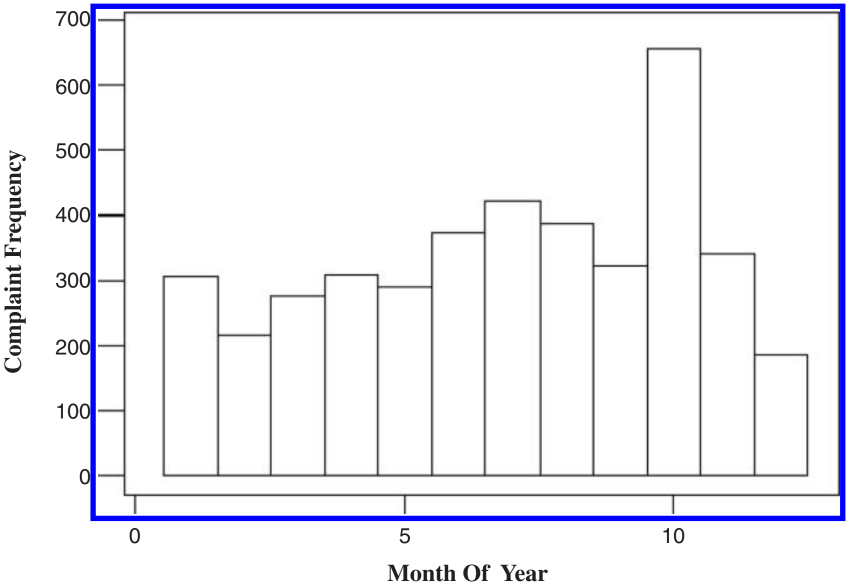


Figure 7. Histogram of customer complaint frequencies for discoloured water during 1997.

peak value around March, which corresponds to the start of a peak temperature and low rainfall event on figure 5 (month 26 of figure 5) before decreasing gradually over the rest of the year. This supports the previous hypothesis that as the local soil dries out, it is subject to heave, and thus results in an increase in burst frequency (assuming no significant other changes).

In respect of customer service complaints, the frequency of complaints for the same period as the burst data of figure 6 is shown in figure 7. In contrast to the histogram for bursts the customer service complaints show a general increase throughout the spring to summer months, with a peak in October. If it is hypothesised that customer service complaints of discoloured water may not be a direct consequence of a mains service failure and the consequent change in the hydraulic regime, but that it may be a consequence of an increase in the demand for water over the summer months. There is a direct link between discolouration and increased demand at the present time. However the October peak is un-explained. Prince 2000 suggests a similar association between change in water demand and discolouration events for distribution systems in Melbourne Australia.

5 CONCLUSIONS

Company databases in an un-manipulated format do not lend themselves to direct analysis to establish any trends in the data. The databases are generally un-related of differing formats and contain many un-populated or company specific default value data fields. Historically this has restricted in-depth data analysis such as may be utilised for performance or cost based modelling.

A methodology for rigorously associating asset and performance databases has been presented. A case study of the type of enhanced relationship analysis that may be undertaken utilising the associated data sets has also been presented. Of specific interest is confirmation that, for an area comprising only 43% cast iron pipes by length, cast iron pipes accounted for 72% of known bursts. The results also suggest that there is relevance between climate conditions and burst rate, and that this relationship could suitably be utilised to schedule leakage detection programmes. Similarly it is possible to examine the potential association between burst rate and customer service complaint.

The confidence gained in results derived from the research carried out to date suggests a change should be made in the practice for incident information recording. It is recommended that an additional single extra data field (the affected assets identification number) should be recorded. This would vastly improve the analysis possibilities of future similar data studies. These conclusions support this recommendation with respect to the associations and performance insights based on the results from a single data set with cast iron pipes.

REFERENCE

Skipworth P.J., Cashman A., Engelhardt M.O., Saul A.J. & Savic D. (2001) "Quantification of mains failure behaviour in a whole life costing approach to distribution system management" *ASCE EWRI conference Orlando, Florida May 2001*

Prince R., Goulter I. & Ryan G. (2001) "Relationship between velocity profiles and turbidity problems in distribution systems" *ASCE EWRI conference Orlando, Florida May 2001*

Rostum J. (2000) "Statistical modelling of pipe failures in water networks" *PhD thesis submitted to the faculty of civil engineering, the Norwegian University of Science and Technology (NTNU)*

Schindler D.F., Garrard T.P. (1999) "GIS as an active management tool in water distribution" *CCWI conference, centre for water systems, University of Exeter September 1999*

Savic D.A., Davidson J.W. & Davis R.B. (1999) "Data mining and knowledge discovery for the water industry" *CCWI conference, centre for water systems, University of Exeter September 1999*

Analysis of district metered area (DMA) performance

S.L. Prescott & B. Ulanicki
Water Software Systems, De Montfort University

N. Shipley
South Staffordshire Water plc

ABSTRACT: This paper presents a few examples from recent work on analysis of district metered areas (DMAs) in a UK water company. The work has been carried out to establish the performance of existing DMAs in terms of leakage levels, head loss and dynamic behaviour. The analysis of each DMA involves three stages – data collection, modelling studies and determination of suitable modifications. Data is collected from several points across the network at 20 second intervals to identify regions of high head loss and sources of dynamic problems. Existing water company models of the DMA are enhanced taking into account new information from the measurements. If problems with stability are identified, a transient model is also constructed to evaluate potential solutions. The study of each area is concluded with a list of recommendations which may include network restructuring or rehabilitation, PRV servicing and optimal pressure supply schedules for leakage minimisation.

1 INTRODUCTION

It is now common practice for UK water companies to split their networks into DMAs to facilitate leakage monitoring and control activities. DMAs are typically small areas (1000–3000 properties) and are closed apart from designated inlets and outlets through which flow is monitored allowing the level of demand and leakage to be estimated. Pressure reducing valves (PRVs) are often used to control the pressure supplied to DMAs and appropriate selection of inlet pressures schedules results in optimised network pressure and therefore less leakage. Pressure control is recognised as a cost effective method to reduce leakage and has been the subject of much research over recent years (Ulanicki et al. 2000, Ulanicka et al. 2001, Prescott 2003).

There is an ongoing debate about operating DMAs on a single or multi feed basis. Multi feed networks are useful to combat excessive head loss in systems and have the advantage of increasing the security of supply. However, such schemes can lead to problems in terms of interacting PRVs, generation of transient behaviour and water quality issues. Also, leakage is more difficult to assess in multi feed areas since they generally have larger properties coverage generating a higher consumption. This will mask any potential leakage within the DMA. Companies therefore prefer single feed DMAs for simplification of control and a better understanding of leakage levels. Consequently, the benefits of multi feed systems are rarely exploited. A thorough understanding of the behaviour of this type of system is required to enable companies to make a more informed decision when selecting the structure of a DMA.

Recently, the authors have carried out a collaborative project to assess the current state of several DMAs and determine optimal pressure schedules. These schedules are determined to minimise pressure across the DMAs and to reduce the effects of any adverse transient behaviour. The analysis and control of each network has been performed using FINESSE – an advanced modelling software package developed by Water Software Systems. The recent addition of a pressure control module to FINESSE provides a tool to generate optimal pressure schedules directly from simulation models.

Three case studies have been selected from the project and will be presented here. The examples cover a range of problems that are typically encountered by water companies and, in this paper, will be referred to as:

- a single input DMA with two target points
- a single input DMA with dynamic problems
- a dual feed DMA where care is required to avoid dynamic problems

This is by no means an exhaustive range of scenarios encountered during the project but gives a flavour of what this sort of work entails. A brief description of the analysis process is given below. Obviously, there needs to be some flexibility in the approach to this type of work since the procedure is governed to a certain extent by what is observed from measurements. However, the analysis of a DMA generally goes through the following stages:

1. Collection of data – inlet flow, PRV inlet and outlet pressure, average zonal node pressure (AZNP), target point pressure, any other suitable/interesting pressures. The pressure measurements were at 20 second intervals and recorded over one week. The presence of dynamic problems can normally be established with this data.
2. Modelling the DMA – a steady state model of the DMA is constructed and calibrated using the data collected and GIS information. Any uncertainties are resolved by collecting more data from the necessary areas. If dynamic problems have been observed, a transient model of the DMA, including the PRV supplying the network is constructed. This is simulated under various conditions (low flow, different pressure set points etc.) to establish the source of these problems.
3. A pressure control scheme for the network is determined from the modelling studies – this will take the form of steady state pressure profiles (time or flow modulated) for leakage reduction and a set of proposals (change of set points, adjustment of PRV speed mechanism etc.) to eliminate dynamic problems.

In the following section, a brief description of the software and models is given. The case studies themselves are described in section 3 and then a few general conclusions are presented.

2 SIMULATION DETAILS

The steady state analysis was carried out in FINESSE – a software package incorporating simulation, demand prediction, pump and valve scheduling, simplification and pressure control. The pressure control module determines inlet pressure schedules to minimise the cost of water in a simulation model. To do this, pressure dependent leakage, represented by a power law, has been incorporated into the model. The parameters of the power law can be set by the user for every node in the network model. This allows different leakage terms to be applied across the network to account for different types of leakage; for example, fixed area bursts or variable area bursts. The user can also set head constraints at each node which provides a mechanism for setting minimum target point pressures. The pressure control module calculates pressure profiles to minimise cost (leakage) while keeping the head within the set constraints at all nodes.

The dynamic modelling was performed using the standard method of characteristics transient model (Wylie & Streeter, 1993) with pressure dependent leakage at each node. This was linked to the behavioural PRV model of Prescott & Ulanicki (2003), given by equation 1.

$$\boxed{\begin{aligned} \dot{x}_m &= \begin{cases} \alpha_{open}\left(h_{set} - h_{out}\right) & , \quad \dot{x}_m \geq 0 \\ \alpha_{close}\left(h_{set} - h_{out}\right) & , \quad \dot{x}_m < 0 \end{cases} \\ q &= C_v\left(x_m\right)\sqrt{h_{in} - h_{out}} \end{aligned}} \tag{1}$$

where h_{in}, h_{out}, h_{set} are the inlet, outlet and set pressures respectively, q is the flow through the PRV, x_m is the opening of the PRV and C_v is the valve capacity characteristic, normally available from PRV manufacturers. The opening and closing rates of the PRV are given by α_{open} and α_{close} respectively. These are dependent upon many factors (including needle valve setting) and are determined by an identification experiment. This model is suitable for representing the behaviour of PRVs in normal operating conditions.

3 CASE STUDIES

In this section, the case studies are described. In each case, the topology of the DMA is given with the most relevant data to describe its behaviour. Pressure control strategies are then presented.

3.1 *Case study 1 – single input DMA with two target points*

The first example describes work carried out on a small, single-feed DMA. The DMA consists of 1047 properties, 1030 of which are domestic and 17 which are small commercial users (shops etc.). The pipes in the DMA are nearly all cast iron, laid between 1956 and 1970. Figure 1 shows the topology of the DMA with the measurement locations and their elevation. There is a large range of elevation in the DMA – the lowest region in the middle with highest points at the ends. The PRV inlet and outlet pressures over a Sunday and the flow through the PRV throughout a week are also shown in figure 1. At night time the flow drops below the range of the flow meter installed so the actual flow as believed to be a little higher than that shown. Since the head loss is low through the night, this should not affect the DMA pressures too much.

Currently, the inlet pressure to the DMA (the PRV outlet) is set at a fixed pressure of 39.2 m. The target point, TP (highest point in the DMA) is 9.7 m higher in elevation and therefore has a maximum pressure of 29.5 m. Since, the TP is close to the inlet, the head loss between them is

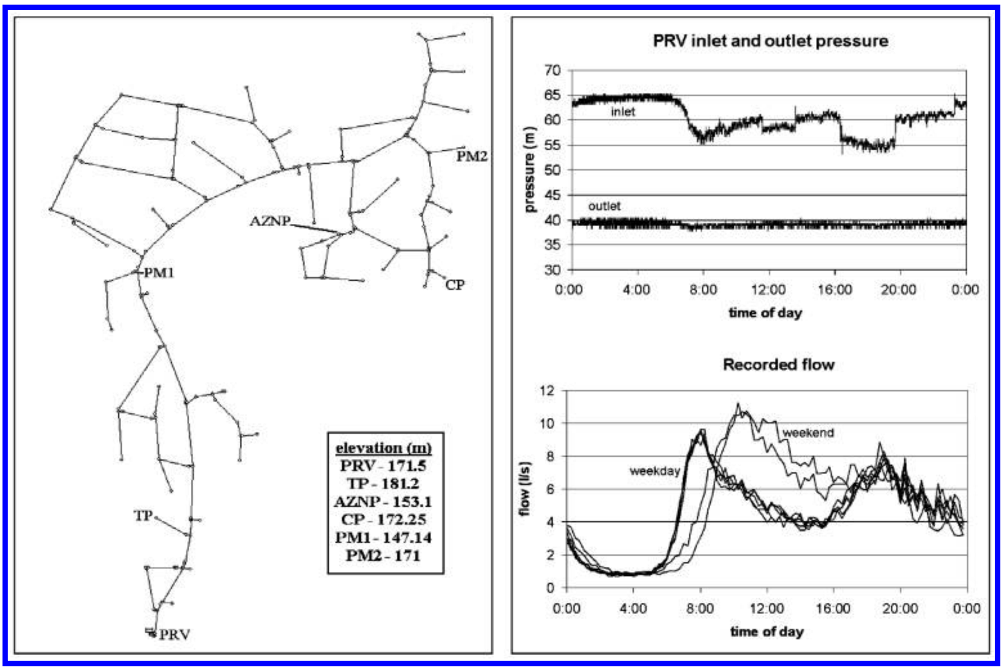


Figure 1. DMA topology and PRV measurements.

small so the TP pressure never drops below 26 m. The critical point, CP, is around 9 m lower in elevation than the TP but has a lower pressure at periods of high demand since there are quite high head losses through the system. The CP pressure ranges from 38 m at low demand to 20 m at high demand. Therefore, two points have to be considered when the control is implemented. The data for a Sunday (day with highest flow) is shown in figure 2 for the TP and CP.

There is little evidence of dynamic problems in the DMA other than a small amplitude oscillation of the PRV. This is more evident through the night when the flow is at its lowest but, even then, the resulting oscillation of the PRV outlet pressure is no more than a couple of meters. Another experiment was performed recording the PRV outlet pressure at 1 second intervals to obtain a more accurate estimate – in this case the outlet pressure had a peak to peak oscillation of just under 3 m. Should this behaviour become worse, servicing the PRV or adjusting the needle valve setting is likely to eliminate it.

Using the pressure control module, optimal pressure profiles for a Sunday were determined to maintain a pressure of 20 m at the TP and CP throughout the day for which data is shown in figures 1 and 2. The pressure control module also calculates the leakage and therefore the total flow into the DMA with the new profiles. Plotting the flow against the pressure profile gives a curve suitable for use in a flow modulation scheme. The optimal time profiles and flow modulation curve are shown in figure 3 along with the resulting pressures at the TP and CP. During low demand, the TP pressure remains at 20 m but the CP pressure is higher since the head loss is small. In periods of high demand, the inlet pressure increases to ensure 20 m at the CP so the TP pressure increases

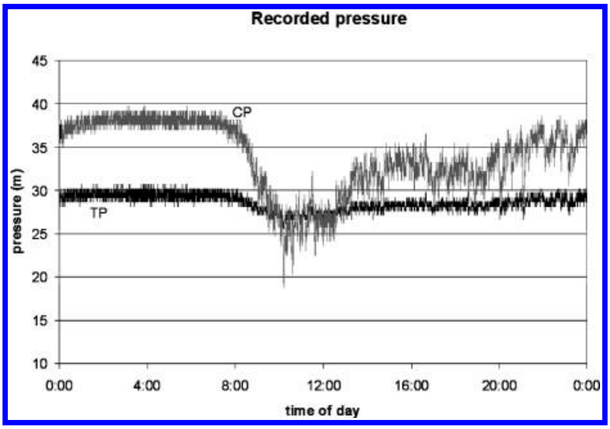


Figure 2. Target and critical point pressures on a typical Sunday.

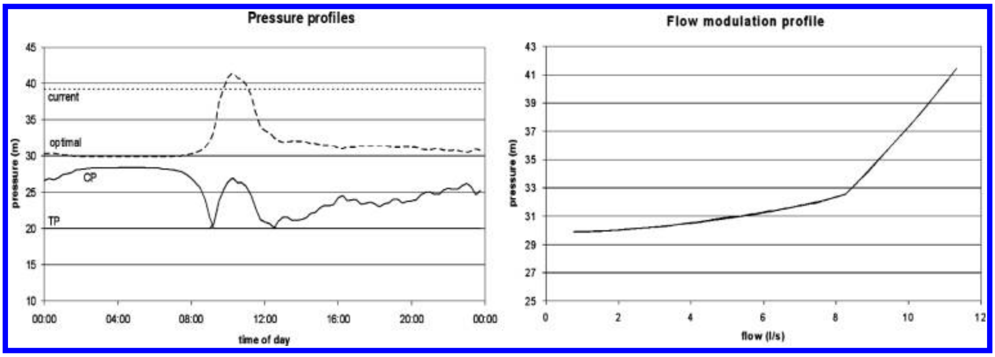


Figure 3. Optimal curves for case study 1.

above its 20 m minimum. This is also reflected in the flow modulation curve where there is a clear "switch" between controlling to the TP and controlling to the CP. If a flow modulation scheme was to be applied to the DMA, a more accurate flow measurement device would be beneficial.

3.2 *Case study 2 – single input network with dynamic problems*

The second DMA is also single-feed and supplies 120 properties (58 domestic, 62 commercial). There are two particularly high users with a combined use of around 400 m^3/day which equates to 75%–80% of total demand.

The layout of the DMA with recorded PRV data and the measurement points is shown in figure 4. The DMA consists of two parallel cast iron mains through the middle with plastic pipes covering the outer sections so there is very low head loss through the system. The pressure at all the measurement points have a similar profile – the pressure difference is simply due to difference in elevation. Therefore, a fixed outlet pressure is sufficient to minimise pressure. However, there are clear "spikes" in the outlet pressure that are believed to caused by similar sudden changes in flow – flow is reduced suddenly (by quickly closing a tap, for example) so the PRV outlet pressure suddenly increases. Over a period of time, often a couple of minutes, the PRV closes to reduce the outlet pressure back to the desired level. This event occurs several times over the week resulting in the pressure spikes in figure 4.

This behaviour can be reproduced in a transient simulation. The graphs of figure 5 give examples from a simulation of a model of this DMA. The demand is set at 7.4 l/s and then suddenly increased by 5 l/s (at the node labelled "sudden demand change" in figure 4), a similar change to that observed in the DMA. Once the system has settled to a steady state, the demand is suddenly decreased by 5 l/s at the same node. The nature of the system response depends primarily on the setting of the needle valve. PRVs typically have a needle valve which controls flow in one direction only. In the responses of figure 5, the needle valve is assumed to have no effect on the rate

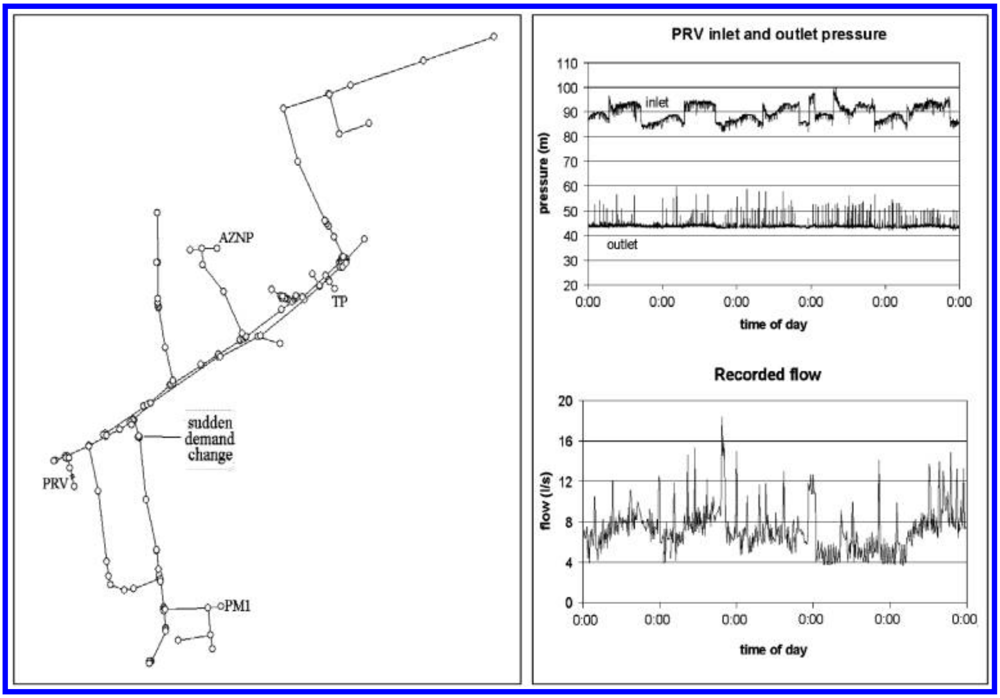


Figure 4. Single input network with rapid flow changes.

at which the PRV can open but can be adjusted for closing speed. In all simulations, $\alpha_{open} = 50 \times 10^{-6}$ and $\alpha_{close} = 1 \times 10^{-6}$ (slow), 5×10^{-6} (medium) and 25×10^{-6} (fast).

The effect of the needle valve setting on the response of the DMA to changes in flow is clear from figure 5. For an increase in flow, the outlet pressure drops sharply then increases as the PRV opens. For fast closing, the PRV closes again quickly, overshoots the steady state and oscillates a couple of times before settling. The same thing happens, although to a lesser extent, for the medium speed closing. For slow closing, the PRV gradually closes to a steady state but takes about 3 minutes to do this. A similar scenario arises when the flow through the PRV decreases except the initial sudden drop in outlet pressure no longer occurs.

Once the cause of the pressure spikes have been identified and eliminated, the DMA will operate efficiently on a fixed outlet pressure. The sudden flow changes are likely to originate from one of the big two users on the DMA (historical data suggests this) so flow loggers will be installed at these locations to observe their demand profile. This data, combined with the transient model will be used to determine the transient behaviour of the system more accurately.

Theoretically, there are two approaches to resolving the problem – to improve the control of the PRV or to remove the sudden changes in flow. From a practical point of view, the control loop on a PRV consists of fixed orifices and a very stiff spring and therefore behaves as a static device and does not have the ability to control this type of dynamics. Removing the sudden changes in flow involves working with the company and either installing a flow control device or asking them to change their operations. This obviously requires extensive communication between the water company and the user.

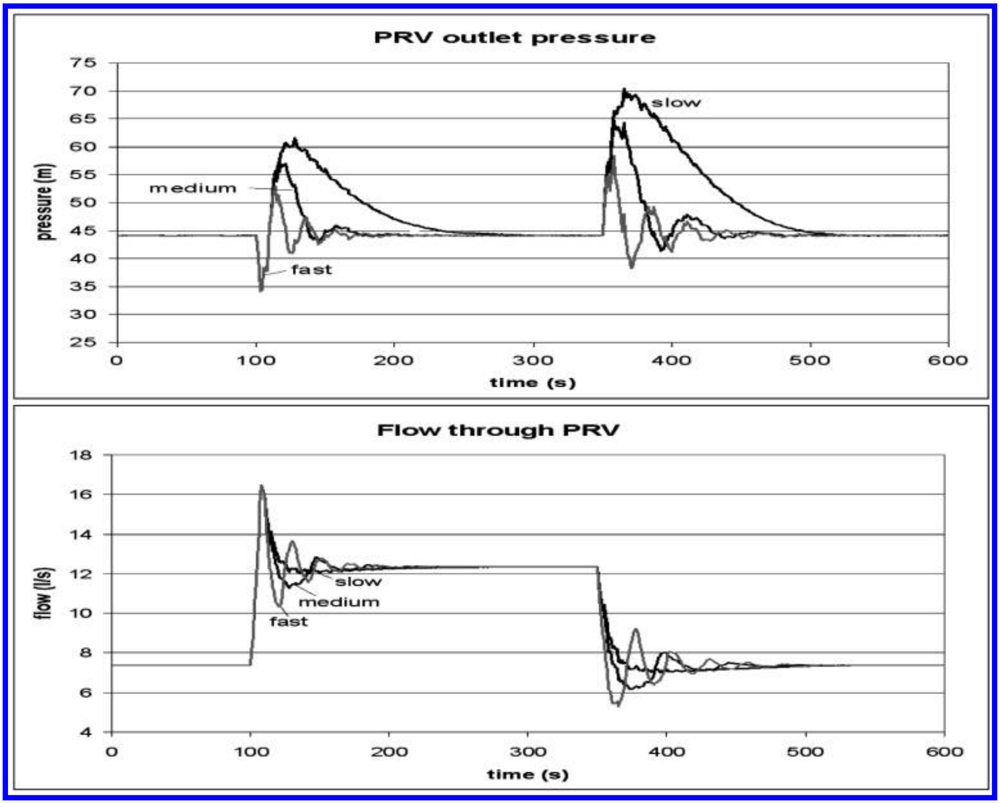


Figure 5. Response of PRV to change in flow.

3.3 *Dual input network*

The final case study is a small dual feed network, shown in figure 6, with 1008 properties (917 domestic, 91 commercial). The DMA has two inlet PRVs but following historical problems with interaction, it is currently supplied through just one. The central part of the DMA has high elevation and largely consists of cast iron pipes with reasonably high head losses. The inlet pressure from the active PRV has to be unnecessarily high to ensure that the pressure is sufficient at the opposite end of the network. Furthermore, since all the flow goes through one PRV and the section of pipes at that end of the DMA, the head losses across that section at peak demand times are excessive.

The flow into the network is also shown in figure 6 along with the pressure at the inlet and outlet of each PRV. It is clear from these that, PRV 2 remains closed almost permanently, PRV1 is not maintaining a steady outlet pressure and there is almost 20 m head loss between the two PRVs at peak demand. The target point, TP, is 13 m higher in elevation than PRV1 and, with the excessive peak demand head loss, the pressure drops to the 15 m limit specified by the regulator. The aim of the pressure control study is to maintain around 20 m at the TP.

Figure 7 shows the optimal PRV outlet profiles and resulting inlet flows by running the pressure control module. The simulated TP remains at exactly 20 m but there is clear evidence that there may be problems with PRV interaction when observing the resulting flow – through the night time, the dominant PRV switches from one to the other. This behaviour is prevalent in multi-inlet DMAs and results in undesired transients and water quality problems. Also, installing controllers on both PRVs is costly so a solution was sought where fixed outlets were used at the PRVs.

To reduce the risk of PRV interaction, it is necessary to select one PRV as the dominant device – a master/slave basis. Since the TP is closest to PRV1 this was selected as the master and PRV2 was selected as the slave. The appropriate pressures were determined using the profiles already obtained as an initial estimate and then modifying these on a trial and error basis using the simulation model.

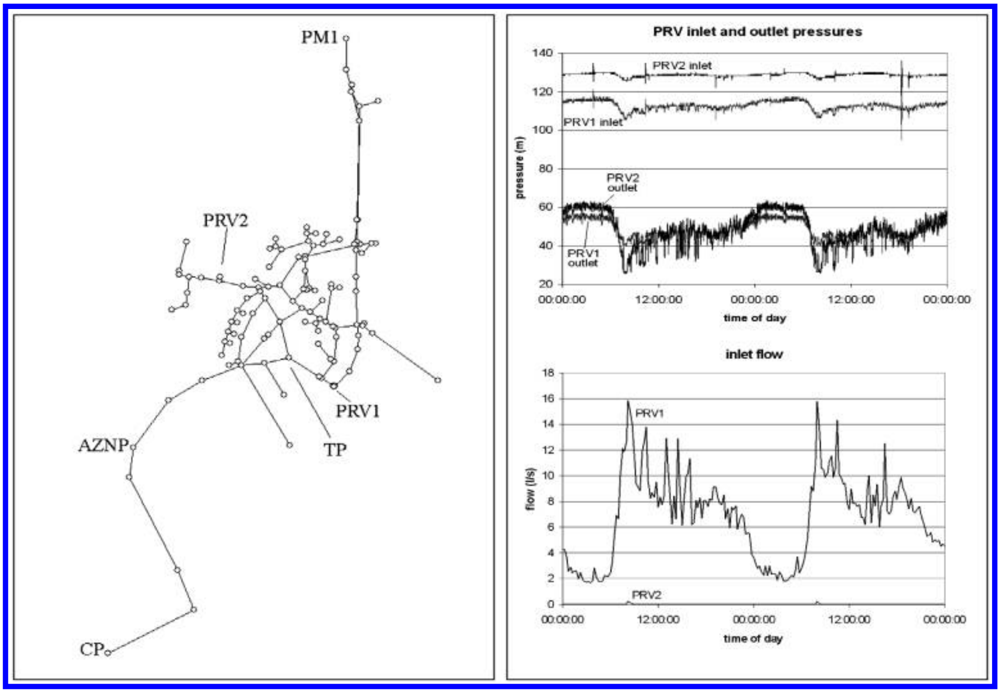


Figure 6. Network topology and PRV data for dual input network.

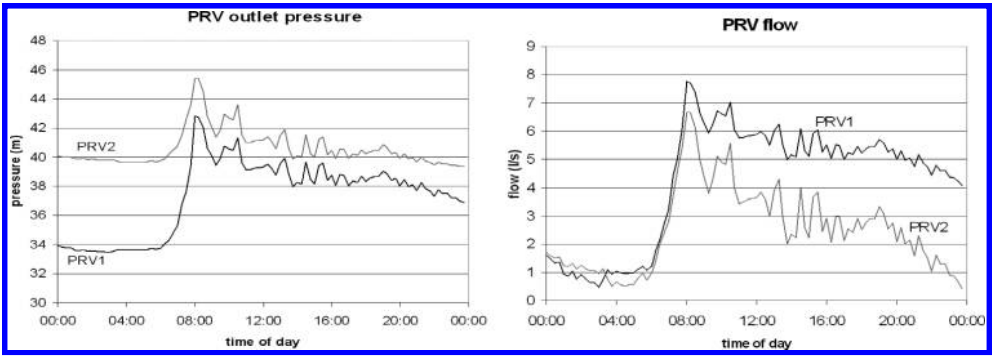


Figure 7. Optimal PRV schedules and resulting flow.

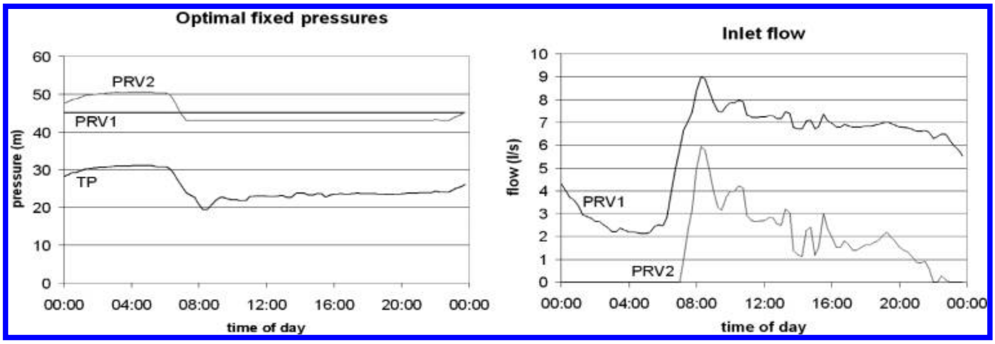


Figure 8. Optimal fixed PRV pressures and resulting flow.

Figure 8 shows the selected pressures: 45 m at PRV1 and 43 m at PRV2. When considering the elevation difference between the two PRVs, this gives a head difference of about 8 m. It can be seen from figure 8 that PRV2 closes through the night since the pressure from PRV1 and low night time head drop maintains a high enough pressure at the PRV2 outlet. When the demand increases, PRV2 outlet pressure drops to its set point and the PRV opens. PRV2 remains active throughout the day until the flow drops low enough for the pressure at its outlet to increase above 43 m. It then closes and remains closed throughout the night.

The overall recommendations for the dual input network are to service/replace PRV1 and set the outlet pressures of 45 m at PRV1 and 43 m at PRV2. This should give ample pressure at the TP (see figure 8) whilst providing a reasonably smooth pressure profile. Furthermore, both PRV's will be active, apart from periods of low demand, so the head losses across the DMA will be significantly reduced.

4 CONCLUSIONS

In this paper, three case studies concerning pressure control of DMAs have been presented. The case studies have covered a range of problems encountered by the water industry and methods to overcome these have been highlighted using examples.

The first example considered steady state control of a simple single-feed DMA. Using the pressure control module in FINESSE, optimal profiles were determined to maintain the desired pressure at two target points identified in the network. The significant target point at any time is dependent upon the flow into the network and a flow modulation curve was derived which combined the curves for each point.

The second case study demonstrated dynamic problems caused by a PRV feeding a network with fast changes in flow. The response of such a system was shown and the effect of the needle valve setting on the behaviour of the system was shown theoretically. Further work will identify the cause of the flow changes which then have to be eliminated through liaison with the user.

The final example showed optimal profiles for a dual input DMA but these showed evidence of PRV interaction at low flows. To eliminate this, the PRVs need to be set up on a master/slave set up and suitable fixed outlet pressures where given to do this. As is usual in this type of DMA, one PRV is active permanently and one controlling only at periods of high demand.

Pressure management of DMAs is known to reduce leakage and calculation of steady state optimal profiles is a reasonably simple task. However, care is needed to identify and remedy dynamic problems to reduce the incidence of bursts and the likelihood of water quality problems.

ACKNOWLEGEMENTS

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REFERENCES

Prescott, S.L. 2003 Pressure management – control devices and determination of inlet pressure profiles. *Water and Sewerage Journal*. Issue 1/2003:17–19.

Prescott, S.L. & Ulanicki, B. 2003 Dynamic modelling of pressure reducing valves. *Journal of Hydraulic Engineering* (accepted for publication).

Ulanicka, K., Bounds, P.L.M., Ulanicki, B. & Rance, J.P. 2001 Pressure control of a very large scale water distribution network with interacting water sources – a case study. In B. Ulanicki, B. Coulbeck, & J. Rance (eds), *Water Software Systems: theory and applications; Proc. CCWI, 3–5 Sept 2001*, Leicester, UK.

Ulanicki, B., Bounds, P.L.M., Rance, J.P. & Reynolds, L. 2000 Open loop and closed loop pressure control for leakage reduction. *Urban Water Journal*. 2(2):105–114.

Wylie, E.B. & Streeter, V.L. 1993 *Fluid transients in systems*. NJ: Prentice-Hall.

Chapter 2 – Leakage detection and management

An assessment of the application of inverse transient analysis for leak detection: Part I – Theoretical considerations

Z. Kapelan, D. Savic & G.A. Walters
Centre for Water Systems, University of Exeter, Exeter, UK

D. Covas, I.N. Graham & Č. Maksimović
Department of Civil and Environmental Engineering, Imperial College, South Kensington Campus, London, UK

ABSTRACT: This paper represents the first of a two-part paper which summarises the research work done on the development of an improved, inverse transient analysis-based, leak detection methodology for water distribution systems (WDS). The work was carried out jointly by Exeter University (EU) and Imperial College (IC) under the EPSRC WITE scheme. In part I (this paper), a summary of the theoretical research work done at EU is presented, while in part II, a summary of the experimental and further theoretical work done by IC is presented. In this paper, after an introduction and brief background, an inverse transient model (ITM) capable of solving the network calibration problem for leak detection and generally, parameter identification, is presented. The ITM is developed by linking a forward transient model (FTM) with a number of optimisation methods. The FTM developed is based on the method of characteristics solution of the basic transient mass-balance and momentum equations. The ITM capability to detect leaks is demonstrated on a network case study. At the end, a summary is provided with some guidelines for further work.

1 INTRODUCTION

Detection of leaks in water and other pipe networks is an important issue. It is not only important from the economics point of view, as is usually presented (i.e. higher the leakage, higher the investment and operational costs). But also because water resources are finite (especially underground resources) and leaks deplete them more than necessary. Furthermore, in pipelines that transport hazardous liquids (oil, gas, waste water, etc.) every leak is a threat to the surrounding environment. Presence of leaks in the networks also lowers the overall hydraulic efficiency and thus the overall reliability of the system. Finally, leaks may cause serious structural damage to neighbouring properties and roads.

In this paper leak detection and roughness calibration approaches developed in the past are briefly reviewed with the emphasis on those that use transient analysis. In the next section, the mathematical model of the inverse transient analysis is briefly outlined. The model is then applied to the case study. Finally, a summary is made in the last paper section.

2 BACKGROUND

A number of different leak detection methods exist today (Simpson and Vitkovsky 1997; Vitkovsky and Simpson 1997). Traditional methods include mass balance where the presence of leaks is identified by measuring all water mass balance components (inputs and outputs) in the analysed part of the pipe network. A number of physically based leak detection systems, coming mainly from the field of oil and gas where leakage problems are most critical, are also available today: hydrostatic testing, flow direction indicators, tracer gases, subsurface radar, earth resistivity changes, infrared spectroscopy, microphonics, odorant and radioactive tracers, etc. All these methods are quite expensive and time consuming to use.

Leak detection in water supply networks by means of solving the inverse steady-state problem was introduced by Pudar and Liggett (1992). Using observed state variables (heads and/or flow) as inputs and assuming orifice type leaks that may occur only at network nodes, they detected leaks by calibrating the network steady-state hydraulic model. After testing the methodology on several networks, they concluded that due to the fact that inverse steady-state problems are not well posed, the proposed methodology would only be commercially possible if massive data was available for the system.

Following Pudar and Liggett's work (1992), Liggett and Chen (1994; 1995) introduced a new leak detection method for water distribution networks by using inverse transient analysis. In contrast to steady-state events, transient events can provide the large amount of data required for successful calibration. Liggett and Chen used pipe friction factors and effective leak areas (leak areas multiplied by leak discharge coefficients) as decision variables. The authors assumed that orifice type leaks occur at network nodes only. The well-known Levenberg-Marquardt (LM) gradient search method was used to solve the optimisation problem together with the adjoint method which was used to calculate the necessary sensitivities (i.e. elements of gradient vector, Jacobian and Hessian matrices).

Following Liggett and Chen's work, Vitkovsky and Simpson (1997) and Vitkovsky et al. (1999; 2000) solved the same inverse transient problem by linking Genetic Algorithms with a forward transient solver. They introduced GAs as the alternative to the LM method following the well-known fact that, although very fast, gradient type methods may fail to converge or may converge to a local minimum rather than the global minimum. On the other hand, GAs, although much slower than the LM method, perform a more robust and comprehensive search and are less likely to fail or not to converge on a global optimum. Vitkovsky and Simpson (1997) analysed two encoding schemes (continuous and discrete) and concluded that a discrete coded scheme produces better results. GAs have also been used by Tang et al. (1999) to solve the inverse transient problem.

3 FORWARD TRANSIENT MODEL

The forward transient model presented here is based on two hyperbolic partial differential equations (Wylie and Streeter 1978) with the momentum equation extended to take unsteady friction effects into account. The governing equations are solved using the fixed grid method of characteristics. Several interpolation schemes are implemented in the model. Still, adjustment of wavespeeds is most frequently used. Two steady/quasi-steady friction models are supported in the current version of the forward transient software: the Darcy-Weisbach model (either constant friction factor f or factor f calculated as a function of relative roughness and Reynolds number) and the Hazen-Williams model. Integration of the steady friction term in the momentum equation is done using the linearisation approach suggested by Karney and McInnis (1992).

The current version of the forward transient software has two unsteady friction models implemented: Brunone et al. model (1991) with modification provided by Vitkovsky et al. (2000) and Trikha's (1975) approximation of Zielke's model (1968). Unsteady friction models were introduced to enable more realistic pressure wave signal simulation (in terms of the signal's amplitude and phase). In addition to this, it turned out that unsteady friction modelling can be utilised to significantly speed-up the process in which the transient simulator is used to find initial (steady-state) heads and flows in the network. For example, an artificially large K_3 parameter value (e.g. $K_3 = 1$) in Brunone et al.'s model may be used to force the forward transient simulator to dampen head and flow oscillations faster than in the case without unsteady friction modelling ($K_3 = 0$), or in the case with realistic K_3 (<0.1) values (see Figure 1).

The boundary condition modelling approach used in this paper is based on concepts introduced by Karney and McInnis (1992) and McInnis (1992). Finally, it is assumed in this paper that leaks of orifice type may occur at network nodes only.

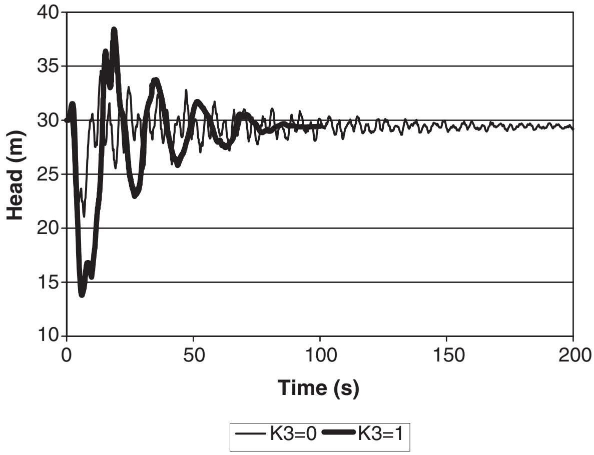


Figure 1. Example of speeding-up steady-state calculations performed by FTM.

4 INVERSE TRANSIENT MODEL

4.1 *Introduction*

The procedure for leak detection in water supply networks by inverse transient analysis consists of the following main steps: (1) Introduction of an artificial transient event(s) in the network, either by changing the flow or head at some location in the network (e.g. valve opening or closure, etc.). The introduced transient event needs to be safe in that pressures and velocities in the network must stay between allowed limits throughout the event; (2) Observation of pressures (and flows, if possible) at a certain number of locations in the system; (3) Periodical calibration of the forward transient model (FTM) for effective leak areas (product of the leak discharge coefficient and the actual leak area), assuming orifice type leaks at a certain number of nodes in the network. Periodical calibration is necessary since leaks may occur (e.g. due to pipe bursts) after the last model calibration was performed.

4.2 *Inverse problem formulation*

The objective function to be minimised in the inverse transient analysis presented here is of the weighted least squares (WLS) type with prior estimates of parameters:

$$E(a) = \left[y^* - y(a)\right]^T W \left[y^* - y(a)\right] + \left(a_0^* - a_0\right)^T V \left(a_0^* - a_0\right) \tag{1}$$

where: E – scalar objective function value; a – vector consisting of N_a unknown calibration parameters (effective leak areas, pipe roughness coefficients, etc.); y^* – vector consisting of $\mathrm{N} = \mathrm{N_x N_t}$ observed system variables (in this case heads); $y(a)$ – vector consisting of system variable values calculated by FTM; a_0^* – vector consisting of $N_p \leqslant N_a$ prior estimates (also called pseudo measurements) of calibration parameters; a_0 – vector consisting of N_p values of calibration parameters (those with prior estimates), $\mathrm{a_0}$ is a sub-vector of a; W and V – weighting matrices; N_x – number of observation locations in the network; N_t – number of observed time steps; N_a – number of unknown calibration parameters; T denotes transpose of a vector/matrix.

Many inverse problems are ill-posed. A problem is ill-posed when it has either no solution, no unique solution or the solution is unstable. In practice, ill-posedness is usually characterised by non-uniqueness and instability. Prior estimates, i.e. prior information on parameters, can be used to condition an ill-conditioned inverse transient problem (Kapelan 2002). An effect of the conditioning of the inverse transient problem search space is shown in Figure 2.

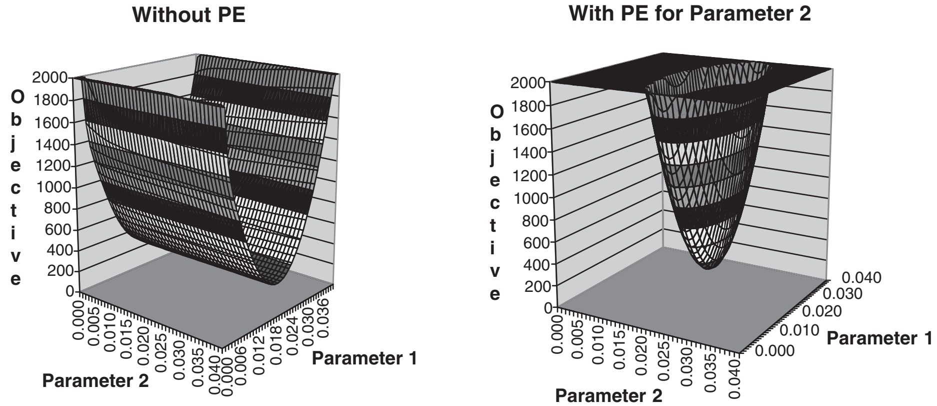


Figure 2. An example of the inverse transient problem conditioning using prior estimates (PE) of calibration parameters.

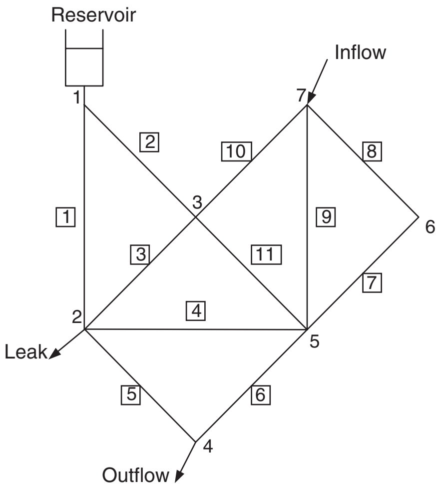
Prior estimates have not been used in the inverse transient analysis so far. Greco et al. (1999) used prior estimates for the calibration of the multiple loading condition steady-state hydraulic network model, but in a significantly different form. Their objective was to minimise the sum of squared prior estimate penalties subject to the set of pressure residual constraints in which pressure residuals at all measurement nodes were held within pre-specified tolerances. In our opinion, prior estimates are better utilised in the form proposed in equation (1) since both pressure residuals and prior estimate penalties, are minimised simultaneously.

4.3 *Optimisation methods*

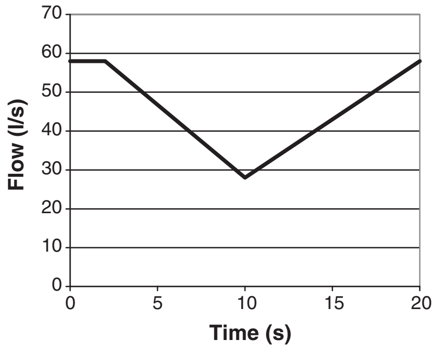
In order to solve the inverse transient problem, a number of existing/new optimisation methods have been linked to the forward transient model described above. Initially, two well know optimisation methods were considered: (1) genetic algorithms (GAs) and (2) the Levenberg-Marquardt (LM) method. After identifying advantages and disadvantages of both methods, a novel hybrid optimisation method was developed to solve the inverse transient problem (Kapelan et al. 2003). The new method was named Hybrid GA or HGA.

The standard GA optimisation method performs search by means of selecting, recombining and replacing chromosomes in the population. The two main operators used in the GA process are crossover and mutation. The crossover operator is considered to have a more global effect, i.e. it can identify one or more solutions in the region near the global optimum, but it lacks the ability to refine those solutions. On the other hand, the effect of mutation is considered to be more of a local type. The mutation operator has two important roles in the standard GA: (1) to maintain population diversity, i.e. to re-introduce genes lost in previous generations; and (2) to effectively refine sub-optimal solutions (local search). Usually, the mutation operator is much more effective in performing the first role than the second one. As a consequence, it is usually said that GA lacks enough selection pressure to effectively perform a local search.

Therefore, it is desirable to modify/hybridise the standard GA method to enable introduction of a controllable level of selection pressure. In the HGA method presented here, this is achieved by introduction of a new GA operator called the local search operator (LSO). The LSO is based on the LM optimisation method. When applied, LSO improves the chromosome by applying the LM method for a limited number of iterations (typically one or two). The LSO is applied with a pre-specified probability every N_g GA generations to N_{chr} chromosomes from the GA population created using standard GA operators. Advantages of the HGA method over the GA and LM methods are further demonstrated on an example case study.



(a) network



(b) flow at node 4

Figure 3. Case study network and transient event initialisation condition.

5 CASE STUDY

The application of the new methodology to the pipe network calibration problem is illustrated considering the problem studied earlier by Chen (1995). The water distribution network is shown in Figure 3a. Relevant pipe data can be found in the above-mentioned reference. The gravity network is fed from a constant head reservoir at node 1 (head equals 30 m). There is one orifice-type leak in the network at node 2 with an effective area of $5*10^{-5}\,m^2$. Inflow at node 7 is constant and equal to 21 l/s.

A transient event is introduced in the network by changing the flow at node 4 with time (see Figure 3b). As can be seen from Table 1, the inverse transient problem is solved for a total of 17 calibration parameters: 11 friction factors and 6 effective nodal leak areas. Measurement data consists of heads at nodes 4 and 7. In order to simulate real measurement data, random Gaussian noise (zero mean and 0.10 m standard deviation) was added to perfect head data (created by running the FTM with all calibration parameter values equal to their true values). A total of 101 measurement time steps are used in the case study, representing a 20-second long transient event. In the example presented here, all friction factors are constant Darcy-Weisbach friction factors.

In order to compare performances of LM, GA and HGA optimisation methods, multiple ITM runs were performed. Results are presented in Figure 4. In this figure, the CPU time necessary to perform optimisation analysis is indicated through the number of FTM runs (horizontal axis). A single FTM run is one in which either heads/flows are calculated or heads/flow derivatives are calculated with respect to one calibration parameter. Also, note that the vertical axis is logarithmically scaled.

First, the above defined problem is solved using the LM method. A run is considered converged when the relative change of all parameters is equal to or less than 10^{-3}, providing that the maximum number of 100 iterations is not exceeded. Two LM runs were performed (LM1 and LM2). In the LM1 run, a starting point far from optimum was chosen. As can be seen from Figure 4 (case LM1), the LM method failed to identify the optimal solution. Detailed insight into the LM search process reveals that the method is, actually, failing to converge due to instability of parameter 3. However, once the LM method is run with the starting point close to the optimum, (LM2 case), the method converges to the optimal objective function value of $2.18\,m^2$ (see Figure 4, case LM2).

Table 1. Optimal parameter values identified in multiple HGA runs (different random seeds).

Par. ID	Param. type*	True value	Optimal value Min	Average	Max	Max/Min
1	FF	0.020	0.0196	0.0222	0.0229	1.17
2	FF	0.020	0.0185	0.0191	0.0211	1.14
3	FF	0.020	0.0400	0.0400	0.0400	1.00
4	FF	0.020	0.0100	0.0100	0.0100	1.00
5	FF	0.020	0.0158	0.0162	0.0166	1.05
6	FF	0.020	0.0210	0.0215	0.0222	1.06
7	FF	0.020	0.0157	0.0165	0.0180	1.14
8	FF	0.020	0.0100	0.0100	0.0100	1.00
9	FF	0.020	0.0400	0.0400	0.0400	1.00
10	FF	0.020	0.0100	0.0100	0.0100	1.00
11	FF	0.020	0.0159	0.0164	0.0180	1.13
12	LA	5.0e−5	4.84e−05	4.85e−05	4.89e−05	1.01
13	LA	0	0	1.77e−11	8.86e−11	–
14	LA	0	0	0	0	–
15	LA	0	0	0	0	–
16	LA	0	0	0	0	–
17	LA	0	2.57e−06	3.03e−06	3.3e−06	1.29

*FF – Darcy-Weisbach friction factor, LA – effective leak area.

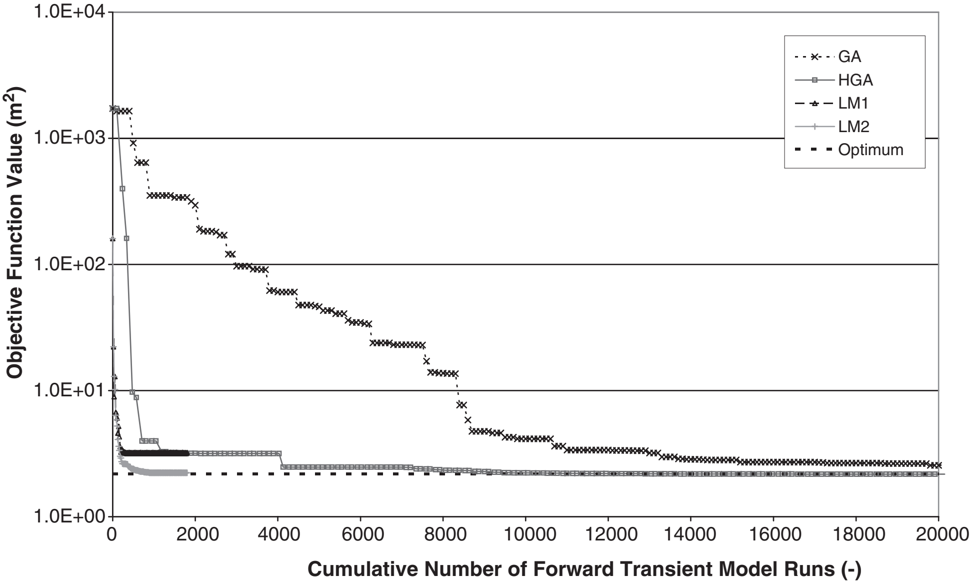


Figure 4. Comparison of optimisation method performances on an inverse transient problem.

In addition to the LM method, the GA optimisation method was used to solve the same inverse problem. Multiple GA runs were performed with different random seeds due to the stochastic nature of the method. The real coded, generational, elitist GA method with a population size of 100 was allowed to evolve for 10,000 generations. The following GA operators were used: bit tournament selection, arithmetic crossover operator with probability 0.70 and mutation by gene operator with probability 0.05. The five GA runs yielded an average optimal solution fitness of 2.30 m^2. Optimal fitness values for the GA runs were within the range 2.23 to 2.36 m^2. A typical GA run history for the best solution is presented in Figure 4 (first 200 generations only). As can be seen, GA is detecting a solution better than the LM1 solution but worse than the LM2 solution. Note that finding a good solution requires a relatively large computational effort.

After application of the LM and GA optimisation methods, the same inverse transient problem was solved using the HGA method. Again, multiple runs with different random seeds were performed. In the HGA runs performed here, all GA parameters were equal to those in the corresponding GA runs and the same LM method that was used on its own was used in all HGA runs. All HGA runs were stopped after 200 generations. The five HGA runs yielded the same optimal solution fitness of 2.18 m^2. A typical HGA run history is presented in Figure 4.

From the case study the following can be noted: (1) The ITM is capable of detecting links in the pipe network in the presence of the reasonable level of measurement noise; (2) When solving the inverse transient problem, the LM method may fail to converge if the starting point is badly chosen. However, when a good starting point is chosen, the LM method outperforms both the GA and the HGA methods; (3) The HGA method is performing better than the GA in terms of both effectiveness and efficiency, i.e. it takes HGA less computational effort to find solutions that are better than GA solutions.

6 SUMMARY

This paper focuses on the development of the mathematical model and software for inverse transient analysis in water distribution networks. First, a forward transient mathematical model is presented in this paper. After that, an inverse transient mathematical model based on the new hybrid search method HGA is presented. The methodology developed is then applied to the case study.

Even though ITM successfully detected leaks in the case study presented here, one has to bear in mind that measurement data used here was of a synthetic type. Therefore, extrapolation of the methodology to laboratory and especially to field data requires further study. This issue is addressed in more detail in the second part of this two-part paper.

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REFERENCES

Brunone, B., Golia, U.M. and Greco, M. 1991. Modelling of Fast Transients by Numerical Methods. In E. Cabrera and M. A. Fanelli, eds., *Proc. International Conference On Hydraulic Transients with Water Column Separation, Valencia, Spain*, 273–280.

Chen, K. and Brdys, M.A. 1995. Set Membership Estimation of State and Parameters of Large Water Supply and Distribution Systems – Integrated Quantity and Quality Models, *Proc. IFAC Symposium on Large Scale Systems, London, UK*.

Greco, M. and Del Guidice, G. 1999. New Approach to Water Distribution Network Calibration. *Journal of Hydraulic Engineering, ASCE* 125(8): 849–854.

Kapelan, Z.S. 2002. Calibration of WDS Hydraulic Models. *PhD Thesis*, Department of Engineering, University of Exeter, p. 334.

Kapelan, Z.S., Savic, D.A. and Walters, G.A. 2003. A Hybrid Inverse Transient Model for Leakage Detection and Roughness Calibration in Pipe Networks. *Journal of Hydraulic Research (IAHR)* (accepted for publication).

Karney, B.W. and McInnis, D. 1992. Efficient Calculation of Transient Flow in Simple Pipe Networks. *Journal of Hydraulic Engineering, ASCE* 118(7): 1014–1030.

Liggett, J.A. and Chen, L.C. 1994. Inverse Transient Analysis in Pipe Networks. *Journal of Hydraulic Engineering, ASCE* 120(8): 934–955.

Liggett, J.A. and Chen, L.C. 1995. Monitoring Water Distribution Systems. The Inverse Method as a Tool for Calibration and Leak Detection, Improving Efficiency and Reliability in Water Distribution Systems. E. Cabrera and A. F. Vela, eds., Boston, USA, *Kluwer Academic*, vol. 14**,** 107–134.

McInnis, D.A. 1992. Comprehensive Hydraulic Analysis of Complex Pipe Systems. *PhD Thesis*, Department of Civil Engineering, University of Toronto.

Pudar, R.S. and Liggett, J.A. 1992. Leaks in Pipe Networks. *Journal of Hydraulic Engineering, ASCE* 118(7): 1031–1046.

Simpson, A.R. and Vitkovsky, J.P. 1997. A Review of Pipe Calibration and Leak Detection Methodologies for Water Distribution Networks, *Proc. 17th Federal Convention, Australian Water and Wastewater Association, Australia*, vol. 1**,** 680–687.

Tang, K., Karney, B., Pendlebury, M. and Zhang, F. 1999. Inverse Transient Calibration of Water Distribution Systems Using Genetic Algorithms. In D.A. Savic and G.A. Walters, eds., *Proc. Water Industry Systems: Modelling and Optimisation Applications, Exeter, UK*, vol. 1**,** 317–326.

Trikha, A.K. 1975. An Efficient method for Simulating Frequency-Dependent Friction in Transient Liquid Flow. *Journal of Fluids Engineering, Transactions of the ASME* 97 97–105.

Vitkovsky, J.P. and Simpson, A.R. 1997. Calibration and Leak Detection in Pipe Networks Using Inverse Transient Analysis and Genetic Algorithms. *Report No. R 157*, Department of Civil and Environmental Engineering, University of Adelaide, p. 97.

Vitkovsky, J.P., Simpson, A.R. and Lambert, M.F. 1999. Leak Detection and Calibration of Water Distribution Systems using Transients and Genetic Algorithms, *Proc. 26th Annual Water Resources Planning and Management Conference, ASCE, Tempe, USA*.

Vitkovsky, J.P., Simpson, A.R. and Lambert, M.F. 2000. Leak Detection and Calibration Using Transients and Genetic Algorithms. *Journal of Water Resources Planning and Management, ASCE* 126(4): 262–265.

Wylie, E.B. and Streeter, V.L. 1978. *Fluid Transients.* McGraw-Hill International Book Company.

Zielke, W. 1968. Frequency-Dependent Friction in Transient Pipe Flow. *Journal of Basic Engineering, ASME* 90 109–115.

An assessment of the application of inverse transient analysis for leak detection: Part II – Collection and application of experimental data

D. Covas, N. Graham & Č. Maksimović
Imperial College London, London, UK

H. Ramos
Instituto Superior Técnico, Lisbon, Portugal

Z. Kapelan, D. Savic & G. Walters
University of Exeter, Exeter, UK

ABSTRACT: This paper constitutes the second (Part II) of a two-part paper which summarises the research work carried out jointly by Exeter University (EU) and Imperial College (IC) under the EPSRC WITE scheme, on the development of an improved inverse transient analysis (ITA) leak detection methodology for water pipe systems. This was approached through the development of a novel hydraulic transient solver and the collection of transient data for the testing and validation of this model. An extensive experimental programme was carried out in two polyethylene (PE) pipelines to collect necessary transient data: the first at Imperial College (London, UK), and the second at Thames Water (London, UK). An improved hydraulic transient solver was developed to calculate hydraulic transients in pressurised multi-pipe systems, incorporating fluid frictional and inertial effects, and pipe-wall linear viscoelasticity. Unlike classic transient solvers, the developed model is capable of accurately predicting transient pressures in PE pipes, which is crucial for the successful application of ITA. The evaluation of the presence, location and size of leaks was carried out using collected data. ITA successful detected the location of leaks, providing that an accurate transient solver was used and leak detection was carried out simultaneously with creep calibration. ITA was applied in a step-wise manner to more accurately pinpoint leaks. Inverse transient analysis is a very promising technique for both leak detection and the diagnosis of existing water supply systems.

1 INTRODUCTION

In the last decade, leakage reduction and control has become a high priority for water supply utilities and authorities. Consequently, there has been a significant interest in the application of the Inverse Transient Analysis (ITA) for leak detection and calibration in water pipe systems (Liggett and Chen, 1994; Vitkovsky *et al.*, 2000; Kapelan *et al.*, 2001). This method has been widely tested with artificial data; however, very little evidence exists of the method validation and testing with laboratory data or field data. Additionally, hydraulic transient simulators used in the implementation of this analysis assume linear elastic behaviour of the pipe material. This assumption is considerably imprecise for the description of transients in plastic pipes. Polymers have a viscoelastic mechanical behaviour that affects the pressure response during transient events by attenuating and dispersing the pressure wave (Covas, 2003). The accuracy of transient solvers is extremely important for the success of ITA.

The aim of the current paper is an assessment of the application of ITA for leak detection using physical data collected in two polyethylene experimental pipelines. Two extensive experimental programmes were carried out at Imperial College London and in cooperation with Thames Water to collect the necessary data. Several experimental tests were run with and without simulated leakage. Leak detection was carried out using two different transient simulators: an 'Elastic Solver' and a 'Viscoelastic Solver'. Whilst the ITA using the 'Elastic Solver' fails detect the correct location of leaks, the use of the 'Viscoelastic Solver' enables the successful leak location.

2 EXPERIMENTAL DATA COLLECTION

Two experimental programmes were carried out at Imperial College (IC) and in cooperation with Thames Water (TW), aiming at the collection of reliable data sets for the investigation of transient-based leak detection, location and sizing techniques.

2.1 *Laboratory data*

The first set of tests was carried out using a specially constructed, 277 m pipeline at Imperial College, London. The pipe was made of high-density polyethylene (PE) with 50.6 mm inner diameter and 277 m long. The rig included a pump and a pressurised vessel at the upstream end, and a globe valve at the downstream end. This valve was used to control the flow and to generate the transient. An electromagnetic flow meter was used to measure the initial flow. The data acquisition system (DAS) was composed of an acquisition board (with 8 channels and a maximum sampling frequency of 9600 Hz per channel), 8 pressure transducers and a computer. Transducers had an absolute pressure range from 0 to 10 bar and an accuracy of 0.3% of full range. Several problems were encountered during data collection phase (e.g. steady state pressure oscillations and a sudden pressure drop after the valve closure), some of which were overcome, whilst others were inherent of the pipe system (i.e. the viscoelastic behaviour of the PE).

2.2 *Quasi-field data*

The second set of tests was carried out in the world's longest experimental PE pipeline (TORUS pipe) with 1.3 km length, located Kempton Park, Thames Water, London. The system consists of two main pipes: (i) an inlet pipe starting at an upstream reservoir with two submersible pumps, 90 m long and made of medium-density PE (MDPE) with 70 mm ID; and (ii) the main pipe, 1.2 km long, buried underground, made of MDPE with 108 mm ID. The main pipe rises above ground at three

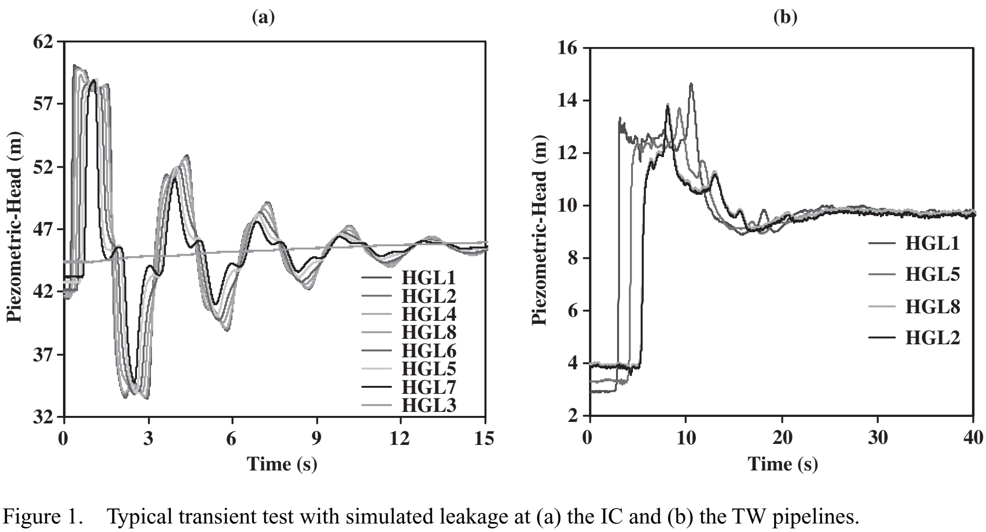

Figure 1. Typical transient test with simulated leakage at (a) the IC and (b) the TW pipelines.

points 500 m, 900 m and 1300 m (from the upstream end) and passes through a 25 m testing station. A gate valve was used to control the flow and a butterfly valve to generate the transient. The flow was measured in an electromagnetic flow meter. The DAS was the same as used in the IC tests. The major problem encountered at this pipeline was a persistent air pocket that considerably deformed the transient pressure wave. This air pocket size was reduced by pressurising the system during several days. Transient data had noise associated with multiple reflections coming from different features of the system.

Several data sets, with and without simulated leaks and for different flow conditions, were collected at the IC and TW pipelines. Two typical transient tests with simulated leakage carried out at the IC and the TW pipelines are presented in Figure 1.

3 INVERSE AND HYDRAULIC TRANSIENT SOLVERS

Inverse Transient Solver (ITS) was implemented to carry out the parameter search as described in the companion paper (Kapelan *et al.*, 2003). The ITS incorporates several optimisation techniques (Levenberg-Marquardt, Genetic Algorithms and hybrid methods), objective functions and several types of parameters (e.g. pipe roughness, leak size, unsteady friction coefficients and creep function parameters).

Given the viscoelastic nature of experimental polyethylene pipelines, a novel Hydraulic Transient Solver (HTS) was developed to calculate hydraulic transients in pressurised PE pipe systems. This model incorporates terms to take into account unsteady friction and pipe-wall viscoelasticity. Whilst the description of unsteady friction losses consists of the incorporation of an additional term in the momentum equation, the viscoelastic effect is described by an additional term in the continuity equation (Covas *et al.*, 2002a; 2002b; 2002c; Covas, 2003). The viscoelastic component is characterised by a creep function, which, by definition, describes the time-variation of strain for a constant stress σ_0, $J(t) = \varepsilon(t)/\sigma_0$. This function can be estimated by a creep test or by dynamical testing of a pipe sample. Alternatively, it can be calibrated using collected transient data. The mechanical model of a generalised viscoelastic solid was used to describe the creep function. The set of partial differential equations was solved by means of the typical Method of Characteristics using a rectangular double grid. A complete description of the HTS and the respective calibration and validation can be found in Covas (2003).

4 INVERSE TRANSIENT ANALYSIS USING IMPERIAL COLLEGE DATA

The ITS is used to assess the application of this technique to locate and size of existing leaks at the IC pipe system using collected transient pressure data. The comparison between leak detection results using 'linear elastic' and 'linear viscoelastic' transient solvers is presented. Results of a sensitivity analysis of leak detectability simultaneously with creep calibration are presented.

4.1 *Leak detection using linear elastic and linear viscoelastic solvers*

The aim of this section is to highlight the importance of the accuracy of the hydraulic transient solver for the successful application of ITA. An example of leak detection using 'linear elastic' and 'linear viscoelastic' transient solvers is presented. Transient data from same test described above were used. The pipe was divided in eight sections (~33.98 m) and described by nine equally spaced nodes (i.e. Nodes 1 to 9). Seven nodes were assumed as potential candidates for leak locations (i.e. Nodes 2 to 8). Leak L2 (~162.50 m) was located near Node 6 (~169.88 m) with $A_{Lef} = 1.52E-05\,m^2$ and 0.43 l/s discharge. Three sample sizes ΔT were analysed corresponding to multiples of the theoretical period of the pressure wave T (i.e. $\Delta T = T$, $2T$ and $3T$).

The ITS using the 'elastic solver' pinpoints to leaks spread along the pipe, particularly at the extremities of the pipeline (Figure 2). These false leak locations are due to the imprecision of the transient solver describing fast transient events, even when a high decay coefficient is used ($k' = 0.15$).

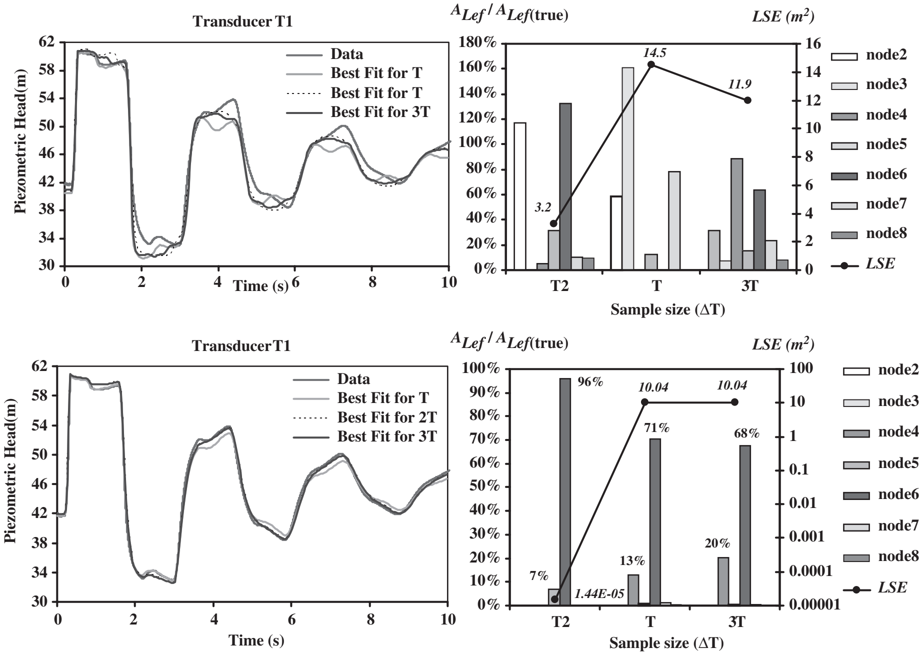

Figure 2. Experimental data vs. optimal numerical solution for $\Delta T = T, 2T, 3T$, Leak 2 with 0.43 l/s using: 'linear elastic' solver (*on the top*) and a 'linear viscoelastic' solver (*on the bottom*).

This results in a huge scatter between data and numerical results, which is crucial for leak detection. The only way that the optimisation algorithm finds to describe observed pressure data at transducer T1 is by allocating false leaks along the pipeline.

The ITS using the 'viscoelastic solver' pinpoints to the nearest node (Node 6) as the actual leak location, well as to the closest nodes upstream, i.e. Nodes 4 or 5 (Figure 2). This is because the leak is between Node 5 and 6. The error in the leak size is less than 5% of actual size for the smallest sample size T, increasing to ~30% for the sample sizes of $2T$ and $3T$. The precise location of the leaks presupposes the discretization of the pipeline into more leak candidates close to the leak location and the application of inverse method in a several-step procedure.

The results show that the 'linear elastic' transient solver is very imprecise in the description of transient events in PE pipes, which hinders the correct location of leaks. In comparison, the 'linear viscoelastic' transient solver (properly calibrated) accurately describes observed transient pressures, which is essential for the successful leak location and sizing (Covas, 2003).

4.2 *Leak detection simultaneously with creep calibration*

The most common engineering situation is to have a leaking pipe with unknown friction and viscoelastic characteristics. Ideally, all model parameters would have to be simultaneously calibrated. Preliminary testing of the ITS has shown that transient pressure data have different sensitivities to different parameters which makes the whole process of simultaneous calibration and leakage detection significantly difficult to carry out, particularly in the distinction between frictional and mechanical damping effects. Additionally, a sensitivity analysis has shown that the creep function can represent well both frictional and mechanical effects, and that, for leak detection purposes, the precise creep function curve is irrelevant as long as leaks are correctly located. Thus, (i) steady-state friction factors were calibrated based on steady state flow conditions; (ii) unsteady friction was assumed to be described by the creep function and (iii) leak detection was carried out simultaneously with creep coefficients calibration.

A sensitivity analysis was carried out to assess the application of this technique for the downstream end flow-rate Q_0 of 1.0 l/s, and three leak diameters (2.7, 4.4 and 6.0 mm) and locations (82.86, 162.48 and 227.38 m from the upstream end), i.e. 9 test cases. The sample size (ΔT) used was equal to one period of the pressure wave, $T = 4L/c$ (L = pipe length and c = wave speed).

ITA was applied in a step-wise manner, starting with a description of leak candidates sparsely distributed throughout the system and gradually defining these candidates around the main potential leak locations obtained in previous steps. This procedure is to minimise the number of leak candidates, fasten the optimisation procedure and gradually define the uncertainty Δx^* associated with leak location. Accordingly, the pipeline with $L = 271.80$ m was divided into 54 sections, equally spaced, with 5.03 m each. In *Step I*, ITA was run for two sets of potential leak locations: (i) 8 leak locations spaced $\Delta x^* = 30.20$ m and (ii) 17 locations with $\Delta x^* = 15.10$ m. In *Step II and III*, ITA was run for another two sets of potential leak locations, defined around the highest leak found in *Step I*: (iii) 10 leak locations spaced $\Delta x^* = 10.06$ m, covering ~90 m near the leak and (iv) 11 leak locations spaced $\Delta x^* = 5.03$ m, covering ~50 m near the leak.

Results obtained for the leak with 6 mm diameter and at 162.48 m are presented in Figure 3: (a) optimal relative leak sizes and respective LSE; (b) optimal creep coefficients and LSE; (c) optimal creep functions and creep function calibrated for laminar conditions without simulated leakage; and (d) optimal piezometric heads and experimental data. Each of the ITS runs is referred to by the corresponding Δx^* between leak candidates (30 m, 15 m, 10 m and 5 m).

Based on the analysis of these results, the following conclusions can be drawn:

(i) The ITS pinpointed to the actual leak location and assessed the leak size for all analysed transient tests, though with different uncertainties associated with the description of leak candidates.
(ii) Leak location uncertainties (defined by ratio between leak location error and total pipe length) varied between 1.8 and 8%; however, most single leaks could be located with an accuracy of 5 m (corresponding to 1.8% of total pipe length).
(iii) The most accurate leak locations and sizes were observed for larger leaks and for leaks closer to the location where the transient event was generated.
(iv) The success of ITA does not depend on steady and unsteady friction coefficients (as it uses the creep function to describe frictional and mechanical damping effects), does not require several measurement sites for the accurate estimation of wave speed (as it incorporates special dynamic effects), and can be generally applied to multi-pipe systems with various leaks and different boundary conditions (as it describes the dynamic behaviour of the system by using a transient simulator).

The field implementation of ITA was simulated in the IC system by generating small transient events with side discharge valves. The method was not as efficient as for a transient generated by the full closure of in-line valves (in this case, at the downstream end), as the continuous flow in the pipe dissipates very quickly the transient event and the leak reflection (Covas, 2003).

5 INVERSE TRANSIENT ANALYSIS USING THAMES WATER DATA

The objective of this section is to show how ITA performs when using data collected in *quasi*-field conditions at the TW pipeline, with several sources of uncertainty associated with the system characteristics and collected data. One example of leak detection simultaneously with creep calibration is presented. For this test, the downstream flow-rate Q_0 is 2.8 l/s and the leak is located at 473.00 m from the downstream end and has a sizes $A_{Lef} = 4.44\text{E} - 05\,\text{m}^2$ (with flow-rate ~0.35 l/s). Unsteady friction effects were neglected.

The methodology was applied in a step-wise manner, as described in *Sub-section 4.2*, in which the *Step I* included the definition of leak candidates equally spaced 5% and 10% of the total pipe length. Accordingly, the pipeline with $L = 1280.00$ m was divided into 61 sections, equally spaced ~20 m, not including the inlet pipe which has 90 m. In *Step I*, ITA was carried out for two sets of potential leak locations defined in this table: (i) 9 leak locations spaced $\Delta x^* = 119.00$ m and

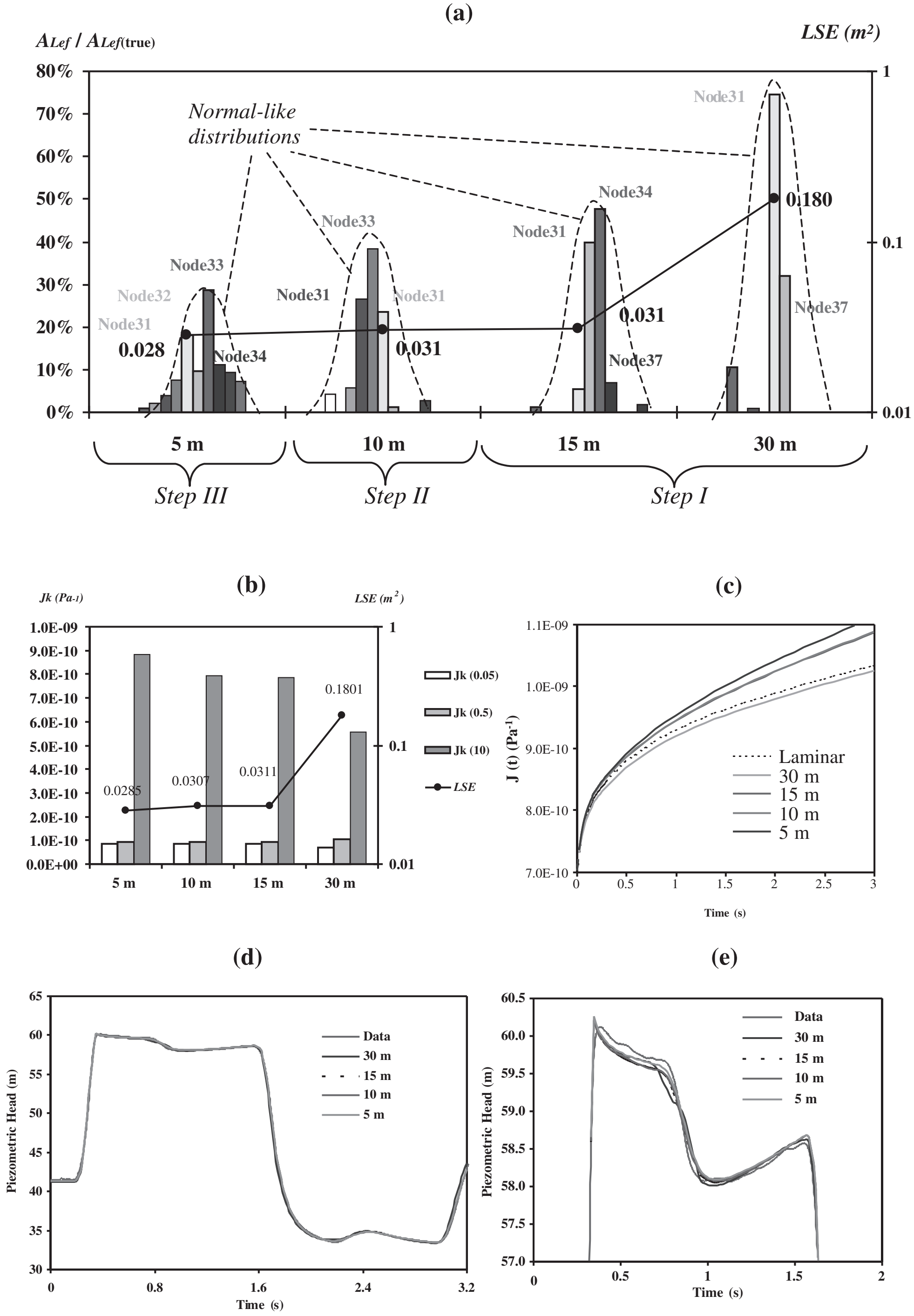

Figure 3. Leak detection results by ITA for $\Delta T = 3$ s, $Q_0 = 1.0$ l/s and a leak with $Q_{L0} = 0.55$ l/s and x = 162.48 m (Node 33): (a) optimal leak locations; (b) optimal creep coefficients; (c) optimal creep functions; (d) and (e) piezometric heads for optimal leak locations vs. experimental data (Covas, 2003).

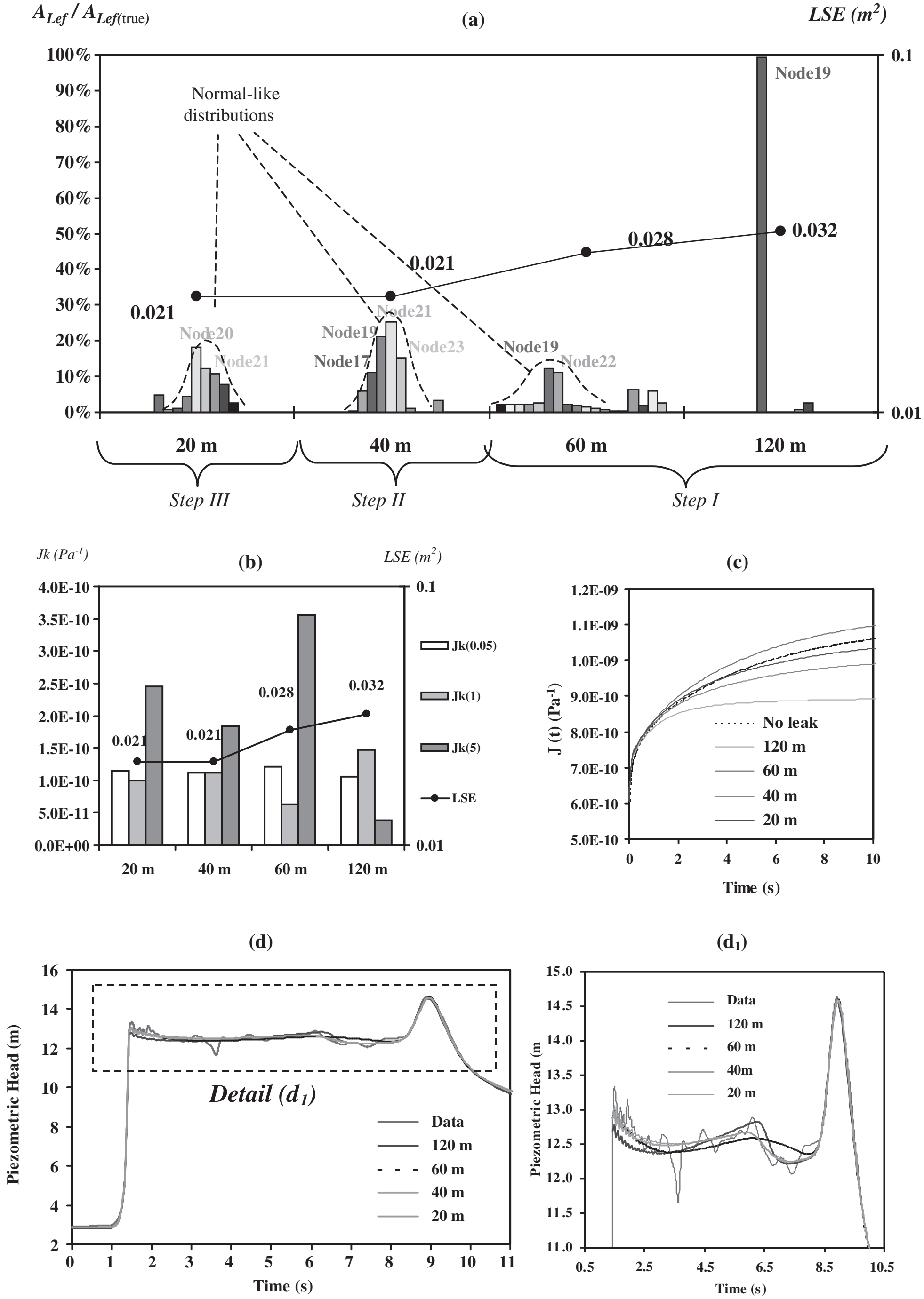

Figure 4. Leak detection results by ITA for $\Delta T = 11\,\mathrm{s}$, $Q_0 = 2.8\,\mathrm{l/s}$ and a leak with $Q_{L0} = 0.35\,\mathrm{l/s}$ and $x = 473.00\,\mathrm{m}$ (Node 20): (a) optimal leak locations; (b) optimal creep coefficients; (c) optimal creep functions vs. calibration for non-leak case; (d) and (e) piezometric heads for optimal leak locations vs. experimental data (Covas, 2003).

(ii) 19 leak locations spaced $\Delta x^* = 59.50$ m. Afterwards in *Step II* and *III*, ITA was run for another two sets of potential leak locations, defined around the highest leak size pointed out in *Step I*: (iii) 11 leak locations with $\Delta x^* = 39.67$ m, covering ~436.00 m near the leak; and (iv) 11 leak locations with $\Delta x^* = 19.83$ m, ~218.00 m near the leak.

Inverse transient analysis was run for $\Delta T = 11$ s, as afterwards the transient wave is completely dissipated and there is no relevant information of the leak (see Figure 1(a)). Transient data collected at the downstream end were used for the optimisation process. Results obtained by the ITS are presented in Figure 4. This figure includes: (a) optimal relative leak sizes (A_{Lef}/A_{Lef_true}) and LSE; (b) optimal creep coefficients and LSE; (c) optimal creep functions and creep function calibrated without simulated leakage; and (d) optimal piezometric heads and experimental data. Each run is referred to by the corresponding approximate distance Δx^* between leak candidates (i.e. 120 m, 60 m, 40 m and 20 m). Based on the analysis of these results, the following conclusions can be drawn:

(i) As the pipeline is divided into smaller sections near the leak location, the ITS tends to spread leaks in the vicinity of the leak with a total flow-rate equal to the leak flow-rate. Consequently, the leak location error diminishes, whereas the leak size error increases. This is because the leak pressure drop is not a sharp change in the pressure.
(ii) Despite the noise in the transient pressure signal, the leak could be correctly located and the ITS did not point to false leaks at the sudden pressure drop due to the air bubble. This is because the response of this feature during a transient event is different from that of a leak.
(iii) The leaks could be located with an accuracy of 24 m, which correspond to 2% of the total pipe length. These results were obtained when there were minor air pockets in the system.

6 CONCLUSIONS

The application of inverse transient analysis (ITA) was tested using physical data collected in the laboratory (IC) and under *quasi*-field conditions (TW). The ITA fails to detect leaks when an inaccurate transient simulator ('Elastic Solver') is used to describe fast transients events in PE pipes. In comparison, ITA has proven to be successful in the detection and location of leaks of a 'reasonable' size, providing an accurate transient solver is used ('Viscoelastic Transient Solver') and the physical and hydraulic characteristics of the system are known.

For leak detection purposes, unsteady friction effects were neglected as these could be well represented by an adequately calibrated creep function of the pipe-wall. Leak location was carried out simultaneously with creep calibration. ITA was applied in a step-wise manner, starting with a description of leak candidates equally spaced at 10% of the total pipe length and gradually reducing it to 2% and 1% near the potential leak locations. Leak location uncertainties depend on the leak size and location, flow regime and location where the transient event is generated, being, in general, less than 2% of the total pipe length.

ITA seems a promising method for identifying the area of the water supply system with leakage, mainly in long pipelines. Although further research is necessary to assess the success of this technique in real-life systems, ITA seems to be particularly useful important for the diagnosis, monitoring and control of existing systems, not only to estimate leak locations and sizes, but also for a better understanding of the causes of pipe bursts induced by transient events.

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REFERENCES

Covas, D. (2003). "Inverse Transient Analysis for Leak Detection and Calibration of Water Pipe Systems – Modelling Special Dynamic Effects." PhD, Imperial College of Science, Technology and Medicine, University of London, London, UK.

Covas, D., Stoianov, I., Graham, N., Maksimovic, C., Ramos, H. and Butler, D. (2002a). "Hydraulic Transients in Polyethylene Pipes.", Proceedings of 1st Annual Environmental & Water Resources Systems Analysis (EWRSA) Symposium, A.S.C.E. EWRI Annual Conference, Roanoke, Virginia, USA (CD-ROM).

Covas, D., Stoianov, I., Graham, N., Maksimovic, C., Ramos, H. and Butler, D. (2002b). "Waterhammer in Polyethylene Pipes: Conceptual Model and Experimental Analysis." Urban Water, Elsevier (*Submitted in 02/2002*).

Covas, D., Stoianov, I., Mano, J., Ramos, H., Graham, N. and Maksimovic, C. (2002c). "The Dynamic Effect of Pipe-Wall Viscoelasticity in Hydraulic Transients. Part II – Model Development, Calibration and Verification." Journal of Hydraulic Research, IAHR (*Submitted in 06/2002*).

Kapelan, Z., Savic, D., Walters, G., Covas, D., Graham, N. and Maksimovic, C. (2003). "An assessment of the application of inverse transient analysis for leak detection: part I – theoretical considerations." Proceedings of CCWI 2003, International Conference on Computers and Control in the Water Industry and the WATERSAVE Network, Imperial college, London, UK, 15–17 September 2003, Pub. Balkema.

Kapelan, Z., Savic, D. A. and Walters, G. (2001). "Use of Prior Information on Parameters in Inverse Transient Analysis for Leakage Detection and Roughness Calibration." Proceedings of Water Software Systems: Theory and Applications, DeMonfort University, Leicester, UK.

Liggett, J. A. and Chen, L. C. (1994). "Inverse transient analysis in pipe networks." Journal of Hydraulic Engineering, ASCE, 120(8), 934–955.

Vitkovsky, J. P., Simpson, A. R. and Lambert, M. F. (2000). "Leak Detection and Calibration Issues using Transient and Genetic Algorithms." Journal of Water Resources Planning and Management, ASCE, 126(4), 262–265.

Pipeline burst detection and location using a continuous monitoring technique

D. Misiunas
Department of Industrial Electrical Engineering and Automation, Lund University, Sweden

J. Vítkovský
School of Civil and Environmental Engineering, University of Adelaide, Australia

G. Olsson
Department of Industrial Electrical Engineering and Automation, Lund University, Sweden

A.R. Simpson & M.F. Lambert
School of Civil and Environmental Engineering, University of Adelaide, Australia

ABSTRACT: Sudden pipe bursts occur in high-pressure water transmission pipelines and water distribution networks. The consequences of these bursts can be very expensive due to the outage time while the burst pipe is repaired, the cost of repair, and damage to surrounding property and infrastructure. As a result, it is advantageous to minimise the detection and location time after the burst occurs. This paper presents a continuous monitoring approach for the detection and location of pipeline bursts using pressure transients. Previous research has shown the potential of fluid transients for pipeline assessment. A sudden pipe burst creates a negative pressure wave that travels in both directions away from the burst point and is reflected at the pipe boundaries. Using pressure measured at one location, the timing of the initial and reflected burst-induced waves determines the location of the burst. The continuous monitoring technique uses the two-sided cumulative sum (CUSUM) algorithm to detect abrupt changes in the pressure data caused by the pipe break. The sensitivity of the algorithm is tuned such that the normal system/measurement noise does not initiate a false alarm. The continuous monitoring technique is verified using results from a laboratory pipeline. Different burst and measurement locations are tested. The results are promising for burst detection and location in real systems.

1 INTRODUCTION

Installed in the first part of the 20th century, municipal water infrastructure systems today are in poor condition and are deteriorating rapidly. As a result, the number of pipe breaks that occur due to corrosion and mechanical stresses is increasing. Pipeline systems with an average annual pipe break ratio per 100 km of 20 to 39 are considered acceptable (Pelletier, 2003). Pipe failure can cause excessive losses both in terms of lost water and other costs resulting from property damage, customer complaints and repair of pipes. It is essential to isolate the damaged section of the network as quickly as possible. Although most of the bursts are quite obvious (the water appearing on the surface), not all of them are detected in a short amount of time. If the burst occurs during the night or in a remote area, it can take a long time for it to be reported and isolated. Furthermore, in some cases, locating the burst point is a complicated and time demanding task. These objectives underlie the need to monitor the system continuously for rapid detection and location of bursts. Once the burst is detected and located it can be isolated and repaired.

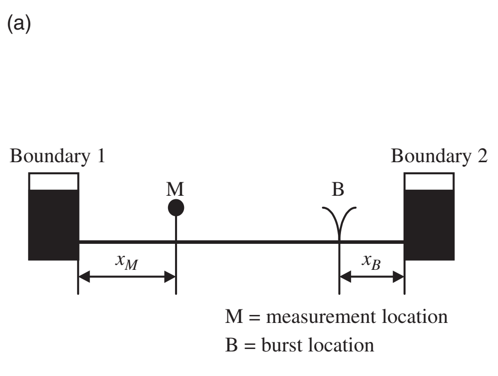
Present practice in the water industry does not provide a comprehensive method for detecting and locating pipe bursts. A number of different techniques for pipe break detection have been applied in the gas and oil industry. Most of them combine continuous monitoring of the physical parameters with some form of mathematical modelling. The methods can be divided into following groups depending on the amount of measurements required:

(i) both pressure and flow measurements at each end of the pipeline (Isermann 1984, Liou and Tian 1995, Mukherjee and Narasimhan 1996, Zhang 2001, Emara-Shabaik *et al*. 2002);
(ii) pressure measurements at each end of the pipeline (Wang *et al*. 1993);
(iii) single pressure measurement (Schlattman 1991, Whaley *et al*. 1992); and
(iv) other measurements, such as sound emission (Rajtar 1997).

Since water distribution mains are considerably less well instrumented than gas or oil pipelines, techniques requiring the least amount of hardware installation are of the most interest. A single-point pressure analysis approach is the least measurement demanding technique. It uses only one pressure measurement point for a pipeline segment. The drawback of existing single point analysis is that, although it detects the burst, it does not locate the burst position. Silva *et al*. (1996) used a number of pressure measurements along the pipeline to detect and locate instantaneous bursts. The algorithm is based on the analysis of the pressure transient caused by the pipe break. Pressure transients have been successfully applied for detecting pre-existing leaks. A transient wave travelling throughout the pipeline, reflecting from the boundaries as well as from the leak can be used to determine the location and size of the leak. The time domain reflectometry technique (Jönsson and Larson 1992, Brunone 1999) is attractive because of its simplicity and the fact that only one pressure measurement point is required. In this paper, the principles of time domain reflectometry are adopted for detecting and locating abrupt pipeline bursts using only one pressure measurement point.

2 BURST LOCATION BASED ON WAVE TIMING

When a burst occurs in a pipeline a negative pressure wave is generated. This negative pressure wave propagates in both directions away from the burst location. Eventually the negative pressure waves reach the boundaries of the pipeline and are reflected. If pressure is measured at two locations, then the burst location can easily be determined from the different negative pressure wave arrival times at each measurement location. However, in many systems only a single measurement location is available. For this case, different features of the reflection pattern are used to determine the burst location. Consider the example pipeline in Figure 1a where a burst occurs at B and the pressure is measured at M. Figure 1b shows the transient pressure measured at M after the burst has occurred.



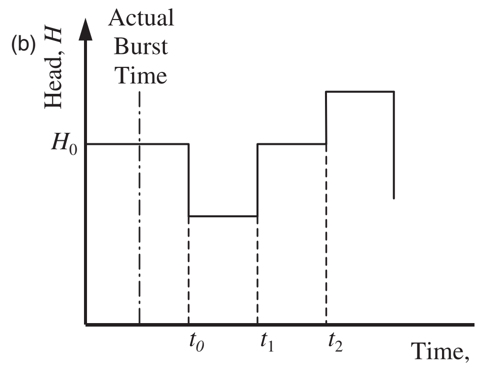


Figure 1. (a) The example pipeline system; (b) The generalised pipe burst transient for the example pipeline system.

Using the two-sided cumulative sum (CUSUM) algorithm for abrupt change detection, the timing of the initial reflections in Figure 1b is determined (t_0, t_1 and t_2). The time differences ($t_1 - t_0$) and ($t_2 - t_0$) correspond to the time taken for either (i) the negative pressure wave to travel from the measurement location (M) to its closest boundary and return, or (ii) the negative pressure wave to travel from the burst location (B) to its closest boundary and return. Therefore the burst could be located at two possible positions; however, since the measurement location is known, the time difference associated with the true burst location can be determined. Once the time difference is known the location of the burst is determined based on the wave speed. In the example pipeline the location of the burst, x_B, is

$$x_B = \tfrac{1}{2} a\left(t_1 - t_0\right) \tag{1}$$

where a is wave speed. The burst location can be confirmed by comparison to a transient model. An approximate burst size can be determined using the Joukowsky pressure rise formula and the orifice equation resulting in

$$C_d A_0 \approx \frac{A \Delta H \sqrt{2g}}{a\sqrt{H_0}} \tag{2}$$

where ΔH = head change due to the burst, H_0 = initial system head; A = pipe cross-sectional area; and $C_d A_0$ = lumped burst orifice parameter. Additionally, the burst size can be estimated using a transient model of the bursting system.

3 ONLINE CHANGE DETECTION APPROACH

For continuous monitoring is it vital to have an algorithm that can reliably detect changes in a system response. The algorithm selected for on-line change detection in the measured signal is illustrated in Figure 2.

The measured signal is first filtered using the adaptive Recursive Least Squares (RLS) filter to reduce the noise content. The filter estimates the signal θ_t from the measurement y_t as

$$\theta_t = \lambda\theta_{t-1} + (1-\lambda)y_t = \theta_{t-1} + (1-\lambda)\varepsilon_t \tag{3}$$

where $\varepsilon_t = y_t - \theta_{t-1}$ is the prediction error and the parameter $\lambda \in [0,1]$ is the forgetting factor. The filtered signal θ_t and the residuals are fed into a two-sided cumulative sum (CUSUM) algorithm (Page 1954, Basseville and Nikiforov 1993) to determine when a change has occurred. Mathematically, the test is formulated as the following time recursion

$$\boxed{\begin{array}{l} g_0^1 = 0 \quad \text{and} \quad g_0^2 = 0 \\ g_t^1 = \max\left(g_{t-1}^1 + \varepsilon_t - \nu, 0\right) \quad \text{and} \quad g_t^2 = \max\left(g_{t-1}^2 - \varepsilon_t - \nu, 0\right) \\ \text{if } \left(g_t^1 > h \text{ or } g_t^2 > h\right) \text{ then } \left(\text{issue alarm and set } t_0 = t,\ g_t^1 = g_t^2 = 0\right) \end{array}} \tag{4}$$

where h and v are appropriately chosen threshold and drift parameters respectively. The RLS filter is restarted after each alarm by setting $\theta_t = y_t$.

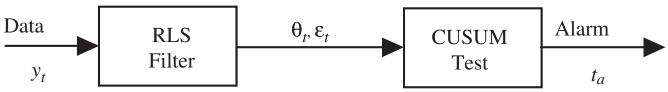


Figure 2. Change detection algorithm structure.

The choice of the design parameters for the CUSUM algorithm is rather arbitrary. There are several guidelines that can be followed. The threshold limits the size of the burst that will be detected. A smaller value of the threshold will result in the detection of smaller bursts. The drift value is dependent on the opening time of the burst orifice. A smaller drift has to be chosen in order to detect slower bursts. It is also important to tune the algorithm in such a way that the rate of false alarms is minimised. The most probable sources of false alarms are the measurement and system noise. The drift value must exceed the noise bandwidth. The RLS filter is used to make the noise bandwidth smaller. The forgetting factor of the filter is chosen with respect to the noise level in the measurement. The systematic approach for choosing CUSUM algorithm parameters is a subject of further research.

4 LABORATORY VALIDATION OF THE CONTINUOUS MONITORING TECHNIQUE

The continuous monitoring approach for burst detection has been verified in a laboratory pipeline at the University of Adelaide. The laboratory apparatus comprises a 37.527 m long copper pipeline of diameter 22.1 mm (Bergant and Simpson 1995). An initial flow in the pipeline is generated by a head difference between two computer-controlled pressurised tanks. The calibrated wave speed of the pipe is 1327 m/s. The burst is simulated using a fast-opening solenoid side-discharge valve. The opening time of the solenoid valve was 4 ms. Additionally, another side-discharge valve that can be actuated manually is used for slower opening times. The setup was such that both tanks were open which is realistic compared to a transmission pipeline in the field. Pressure was measured using a PDCR810 Druck transducer located at a position of 0.1784 along the pipeline. The data acquisition was performed using Visual Designer software with a sampling rate of 2000 Hz.

Tests 1 to 4 consider a fast opening burst located at various positions along the pipeline. The burst positions were at 0.1784, 0.4985, 0.7476, and 0.9936 along the pipeline for tests 1 to 4 respectively.

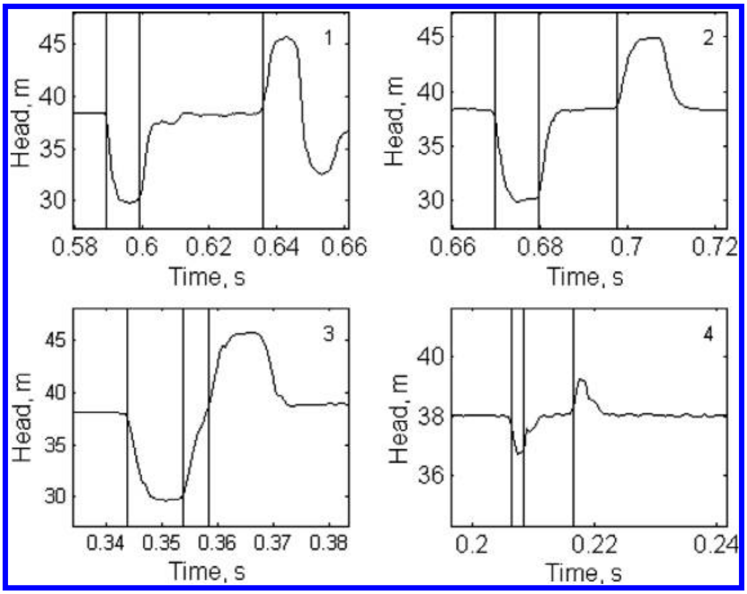


Figure 3. Burst detection for different leak positions along the pipe: 0.1784 (Test 1), 0.4985 (Test 2), 0.7476 (Test 3), and 0.9936 (Test 4). The CUSUM algorithm parameters are $v = 0.1$, $h = 0.4$, $\lambda = 0$. Pressure measurement was located at a position of 0.1784. Vertical lines indicate change points detected by the CUSUM algorithm.

The calibrated size of the burst in each case was $C_dA_0 = 1.7665 \times 10^{-6}\,m^2$. The experimental pressure traces and the online burst detection algorithm results are shown in Figure 3 and Table 1. In each test case the burst was quickly and accurately located. However, the burst occurring in the vicinity of the right-hand tank (at a position of 0.9936) was less accurately located. A reason for this is explained in the following section. In cases 1, 2 and 3 the estimate of the burst orifice size is within approximately $\pm 1\%$ of the correct value. For test 4 the estimated burst orifice size is largely underestimated, again reasons for this are given in the following section.

Test 5 assesses the effectiveness of the online burst detection method when the burst opening occurs over a greater time period. In this case, a manually actuated side discharge valve was used to generate the burst at a position of 0.1784 along the pipeline. The calibrated size of the burst orifice is $6.0192 \times 10^{-7}\,m^2$. Pressure measurement was taken at a position of 0.7476 along the pipeline. The burst was detected, but algorithm was not able to find the correct location. After changing the CUSUM parameters the correct position of the burst was obtained. The values shown in Table 1 reflect the revised CUSUM parameters. Figure 4 shows the measured pressure trace for the slower burst and the online burst detection algorithm results. Since the noise level in the experimental data is low, no filtering is performed ($\lambda = 0$). Although the algorithm works for sudden

Table 1. Summary of experimental burst detection tests.

Test No.	Parameter	Actual	Estimated	Relative error
1	x_B* (−)	0.1784	0.1776	−0.45%
	C_dA_0 (m^2)	1.7665×10^{-6}	1.7727×10^{-6}	0.35%
2	x_B* (−)	0.4985	0.5048	1.26%
	C_dA_0 (m^2)	1.7665×10^{-6}	1.7505×10^{-6}	−0.91%
3	x_B* (−)	0.7476	0.7436	−0.54%
	C_dA_0 (m^2)	1.7665×10^{-6}	1.7618×10^{-6}	−0.27%
4	x_B* (−)	0.9936	0.9646	−2.92%
	C_dA_0 (m^2)	1.7665×10^{-6}	2.7006×10^{-7}	−84.71%
5	x_B* (−)	0.1784	0.1768	−0.90%
	C_dA_0 (m^2)	6.0192×10^{-7}	5.3829×10^{-7}	−10.57%

*Burst location x_B is given as a fraction of the total pipeline length.

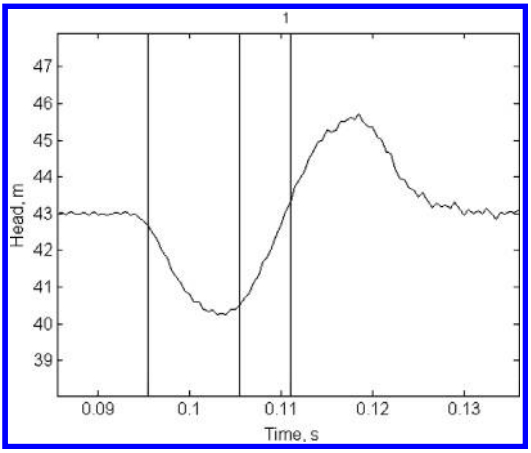


Figure 4. Pressure trace for a slower burst (Test 5). The burst occurs at a position of 0.1784 along the pipeline. Measurements are made at a position of 0.7476 along the pipe. The CUSUM parameters are $v = 0.05$, $h = 0.2$, $\lambda = 0$. Vertical lines indicate change points detected by the CUSUM algorithm.

bursts, further development is required in order to be able to locate bursts with different opening times reliably.

Table 1 presents a summary of the online burst detection algorithm results presented in this section.

5 CONTINUOUS MONITORING TECHNIQUE CONSIDERATIONS

There are a few considerations that remain for the continuous monitoring technique for burst detection and location. Some of the considerations presented in this section are (i) occurrence of bursts near boundaries, (ii) the speed of burst opening, (iii) measurements located at the centre of the pipeline, and (iv) transients caused by normal pipeline operation.

The first two considerations are related and depend on the interaction of a burst with the boundary before the burst has fully opened. In this case, as the burst begins to open a negative pressure wave is generated that propagates away from the burst. If nothing interacts with the burst while it opening then the full pressure decrease is realised. However, if the burst is located near a boundary (like a reservoir) such that the initial negative pressure wave is reflected off the reservoir and arrives back at the burst before the burst has fully opened then the full pressure decrease is not realised. This effect is shown in Figure 5 where the burst is located at a distance x_B from the boundary. The reflection time t_B is equal to

$$t_B = \frac{2x_B}{a} \tag{5}$$

If the burst opening time t_P is greater than t_B then some interaction will occur. An alternative way to explain the relation in Eq. 5 is that the correct size of the burst will not be determined if the distance from the leak to the boundary is

$$x_B < \tfrac{1}{2} t_B \cdot a \tag{6}$$

An example of this is Test 4 in the previous section. Additionally, the minimum of the negative pressure wave is delayed causing a timing error from the change detection algorithm. However, it is likely that the timing error will be constant for all changes required by the burst location calculation and thus not cause much error in the estimated burst position (see estimated burst location in Test 4). Ultimately the successful detection, location and sizing of a burst will depend on both the location of the burst and the burst opening time. The algorithm is tuned to give a good performance for a certain range of bursts. In this respect, for very small size or very slow burst opening the burst will be detected but not correctly located or sized. However it is acknowledged that serious bursts may occur slowly to begin with, but then fail quickly resulting in sharp pressure fronts (that will favour the change detection algorithm).

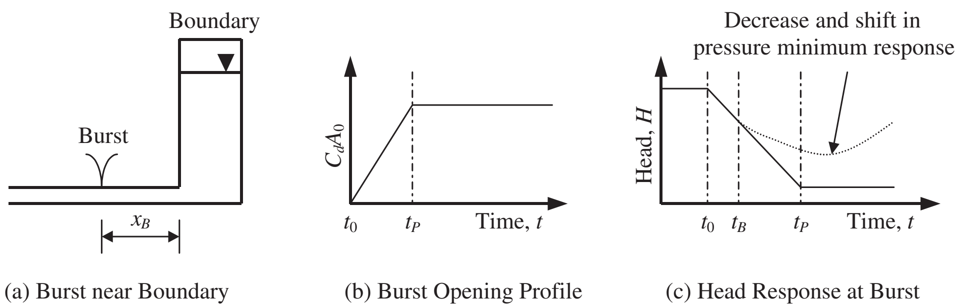


Figure 5. The occurrence of a burst near a pipeline boundary.

Another potential problem is making pressure measurements at the centre of the pipeline. In this case the arrival of the pressure reflections coincide making determination of the true burst location difficult. This problem can be avoided by making pressure measurements at positions other than at the centre of the pipeline.

Finally, the pressure transients caused by the normal operation of the system also have to be taken into consideration. Pump startup and shutdown or valve operation are potential sources of false alarms. One way to deal with this problem is to temporarily disable the burst detection system for the duration of these events. Another option is to model expected transient from the system operation and compare it with the measured one. The discrepancy between predicted and actual pressure traces would indicate a burst event.

6 SUMMARY AND CONCLUSIONS

The continuous monitoring approach presented in this paper shows promise for the efficient detection and location of bursts in pipelines. Experimental data confirms the approach taken for fast bursts. Additional research is required to assess the effect of burst speed on the algorithm, although most burst failures are expected to be rapid.

Even though slower bursts and bursts occurring near boundaries cause problems for this technique, the technique will still detect bursts. It is possible that ancillary techniques, such as inverse transient analysis (Liggett and Chen 1994), transient damping methods (Wang *et al*. 2002), reflection-based methods (Brunone 1999) which use more of the transient signal, could be employed to determine the location and size of the burst.

Additionally, in realistic transmission pipelines the open-open boundary condition may not be the only case. In many such pipelines partially open valves and pumps exist at the boundaries of the system. Further research is required to deal with these phenomena.

REFERENCES

Basseville, M. & Nikiforov, I.V. 1993. *Detection of Abrupt Changes: Theory and Applications*. Englewood Cliffs, NJ: Prentice-Hall.

Bergant, A. & Simpson, A.R. 1995. *Water Hammer and Column Separation Measurements in an Experimental Apparatus*. Research Report R128, School of Civil & Environmental Engineering, University of Adelaide, Australia.

Brunone, B. 1999. Transient test-based technique for leak detection in outfall pipes. *Journal of Water Resources Planning and Management, ASCE*, 125(5), September/October: 302–306.

Emara-Shabaik, H.E., Khulief, Y.A. & Hussaini, I. 2002. A non-linear multiple-model state estimation scheme for pipeline leak detection and isolation. *Proc. Inst. Mech. Eng. Part I: Journal of Systems and Control Engineering*, 216(6): 497–512.

Isermann, R. 1984. Process fault detection based on modelling and estimation methods – A survey. *Automatica*, 20(4): 387–404.

Jönsson, L. & Larson, M. 1992. Leak detection through hydraulic transient analysis. In: *Pipeline Systems*, B. Coulbeck and E. Evans (eds.), Kluwer Academic Publishers: 273–286.

Liggett, J.A. & Chen, L.-C. 1994. Inverse transient analysis in pipe networks. *Journal of Hydraulic Engineering, ASCE*, 120(8): 934–955.

Liou, J.C.P. & Tian, J. 1995. Leak detection – transient flow simulation approaches. *Journal of Energy Resources Technology, Trans. ASME*, 117(3), September: 243–248.

Mukherjee, J. & Narasimhan, S. 1996. Leak detection in networks of pipelines by generalized likelihood ratio method. *Industrial & Engineering Chemistry Research*, 35(6), June: 1886–1893.

Page, E.S. 1954. Continuous Inspection Schemes. *Biometrika*, 41: 100–115.

Pelletier, G., Mailhot, A. & Villeneuve, J.-P. 2003. Modelling water pipe breaks – Three case studies. *Journal of Water Resources Planning and Management, ASCE*, 129(2), March/April: 115–123.

Rajtar, J.M. & Muthiah, R. 1997. Pipeline leak detection system for oil and gas flowlines. *Journal of Manufacturing Science and Engineering, Trans. ASME*, 119(1), February: 105–109.

Schlattman, D.T. 1991. Pressure analysis improves lines' leak-detection capabilities. *Oil & Gas Journal*, 89(52), December: 98–101.

Silva, R., Buiatta, C., Cruz, S. & Pereira, J. 1996. Pressure wave behaviour and leak detection in pipelines. *Computers in Chemical Engineering*, 20: S491–S496.

Wang, G., Dong, D. & Fang, C. 1993. Leak detection for transport pipelines based on autoregressive modelling. *IEEE Transactions on Instrumentation and Measurement*, 42(1), February: 68–71.

Wang, X.-J., Lambert, M.F., Simpson, A.R., Liggett, J.A. & Vítkovský, J.P. 2002. Leak detection in pipeline systems using the damping of fluid transients. *Journal of Hydraulic Engineering, ASCE*, 128(7), July: 697–711.

Whaley, R.S., Nicholas, R.E. & Van Reet, J. 1992. A tutorial on software based leak detection methods. *Pipeline Simulation Interest Group*, October 1992.

Zhang, J. 2001. Statistical pipeline leak detection for all operating conditions. *Pipeline and Gas Journal*, 229(2), February: 42–45.

Optimisation of the use of valves in a network water distribution system for leakage minimisation

L.S. Araujo
Universidade Federal do Ceará, Fortaleza, Brasil

H.M. Ramos
Instituto Superior Técnico, Lisboa, Portugal

S.T. Coelho
Laboratório Nacional de Engenharia Civil, Lisboa, Portugal

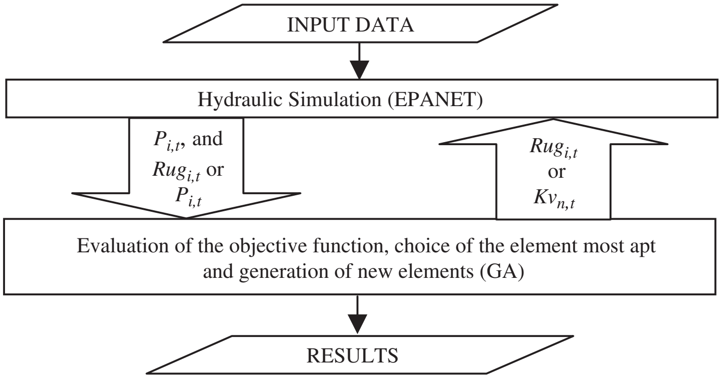
ABSTRACT: The authors developed a model with the capacity to support decisions regarding the quantification, localisation and opening adjustment of valves in a network system, with the objective to minimise pressures and consequently water losses. Two operational modules are established: (1) for the evaluation of an objective-function to optimise the localisation of valves in the network system; (2) for the opening valve adjustment in order to optimise pressures through the evaluation of an aptitude function. New values of these parameters are generated randomly by using the technique of genetic algorithms; in order the system reaches its optimum value in terms of pressure minimisation (*i.e.*, consequently leaks are minimised), and EPA's programmable EPANET routines as a general-purpose for network analysis.

1 INTRODUCTION

In these last decades, one of the main concerns of the water systems managers, of all the world, have been the minimisation of water losses, that frequently reach values of 30% or even 40% from all water that supply the drinking systems. Nowadays, the problem of the water losses and its control in water distribution systems assume more and more importance in the current trend to privilege the sustainability of consumptions and the environmental protection. It is a subject of considerable media and politics visibility, mainly when occur periods of scarcity of water resources or when the water supply are not sufficient in areas with fast growth.

In order to guarantee an adequate technique performance it demands a global evaluation of the system, which includes different scenarios by means of operational conditions for different restrictions of each component. In this way, only through integrated analysis based on several support instruments regarding the system behaviour, it will be able to answer the necessary requirements to attain the maximum efficiency.

Jowitt & Xu (1990), Vitkovský *et al.* (2000), Alonso *et al.* (2000) and Ulanicka *et al.* (2001) present techniques for minimisation the pressure as a conditional parameter of leak indicators in the water network systems. Regarding the methodology to be used in the reduction of pressures in the system, Jowitt & Xu (1990), Reis *et al.* (1997), Kalanithy & Lumbers (1998), Tucciarelli *et al.* (1999), Reis & Chaudhry (1999) and Ulanicka *et al.* (2001) have suggested the best solution including the use of elements which provoke head losses, such as pressure reducing valves. Vairavamoorthy & Lumbers (1998) analyse the optimum localisation of valves. The optimum number of valves is other crucial step in this process.



Figurre 1. Main structure of the general methodology ($P_{i,t}$: Pressure in node i, $Rug_{i,t}$: Roughness in pipes i and $Kv_{n,t}$: Coefficient of head or pressure loss in the valve and t: time)

Savic & Walters (1995ä, b) and Reis & Chaudhry (1997) had developed models to locate, in an optimum way, valves in water distribution systems. However it is important to enhance that the first ones had worked with valves operating in completely open or closed position and using the static simulation. Reis & Chaudhry (1997), even had investigated the localisation of valves in static simulation, also analysed several scenarios of consumption during the optimisation process. The total number of valves was pre-specified.

With the objective to obtain a solution that allows simultaneously to optimise the number of valves and its localisation, and later a refinement in the adjustment of the valves opening for simulation in extended period[1], basically it has been developed a methodology constituted by two phases: (1) for the evaluation of an objective-function established to optimise the number and the location of valves; (2) adjustment of valves opening in order to optimise pressures through the evaluation of another aptitude-function. The new values for these parameters are generated randomly using the technique of genetic algorithms, in order the system reaches its optimum value for minimum pressures (*i.e.*, minimising leaks), and EPA's programmable EPANET routines with the general purpose to analyse network systems. Figure 1 shows the main structure of the presented methodology.

2 METHODOLOGY

The hydraulic simulation of the system is developed by routines based on EPANET 2.0. The choice of this tool is due to be a widely tested robust model and, with a large community of users in all the world (*e.g.*, Martínez *et al*. (1999), Hernández *et al*. (1999); Sakarya & Mays (2000); Prescott & Ulanicki (2001); Araujo *et al*. (2002, 2000a). For each user, it is allowable to access to all capacities of simulation of the model through programmable routines in C, Basic or Paschal in MS Windows® environment (there is a Portuguese version of EPANET 2.0 (*i.e.*, www.dh.lnec.pt/Nes/epanet)).

The hydraulic simulator needs, for each time interval, to know the values of the intervening variables in the modelling process, namely values of roughness in each branch pipe and head loss coefficients or pressure for each valve. A Genetic Algorithm (GA) develops the generation of these values that it is also responsible for the optimisation of the process as a whole.

[1] Sometimes also called by "dynamic simulation", for the network operation in time, with conditions of changeable consumption and exploitation, for succession of static simulations.

GAs are a part of the meta-heuristic[2], non-deterministic techniques, of search, optimisation and learning of industrial equipment, which manipulate a space of possible solutions using adaptive mechanisms to the "Natural Selection of Darwin". Thus, the Darwinian theory of the natural selection and the paradigm of the survival of the most apt had inspired the development of relatively recent computational techniques, as artificial intelligence and the evaluative computation. Among these techniques, is distinguished the developed by Holland (1975) and well known by the technique of Genetic Algorithms. This technique is robust and efficient in irregular, multidimensional and complex spaces of search and, according to Goldeberg (1989, 1994), it presents characteristics such as they do not require derivatives, operates in a population of points, works on representative form of parameters (normally binary representation), uses non-deterministic rules or, probabilistic, and, for each element of a population, it requires information only on the value of an aptitude-function. These techniques have been commonly used, with sufficient success, in several fields of sciences, inclusively in the resolution of optimisation problems of water distribution systems.

This research uses a generational-conventional GA – in which all the population is substituted by new elements generated by the process of selection and application of operators: "mutation" and "crossover". As a "living together" between parents and children does not exist; these substitute totally the parents, having the possibility of loss good elements. This inconvenience is overcome by the application of "elitism", or is equivalent to preserve the best element of a generation by passing, directly, a copy for the next generation. In order to keep a good level of competition and prevent a convergence premature, during the computer running of the GA, it was adopted the technique of "linear scheduling".

2.1 *First phase*

This first stage consists in the optimisation of the quantification and localisation of possible valves in the water distribution system. The consideration of pseudo valves in the network, in each pipe branches, is simulated as an additional roughness that minimise pressures in the nodes. The scenarios of localisation of these pseudo valves and its respective head losses (*i.e.*, opening degree) are generated randomly, in order the system reaches the objective of minimum pressures, with an optimum number and localisation of valves (*i.e.*, number of pipes with physic roughness (*i.e.*, head loss) greater than 90% (adopted here as the optimum penalty) of the real roughness). The proposal methodology is applied to network systems operating in steady-state or extended regime.

The mathematical formulation, for this component of the optimisation, is based on the following aptitude-function:

- *Optimize* $$f(p_i, nv)\Big|_{t=1}^{T} = nv_t \Big/ \left\{ \sum_{i=1}^{N} \left[\frac{\left(P_{cal,i,t} - P_{\min}\right)}{P_{\min}} \right]^2 * nv_t + nv_t \right\}^2 \Big|_{t=1}^{T} \qquad (1)$$

with T the total number of intervals to simulate (normally equal to 24 intervals of 1 hour); N the total number of nodes; $P_{cal,i,t}$ the pressure calculated in the node i for the hour t; $P_{\min}$ the minimum pressure, pre-established by the user, for any node of the network and nv_t the number of valves calculated for instant t (*i.e.*, number of pipes with roughness greater than the original roughness – *i.e.*, small Hazen-Williams coefficients) being, therefore, a conditioning of the formulated problem, in order to have lesser possible number of pipes with Hazen-Williams coefficients below to the real ones (*i.e.*, minor number of possible localisations for the valves).

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[2] Heuristic – all the approach methods created specifically to solve a type of problem in polynomial time. Meta-heuristic is a heuristic type of current use that give good solutions but, also, they do not guarantee the optimum solution (VIANA (1998)).

2.2 *Second phase*

This module of optimisation has the purpose to establish adjustment for the valves opening (Kv) – coefficients of head losses for Throttle Control Valves (TCV) or outlet pressure for Pressure Regulating Valves (PRV), in order to propitiate, to the manager of the system, more subsidies, depending on the number and localisation of really necessary valves which are economically and technical viable or, simply, to optimise opening adjustments of already existing valves in the network. In this particular case, the necessity of adopting the first phase of this model is optional.

The mathematical formulation, for this component of optimisation, is based on the following aptitude-function:

- *Optimize* $$f(p_i)\Big|_{t=1}^{T} = 1\Big/\sum_{i=1}^{N}\left[\frac{\left(P_{cal,i,t} - P_{\min}\right)}{P_{\min}}\right]^2\Big|_{t=1}^{T} \quad (2)$$

with T; N; $P_{cal,i,t}$; and $P_{\min}$ the same mean presented for eq.(1).

3 APPLICATION

The case study chosen to test the proposal methodology is a widely divulged example in several papers, namely in Jowitt & Xu (1990), Savic & Walters (1995a, b) and Reis & Chaudhry (1997). The same value presented by the former researchers for the exponent β (1.18) of the pressure in the leak equation is adopted. It was distributed, between nodes, values for the leaks coefficient (K_f), in order the losses could be considered in the resolution of the hydraulic balance of the network. For the distribution of these coefficients, as well as of the new values of the consumption factors (Fc), it was adopted the methodology developed by Araujo *et al.* (2003). Figure 2 shows the network of the case study. In Appendix, the physical data and the distribution of the base consumptions by nodes in the network and their respective leak coefficients (K_f) are presented.

The simulation of the typified network was made for a period of 24-hour, with intervals of 1 hour. The objective consists to minimise the pressure, but to not let it be lower than 30 m in any node of the system. The program was run in a computer equipped with a processor of 1.0 GHz and 512 KB of memory RAM. For the genetic algorithm, a population with 100 individuals was used as well a probability of crossover of 0.8, a probability of mutation of 0.001 in the first phase and of 0.01 in the second phase. It was also used "elitism" and "linear scheduling" with constant values of 1.25.

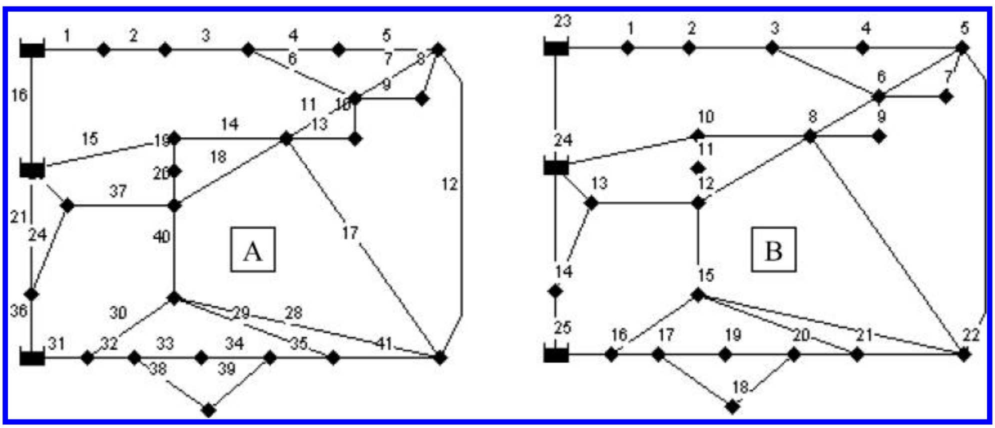

Figure 2. Network with identification of the pipes (A) and nodes (B) (adapted from Jowitt & Xu, 1990).

The results obtained in the first phase are used as input data for the second phase in this proposed model.

3.1 *First phase*

After running this phase, the program generates results that can be used by EPANET program in order to propitiate to each user the possibility to define where and how many valves will be used to model the network in the second phase. Table 1 shows the pipes that had been altered, in its roughness, for each time interval of the simulated hour. A simple statistics evaluation for the discharges and pressures are also presented. A specific analysis of this table allows to get interesting conclusions: if one valve has been installed it would indubitably be in pipe 37; the best combination, in all aspects, is of hour 2 with valves in pipes 1, 31, 37, 40 and 41, an average pressure of 32 m, with standard deviation 1.4 and reduction of 6.8 (l/s) in the leak discharge, modifying these from 29 to 22 (l/s). Based on theses analysis, it was decided to make a deepener evaluation, running the second phase of the model, with the combinations presented in Table 2. The combination with 3 valves, although does not appear isolate in Table 1, it was chosen because it appears in

Table 1. Compilation of results generated by the model in the first phase, for the case study.

Pipe	1	5	15	18	28	30	31	32	37	40	41	23	Flow (l/s)				Pressure (m)			
Time (h)	Hazen-Williams coefficients												Total Inflow	Initial Leaks	Modelled Leaks	Leaks reduction	Maximum	Averages	Minimum	Stand. Desv.
01	100	99	90	100	1	113	1	129	15	86	91	103	120	28	23	5.5	37	33	30	2.0
02	3	103	95	103	91	114	1	132	6	1	1	100	120	29	**22**	**6.8**	**35**	**32**	**30**	**1.4**
03	100	104	63	110	1	125	5	127	7	86	90	101	90	29	23	5.9	38	34	30	2.8
04	100	100	38	99	1	125	5	127	10	1	90	99	90	29	22	6.7	38	32	30	2.2
05	103	110	25	109	91	124	1	1	16	86	90	103	90	29	23	6.0	38	33	30	2.3
06	100	106	51	17	2	124	5	127	99	48	1	1	90	28	23	5.2	38	33	30	2.6
07	99	102	90	102	91	2	1	140	25	86	91	102	150	28	23	4.8	38	33	30	2.2
08	8	103	94	108	90	117	6	127	53	86	91	105	150	26	23	2.8	37	34	30	1.8
09	100	101	90	100	1	114	5	127	56	86	90	99	210	26	23	2.5	37	33	30	2.0
10	7	99	8	109	90	116	6	132	103	25	91	100	210	26	23	3.1	37	33	30	2.0
11	101	100	91	107	1	123	5	135	42	86	91	100	195	26	23	3.1	37	33	30	1.9
12	8	100	16	44	15	119	6	134	100	33	94	100	195	27	22	4.5	37	33	30	2.1
13	100	104	94	39	29	118	5	128	42	30	94	100	165	27	23	3.8	37	33	30	2.1
14	104	1	94	100	1	116	5	127	28	86	94	100	165	27	23	3.8	37	33	30	1.8
15	99	104	97	18	91	117	1	127	103	89	91	99	165	27	24	2.8	38	34	30	2.6
16	100	100	91	99	90	124	5	128	99	1	90	100	165	26	24	2.3	39	34	30	2.8
17	8	99	99	104	90	115	5	127	40	86	91	99	180	26	23	3.1	37	34	30	2.1
18	8	100	91	99	1	124	1	138	35	88	99	101	180	27	22	4.5	38	32	30	1.8
19	101	101	14	109	91	114	5	126	47	86	91	110	165	27	23	3.2	38	34	30	2.3
20	100	100	92	103	1	124	5	127	22	86	91	102	165	27	23	4.2	38	33	30	2.2
21	101	104	92	103	90	117	5	126	24	86	90	103	150	27	23	3.8	38	34	30	2.4
22	100	99	91	100	91	125	5	126	15	86	90	99	150	28	24	4.2	38	34	31	2.4
23	100	100	96	106	94	125	5	126	12	86	91	103	120	28	23	4.8	38	34	30	2.5
24	104	104	20	102	26	115	5	130	24	3	94	100	120	28	22	5.8	37	32	30	2.0
Real	110	110	100	110	100	125	6	140	110	95	100	110								

Table 2. Combinations of valves to be considered in the second phase of the modelling.

Combinations	No. of valves	No. of pipes						
1	1	–	–	–	–	37	–	–
2	2	01	–	–	–	37	–	–
3	3	–	–	–	–	37	40	41
4	4	01	–	28	31	37	–	–
5	5	01	–	–	31	37	40	41
6	6	01	15	28	31	37	40	–

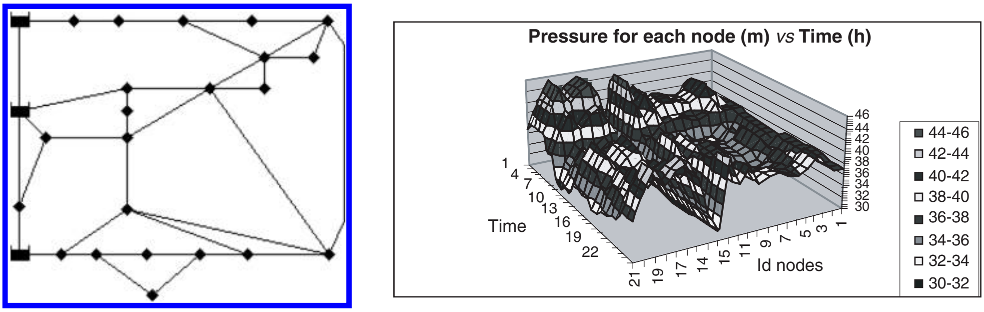

Figure 3. Distribution of pressures by nodes for each time interval in the network without valves.

several works presented in conferences and specialised publications. However, the combination with 6 valves is to show the set of pipes that had its roughness modified at least 6 times.

3.2 *Second phase*

This part of the proposed methodology has the objective to establish values of opening-adjustment for the valves, in order to minimise the pressures in the network. Either it can be used independently, since there are installed valves, or complementarily to the first phase, as in this case study. In order to verify the proposed combinations on Table 2, from the first phase, Throttle Control Valves (TCV) had been used, for discharge control. Furthermore it was also considered control parameters for each interval of simulated hour (T = 24). The control parameters constitute the variables, inherent to each valve, to be modified for the GA, in order to minimise the pressures in the network.

The analysis of the results generated in this phase allows a more refined condition of the real situation of the network regarding the use of valves. In Figure 3, the behaviour of the network can be observed when valves are not used. A sufficiently reasonable variation of the pressures is noticed, relatively to the time interval and the node, with values between 30 and 46 m. In Figures 4, 5, 6, 7, 8 and 9 can be seen the pressures behaviour when valves are installed. It is interesting to observe that for any amount of valves, a reduction in the maximum platform of the pressures was obtained, changing from 46 to 38 m. Above this value, the behaviour depends on the number and the localisation of the valves. Combinations with 6 and 4 TCV had presented the best results for this parameter.

Analysing the behaviour of leaks, it is noticed, in the left side of Figure 10, the combination with 6 TCV presents the best results, losing only, at some instants, to the combination with 4 TCV, which can be evidenced when observing the right side of the same figure. The platform for leakage was changed from 27.3 (l/s) to 22.1 (l/s). In Figure 11, is enhanced the supremacy of the combination with 6 TCV that it obtains an average reduction of 5.2 (l/s) in the leak discharge, followed by the combination of 4 TCV, with an average reduction of 5.1 (l/s). It is observed that, regarding

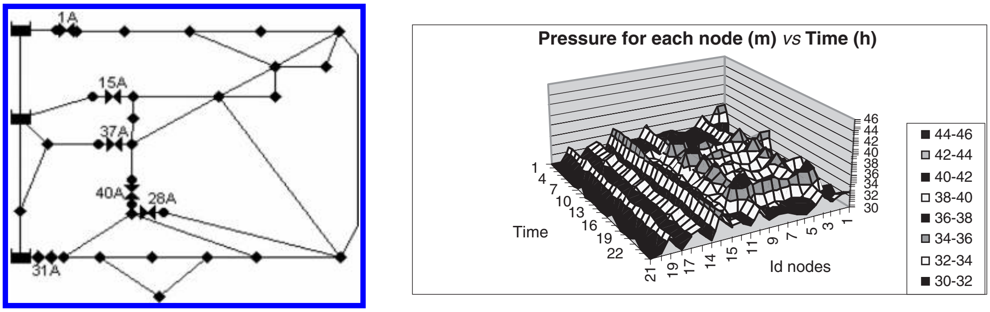

Figure 4. Distribution of the pressures by nodes for each time interval in the network with 6 valves.

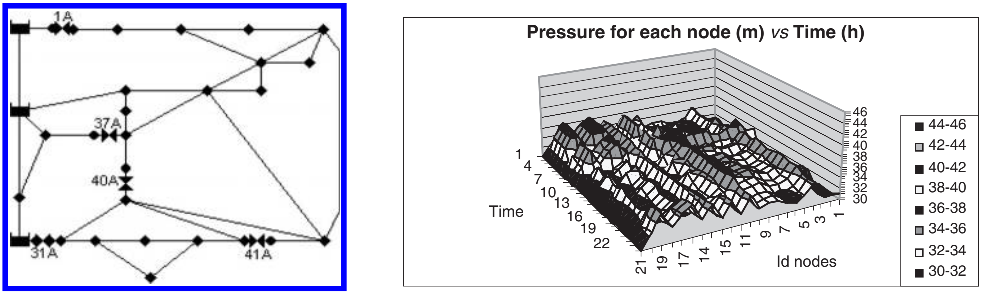

Figure 5. Distribution of the pressures by nodes for each time interval in the network with 5 valves.

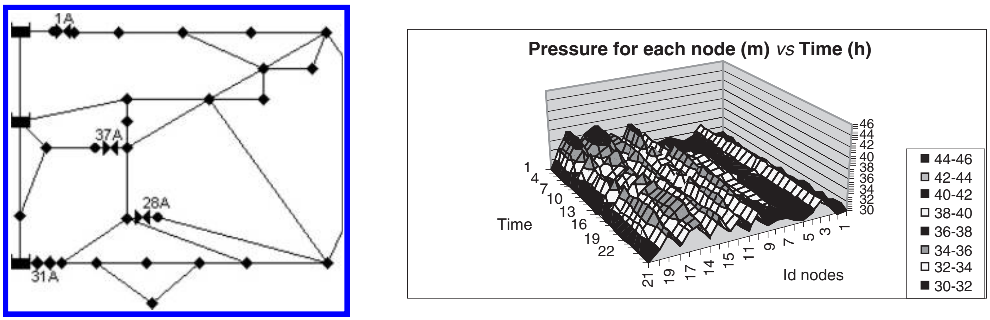

Figure 6. Distribution of the pressures by nodes for each time interval in the network with 4 valves.

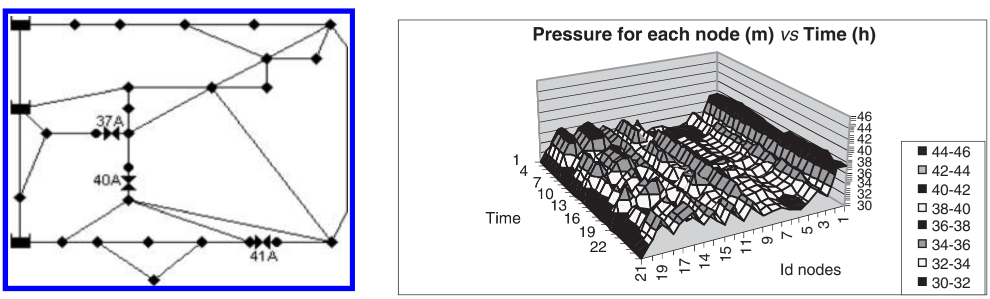

Figure 7. Distribution of the pressures by nodes for each time interval in the network with 3 valves.

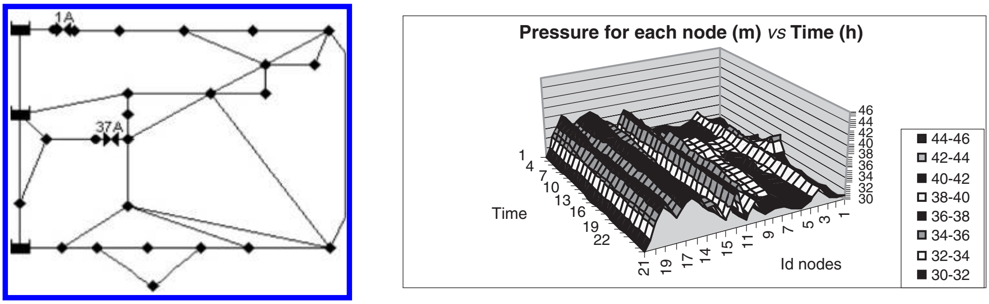

Figure 8. Distribution of the pressures by nodes for each time interval in the network with 2 valves.

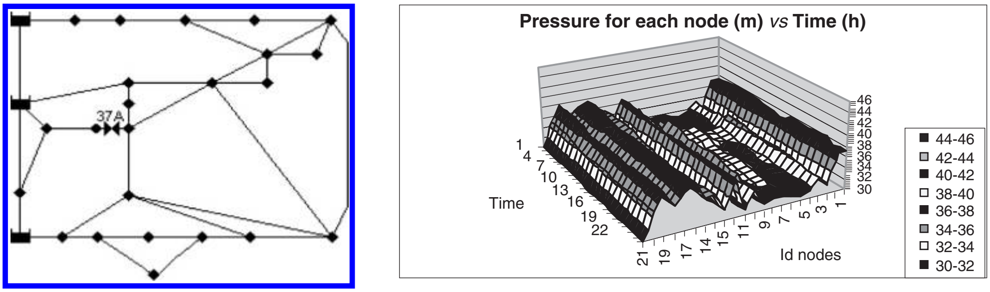

Figure 9. Distribution of the pressures by nodes for each time interval in the network with 1 valve.

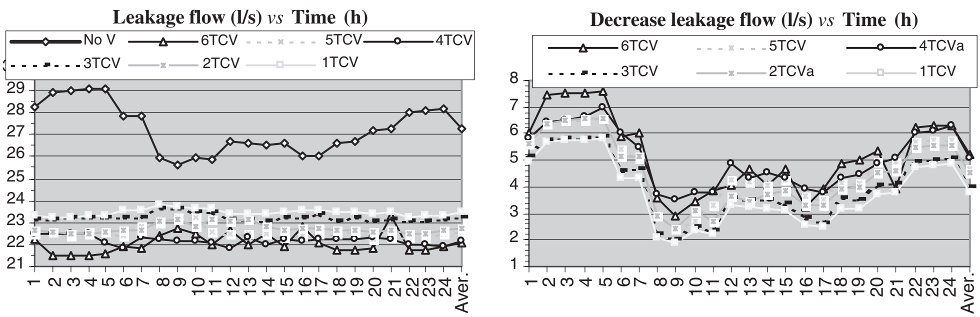

Figure 10. Distribution of the leak discharges and reductions for each node along the time.

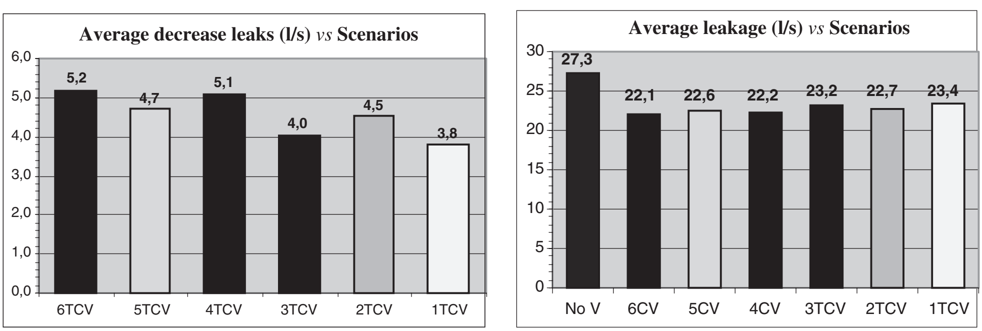

Figure 11. Comparison between average leaks and reduction leakage, for conditions without control and with control for several combinations of valves.

this variable, the amount of installed valves is lesser important than its localisation. Therefore, it is verified that the solution with 2 valves "well" located, gives better results than with 3 "bad" located valves. Furthermore, 4 valves can be a solution well attractive than a solution with 6, for both analysed variables: pressures and essentially, leaks.

4 CONCLUSIONS

An efficient strategy of control and minimisation of pressures is a good operational tool for leakage reduction in networks of water supply systems, without compromising the system operationality. However, when methodologies of location and optimisation opening-adjustment of valves are associated to the control strategy, the profits are indubitably more significant.

The proposed model in this work optimises the number and the location of head loss valves, as well as their opening adjustments, for an effective control of leakage discharges by a reduction of extreme pressures. The model shows to have the capacity to find a solution that fully satisfies the management of the extreme pressures without introducing significant constraints to the efficiency and functionality of this network system.

ACKNOWLEDGEMENTS

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BIBLIOGRAPHY

Alonso, J.M., Fernando, A., Guerrero, D., Hernández, V., Ruiz, P.A., Vidal, A.M., Martinez, F., Vercher, J. & Ulanicki, B. 2000. Parallel Computing in Water Network Analysis and Leakage Minimization. Journal of Water Resources Planning and Management, ASCE, July/August, pp. 251–260.

Araujo, L.S., Ramos, H.M. & Coelho, S.T. 2002. Integrated management of water distribution systems for a better control of losses (in Portuguese: Gestão Integrada de Sistemas de Distribuição de Água para um Melhor Controlo de Perdas). 6° Congresso da Água, 12/22 March, 2002. Porto, Portugal.

Araujo, L.S., Ramos, H.M. & Coelho, S.T. 2002a. Optimisation of valves location in a network distribution system for leaks minimisation (in portuguese: Optimização da Localização de Válvulas numa Rede de Distribuição para a Minimização de Fugas). 10° ENaSB/SILUBESA, 16/19 September, 2002a. Braga, Portugal.

Araujo, L.S, Ramos, H.M. & Coelho, S.T. 2003. Estimation of Distributed Pressure-Dependent Leakage and Consumer Demand in Water Supply Networks. Advance in Water Supply Management. Imperial College London, UK, 15–17 September 2003.

Goldberg, D.E. 1989. Genetic Algorithms in Search, Optimization and Machine Learning, Adison-Wesley, USA.

Goldberg, D.E. 1994. Genetic and Evolutionary Algorithms Come of Age. Communications of the ACM, Vol. 37, N° 3, pp. 113–119.

Hernández, V., Martínez, F., Vidal, A.M., Alonso, J.M., Alvarruiz, F., Guerrero, D., Ruiz, P.A. & Vercher, J. 1999. HIPERWATER: A high Performance Computing EPANET-Based Demonstrator for Water Network Simulation and Leakage Minimisation. Water Industry Systems: Modelling and Optimization Applications, Vol. 1, Research Studies Press Ltd., Baldock. Hertfordshire, England, pp. 141–153.

Holland, J.H. 1975. Adaptation in Natural and Artificial Systems. University of Michigan Press, Ann Arbor.

Jowitt, P.W. & Xu, C. 1990. Optimal Valve Control in Water Distribution Networks. Journal of Water Resources Planning and Management, ASCE, July/August, pp. 455–472.

Kalanithy, V. & Lumbers, J. 1998. Leakage Reduction in Water Distribution Systems: Optimal Valve Control. Journal of Hydraulic Engineering, ASCE, November, pp. 1146–1154.

Martínez, F., Conejos, P. & Vercher, J. 1999. Developing an Integrated Model for Water Distribution Systems Considering both Distributed Leakage and Pressure-Dependent Demands. Proceedings of the 26th ASCE Water Resources Planning and Management Division Conference. July. Tempe, Arizona.

Prescott, S.L. & Ulanicki, B. 2001. Time Series Analysis of Leakage in Water Distribution Networks. Water Software Systems: Theory and Applications. Vol. 2, Research Studies Press Ltd., Baldock, Hertfordshire, England, pp. 17–28.
Reis, F.R. & Chaudhry, F.H. 1999. Hydraulic Characteristics of Pressure Reducing Valves for Maximum Reduction of Leakage in Water Supply Networks. Water Industry Systems: Modelling and Optimization Applications, Vol. 1, Research Studies Press Ltd., Baldock, Hertfordshire, England, pp. 259–267.
Reis, F.R., Porto, R.M. & Chaudhry, F.H. 1997. Optimal Location of Control Valves in Pipe Networks by Genetic Algorithm. Journal of Water Resources Planning and Management, November/December. pp. 317–326.
Rossman, L.A. 2000. EPANET 2 Users Manual. Water Supply and Water Resources Division, National Risk Management Research Laboratory, Cincinnati, OH 45268, September. (traduzido para português por D. Loureiro e S.T. Coelho, LNEC 2002 www.dh.lnec.pt/Nes/epanet)
Sakarya, A.B.A. & Mays, L.W. 2000. Optimal Operation of Water Distribution Pumps Considering Water Quality. Journal of Water Resources Planning and Management, ASCE, July/August. pp. 210–220.
Savic, D.A. & Walters G.A. 1995a. Integration of a Model for Hydraulic Analysis of Water Distribution Networks with an Evolution Program for Pressure Regulation. Microcomputers in Civil Engineering, Vol. 10. pp. 219–229.
Savic, D.A. & Walters G.A. 1995b. An Evolution Program for Optimal Pressure Regulation in Water Distribution Networks. Engineering Optimization. Vol. 24, N° 3. pp. 197–219.
Tucciarelli, T., Criminisi, A. & Termini, D. 1999. Leak Analysis Systems by Means of Optimal Valve Regulation. Journal of Hydraulic Engineering, ASCE, March, pp. 277–285.
Ulanicka, K., Bounds, P., Ulanicki, B. & Rance, J. 2001. Pressure Control of a Large Scale Water Distribution Network with Interacting Water Sources: A Case Study. Water Software Systems: Theory and Applications, Vol. 2, Research Studies Press Ltd., Baldock, Hertfordshire, England, pp. 41–53.
Vairavamoorthy, K. & Lumbers, J. 1998. Leakage Reduction in Water Distribution Systems: Optimal Valve Control. Journal of Hydraulic Engineering, ASCE, November, pp. 1146–1154.
Viana, G.V.R. 1998. Meta-heuristic and parallel programming in combinatory optimisation (in Portuguese: Meta-heuristicas e Programação Paralela em Otimização Combinatória). Edições UFC, Fortaleza, Brasil.
Vítkovský, J.P., Simpson, A.R. & Lambert, M.F. 2000. Leak Detection and Calibration using Transients and Genetic Algorithms. Water Resources Planning and Management, ASCE, July/August, pp. 258–262.

APPENDIX

Table A1. Data of the pipes and nodes for the system presented in Figure 2.

ID_Pipe	Length (m)	Diameter (mm)	Roughness	ID_Node	Elev. (m)	Base Cons. (ls^{-1})	Emitter (K_f) ($ls^{-1}m^{-1/2}$)
P_01	606	457	110	N_01	18,0	5,0	0,012055
P_02	1930	457	110	N_02	18,0	10,0	0,033656
P_03	5150	305	10	N_03	14,0	0,0	0,032088
P_04	326	152	100	N_04	12,0	5,0	0,005562
P_05	844	229	110	N_05	14,0	30,0	0,018383
P_06	1274	152	100	N_06	15,0	10,0	0,019238
P_07	1115	229	90	N_07	14,5	0,0	0,005300
P_08	500	381	110	N_08	14,0	20,0	0,018853
P_09	615	381	110	N_09	14,0	0,0	0,003532
P_10	300	299	90	N_10	15,0	5,0	0,019837
P_11	743	381	110	N_11	12,0	10,0	0,006270
P_12	1408	152	100	N_12	15,0	0,0	0,024410
P_13	443	229	90	N_13	23,0	0,0	0,016842
P_14	249	305	105	N_14	20,0	5,0	0,019490
P_15	3382	305	100	N_15	8,0	20,0	0,028884
P_16	454	457	110	N_16	10,0	0,0	0,013467
P_17	931	229	125	N_17	7,0	0,0	0,010957
P_18	1600	457	110	N_18	8,0	5,0	0,005286
P_19	542	229	90	N_19	10,0	5,0	0,009203

(*continued*)

Table A1. (*continued*)

ID_Pipe	Length (m)	Diameter (mm)	Roughness	ID_Node	Elev. (m)	Base Cons. (ls^{-1})	Emitter (K_f) ($ls^{-1}m^{-1/2}$)
P_20	777	229	90	N_20	7,0	0,0	0,010819
P_21	2782	229	105	N_21	10,0	0,0	0,020118
P_24	1014	381	135	N_22	15,0	20,0	0,034997
P_28	2334	229	100				
P_29	832	152	90				
P_30	914	229	125				
P_31	1097	381	6				
P_32	822	305	140				
P_33	1072	229	135				
P_34	864	152	90				
P_35	711	152	90				
P_37	762	457	110				
P_38	411	152	100				
P_39	701	229	110				
P_40	1996	229	95				
P_41	2689	152	100				
P_23	1767	475	110				
P_36	304	381	135				

Table A2. Data of the consumption factors (*Fc*) and 23, 24 and 25 reservoirs water levels.

Time	1	2	3	4	5	6	7	8	9	10	11	12
Fc	0,61	0,61	0,41	0,41	0,41	0,41	0,81	0,81	1,23	1,23	1,13	1,13
23	55,2	55,3	55,5	55,6	55,7	55,8	55,9	56,0	55,7	55,4	55,2	55,1
24	55,2	55,3	55,3	55,4	55,4	55,5	55,5	55,5	55,3	55,2	55,0	54,8
25	55,0	55,1	55,2	55,3	55,4	55,4	55,5	55,5	55,5	55,0	54,8	54,7
Time	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
Fc	0,92	0,92	0,92	0,92	1,03	1,03	0,92	0,92	0,82	0,82	0,61	0,61
23	54,9	54,7	54,6	54,6	54,5	54,5	54,6	54,7	54,8	54,9	55,0	55,2
24	54,8	54,8	54,7	54,6	54,6	54,5	54,7	54,7	54,7	54,8	54,9	55,0

Introduction of a fully dynamic representation of leakage into network modelling studies using Epanet

R. Burrows, G. Mulreid & M. Hayuti
Department of Civil Engineering, The University of Liverpool, UK

J. Zhang & G.S. Crowder
Crowder & Co Ltd, Birkenhead, UK

ABSTRACT: The paper presents a practical implementation of fully pressure-dependent leakage specifications in routine network modelling studies of the water distribution system. This enhancement yields a capability to construct models which, following calibration, are capable of responding automatically in their leakage figures to the pressures created in any chosen operational (or planning based) scenario. The procedure is demonstrated using the EPANET-2 software package, which is freely available from the world-wide web (www), and is applied to 58 small "District-Metered-Area" (DMA) networks in the UK.

The direct benefits to the water utility are that leakage will self adjust automatically according to the designated flow scenario (ie hottest day, time horizon forecast etc), it being fully driven by the system pressures arising. This enables operational and planning scenarios to be run with only the primary elements of demand (ie domestic and trade/industrial) pre-determined. Leakage is distributed across the system according to number of connections (or length of mains) as well as the local pressures. Leakage levels are first set from field monitoring and model calibration and can be updated as the status of the infrastructure changes. The procedure leads to the highest achievable accuracy in total daily leakage assessment, and post-processing of demand using the pressure dictated leakage, significantly improves the quality of domestic consumption statistics (per capita consumption [PCC] and profile). Pressure management, involving PRV installation and possible re-zoning, is simulated in full accord with the hydraulic situation (without need for leakage data re-evaluation) and so produces highly reliable estimates of water savings, etc.

1 INTRODUCTORY BACKGROUND

Hydraulic (flow/pressure) modelling of water distribution systems using computer based network models is now firmly established within the UK water industry, following first introduction upon the PC platforms from the late 1980s. Early implementations were focused upon strategic (trunk main) systems, capable of simplification in respect of pipe system complexity etc. Increase in computer power has more recently enabled the construction of all-mains models, extending into the streets supplying individual properties. Most UK Water Companies now possess models of their local distribution systems, down to the pipes comprising the individual District-Metered-Areas (DMA) used for local performance monitoring. The authors have discussed aspects of the network modelling and information system needs for holistic distribution management strategies in earlier articles [Burrows et al (1999, 2000), Crowder and Burrows (1998), Crowder (1999)], together with more recent contributions on pressure dependent demand [Tanyimboh et al (2001), Tabesh et al (2002), Hayuti (2002)].

A variety of propriety software systems have been adopted for these studies, including the US Environmental Protection Agency's public domain package, EPANET. The vast majority of these products are "demand-driven" in the sense that flows taken from the network system at internal nodes are pre-specified at data entry, along with boundary conditions (at sources etc) and time-based operational factors (pump switching, valve adjustments etc). The network analysis routine then solves for the pressure distribution across the nodes of the network and the flows in individual pipes. In the UK, it has become customary for these demands to be specified in terms of household (HH, domestic), various types of non-household (NHH) use, and leakage components. In most applications it is acceptable and appropriate that the "true" demands (HH and NHH), representing all intended water withdrawals are pre-specified for a given investigation. Leakage on the other hand is highly pressure-dependent and so cannot be known accurately in advance until the system pressures are synthesised by the network modelling procedure itself. In principle, this calls for iterative readjustment of leakage input data (in terms of its magnitude and time-profile at network nodes) until it demonstrates consistency with the nodal pressure regime output by the model. Such iterative treatment could arguably be appropriate also to at least a proportion of the HH or NHH demands where the capability to withdraw water at the requisite rate might be compromised at particular nodes at certain times if the pressures drop below critical thresholds. Networks forced to operate under "excessive" demands (which might include major burst or fire-fighting for example) or under restricted capacity (due to pipe section isolation for repair) might warrant this treatment. The present contribution is seen as a precursor to the further (practical) implementation of such pressure related performance investigations.

Returning to leakage, for systems where such losses are small (<10% total consumption, say), adopting a fixed (pre-determined) leakage for routine planning or operational applications can perhaps be justified. On the other hand, where leakage rates are higher and for investigative studies likely to result in significant change to the pressure regime across the network (pressure management studies, introducing pressure reduction etc) retention of earlier (pre-determined) leakage specifications will not be robust. With the emphasis in the UK industry now switching from model calibration, enabling appraisal of current asset performance, to operational use involving investigation of other-than-normal conditions, the automated re-adjustment of leakage offers both time and prediction accuracy benefits.

Some software modelling products are configured in such a way that fully pressure dependent withdrawal at nodes can be incorporated without difficulty, so removing this shortcoming in the treatment of leakage from the system. In what follows, an account is given of the implementation of (automated) dynamic leakage modelling in the EPANET-2 software [Rossman (2000)]. The underlying technical advancement is the introduction of the so-called EMITTER in EPANET-2, which represents a pressure dependent outflow from any node. This can be so configured, therefore, to represent nodal leakage, accumulated from the pressures to which the associated mains lengths and/or property connections are subjected. The extrapolation of nodal pressures (from the network modelling software) to conditions at individual property connections can be achieved using GIS/database information systems such as NETBASE [Crowder (1999)].

2 METHODOLOGY

The following step-by-step procedure for implementation of dynamic leakage draws from the existing practices for leakage evaluation and domestic consumption (PCC and profile) determination, and subsequent model calibration, at DMA level across the distribution systems in the UK presently.

1. Conduct demand allocation and leakage, PCC and household diurnal profile reconciliation as currently. Application to be at DMA level and to extend to meter-delineated sections of trunk main, possibly including "dummy" DMAs of the distribution system, as necessary.
2. Calibrate the network (DMAs and trunk mains) as normal.

3. From final calibrated model output at time of Minimum Night Flow (MNF), abstract operational pressures: (a) at each property/connection, for connection based leakage allocations; or (b) over lengths of main allocated to network "nodes", for mains length based leakage allocation. A combination of the two approaches should also be feasible where preferred.
4. Introduce EMITTER specifications, through their discharge coefficients, $C_{node\text{-}I}$ at model network nodes, node-I, so as to represent the pressure dependency of leakage in the network model.
5. For an EMITTER, leakage can be expressed as

$$\text{Leakage}_{\text{node-I}} = C_{\text{node-I}} \times (\text{Pressure}_{\text{node-I}})^{N1} \tag{1}$$

where N1 is the chosen pressure exponent for the network, $C_{node\text{-}I}$ is the nodal leakage discharge coefficient.

The value of the exponent will be decided on the basis of the integrity of the system, in respect of the perceived balance between "burst" and "background" leakage losses. Where leakage is dominated by bursts (of fixed "orifice" opening) N1∼0.5, whilst according to the FAVAD (fixed and variable area discharge) approach [Lambert (1997)] for varying sized openings, typified by opening pipe joints creating numerous small "background" losses N1∼1.5–2.5. A typical figure for UK networks is often stated as 1.15 and so a round figure of 1.0 may well be a reasonable approximation, which has the added benefit that the leakage then varies *linearly* with pressure. The only restriction in EPANET modelling is that the leakage functionality (N1) has to be applied across the entire network modelled.
6. Taking the case of leakage allocation according to number of connections, it is convenient to apply the leakage function above to the case of each connection (ci) to node-I. The discharge coefficient can be expressed in terms of a "unit leakage", C_u, for the default pipe condition (to be determined) and Leakage Weighting Factor (LWF). C_u represents the rate of leakage per connection under 1.0 metres of pressure for pipe-work of good order (LWF = 1.0) and $LWF_{node\text{-}I}$ represents the condition of the pipes which connect node-I to the properties assigned to it. It is anticipated that a table of LWF for pipes of different material and age would be produced and that the default would be LWF = 1.0. It now follows that,

$$\begin{aligned}\text{Leakage}_{\text{node-I}} &= (C_{\text{node-I}}) \times (\text{Pressure}_{\text{node-I}})^{N1} \\ &= C_u \times (\text{LWF}_{\text{node-I}}) \times \Sigma\{(\text{Pressure}_{ci})^{N1}\}_{\text{all connections}}\end{aligned} \tag{2}$$

and the only unknown term is C_u.

If it is intended to apply the pressure dependency only at the nodal level, and taking there to be N_{ci} connections to node-I, the nodal leakage becomes

$$\text{Leakage}_{\text{node-I}} = C_u \times (\text{LWF}_{\text{node-I}}) \times N_{ci} \times (\text{Pressure}_{\text{node-I}})^{N1} \tag{3}$$

7. The sum of all these nodal leakage contributions across the whole DMA, or meter delineated trunk mains, must now be set to match the Unaccounted For Water (UFW) following a water balance calculation at the time of MNF.

This is achieved by applying suitable night usage to the various metered demand categories (consistent with their adopted diurnal profiles). For unmetered (domestic) consumption, this requires imposition of the chosen level for Legitimate Overnight Unmetered Consumption (LOUC) per property (a typical figure for UK households has often been taken as 1.7 litres/connection/hour) or the MNF consumption arising from the adopted domestic consumption profile if this has been determined.

$$UFW = [\sum \{Leakage_{node-I}\}_{all\ nodes}]_{MNF} = [\sum \{C_{node-I} \times (Pressure_{node-I})^{N1}\}_{all\ nodes}]_{MNF}$$

$$= (C_u) \times [\sum\{(LWF_{node-I}) \times \sum\{(Pressure_{ci})^{N1}\}_{all\ connections}\}_{all\ nodes}]_{MNF} \quad (4)$$

The value of C_u, as the only unknown in equation [4], can now be determined.

This gives a measure of the average leakage rate per connection in the network under a 1.0 metre pressure.

Simplifications are possible, for example, if applying the analysis at nodal level only (as is necessary if GIS-based modelling systems providing linkage between nodes and individual properties is not available), and if no differential (LWF) factors are defined, equation [4] simplifies, such that only the nodal pressures are necessary to compute C_u,

$$UFW \sim (C_u) \times [\sum\{N_{ci} \times (Pressure_{node-I})^{N1}\}_{all\ nodes}]_{MNF} \quad (5)$$

With C_u determined, it now becomes possible (using equation [2] at MNF) to establish the differing values of Emitter coefficient (C_{node-I}) to be inserted at each of the nodes in the EPANET-2 model.

Note that this Emitter specification replaces the leakage demand category and its predetermined profiles, which must be removed from the EPANET-2 input data files.

8. Re-run the EPANET-2 model with the EMITTER based leakage specification so as to produce output of the total nodal supplies, from which the actual hour-by-hour pressure dependent leakage can be abstracted (by deducting all the imposed customer demands).
9. From this revised leakage profile, re-work the DMA level flow balance to revise the appropriate PCC and household (HH) diurnal profile.
10. Introduce the updated PCC/profile into the EPANET model and check for flow and pressure calibration, making calibration adjustments as necessary. It is possible that steps (7–9) may need to be repeated if pressures have changed significantly. This is likely to be restricted, however, to situations where leakage losses are a large proportion of system throughput.
11. Once calibration is achieved with the enhanced model, it can now be applied to miscellaneous scenarios, whence leakage will self re-adjust automatically to the pressure regime arising, without any intervention. The quality-enhanced leakage and PCC specifications can then be returned to other modelling platforms if desired.
12. User oriented routines can be developed such as those in *NETBASE-Modelling* (Crowder 1999) to deliver client requested reports and thematic outputs and existing advanced reporting of the network's hydraulic performance (ie Average Zone Pressure and Hydraulic Characteristics) can be upgraded. More focused and rigorous evaluation of potential benefits from pressure management is a prime deliverable of this technical enhancement.

3 APPLICATION

The above methodology has been programmed in the Delphi language [Borland (1998)] to provide a utility which interacts seamlessly and automatically with EPANET-2 to transform a model input file constructed in the standard manner, with leakage pre-defined following a normal MNF analysis, into a model with leakage characterised through nodal emitters. The present version of the user interface is shown in figure 1.

The approach has been applied (with N1 = 1.0, see eqn. 1) by Mulreid (2002) to a total of 58 DMAs in the UK, comprising both single feed and multiple feed networks. It has been impemented here only at the nodal level (see step 7 above) so making no requirement other than availability of a valid (calibrated) network model defined by an EPANET input file.

Typical output is shown in figures 2 and 3, indicating respectively, the adjusted leakage profile and the re-evaluated domestic (household, HH) profile. The corresponding adjustment to the

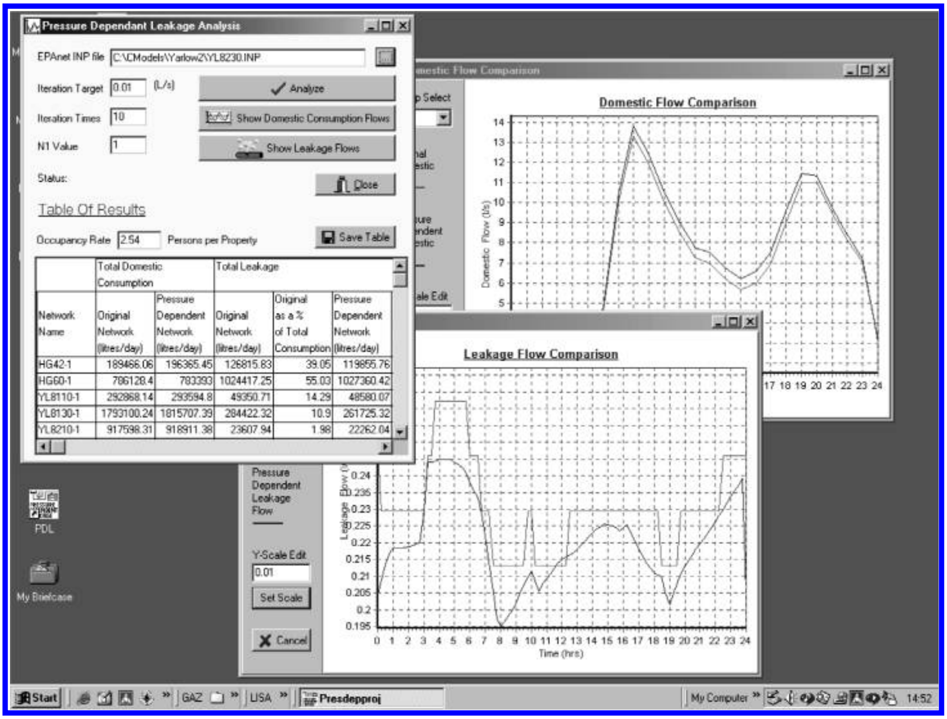

Figure 1. User interface for pressure-dependent leakage modelling showing graphical and tabular results output.

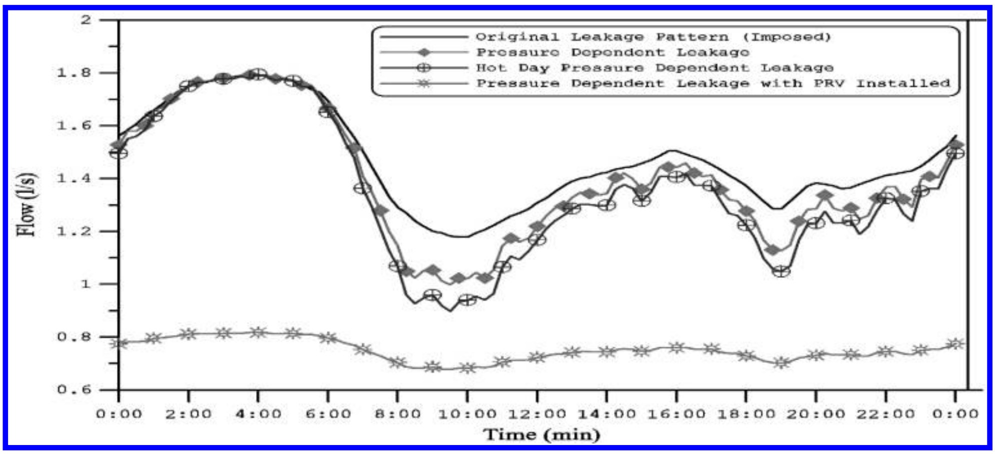

Figure 2. Diurnal leakage profiles for test network no. 1.

Per Capita Consumption figures are also evaluated and these are listed in the summary, Table A1, contained in the Appendix. Note that, in figure 2 the oscillations in the pressure dependent leakage plots reflects the variations in the observed pressure transducer records at the 15-minute sampling rate, whilst the original leakage profile was determined from hourly-averaging (during the MNF based leakage-evaluation/demand-reconciliation process), and hence is "smoothed" in appearance.

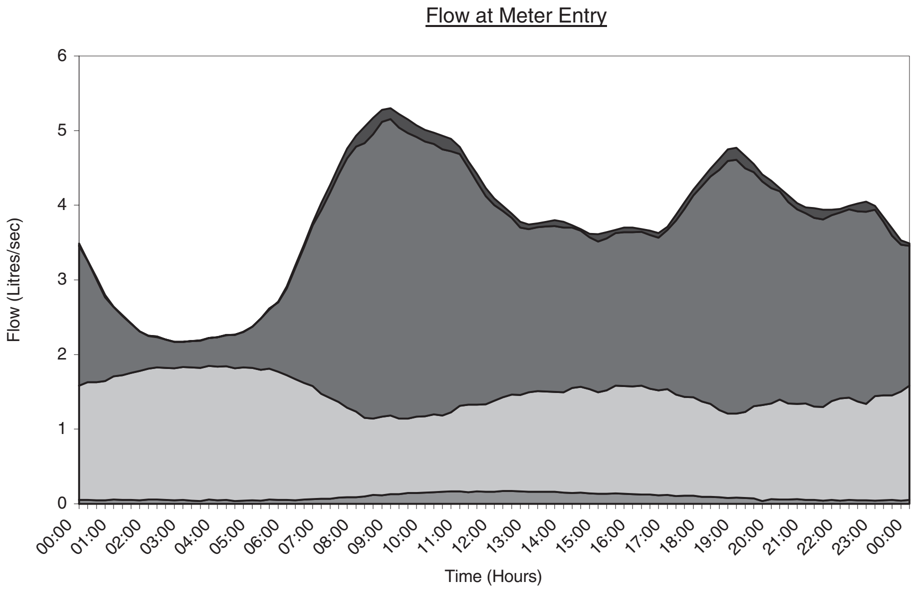

Figure 3. Diurnal profiles of the different components (HH and NHH) of consumption and leakage for DMA network no. 1.

For background information, the test network (N° 1) is a small DMA of 568 properties represented in a pipe system of 45 nodes and 47 pipes. Figure 2 also shows the reduced leakage arising from: (i) operation of the test network under a demand increased by 30% to loosely represent a hot-day scenario; and (ii) operation of the system when pressure managed, with a pressure reducing valve (PRV) creating a pressure drop (of 18 m at peak-hour) immediately downstream of the network inlet meter. Both these cases exhibit reduction in the leakage losses as a direct result of the reduced pressures over the network under these scenarios and these adjustments arise automatically as a result of the EMITTER representation of leakage in the EPANET-2 models. With conventional models several trial-and-error iterations of leakage (magnitude and profile) would be necessitated as a manual intervention and this would not necessarily establish the precise distribution of leakage across the network.

In figure 3, the dark shading shows the net change in HH consumption resulting from the introduction of pressure dependent leakage to the "demand-allocation" process. Table 1 summarises the overall result of introduction of pressure dependent leakage into the 58 test networks, leakage on average reduces by 1.4% and correspondingly PCC increases by 0.6%. Table 2 breaks this analysis into networks grouped in terms of the scale of DMA leakage present (ie $< 5\%$, 5–10%, etc) and from this it appears that there is no clear inter-dependency.

4 CONCLUSIONS

Benefits of introduction of dynamic (pressure dependent) leakage to network modelling as presented herein, to the water utility are:

(i) It enables operational and planning scenarios to be run with only the primary elements of demand (ie domestic and trade/industrial) pre-determined;
(ii) Leakage will self adjust automatically according to the designated flow scenario (ie hottest day, time horizon forecast etc), it being fully driven by the system pressures arising;
(iii) Leakage is distributed across the system according to number of connections (or length of mains) as well as the local pressures. The approach is fully compatible with "Background

Table 1. Outcome of introducing pressure-dependent leakage: all 58 DMAs.

	Total domestic consumption		Total leakage					Total metered consumption	Per capita consumption (PCC)		
Model number	Original network (litres/day)	Pressure dependent network (litres/day)	Original network (litres/day)	Original as a % of total consumption	Pressure dependent network (litres/day)	Pressure dependent as a % of total consumption	% Change	(litres/day)	Original network (litres/day)	Pressure dependent network (litres/day)	% Change
Model average	561717.75	565334.17	151984.02	16.54	148473.96	16.24	−1.42	110957.24	144.51	145.46	0.62

Table 2. Outcome of introducing pressure-dependent leakage: all 58 DMAs.

		Total domestic consumption		Total leakage					Total metered consumption	Per capita consumption (PCC)		
Average model leakage %	Number of DMA's	Original network (litres/day)	Pressure dependent network (litres/day)	Original network (litres/day)	Original as a % of total consumption	Pressure dependent network (litres/day)	Pressure dependent as a % of total consumption	% Change	(litres/day)	Original network (litres/day)	Pressure dependent network (litres/day)	% Change
<5	18	662552.03	662645.72	11707.01	1.80	11602.64	1.79	−0.95	99658.25	137.14	137.17	0.02
5 to 10	9	485589.91	487441.77	51456.10	7.45	49593.16	7.28	−1.70	117394.02	136.23	136.95	0.42
10 to 20	13	598412.20	601288.57	126759.57	14.60	123850.64	14.41	−1.44	158270.41	153.58	154.01	0.27
20 to 40	12	587605.84	597135.95	315815.18	31.55	306812.07	30.77	−2.06	85016.22	144.22	146.77	1.49
>40	6	352230.86	358854.05	452462.85	48.81	445930.83	48.17	−1.33	78959.19	161.81	163.80	1.71

and Burst Estimation (BABE) modelling", based on the "N1" leakage index (ie leakage = function [{pressure}^{N1}]);

(iv) Leakage levels are first set from field monitoring (Minimum Night Flow (MNF) analysis and Network (DMA) Model calibration) and can be updated as the status of the infrastructure changes, via successive MNF assessments;

(v) The procedure leads to the highest achievable accuracy in Total Daily Leakage assessment and highest possible precision in the determination of T-Factor (for peak-hour to daily-leakage computation) and Average Zone Pressure (AZP) determination;

(vi) Post-processing of Demand Data (domestic PCC and diurnal profiles) via the re-worked MNF and 24-hour analyses, using pressure dependency of leakage, will significantly improve the quality of Domestic Consumption statistics (PCC and profile) over their initial estimates based on surrogates for AZP:

(vii) Over the range of 58 DMAs studied herein, the adoption of pressure dependent leakage as part of the normal "MNF" analysis results in a 1.4% reduction in total daily leakage figures and a corresponding 0.6% increase in domestic consumption (PCC) estimates: and

(viii) Pressure Management, involving PRV installation and possible re-zoning, will be simulated in full accord with the hydraulic situation (without need for leakage data re-evaluation) and so produce highly reliable estimates of water savings, etc.

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REFERENCES

Borland International. 1998. "Delphi-4 for Windows 95 and Windows NT" Borland International Inc., P.O.Box 660001, Scotts Valley, CA 95067-0001, U.S.A.

Burrows R., Crowder G.S. and Zhang J. 1999. "Application of network modelling for operational management of water distribution systems". Proc CCWI'99 Int. Conference, "Water Industry Systems: Modelling and Optimisation Applications" eds Savic D. and Walters G. (Research Studies Press Ltd), Vol. 1, 229–242.

Burrows, R., Crowder, G.S. and Zhang, J. 2000. "Utilisation of network modelling in the operational management of water distribution systems", Urban Water, Vol. 2, 83–96.

Crowder G.S. and Burrows R. 1998. "Developing a holistic approach to water distribution management", Proc. IQPC Water Industry Conference on "Modelling the Distribution System", IQPC London (http://www.iqpc.co.uk), July, V 1–8.

Crowder & Co. Ltd. 1999. "NETBASE for Windows: Hydraulic Model Management", User Guide', 17–21 Price Street, Birkenhead, L41 6JN, UK, (e-mail crowders@compuserve.com).

Hayuti H.M. 2002. "Performance assessment of water distribution systems", MSc(Eng) Dissertation, Department of Civil Engineering, University of Liverpool, September, 115p.

Lambert A.O. 1997. "Pressure management/leakage relationships: theory, concepts and practical applications". IQPC Seminar, London, April.

Mulreid G. 2002. "The incorporation of pressure dependency in water supply leakage modelling", Mater of Research (M.Res) Dissertation, Department of Civil Engineering, University of Liverpool, September, 140p.

Rossman L.A. 2000. "EPANET-2 Users Manual", NRMRL, Office of Research and Development, U.S. EPA, Cincinnati, OH 45268. (http://www.cee.odu.edu/cee/model/epanet.html).

Tabesh M., Tanyimboh T.T. and Burrows R. 2002. "Head driven simulation of water supply networks", Int Journal of Engineering (Iran),Vol. 15, No. 1 Feb, 11–22 (ISSN 1025–2495).

Tanyimboh, T.T., Tabesh, M. and Burrows, R. An appraisal of the source head methods for calculating the reliability of water distribution networks, ASCE J. Wat. Res. Plan and Man., Vol. 127, No. 4, July, 2001, 206–213.

APPENDIX

Table A1. Summary data for all 58 DMA networks.

	Total domestic consumption		Total Leakage					Total metered consumption	Per capita consumption (PCC)		
Model number	Original network (litres/day)	Pressure dependent network (litres/day)	Original network (litres/day)	Original as a % of total consumption	Pressure dependent network (litres/day)	Pres Dep as a % of total consumption	% Change	(litres/day)	Original network (litres/day)	Pressure dependent network (litres/day)	% Change
1	189466.06	196365.45	126815.83	39.05	119855.8	36.9	−5.49	8494.54	131.33	136.11	3.64
2	786128.4	783393	1024417.3	55.03	1027360	55.19	0.29	51091.1	139.16	138.68	−0.35
3	292868.14	293594.8	49350.71	14.29	48580.07	14.06	−1.56	3244.6	159.26	159.65	0.25
4	1793100.2	1815707.39	284422.32	10.9	261725.3	10.03	−7.98	532014.6	170.52	172.67	1.26
5	917598.31	918911.38	23607.94	1.98	22262.04	1.87	−5.7	251823.01	168.58	168.82	0.14
6	490578.26	490397.81	19928.6	3.87	20114.12	3.9	0.93	4943.82	118.64	118.59	−0.04
7	1073629.2	1073647.09	6136.13	0.53	6122.79	0.53	−0.22	81012.87	128.24	128.25	0
8	464071.82	463719.3	4722.71	0.96	5076.88	1.04	7.5	20934.38	181.8	181.66	−0.08
9	633556.4	651313.47	85495.41	8.54	67694.61	6.76	−20.82	282071.66	177.66	182.64	2.8
10	239962.75	253199.18	553692.67	61.6	540524.1	60.13	−2.38	105210.95	173.66	183.24	5.52
11	393295.5	394005.86	72849.35	15.03	72139.02	14.88	−0.98	18499.86	137.64	137.88	0.18
12	253791.54	280490.41	264682.22	43.31	238407.7	39.01	−9.93	92705.94	152.55	168.59	10.52
13	145261.62	144402.97	10885.97	5.57	11744.63	6	7.89	39439.35	137.81	136.99	−0.59
14	36214.56	34085.26	29278.08	41.9	30762.02	44.02	5.07	4382.78	237.63	223.66	−5.88
15	528732.58	558960.29	349461.29	36.89	325116.3	34.32	−6.97	69040.12	237.63	251.21	5.72
16	799614.18	802331.34	287552.59	24.13	284854.4	23.9	−0.94	104513.72	132	132.44	0.34
17	757947.96	769452.3	365772.42	29.58	354334.8	28.66	−3.13	112665.48	146.64	148.86	1.52
18	278992.48	279409.96	171471.38	34.05	171032.4	33.96	−0.26	53147.3	143.96	144.17	0.15
19	1153600.1	1152796.39	104521.18	7.6	105344.7	7.66	0.79	117886.37	123.35	123.26	−0.07
20	424971.29	426818.41	95934.53	17.14	94075.99	16.81	−1.94	38678.97	143.99	144.61	0.43
21	481041	486074.49	284053.25	32.8	279119.9	32.23	−1.74	100952.96	152.36	153.96	1.05
22	206978.36	211452.65	54565.16	8.12	49900.59	7.42	−8.55	410553.34	142.71	145.8	2.16
23	538938.58	545560.66	408436.7	41.56	401888.4	40.89	−1.6	35395.25	139.78	141.49	1.23
24	1129995.5	1172666.17	901809.07	39.94	859226.7	38.06	−4.72	225962.63	139.59	144.86	3.78
25	603709.31	633289.21	270972.97	29.53	241269.6	26.29	−10.96	43009.74	131.83	138.28	4.9
26	1016774.8	1021436.23	318532.03	16.54	313798.2	16.3	−1.49	590387.17	190.53	191.4	0.46
27	440844.77	421947.46	169298.53	24.88	188295	27.68	11.22	70205.68	123.62	118.32	−4.29
28	1164685.6	1167365.09	235498.18	14.17	232802.4	14	−1.14	262284.91	234.67	235.21	0.23

(*Continued*)

Table A1. (*continued*).

Model number	Total domestic consumption: Original network (litres/day)	Total domestic consumption: Pressure dependent network (litres/day)	Total leakage: Original network (litres/day)	Total leakage: Original as a % of total consumption	Total leakage: Pressure dependent network (litres/day)	Total leakage: Pres Dep as a % of total consumption	Total leakage: % Change	Total metered consumption (litres/day)	Per capita consumption (PCC): Pressure original network (litres/day)	Per capita consumption (PCC): Dependent network (litres/day)	Per capita consumption (PCC): % Change
29	313894.37	316474.59	252711.94	34.36	250318.2	34.04	−0.95	168778.99	118.94	119.92	0.82
30	1053662.5	1056184.95	272105.86	16.66	269548.1	16.51	−0.94	307085.01	139.72	140.05	0.24
31	283063.97	283636.1	94977.79	25.09	94383.78	24.93	−0.63	523.87	124.1	124.35	0.2
32	822217.25	818510.29	89845.2	9.3	93569.82	9.69	4.15	53769.85	122.57	122.02	−0.45
33	281390.47	281375.76	24636.1	7.89	24649.12	7.89	0.05	6211.28	137.96	137.95	−0.01
34	400090.03	398920.31	37275.55	7.03	38520.72	7.26	3.34	93015.91	122.49	122.13	−0.29
35	508472.5	509488.77	116138.88	17.8	115123	17.64	−0.87	28028.54	141.77	142.06	0.2
36	1243967.9	1245024.04	514885.14	28.26	513938	28.21	−0.18	62899.56	148.63	148.76	0.08
37	474449.98	475542.95	37791.36	7.12	36682.58	6.91	−2.93	18287.2	117.55	117.82	0.23
38	252765.04	252661.11	18088.97	5.91	18231.67	5.95	0.79	35311.19	144.01	143.95	−0.04
39	110513.7	110604.18	19302.82	13.99	19195.65	13.92	−0.56	8123.12	178.32	178.46	0.08
40	113443.16	113600.35	19518.62	13.14	19307.71	13	−1.08	15571.53	76.87	76.98	0.14
41	21714.62	21718.43	352.51	0.36	336.74	0.35	−4.48	75165.86	131.52	131.55	0.02
42	2180258.1	2180800.31	13561.34	0.46	12965.91	0.44	−4.39	780009.48	207.74	207.79	0.02
43	258349.34	256395.76	434270.16	49.48	436642.3	49.75	0.55	184969.12	128.1	127.13	−0.76
44	79475.83	79328.89	18667.38	17.53	18878.39	17.72	1.13	8370.79	147.59	147.32	−0.18
45	168665.21	168913.72	38040.75	9.28	37629.34	9.18	−1.08	203319.9	111.98	112.14	0.15
46	564228.81	564807.81	31326.8	3.9	30697.69	3.83	−2.01	206954.57	185.89	186.08	0.1
47	365821.24	365903.9	3325.54	0.78	3228.91	0.76	−2.91	57919.91	117.47	117.5	0.02
48	659430.25	659702.75	107512.92	13.29	107255.2	13.26	−0.24	41906.38	163.69	163.76	0.04
49	490578.26	490387.98	19940.76	3.87	20136.16	3.91	0.98	4943.82	118.64	118.59	−0.04
50	154741.32	154895.73	3120.77	1.73	2937.77	1.63	−5.86	22610.66	106.88	106.99	0.1
51	490578.26	491239.42	19928.6	3.87	19261.41	3.74	−3.35	4943.82	118.64	118.8	0.13
52	541194.59	541386.65	2981.23	0.53	2787.15	0.5	−6.51	18414.25	131.04	131.09	0.04
53	701494.92	699938.45	10520.13	1.42	12083.73	1.63	14.86	30071.41	119.2	118.93	−0.22
54	490578.26	491797.06	19928.6	3.87	18701.24	3.63	−6.16	4943.82	118.64	118.93	0.25
55	541194.59	541427.13	2981.23	0.53	2746.11	0.49	−7.89	18414.25	131.04	131.09	0.04
56	701494.92	700124.26	10520.13	1.42	11896.92	1.6	13.09	30071.41	119.2	118.96	−0.2
57	1073629.2	1073874.48	6136.13	0.53	5889.25	0.51	−4.05	81012.87	128.24	128.27	0.02
58	1922.02	1921.84	515.41	0.38	518.16	0.38	0.53	133318.22	126.12	126.1	−0.01
Model average	561717.75	565334.17	151984.02	16.54	148473.96	16.24	−1.42	110957.24	144.51	145.46	0.62

Estimation of distributed pressure-dependent leakage and consumer demand in water supply networks

L.S. Araujo
Universidade Federal do Ceará, Fortaleza, Brasil

S.T. Coelho
Laboratório Nacional de Engenharia Civil, Lisboa, Portugal

H.M. Ramos
Instituto Superior Técnico, Lisboa, Portugal

ABSTRACT: The research described in this paper explores the suitability of a method for estimating the leakage-associated parameters and reaching a more plausible demand description than that afforded by traditional *static* estimates, when modelling the hydraulic behaviour of a water distribution system. Leakage is represented as a uniformly distributed, pressure-dependent demand and is deducted from the metered total system demand. The method relies on a genetic algorithm as the optimisation technique and uses the Epanet 2.0 programmable toolkit in order to create a general-purpose network analysis program.

1 INTRODUCTION

For most real-life simulation models of water distribution networks, the modelling effort starts from a physical network description, a flow measurement recorded at the network's inlet(s), and an estimation of the spatial distribution of this lumped demand across the network. At best, the location of the demand nodes is known, but how the total inflow into the network is allocated among them can only be estimated – even when there is domestic metering, the data is seldom directly usable for this purpose.

A documented technique for a more consistent description of demands is to assign part of the total metered inflow to leakage, and account for this as a pressure-dependent demand. In water network analysis, leakage is often modelled based on the equation of flow through an orifice:

$$q = k(p_i - p_o)^{\beta} \tag{1}$$

where q is flow, k a coefficient that depends on the shape and size of the orifice, and p_i and p_o the values of pressure upstream and downstream of the orifice. The value of p_o is usually assumed to be zero. The exponent β equals 0,5.

Several variations and improvements on the above equation have been proposed. Jowitt & Xu (1990) and Vairavamoorthy & Lumbers (1998) have used the following expression, obtained from field data:

$$q_{ij} = c_i L_{ij}\left(\frac{hi - hj}{2}\right)^{1,18} \tag{2}$$

where q_{ij} is flow on the pipe from node i to node j; c_i is the leakage coefficient per unit length (for the service pressure, and as a function of the age and condition of the pipes and the soil characteristics), L_{ij} is the pipe length, and h_i, h_j the pressure at nodes i and j respectively. Jowitt & Xu (1990) estimate the value of c_i from night flows, while Vairavamoorthy & Lumbers (1998) assess it based on the local average service pressure. The expression uses an exponent of 1.18, which is rather higher than the expectable 0.5 from Eq. 1, but is nevertheless reflected in the experimental results it draws from. It is speculated that the effect may be caused by changes in the shape and size of the cracks and orifices due to the internal/external pressure differences.

Tucciarelli et al. (1999) propose the following variation:

$$Q_i = (H_i - z_i)^{\alpha} \sum_{j=1}^{M_i} \frac{\pi}{2} D_{ij} \theta_{ij} L_{ij} \tag{3}$$

where Q_i is flow, H_i is total hydraulic charge at node i, z_i is the elevation at the node, α the loss coefficient, M_i the number of pipes connected to node i, D_{ij} the diameter of the pipe from i to j and θ_{ij} the leakage surface per unit surface area of the pipe from i to j.

Martinez et al. (1999) use Eq. 2 in order to express the share of leakage corresponding to the pipes actually considered in the model and equation Eq. 4 below for those pipes that have been suppressed in the model and whose leakage are concentrated on the nodes:

$$q_{2,i} = c_2^s \left[\sum_{t=1}^{T} Q_{c,i} \right] p_i^{\beta} \tag{4}$$

where c_2^s has the same meaning as c_i, for a particular network sector s, T is the total number of simulated time steps, $Q_{c,i}$ is the total demand at node i, p_i is pressure and β is 1.18 as in Eq. 2.

Alonso et al. (2000) use the following expression:

$$q_j = K_j p_j^{1,18} \tag{5}$$

where q_j is leakage flow at node j, p_j the service pressure at node j and K_j is a fixed leakage coefficient for the node, estimated as a function of pipe and soil characteristics.

For hydraulic analysis, the model described in this paper relies on the Epanet 2.0 water distribution simulator programmable toolkit (Rossman, 2000). The library includes a nodal emitter function that implements an expression of the same type as equation (5), previously used by Araujo et al. (2002; 2002a), where K_j must be specified and the exponent may be set by the user –1.18 is the value adopted here.

2 PROPOSED METHOD

The method proposed here starts from a bulk evaluation of leakage volumes from night-flow analysis. The network is then simulated for the minimum-flow situation and the leakage volume thus estimated is assigned to a uniformly distributed pressure-dependent demand, which is distributed along all pipes of the network and duly concentrated on their start and end nodes for modelling purposes. The remainder of the total metered flow is concentrated on the consumer demand nodes according to the initial allocation.

A genetic algorithm[1] (GA) is used for the iterative process in order to stabilise a pressure-demand solution (a given hydraulic solution will generate a particular pressure-dependent demand,

[1]GA are non-deterministic meta-heuristic methods used in search, optimization and machine learning. They manipulate a space of possible solutions using mechanisms that mimic the Darwinian natural selection laws. They are efficient and robust in irregular, multidimensional and/or complex search spaces, and have been successfully deployed in several domains of water supply network optimisation.

which in turn forces the pressure distribution to change, and so on). After the pipe leakage coefficient has been estimated in this manner, it is considered as fixed for the ensuing analysis.

The network is subsequently simulated for an extended period, specified from the same flow records. The network is loaded with two types of demands: a leakage-related demand in all nodes, which is pressure-dependent, and the initial consumer demand. For each successive time step, the demand pattern used for the consumer demand nodes is recalculated in order to adjust total system demand to the metered inflow. Again, a genetic algorithm is deployed for this iterative process.

The model is implemented in Visual Basic, making use of Epanet 2.0 for hydraulic simulation, as previously mentioned.

The basic assumptions for this procedure are as follows:

- the only flow data available are continuous records of the inflow to the modelled network sector or district metering area (DMA);
- nodal demand is unknown and its spatial allocation is carried out based on qualitative estimation, such as household counts or ground occupation – it is expressed in terms of a fraction of the total inflow;
- leakage is an exclusive function of pressure;
- the leakage coefficient c is the same for all the pipes in the DMA, and is independent from the pipe cross-section; and
- at the minimum night flow, total flow can be divided into an estimated percentage which is due to leakage only, and the remainder is effective demand.

The method then consists of the following steps:

1. From the inflow record to be replicated, find the time t_{min} at which the minimum flow takes place (usually a minimum night flow).
2. At that time, $Q_{T,tmin}$ is the minimum total flow and leakage flow $Q_{F,tmin}$ can be estimated at a given percentage F of that value, say 80%:

$$Q_{F,tmin} = \%F * Q_{T,tmin} = 0.80 * Q_{T,tmin} \tag{6}$$

3. The program, through the application of the GA, finds the value of c for which the following relationship is true:

$$Q_{F,t\min} = \sum_{i=1}^{N} q_{f,i} = \sum_{i=1}^{N} (P_i^{1,18} * c * \sum_{j=1}^{M} 0,5 * L_{ji}) = 0.80 * Q_{T,t\min} \tag{7}$$

   where $q_{f,i}$ is the leakage flow at node i for t_{min}, N the total number of nodes in the network, M the set of links connected to node i, L_{ji} the length of the link connecting node j to node i and P_i is pressure head at node i, for $t = t_{min}$. The c coefficient, thus estimated, remains a fixed quantity for the remainder of the simulation, as it is a feature of the pipes of the network.
4. Once c is known, there is now a given amount of leakage which is dependent on the available pressure at each time t, and which equally distributed along all the pipes of the network, and split between their start and end nodes. Nodal demand must also be reallocated, in order to make up for such differences and add up to the boundary condition defined by the network inflow measurement.
5. The following relationships must be verified, where $Q_{T,t}$ is the total network inflow at time t, $Q_{Te,t}$ is total effective network demand at time t, q_{bi} is the nominal or *base* demand (without leakage) at node i, $f_{c,t}$ is the demand factor for time t (i.e., the value of the dimensionless demand pattern at time t, which replicates the shape of the inflow record), P_i is pressure head at node i, for $t = t_{min}$, and $K_{f,i}$ is the equivalent emitter coefficient for node i:

$$Q_{T,t} = Q_{Te,t} + Q_{F,t} \tag{8}$$

$$K_{f,i} = c * \sum_{J=1}^{M} 0,5 * L_{ji} \tag{9}$$

$$Q_{T,t} = \sum_{i=1}^{N} \left(q_{bi,t} * f_{c,t} \right) + \sum_{i=1}^{N} \left(K_{f,t} * P_{i,t}^{1,18} \right) \tag{10}$$

6. A new, different demand pattern is determined at each node in order for Eqs 8–10 to be satisfied – i.e., a new value for $f_{c,t}$ at each node and at each time t must be found.

In the formulation of the GA for steps 3 and 5–6, the following fitness functions were derived:

- First stage, c optimisation:

$$Min \quad f(\Delta Q_F)_{t\,\min} = (\%F * Q_{T,t\,\min} - Q_{F,t_{\min},Mod}) / \%F * Q_{T,t\,\min} \tag{11}$$

- Second stage, demand pattern recalculation ($f_{c,t}$):

$$Min \quad f(\Delta Q_{T,t})\Big|_{t=1}^{H \neq t_{\min}} = (Q_{T,t} - Q_{T,t,Mod})^2 / Q_{T,t}\Big|_{t=1}^{H \neq t_{\min}} \tag{12}$$

where the index $_{Mod}$ denotes the value calculated by the model.

The application described in this paper makes use of a conventional generational GA (Fig. 1) where the entire population is replaced by new elements generated through the selection process

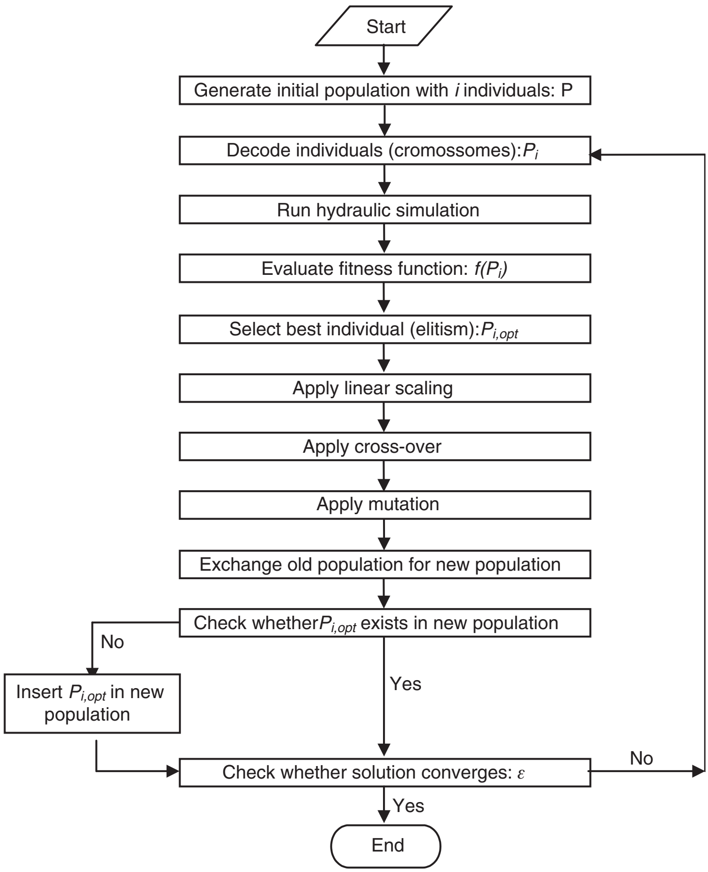

Figure 1. Genetic algorithm application flowchart.

and the application of the mutation and crossover operators. There is no coexistence of parents and children (each generation completely replaces the previous one). This allows for the loss of good individuals, but is compensated by an elitism process that preserves the best individual by passing on a copy to the following generation. Linear scaling is employed in order to ensure adequate competition and avoid premature convergence during the GA's execution.

3 MODEL VALIDATION

3.1 *Case study: simple network*

A simple artificial network was used in order to test and validate the model. As previously seen, the process consists essentially of two stages: calculation of a value for c, and redistribution of demands through the evaluation of new demand patterns (made up of demand factors $f_{c,t}$) at all nodes, so that an equilibrium is reached between nodal pressures, pressure-dependent leakage and nodal demands, in order to comply with the total inflow, which is a boundary condition.

In order to set up the test network, an arbitrary fixed c value is allocated to all its pipes: $c = 5{*}10^{-5}$. The emitter coefficient K_f is established for all the nodes as follows:

$$K_f = c * \sum_{j=1}^{M_i} 0{,}5 * L_{ji} \tag{13}$$

An arbitrary base demand q_{bi} is allocated to each node i, as well as demand factors $f_{c,t}$ for a given extended period, thus defining a demand pattern at each node.

The network is simulated with both nodal demands and the emitters, and the total inflow is registered for the entire extended period simulation. For the time step when the minimum total inflow takes place, total leakage is added up and the percentage it represents of total inflow is calculated. The procedure can then be applied to the test network, using only the inflow record, and the percentage of leakage corresponding to the minimum flow that takes place during the entire period. The inflow record is divided by its mean and used as the initial overall demand pattern, and arbitrary nodal base demands are allocated.

Illustrating with a static simulation, Fig. 2 shows the test network with only nodal demands and no leakage, considering $P_{1,2,3\text{ e}4} = 60\,\text{m}$ and $f_{c,t} = 1.0$. It is known that only nodes 2 and 3 have consumer demand. Leakage is calculated for $c = 5{*}10^{-5}$, thus:

$k_1 = 5 \bullet 10^{-5} \bullet 0.5(1000 + 1000) \Rightarrow k_1 = 0.05$ which yields $q_{f1} = 0.05 \bullet 60^{1,18} \Rightarrow q_{f1} = 6.27$
$k_2 = 5 \bullet 10^{-5} \bullet 0.5(1000 + 1000) \Rightarrow k_2 = 0.05$ which yields $q_{f2} = 0.05 \bullet 60^{1,18} \Rightarrow q_{f2} = 6.27$
$k_3 = 5 \bullet 10^{-5} \bullet 0.5(1000 + 1000) \Rightarrow k_3 = 0.05$ which yields $q_{f3} = 0.05 \bullet 60^{1,18} \Rightarrow q_{f3} = 6.27$
$k_4 = 5 \bullet 10^{-5} \bullet 0.5(1000 + 1000) \Rightarrow k_4 = 0.05$ which yields $q_{f4} = 0.05 \bullet 60^{1,18} \Rightarrow q_{f4} = 6.27$

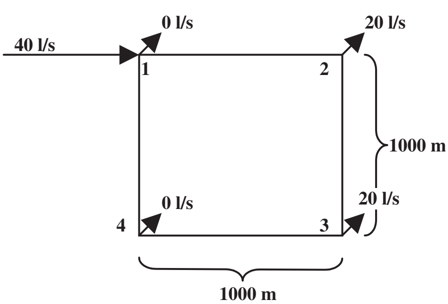

Figure 2. Test network, with nodal demand and no leakage.

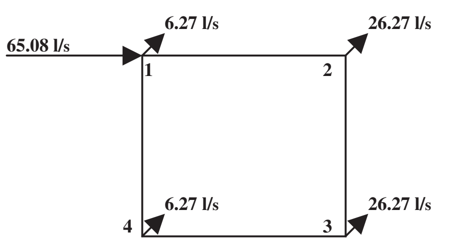

Figure 3. Test network with leakage and total flows.

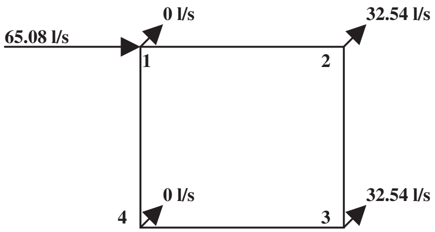

Figure 4. Only the inflow is known: arbitrary base demands are allocated to nodes 2 and 3.

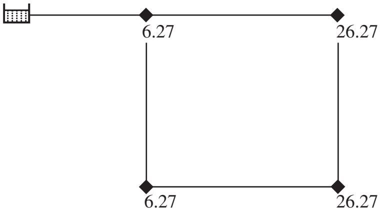

Figure 5. Final demand and leakage distribution.

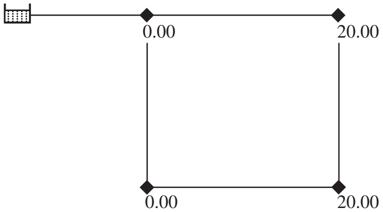

Figure 6. Nodal demand only.

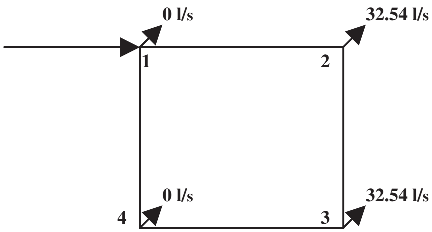

Figure 7. Extended period simulation base network.

The leakage values are added up to the nodal demands (Fig. 3) and the leakage percentage in the total inflow, calculated with the above values, yields 38.53%.

In order to apply the procedure, the only known quantity now would be the total inflow, 65.08 l/s, which would now be arbitrarily allocated to the nodes that are known to have consumer demand, as in Fig. 4. It is also "estimated" that leakage is 38.53% of the total.

Applied to this hypothesis, the model $c = 0.0000499910$ and $f_c = 0.615$ for both nodes. Fig. 5 shows the final demand + leakage distribution, which coincides with Fig. 3, as expected. Nodal demands, without leakage (Fig. 6), equal those in Fig. 2.

For a 12-hour extended period simulation test, a dimensionless demand pattern is assumed (which would be obtained from the inflow measurement, divided by its own mean), with the static scenario above coinciding with the minimum flow situation, which takes place at $t = 3.00$ h ($t_{min} = 3$). The base demand allocated to demand nodes 2 and 3, q_{b2} and q_{b3}, is shown on Fig. 7. Table 1 shows "recorded" inflow and the resulting demand factors. Table 2 shows total outflow at each node (demand + leakage), calculated for an arbitrary $c = 0.00005$.

The model was run for the known leakage level of 38.53% at the minimum flow hour, yielding a new c value of 0.0000500037 and new unit demand factors (f_c) as shown on Table 2A. On Fig. 8, modelled leakage flows are shown against the previously determined values. Tables 3 and 3A show the values of nodal pressure, both previously determined and modelled. The results confirm the validity of the proposed method.

3.2 *Case study – literature example*

The network shown on Fig. 9A was used by Jowitt & Xu (1990) in order to illustrate a leakage estimation application, considering $c = 0.00001$, $\beta = 1{,}18$, with the demand factors shown on Fig. 9B, a total metered inflow as in Fig. 9C and leakage flows as shown on Fig. 9D. The same network has also illustrated applications by Reis et al. (1997) and Vairavamoorthy & Lumbers (1998).

The model was run for a leakage level of 32.44%, established on the basis of the minimum night flow value ($t_{min} = 4$ h) of $Q_T = 90$ l/s (Fig. 8C) and the leakage estimates from Fig. 9D, which yielded a value of 29,2 l/s. The results showed a recalculated c value of 0.00000951.

Table 1. "Recorded" inflow and demand factors, for a 12-hour extended period scenario.

Time (h)	1	2	**3**	4	5	6	7	8	9	10	11	12
$Q_{T,t}$ Metered	72.84	80.59	**65.08**	72.84	80.59	84.45	88.31	84.45	72.84	68.96	65.08	72.84
$f_{c,t}$ Calculated	1.119	1.238	**1.000**	1.119	1.238	1.298	1.357	1.298	1.119	1.060	1.000	1.119

Table 2. Total outflow at each node, calculated for an arbitrary $c = 0.00005$.

Node ID	Base	1	2	**3**	4	5	6	7	8	9	10	11	12
1	0.00	6.27	6.27	**6.27**	6.27	6.27	6.27	6.27	6.27	6.27	6.27	6.27	6.27
2	32.54	30.18	34.08	**26.27**	30.18	34.08	36.03	37.97	36.03	30.18	28.23	26.27	30.18
3	0.00	6.22	6.16	**6.27**	6.22	6.16	6.13	6.1	6.13	6.22	6.24	6.27	6.22
4	32.54	30.18	34.07	**26.27**	30.18	34.07	36.02	37.96	36.02	30.18	28.22	26.27	30.18
Inlet	0.00	−72.84	−80.59	**−65.08**	−72.84	−80.59	−84.45	−88.31	−84.45	−72.84	−68.96	−65.08	−72.84

Table 2A. Total consumption at each node, as modelled, for a leakage level of 38.53% and $t_{min} = 3$, with new unit demand factors (f_c).

Node ID	Base	1	2	**3**	4	5	6	7	8	9	10	11	12
1	0.00	6.27	6.27	**6.27**	6.27	6.27	6.27	6.27	6.27	6.27	6.27	6.27	6.27
2	32.54	30.17	34.01	**26.27**	30.16	34.01	35.98	37.99	36.04	30.16	28.24	26.30	30.14
3	0.00	6.22	6.16	**6.27**	6.22	6.16	6.13	6.10	6.13	6.22	6.24	6.27	6.22
4	32.54	30.16	34.00	**26.27**	30.16	34.00	35.97	37.98	36.03	30.16	28.24	26.30	30.14
Q_T modelled	0.00	−72.82	−80.45	**−65.08**	−72.81	−80.45	−84.36	−88.34	−84.48	−72.82	−68.99	−65.13	−72.77
New f_c		0.737	0.858	**0.615**	0.737	0.858	0.92	0.984	0.922	0.737	0.677	0.615	0.736

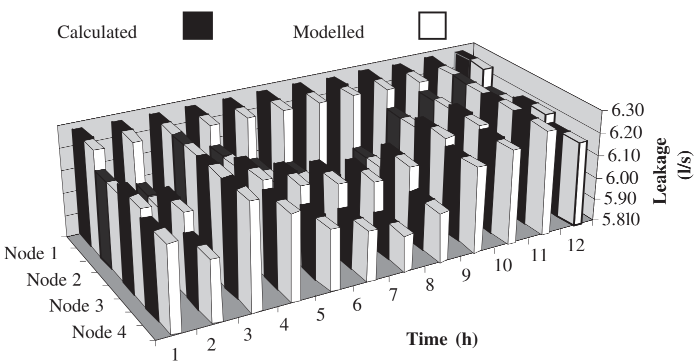

Figure 8. Modelled leakage flows are shown against the previously determined values.

The new unit demand factors (f_c) are shown on Fig. 10 together with the original ones. Fig. 11 plots the modelled leakage flows as compared to the original values. In both cases there are significant adjustments applied to the original values, in order to make them compatible with the pressure-dependent leakage flows estimated as mentioned above.

Table 3. Nodal pressures for $c = 0.00005$, previously determined.

Node ID	1	2	**3**	4	5	6	7	8	9	10	11	12
1	60.00	60.00	**60.00**	60.00	60.00	59.99	59.99	59.99	60.00	60.00	60.00	60.00
2	59.28	58.47	**60.00**	59.28	58.47	58.04	57.60	58.04	59.28	59.65	60.00	59.28
3	59.59	59.13	**60.00**	59.59	59.13	58.89	58.64	58.89	59.59	59.80	60.00	59.59
4	59.25	58.42	**60.00**	59.25	58.42	57.97	57.51	57.97	59.25	59.64	60.00	59.25

Table 3A. Nodal pressures for $c = 0.000050001$, modelled.

Node ID	1	2	**3**	4	5	6	7	8	9	10	11	12
1	60.00	60.00	**60.00**	60.00	60.00	60.00	59.99	60.00	60.00	60.00	60.00	60.00
2	59.28	58.49	**60.00**	59.28	58.49	58.06	57.59	58.04	59.28	59.65	60.00	59.29
3	59.59	59.14	**60.00**	59.59	59.14	58.90	58.64	58.89	59.59	59.80	60.00	59.59
4	59.25	58.44	**60.00**	59.26	58.44	57.99	57.51	57.97	59.25	59.63	60.00	59.26

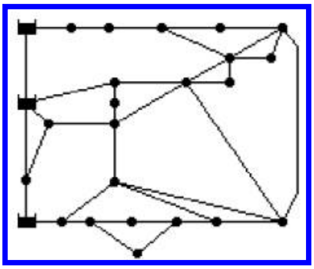

Figure 9A. Jowitt & Xu (1990)'s modelled network.

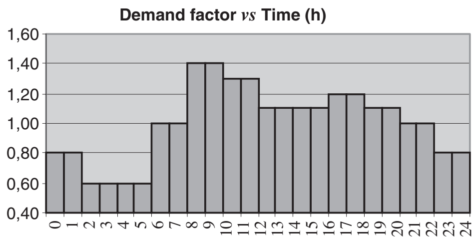

Figure 9B. Demand factors.

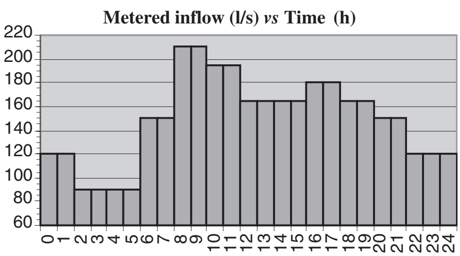

Figure 9C. Total metered inflow.

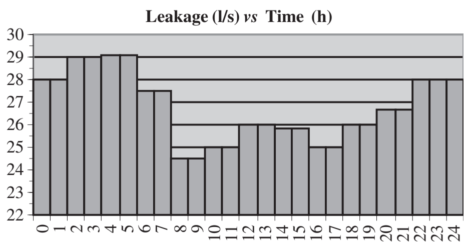

Figure 9D. Leakage flow, original estimate.

4 MODEL PERFORMANCE

In a P4 PC running at 1 GHz, with 512 Mb RAM, the GA deployed in the above simulations ran the entire 24-hour cycle within one hour, with an average 100 iterations for each time step. The GA operated typically on a population of 50 individuals, with a crossover probability of 0.75, a mutation probability of 0.01 and a linear scaling of 1.2.

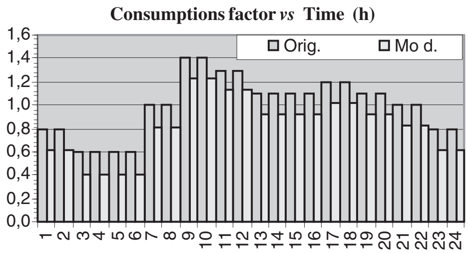

Figure 10. Original (Orig.) demand factors and those resulting from the model (Mod.).

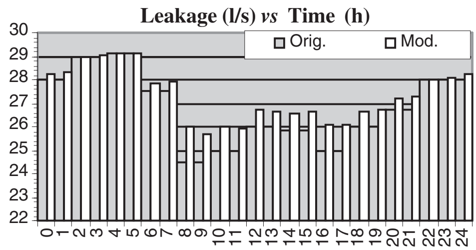

Figure 11. Original (Orig.) leakage flow values and those resulting from the model (Mod.).

5 CONCLUDING REMARKS

The method proposed allows for a more plausible description of demands (leakage and consumer) and pressures in network analysis, by comparison with classical demand allocation, and leads to improvements in model calibration, especially in systems with significant headloss.

Starting from a global leakage flow estimate taken from a night flow readout, leakage coefficients and pressure-dependent leakage flows are established for all the nodes of the network. Consumption at each node is re-balanced throughout the extended period so as to comply with the total inflow measurement boundary condition.

The method constitutes an improved basis for such applications as the optimisation of system pressures through valve or pump operation.

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REFERENCES

Alonso, J.M., Fernando, A., Guerrero, D., Hernández, V., Ruiz, P.A., Vidal, A.M., Martinez, F., Vercher, J. & Ulanicki, B., 2000. *Parallel Computing in Water Network Analysis and Leakage Minimization*. J. Wat. Res. Plan. Management, ASCE, July/August, pp. 251–260.

Araujo, L.S., Ramos, H.M. & Coelho, S.T., 2002. *Gestão Integrada de Sistemas de Distribuição de Água para um Melhor Controlo de Perdas*. 6° Congresso da Água, Porto, Portugal.

Araujo, L.S., Ramos, H.M. & Coelho, S.T., 2002a. *Optimização da Localização de Válvulas numa Rede de Distribuição para a Minimização de Fugas*, 10° ENASB/SILUBESA, 16–19/09/2002. Braga, Portugal.

Goldberg, D.E., 1989. Genetic Algorithms in Search, *Optimization and Machine Learning*, Adison-Wesley, USA.

Goldberg, D.E., 1994. *Genetic and Evolutionary Algorithms Come of Age*. Communications of the ACM, Vol. 37, n° 3, pp. 113–119.

Hernández, V., Martínez, F., Vidal, A.M., Alonso, J.M., Alvarruiz, F., Guerrero, D., Ruiz, P.A. & Vercher, J. 1999. *HIPERWATER: A High Performance Computing EPANET-Based Demonstrator for Water Network Simulation and Leakage Minimisation*. Water Industry Systems: Modelling and Opt. Applications, Vol. 1, Research Studies Press Ltd., pp. 141–153.

Jowitt, P.W. & Xu, C., 1990. *Optimal Valve Control in Water Distribution Networks*. Journal of Water Resources Planning and Management, ASCE, July/August, pp. 455–472.

Kalanithy, V. & Lumbers, J., 1998. *Leakage Reduction in Water Distribution Systems: Optimal Valve Control*. Journal of Hydraulic Engineering, ASCE, November. pp. 1146–1154.

Lambert, A., Mayers, S. & Trow, S., 1998. *Managing Water Leakage: economic and technical issues*. Financial Times Energy. http://www.ftenergy.com

Martínez, F., Conejos, P. & Vercher, J., 1999. *Developing an Integrated Model for Water Distribution Systems Considering both Distributed Leakage and Pressure-Dependent Demands*. Proc. 26th ASCE Wat. Res. Planning Man. Div. Conference. July. Tempe, Arizona

National Water Council Standing Technical Committee Report 26, 1980. *Leakage Control Policy and Practice*. Technical Working Group on Waste of Water. ISBN 0 904561 95 X.

Reis, F.R. & Chaudhry, F.H., 1999. *Hydraulic Characteristics of Pressure Reducing Valves for Maximum Reduction of Leakage in Water Supply Networks*. Water Industry Systems: Modelling and Optimization Applications, Vol. 1, Research Studies Press Ltd., pp. 259–267.

Reis, F.R., Porto, R.M. & Chaudhry, F.H., 1997. *Optimal Location of Control Valves in Pipe Networks by Genetic Algorithm*. J. Wat. Res. Plan. Management, Nov/Dec, pp. 317–326.

Rossman, L.A., 2002. *EPANET 2 Users Manual*. Water Supply and Water Resources Division, National Risk Management Research Laboratory, Cincinnati, OH 45268, September, 2000.

Tucciarelli, T., Criminisi, A. & Termini, D., 1999. *Leak Analysis Systems by Means of Optimal Valve Regulation*. Journal of Hydraulic Engineering, ASCE, March. pp. 277–285.

Vairavamoorthy, K. & Lumbers, J., 1998. *Leakage Reduction in Water Distribution Systems: Optimal Valve Control*. Journal of Hydraulic Engineering, ASCE, November, pp. 1146–1154.

Viana, G.V.R., 1998. *Meta-heuristicas e Programação Paralela em Otimização Combinatória*. Edições UFC, Fortaleza. 250 p.

Application of integrated GIS and hydraulic models for unaccounted for water studies in water distribution systems

M. Tabesh
Department of Civil Engineering, University of Tehran, Tehran, Iran

M.R. Delavar
Department of Surveying Engineering, University of Tehran, Tehran, Iran

ABSTRACT: One of the most important problems in water distribution systems is analysis of unaccounted for water (UFW). This analysis is performed with some different purposes, for instance, leakage control, assessment of network conditions and leakage estimation. To analyze the water losses, real hydraulic conditions and pressure values are required. These data are produced by a hydraulic model (EPANET) in this research. Then with a proper leakage-pressure relationship, values of leakage are computed. Leakage-pressure relationship parameters are obtained from a GIS model.

GIS database for a water distribution system is developed by Arc/Info and Arc/View softwares. Then using an interface program all the required data are extracted from the GIS and an EPANET input file is built. After running the hydraulic model, the EPANET output file is exported to the GIS model using another subroutine. Using this integrated model, data from a real network in the north of Iran is analyzed and values of leakage are calculated. The results show good correlation with the measured values.

1 INTRODUCTION

In recent years by developing Information Technology (IT) and Management Information Systems (MIS) possibility of integration of individual information systems have been increased. In water systems, which include various data sets related to the network, properties, etc., storage, retrieval and edit of the information are very important for optimum management of the systems.

A number of geospatial information systems (GIS) were developed in the last decade not only for managing the information, but also as a decision support system for management of the network (Jaccobs et al. 1993, Feinberg & Uhrich 1997). Development of a GIS model and production of the required information in water utilities are very time consuming and expensive. It has been clear in recent years that application of GIS in water and wastewater systems without any link with hydraulic simulation models cannot support all required management targets. Ramirez (1997) developed an interface to link a hydraulic model and ArcView. These efforts have been continued by Alzamora et al. (1997) and Tabesh et al. (2003). More recently Bostanian (2001) used an integrated model for unaccounted for water studies in water distribution system. Also, Maksimovic and Carmi (1999) presented a method to evaluate the vulnerability of a water distribution network to the pressure and age of pipes.

Several relationships and procedures have been developed for leakage calculation in water networks (UK/WI 1994, Tabesh 1998). Lambert (1994) introduced the background and bursts estimation concept (BABE) to determine the leakage parameters. Besides minimum night flow estimation concept (BABE) to determine the leakage parameters. Besides minimum night flow measurement procedure (UK/WI 1994), the concept of fixed and variable discharge area of leakage (FAVAD) introduced by May (1994) to calculate a District Metered Area (DMA) leakage. Furthermore, a few leakage-pressure relationships have been introduced to calculate pipe leakage (Germanopoulos 1985, Vela et al. 1991). Despite these researches, an integrated GIS and hydraulic simulation model which can be applied in leakage calculation of a network is still required.

In this paper a GIS software (ArcView) is connected to a hydraulic simulation software (EPANET) by two interface subroutines. Using this integrated model after any change in network, the required information is exported from the GIS to the EPANET and after hydraulic simulation, the results are exported to the GIS, again. Then, this integrated model is applied to calculate the leakage in water systems using some leakage-pressure relationships.

2 METHODOLOGY

The methodology implemented in this paper consists of three sections including development of a GIS database and two user interface subroutines to connect GIS and hydraulic analysis models and a procedure for leakage calculation in water distribution systems using the integrated model.

2.1 *Development of a GIS database*

In order to input network data into a GIS environment, the data are organized into different layers (such as nodes, pipes, valves, tanks). There is a need to code spatial relations of data. Such a coding is being done using different models. In vector spatial data method (in which the features are shown and stored as point, line and polygon), a topologic model is used. Topological construction is done using some GIS softwares such as Arc/Info. Some other GIS softwares like ArcView are used to display, edit, develop databases and query from databases.

After producing network map in a CAD environment, these maps will be converted to one or more coverage using the Arc/Info. Each coverage includes four themes. A theme is a file, which contains all features of the coverage having common class. Each theme is regarded as a data layer. For each theme an attribute table is created in topological construction phase.

To transfer themes to the ArcView environment, a view is defined first. View covers information, which is required by the ArcView. However, view does not include the data/database itself, it reflects the database forms. After creation of view, the required themes are positioned in these files. Therefore, tables of themes can be edited and developed and databases can be completed.

ArcView is able to edit and develop attribute tables of themes. The required information can be directly introduced into the ArcView environment for each table. It is also possible to add attribute data to the tables using database software such as Excel to convert data to a database format (e.g. DBF). Then, it is added to an available attribute table using capabilities of the ArcView.

2.2 *Data transfer between database and the hydraulic analysis program*

There are a variety of attribute data, which are used as data layers in a customized GIS for water distribution systems. Some of these data such as nodal demands and friction coefficients of pipes have to be updated regularly. Data transfer between GIS and hydraulic analysis models is done using two interface subroutines.

Tables of attribute data for each layer are extracted in Delimited Text formant from the ArcView environment. The table consists of huge amount of information. For example, the attribute data for pipes include a variety of fields such as code, length, diameter, type, friction factor, start and end nodes, address, laying date, time of last repair, pipe depth, etc. In addition, the attribute data include the pipe breaks, vulnerable routes to leak and the results of different hydraulic analyses. The format of input files in the hydraulic analysis software (EPANET) includes data related to nodes, pipes, valves, pumps, and tanks in the network.

An interface subroutine was developed to extract the required information from the ArcView using a unified file at an acceptable format to be inputted to the EPANET. On the other hand, after hydraulic simulation, the results such as pressure and density of chlorine at nodes as well as the velocity of water in pipes are stored in some tables in the EPANET output file. At this stage, another subroutine program was developed to transfer this information using Excel software to dBase format and finally joined them to attribute tables in the ArcView. By this method analysis of water distribution networks can be performed in very quick and reliable way. Also, decision making in emergency cases can be supported using a number of analysis and representation capabilities of GIS.

2.3 *Leakage calculation*

Leakage in a network is estimated by the FAVAD method. In this method leakage at time t, L_t, is obtained by the following equation (May 1994).

$$L_t = NFM_0 \, [\, AZP_t / AZNP_0]^{N} \tag{1}$$

where NFM_0 is the minimum night flow measured in the DMA, $AZNP_0$ is the average zonal night pressure and AZP_t is the average zonal pressure at time t. N is related to variations of leakage discharge areas and pressure. Using the ratio of the fixed discharge areas to the variable discharge areas at pressure P_0, R_{FVL}, N is obtained from Figure 1 when pressure varies from P_0 to P_1.

The minimum night flow is obtained from two successive measurements at fixed time intervals (e.g. 15 min.). This parameter is measured by one or more flow meters at the lowest demand period (i.e. midnight). To reduce the effects of irregular variations of NFM this procedure is repeated at a few successive nights and the minimum flow is chosen as NFM_0. $AZNP_0$ is determined by pressure gages during minimum demand time (e.g. from 24 p.m. to 4 a.m.). Having the value and elevation of each pressure gage, $AZNP_0$ is calculated as follows.

$$AZNP_0 = \frac{\sum P_i^{av} \times EL_i}{\sum EL_i} \quad \text{(i = pressure gage number)} \tag{2}$$

in which P_i^{av} and EL_i are the average pressure and elevation at gage i, respectively. AZP_t is also calculated in a same way.

The best points for pressure measurement are determined in a trial and error procedure using a hydraulic simulation model. The integrated GIS and hydraulic simulation model is very helpful to speed this procedure. For this aim some hypothetical breaks are defined on the network and after

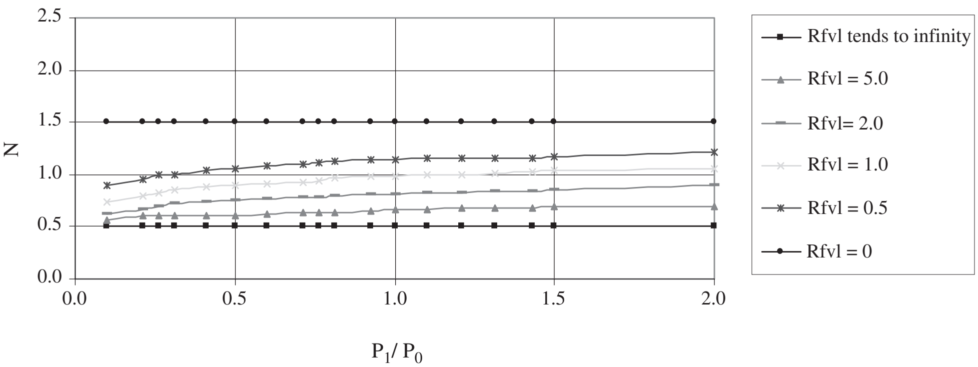

Figure 1. Power N in FAVAD leakage-pressure relationship (May 1994).

hydraulic analysis, the results (including nodal pressures and velocity in pipes) are exported to the GIS model. Comparing these results and the network specifications before breaks, sensitive points to these breaks are determined to install the pressure gages (Tabesh & Yaraghi 2003).

In the second part, assuming uniform distribution for leakage in a pipe, value of leakage at each pipe is calculated as follows (Germanopoulos 1985).

$$Q_{L,ij} = C.\ L_{ij}\ .(P_{ij}^{av})^{1.18} \tag{3}$$

where $Q_{L,ij}$ is the leakage discharge from pipe ij, C is a fixed coefficient related to the network characterizations, type and age of pipes, L_{ij} is length of pipe ij and P_{ij}^{av} is pressure averaged in the start and end nodes of each pipe. After hydraulic simulation by the EPANET, calculated pressure values are exported as new attribute data into the GIS model. Then, leakage from each pipe is calculated by Eq. (3) in the ArcView.

3 CASE STUDY

To calculate the proposed methodology, a DMA in one of the northern cities of Iran has been chosen. Information data related to the DMA and its network is presented in Tables 1 and 2. With closing 5 gate valves, existing connections to the vicinity areas have been cut and by metering the only remaining input route to the DMA, the net inflow is measured.

Before entering the network map to the GIS, the required corrections were made on the data and then the DMA data was organized based on different layers of pipes, valves, properties, wells, buildings and streets. After preparing the network map file, topology was built for each layer in the Arc/Info. Then all built coverages were imported to the ArcView. The main data was related to pipes and valves. The concerned database for pipes includes diameter, type, friction coefficient, depth, date of laying, date of latest repair, etc. Nodes layer consists of coordination, elevation, pressure and base demand. Valves layer also includes type, code, depth, nodal numbers in each side, diameter, friction coefficient, etc. For properties layer, information of type and diameter of the service pipe, mean consumption, address, type of meter, etc., were imported.

To determine points of pressure measurement, after considering some assumed breaks in some parts of the network, the hydraulic performance of the system was analyzed and the results were imported to the pipe layer information in the GIS. Then, the subtracted pressure values before and after each break were entered to this field and the highest value was identified. From this layer the most sensitive nodes were illustrated. According to this procedure 12 points were determined in this DMA and pressure variations were measured during one week for 2 hours intervals.

The required parameters in the FAVAD concept to calculate the DMA leakage are defined as follows. Based on the pipe breaks data, the values of fixed and variable discharge areas for different leak shapes (i.e. complete pipe break, whole and crack) are equal to 2561.01, 208.62 and 143.48 cm^2,

Table 1. DMA information data.

Area	76.5 hec.
No. of loops	1
No. of pipes	441
No. of valves	92
No. of wells	19
No. of properties	1533
Average property consumption	0.74 m^3/day

Table 2. Pipe data.

Type	Diameter (mm)	Length (m)
AC	100	9877
AC	150	1774
AC	200	1671
PE	63	1562
PE	110	935
GA	30	816
GA	40	3234
GA	60	3242
GA	80	398

respectively. The coefficient of R_{FVL} is calculated as follows:

$$R_{FVL} = [(2561.01 + 208.62)/143.48] = 19.3$$

Therefore, based on Figure 1, N is obtained as 0.5. According to the night flow measurements from 2 to 4 a.m. and the weighted mean of pressure values at 2 a.m. (Table 3), $NFM_0 = 89.2\ m^3/hr$ and $AZNP_0$ is calculated as 2.81 atm.

$AZNP_0$ can be also calculated by the GIS model in more exact way as follows. Using the pressure gage results, the EPANET model is calibrated. Knowing the night demand at each node, the network is simulated. Nodal pressure values are exported to the GIS attribute data sets. Multiplication of nodal pressure and elevation values are added as new information. Then by dividing these data to summation of the nodal elevation values the exact $AZNP_0$ value is calculated.

AZP_t is evaluated in a same way. Values of AZP_t and L_t can be seen in Table 4. For example, at 4 a.m. these parameters are calculated as follows:

$$AZP_t = 46.03/18.4 = 2.5 \text{ atm.} \quad \text{and} \quad L_t = 89.2 * (2.5/2.81)^{0.5} = 84.13\ m^3/hr$$

According to a field measurement based on the minimum night flow procedure and applying correction factors of flow meters and leak factor at mean pressure, the value of leakage in the DMA was obtained as 18.1 lit/sec. It can be seen that there is about 10% difference between the calculated and measured values, which is acceptable.

To determine the leakage at each pipe, C coefficient in Eq. (3) should be evaluated at first. In this regard the network is divided to a few smaller DMAs and leakage value at one DMA is obtained by night flow measurements. The nominated DMA should have the most similarity with the entire area. Having the nodal pressures and DMA leakage value, C is assumed and leakage value of each pipe is calculated. In a trial and error procedure, by comparing the calculated and measured leakage values the procedure is continued. After final step, the exact C coefficient is obtained as 0.000025 for the DMA. Finally, the values of leakage at each pipe are calculated by the GIS model and suspected routines to leakage with different priorities are identified. Figure 2 shows the pipe leakage calculation procedure inside the GIS model. Figure 3 illustrates the pipes with leakage more than 0.1 lit/sec. This map is very useful for leak detection program within the DMA.

Table 3. The mean pressure values measured at 2 a.m.

Pressure gage number	Mean pressure value (atm) (EL)	Elevation Elevation of pressure gage (m)(P)	El * P
1	3	10.8	32.4
2	3.2	10	32
3	4	3.9	15.6
4	3.17	−3.2	−10.14
5	1.11	−3.2	−3.55
6	2	−9.9	−19.8
7	3.17	−12.8	−40.57
8	2.67	−4.5	−12.01
9	2.4	−0.8	−1.92
10	5.2	4.5	23.4
11	1.6	8.7	13.92
12	1.5	14.9	22.35
Σ		18.4	51.68

$$AZNP_0 = \frac{51.68}{18.4} = 2.81 \text{ atm.}$$

Table 4. Diurnal values of AZP_t and L_t in the DMA.

Time (hr)	Σ P * EL	AZP_t (atm)	L_t (m^3/hr)
2	51.68	2.81	89.2
4	46.03	2.5	84.13
6	48.9	2.65	86.6
8	29.4	1.6	67.3
10	14.02	0.76	46.39
12	3.26	0.177	22.4
14	4.63	0.25	26.7
16	3.23	0.176	22.26
18	4.27	0.232	64.25
20	55.57	1.98	85
22	38.76	1.37	62.27
24	68.13	2.4	82.4
Σ			700.29

$$\boxed{\frac{\sum L_t \times 2}{24} = 58.35\ m^3/hr = 16.2 \text{ lit/sec}}$$

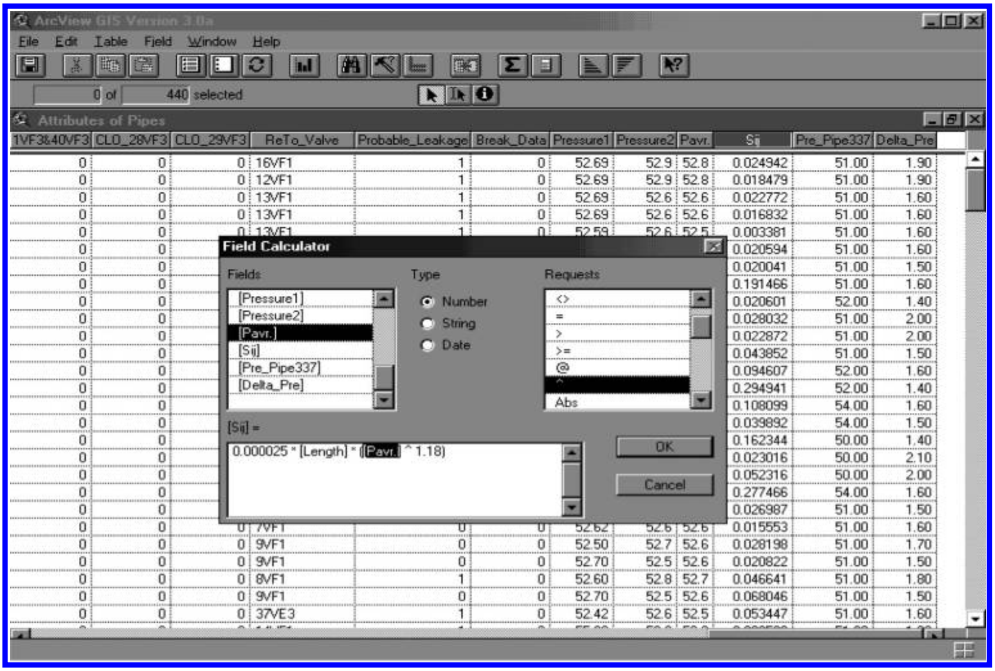

Figure 2. Pipe leakage calculation in the ArcView.

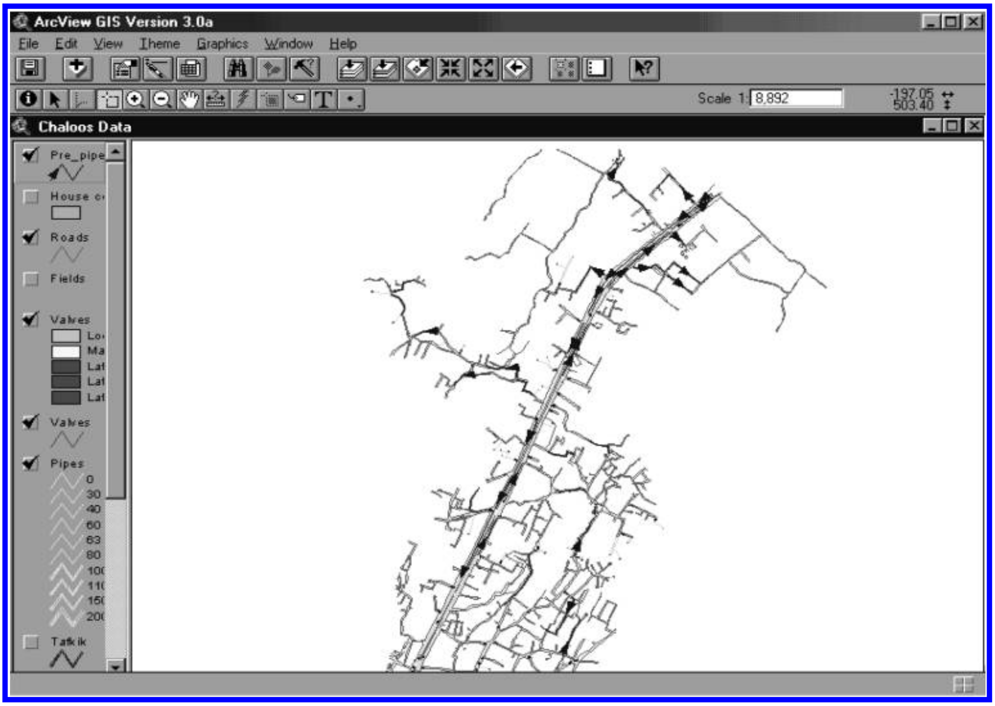

Figure 3. Some parts of the DMA with leakage more than 0.1 l/s.

4 SUMMARY AND CONCLUSIONS

In this paper the ArcView and EPANET softwares were combined by two interface subroutines. Using the information of properties and network, the maps and attribute data were produced by the Arc/Info and ArcView, respectively. The effects of every normal or abnormal situation, which may be occurred in the system, are identified by the EPANET simulation model. By transferring the new hydraulic results to the GIS model more exact analyses can be performed. The application of this integrated model to leakage calculation of a DMA in a city in the north of Iran was successfully examined and presented. Without this combined model, application of GIS in water distribution systems cannot evaluate the changed hydraulic situation of the network properly.

REFERENCES

Alzamora, F.M., Ayala, H.J.B. & Roca, N.M. 1997. Connecting ArcView 3.2 to FPANET2 a full environent to manage water distribution systems using models. In Ulanichi, B., Coulbeck, B. & Rance, J. (eds.), *Water Softwater Systems: Theory and Applications, Vol. 2.*

Bostanian, M.B. 2001. *Application of combined GIS and hydraulic model in unaccounted for water studies of water distribution networks*. MSc. Thesis, Civil Engineering Department, University of Tehran, Tehran, Iran.

Feinberg, D. & Uhrich, S.W. 1997. Integrating GIS with water and wastewater hydraulic models. *Esri User Conference, http://www.Esri.com.*

Germanopoulos, G. 1985. A technical note on the inclusion of pressure dependent demand and leakage terms in water supply network models. *Civil Engineering Systems,* 2(September): 171–179.

Jacobs, P. & Goulter, I.C. & Davidson, 1993. Water-distribution GIS from fragmented and incomplete information. *J. Computing in Civil Engineering, ASCE,* 7(3).

Lambert, A. 1994. Accounting for losses: The bursts and background concept. *J. JWEM,* 8(April): 205–214.

Maksimovic, C. & Carmi, N. 1999. GIS supported analysis of pressure dependant vulnerability of distribution networks to leakage. In Savic, D.A. & Walters, G.A. (eds.), *Water Industry Systems, Proceedings of CCWI99, University of Exeter, Vol. 2: 85–96.*

May, J. 1994. Pressure dependent leakage. *World Water & Environmental Engineering Management*, October.

Ramirez, A. 1997. Interfacing potable water hydraulic models with Arc/Info and ArcView data sets: From start to finish. *http://www.esri.com/library/userconf/proc97/abstract/a454.htm.*

Tabesh, M., Bostanian, M.B. & Delavar, M.R. 2003. Producing a combined GIS and hydraulic model for better management of water supply systems. In D. Mostofinejad (ed.), *Proceedings of the 6th International Conference on Civil Engineering, Isfahan University of Technology, Isfahan, Iran, 5–7 May.*

Tabesh, M. & Yaraghi, P. 2003. Analysis of physical losses in water distribution networks using BABE method. In D. Mostofinejad (ed.), *Proceedings of the 6th International Conference on Civil Engineering, Isfahan University of Technology, Isfahan, Iran, 5–7 May.*

Tabesh, M. 1998. *Implications of the pressure dependency of outflows on data management, mathematical modelling and reliability assessment of water distribution systems*. Ph.D. Thesis, Department of Civil Engineering, University of Liverpool, UK.

U.K. Water Industry. 1994. *Managing leakage. Report G: Managing water pressure*. WRc plc/ Water Service Association/ Water Companies Association, Swindon, UK.

Vela, A., Perez, R. & Espert, V. 1991. Incorporation of leakage in the mathematical model of a water distribution network, *Proceedings of the 2nd International conference on Computer methods in Water Resources, 20–22 February: 245–257*, Southampton: Computational Mechanics Publication.

Chapter 3 – Analysis, design and rehabilitation of distribution networks

Least-cost design of water distribution networks under uncertain demand

A.V. Babayan, D.A. Savic & G.A. Walters
Centre for Water Systems, University of Exeter, Exeter, UK

ABSTRACT: The problem of least-cost design of a water distribution network with a given level of reliability under uncertain demand is considered. The original stochastic formulation is replaced by a deterministic one, using standard deviation as the natural measure of the variability of the pressure in the nodes caused by uncertainty in demands. Such an approach allows us to use effective numerical methods to quantify the influence of the uncertainty on the robustness of the water distribution system and to avoid using Monte Carlo simulation during optimisation process. The proposed methodology was tested on the New York tunnel problem. The optimum networks was found for different levels of reliability and different forms of probability distribution functions for demands. The robustness of the optimal networks found was compared with known solutions for determenistic formulation, using Monte Carlo simulation.

1 INTRODUCTION

The vast majority of the mathematical models use deterministic approaches to describe various processes and systems. Meantime all real life problems incorporate uncertainty in one way or another. It could be uncertainty in measurement, uncertainty in estimation of parameters, uncertainty in which processes one should include into the model etc. Such contradiction between "mathematical determinism" and "natural uncertainty" can seriously affect the reliability of the results of mathematical modelling. So developing of methodology which allow us to take into account uncertainty when predicting the behavior of the system is of great practical interest.

In the process of design of water distribution networks one of the most uncertain quantities is the value of demands at the network's nodes. The good engineer can estimate peak demand for existing consumers with an error of 4–6% (Obradovic and Lonsdale 1998), which does not appear excessive. However, the situation becomes much worse when one needs to *predict* the consumption for the time horizon 10 or 20 years into the future, which is usually the case for rehabilitation or master planning studies. Besides engineering factors, such estimation should take into account political, economical perspectives etc. It is clear that in this situation significant errors are unavoidable. Natural caution usually leads to overestimation of demands, which results in excessive cost for a project.

Therefore, it is important to take uncertainty of demand into account when designing new water systems or extending/rehabilitating existing ones. This usually means that one has to include demand uncertainty considerations as corresponding constraints into the problem formulation (Lansey et al., 1989; Xu and Goulter, 1999). The main obstacle here is that constraints have to be imposed as a minimum pressure head requirement at nodes, that in turn depend on uncertain variables (demand). The problem is difficult to solve because the problem is non-linear and the relationship between the demand and pressure is an implicit one.

Water distribution system design optimisation is one of the most heavily researched areas in the hydraulics profession (see Walski, 1985; Goulter, 1992; Lansey, 2000 for detailed review). Recently, genetic algorithms (GA) have become the preferred water system design optimisation technique for many researchers (Dandy et al., 1996; Savic and Walters, 1997) because they demonstrate good ability to deal with complex, nonlinear and discrete optimisation problems. This method uses full network model simulation to evaluate how good a design solution is and, therefore, may require excessive computing time when real networks are considered. To quantify the influence of uncertainty in demand, Monte-Carlo simulation is usually implemented. This, however, means that one has to perform a large number (e.g. tens of thousands) of simulation runs even for relatively simple networks. This is unacceptable when using GAs for optimization, as one needs to calculate the fitness function for large number of network configurations.

In this paper the original stochastic formulation is replaced with a deterministic one, using standard deviation as a natural measure of the variability of the pressure in the nodes caused by uncertainty in demands. Such an approach allows us to use effective numerical methods to quantify the influence of the uncertainty on the robustness of the water distribution system design.

2 PROBLEM FORMULATION

Consider the design of a water distribution network as a least-cost optimisation problem with given reliability under demand uncertainty. Pipe layout, connectivity and minimum pressure head constraints at pipe junctions (nodes) are assumed given and pipe diameters are considered as decision variables. Let us also assume that demands at nodes are independent, uncertain, random variables with given probability distribution functions (p.d.f.). The computed hydraulic head at a node is an (implicit) function of demands, therefore, the pressure in each node is a random variable itself for which mean and standard deviation can be calculated, using conventional formulas (Ross, 1987). Note that in general one can expect correlation between demands in some groups of nodes (e.g. due to some changes in the weather). If this correlation is strong, group nodal demands can be described using one statistical vector variable whose components change simultaneously according to a common p.d.f. (may be, with different parameters).

The objective is to design a water distribution network that minimises the cost and meets the pressure requirements in terms of given reliability under uncertain demand. The problem will be solved if the solution satisfies: (a) minimum cost requirement; (b) probability that pressure in all nodes exceeds required minimum value is more than some prescribed value.

The mathematical formulation of the problem is given as:

$$\boxed{f(D_1, D_2, \ldots, D_N) = \sum_{i=1}^{N} c(D_i, L_i) \to \min} \tag{1}$$

$$P\left(H_j \geq H_j^{\min}, j = 1, \ldots, M\right) \geq \varphi \tag{2}$$

where $c\ (D_i,L_i)$ – cost of pipe i with diameter D_i (which is chosen from a discrete set of diameters) and length L_i, H_j – head at junction j, $P(\cdot)$ – probability of some event, $H_j^{\min}$ – minimum required head at node j, φ – desirable reliability of the network (ee. 95% or 99%) N – total number of pipes in the system, and M – number of junctions. Further we will denote any fixed vector of diameters $[D]$ as a "configuration" of the system. Inequality (2) means that if we generate a set of vectors of demand $[Q^e]$ from the given distribution condition $H_j \geqslant H_j^{\min}$ will be satisfied for, say, 95% cases (depending of value of the φ).

To calculate the state of the system for any given configuration and fixed demands $[Q^e]$ one has to solve the non-linear system of equations consisting of continuity equations at nodes and energy conservation equations around each elementary loop:

$$\sum Q^{in} - \sum Q^{out} = Q \tag{3}$$

$$\sum h_f - \sum E_p = 0 \tag{4}$$

where Q^{in} – flow into the junction, Q^{out} – flow out of the junction, Q – demand at a junction node, h_f – head-loss term which can be expressed using Hazen-Williams or Darcy-Weisbach formula, E_p – energy put into a liquid by a pump. Non-linearity here is associated with the h_f term:

$$h_f = \omega \left(\frac{Q^p}{C} \right)^a \frac{L}{D^b} \tag{5}$$

where ω – numerical conversion constant which depends on the units used, C – pipe Hazen-William roughness coefficient, Q^p – flow through the pipe, L and D – length and diameter of the pipe correspondingly. The constants a and b are usually chosen as 1/0.54 and 2.63/0.54 correspondingly, but some other values are also used (Obradovic and Lonsdale, 1998).

It follows from equation (5) that pressure heads at nodes are non-linear and implicit functions of pipe diameters and volumetric demand.

3 SOLUTION TECHNIQUE

Due to intrinsic discreteness and non-linearity of constraints, the optimisation problem (1)–(3) is difficult to solve even for deterministic case. We will use the genetic algorithms (GA) which have proved their capability to yield good approximate solutions for such kinds of problems (Michalewicz, 1992). The method mimics the natural selection process and allows better solutions to evolve through inheritance brought about by genetic operators (cross-over and mutation). Unlike standard non-linear programming techniques (Duan et al., 1990) or linearization (Alperovits and Shamir, 1977) it does not require simplification of the original problem or moving it to the different (continuous) solution space. Another great advantage of GAs is its universality; i.e. it can be applied quite straightforwardly to any network.

Using GA requires calculating the fitness function for many possible (but not always feasible) network configurations. For deterministic problems (when demands are certain and condition (2) is in the form $H_j \geqslant H_j^{\min}$) one needs to compute the state of the system only once for every network considered. But when uncertainty is included the situation becomes more complicated. The condition (2) can be rewriting in the form

$$\int_{\mathbf{H} \geq \mathbf{H}^{\min}} \mu(\mathbf{H}) d\mathbf{H} \geq \varphi \tag{2*}$$

where $\mu(\mathbf{H})$ – p.d.f. for the heads in the nodes. So to check whether a possible solution is feasible it is necessary to find the value of the multidimensional integral in (2*). As heads at nodes depend on demand implicitly the function $\mu(\mathbf{H})$ is unknown. So there is no way to compute or estimate integral in (2*) analytically.

The common technique to estimate the probability (2) is by using Monte Carlo simulation. An idea is to generate randomly a set of the vectors of demands from the given distribution and calculate the heads in nodes for every member of this set. If the total number of vectors generated is N_Q and the condition $H_j \geqslant H_j^{\min}$ is satisfied for N_S of them the sought probability $P(H_j \geqslant H_j^{\min})$ is accepted as N_S/N_Q. This method is quite reliable and widely used (Bargiela and Hainsworth, 1989). However it requires a large number of calculations of state estimates for every network configuration, which is especially unacceptable in case of GA.

So, in order to use GA for optimisation we need to replace condition (2) with another, which is easier to compute but provides the desirable level of reliability of the final solution. The simplest way is to replace (2) with condition

$$H_j \geq H_j^{\min} + a_j \tag{6}$$

where a_j is a "margin of safety" for every node, ensuring robustness of the system. The most difficult part for the designer is obviously specifying values of a_j for every particular node. Too cautious an estimate will lead to a too expensive solution while a too optimistic one can cause system failure. Moreover, the variation in node head due to demands fluctuations depends on the network configuration as well, so for the same node different "safety margins" can provide the same level of reliability.

The measure of diversity of a statistical variable is its standard deviation, so it is naturally to use it as the base for defining a_j:

$$H_j \geq H_j^{\min} + \alpha\sigma(H_j(\mathbf{Q})) \tag{7}$$

where $\sigma(H_j(\mathbf{Q}))$ – standard deviation of the head in node j (which depends on vector of demands $[\mathbf{Q}]$ in the whole network), α – parameter which determines the level of reliability of the system.

Head standard deviation σ at each node can be calculated using the following theoretical formula (Ross, 1987):

$$\begin{aligned}\sigma\left(H_j(\mathbf{Q})\right) &= \int_{-\infty}^{+\infty}\ldots\int_{-\infty}^{+\infty}\left(\left(H_j(\mathbf{Q}) - \xi\left(H_j(\mathbf{Q})\right)\right)^2 \prod_{k=1}^{M} \eta_k(Q_k)\right) dQ_1 \ldots dQ_M \\ \xi\left(H_j(\mathbf{Q})\right) &= \int_{-\infty}^{+\infty}\ldots\int_{-\infty}^{+\infty}\left(H_j(\mathbf{Q}) \prod_{k=1}^{M} \eta_k(Q_k)\right) dQ_1 \ldots dQ_M\end{aligned} \tag{8}$$

Here $\xi(H_j(\mathbf{Q}))$ – mean of head in node j, $\eta_k(Q_k)$is the known p.d.f. for the demand at the node k. Unfortunately, because of implicit relationship between demands and heads it is impossible to calculate (8) analytically. Moreover, although one can compute the value $H_j(\mathbf{Q})$ for any fixed vector $[\mathbf{Q}]$, it is virtually impossible to use quadrature formulae for numerical estimation of these integrals for any real problem, as the dimension of the integrals is equal to the number of nodes M in the network. So further simplifications are necessary.

To estimate standard deviation we will use the same technique as (Babayan et al., 2003). It is based on assumptions that (1) head fluctuations at some node are usually caused by uncertainty in the demand at that node; (2) in most cases only a small proportion of network nodes are critical in terms of head constraint violations (see eqn. (2)). The first assumption allows us to replace integrals in (8) to one-dimensional ones:

$$\begin{aligned}\sigma\left(H_i(Q_i)\right) &= \int_{-\infty}^{+\infty}\left(\xi\left(H_i(Q_i)\right) - H_i(Q_{ik})\right)^2 \eta_i(Q_i)\, dQ_i \\ \xi\left(H_i(Q_i)\right) &= \int_{-\infty}^{+\infty} H_i(Q_i)\,\eta_i(Q_i)\, dQ_i\end{aligned} \tag{9}$$

so we can use a fast and effective (for example Gaussian) quadrature formulae (Krylov, 1962) for numerical integration.

The second assumption means, that only some nodes in network are "critical", in that demand uncertainty in these nodes decreases reliability of the whole network. So one can build up the list of such nodes and consider the demand in the rest of the network as certain. It allows significant reduction of the computation time.

Finally the following algorithm is proposed to solve the problem (1), (6) using GA:

1. *Analyze network vulnerability for uncertain demands*
   Take some initial configuration of the network and consider demand at node *i* as uncertain and compute the deviation for pressure at each node in the network using formulae (9). If constraint (6) is violated for at least one of the nodes then node *i* is considered to be "critical" and is added to the set of "critical nodes" (Ω). At the next stage only demand at nodes from Ω will be considered as uncertain.
2. *Run GA to find optimal design.*
   Use a single-objective GA to get the minimum of the function

$$g([D]) = f([D]) + p_\sigma([D]) \tag{10}$$

where $f([D])$ is cost function from (1), $p_\sigma([D])$ – penalty function to guarantee that the solution obtained satisfies conditions (2). To compute the penalty function values we will compute $\sigma(H_i(Q_i))$ for the current system configuration $[D]$ using (9). Recall that now only demand in nodes from Ω is assumed to be uncertain, that is for $i \notin \Omega$ $\sigma(H_i(Q_i)) = 0$ (and we actually do not need to compute it). We will take p_σ as

$$p_\sigma = \omega \sum_{i\in\Omega}\sum_{j}\left(H_j^{\min} + \alpha\sigma\left(H_j(Q_i)\right) - H_j\right) \; for \;\; j : H_j < H_j^{\min} + \alpha\sigma\left(H_j(Q_i)\right) \tag{11}$$

where ω is the penalty rate. The total number of state estimate calculations necessary to obtain the fitness function value (11) using formulae (1), (10) and (11) is $|\Omega| n$ (where $|\Omega|$ – size of Ω, n – number of points in the quadrature formula used) for each member of the population. In general, as the GA run progresses, new critical nodes may have to be added to the set Ω identified for the initial network configuration. To address this potential problem, an automatic procedure can be run periodically during the GA run.

4 CASE STUDY

The New York tunnels problem well-known from literature (Schaake and Lai, 1969) was chosen to serve as illustrative examples. The objective of this study was to determine the most economically effective design for additions to the existing water distribution systems of the City of New York. The input data for existing pipe, discrete set of available diameters, and associated unit pipe costs, were taken from the paper Savic and Walters, (1997).

Two types of probability distribution function of demands were considered: the truncated Gauss (normal) and uniform.

$$\boxed{pdf_n(Q) = \begin{cases} \dfrac{1}{0.9974}\dfrac{1}{\sqrt{2\pi}\sigma}e^{-\frac{(Q-a)^2}{2\sigma^2}} & |Q-a| <= 3\sigma \\ 0 & |Q-a| > 3\sigma \end{cases}} \tag{12}$$

Here a and σ are the given mean and standard deviations respectively, and the multiplier 1/0.9974 is introduced to satisfy condition

$$\int_{-\infty}^{\infty} pdf_n(x)dx = 1$$

Table 1. The parameters of the GA used.

Genetic Algorithm	Generational Elitist
Population size	200
Number of iterations	2000
Crossover	Multipoint crossover
Crossover Rate	0.8
Mutator	Random non uniform by gene
Mutation Rate	0.9
Selector	Roulette

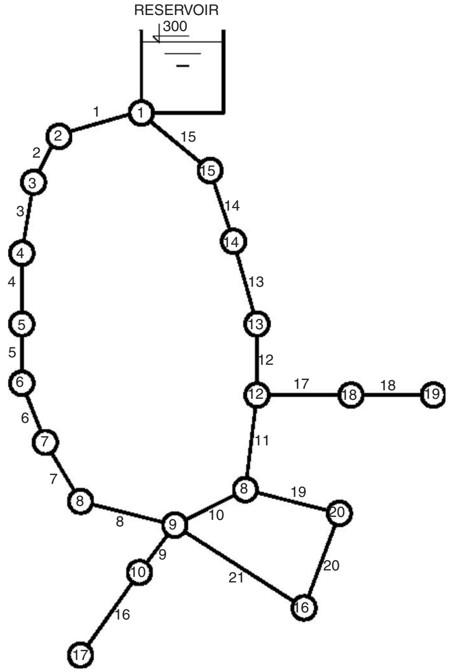

Figure 1. Layout for New York city water supply system.

For the test we took the mean demand a for each node as demand from Savic and Walters, (1997) (NY tunnels problem). The deviation was taken as $\sigma = 0.1a$.

P.d.f. for uniform distribution was taken in the form:

$$\boxed{pdf_u(D) = \begin{cases} \dfrac{1}{2b} & |D-a| <= b \\ 0 & |D-a| > b \end{cases}} \tag{13}$$

where a and b are the given mean and boundaries for demand correspondingly. We used the same values for mean demand a for previous p.d.f. and $b = 0.15a$. It means that demands are specified with 15% error and can take any values from $0.85a$ to $1.15a$ with equal probability. The main parameters of the GA used to obtain solution are summarised in the Table 1. To calculate integrals (9) for each node from the set of critical nodes the 5-point Gaussian-Hermite and Gauss-Legendre

Table 2. Solutions for different values of α. Truncated Gauss distribution.

	$\alpha = 0$	$\alpha = 3$	$\alpha = 4$	$\alpha = 5$	$\alpha = 6$
Cost (mln $)	38.13	48.36	50.4	52.59	56.65
Reliability (%)	35.3	91.4	94.7	97.7	99.4

Table 3. Solutions for different values of α. Uniform distribution.

	$\alpha = 0$	$\alpha = 3$	$\alpha = 4$	$\alpha = 5$	$\alpha = 6$
Cost (mln $)	38.13	45.76	48.89	51.70	54.32
Reliability (%)	30.7	89.2	95.3	98.4	99.5

Table 4. Reliability of known solutions for deterministic problem formulation.

	Quindry et al. (1981)	Gessler (1982)	Bhave (1985)	Morgan & Goulter (1985)	Murphy (1993)
Cost (mln $)	63.58	41.80	40.18	39.20	38.80
Reliability (normal) (%)	33.3	36.9	35.6	34	34.6
Reliability (uniform) (%)	32.1	36.5	34.5	34.2	33.7

quadrature formulas were used ($n = 5$) for truncated and uniform distributions correspondingly (Krylov, 1962). These formulas have the 6th degree of accuracy, which is quite enough for our purposes.

The same pipe layout (Fig. 1), minimum head and Hazen-Williams coefficients were used as in Savic and Walters, (1997).

The analysis of the initial configuration of the network shows that nodes 16, 17, 18, 19 and 20 should form the set of critical nodes Ω. To calculate the fitness function (11) the EPANET solver was run $|\Omega|n = 5*5 = 25$ times for every network configuration (organism) considered. The following values of parameter α (condition (7)) were considered: $\alpha = 6, 5, 4, 3$ and 0 (the case of $\alpha = 0$ corresponds to optimisation without taking into account uncertainty). The GA was run 5 times for every α and the best solution was chosen.

The robustness of final solutions for different α was checked using full Monte Carlo simulation with 100000 sampling points. The results for truncated Gauss and uniform p.d.f. for demands are shown in Tables 2 and 3 correspondingly. The same analysis was carried out for solutions for deterministic formulation of the problem known from literature (Table 4). As one can see if uncertainty is not taken into account, the reliability of the resulting system is approximately the same for all solutions and does not exceed 40% which is unacceptably low. At the same time, the cost of the most reliable network with robustness level 99% is still less then the cost of the most expensive configuration.

5 CONCLUSIONS

In this paper the methodology is proposed for designing water distribution networks under uncertain demand with a given level of reliability. The original probabilistic formulation is replaced with a simplified deterministic one, which allows for the solutions to be obtained in reasonable time using GA. Also the robustness of the optimal networks found with the proposed methodology was compared with known solutions for determenistic formulation, using Monte Carlo simulation. The results clearly demonstrate that the reliability of solutions, which do not take uncertainty into account, is usually unacceptably low. The methodology proposed can be used, with some changes, for various source of uncertainty (e.g. pipe diameters, roughness etc.) with different p.d.f.

REFERENCES

Alperovits, E. & Shamir, U. 1977. Design of Optimal Water Distribution Systems. *Water Resources Research* 13(6): 885–900.

Babayan A.V., Savic, D.A. & Walters G.A. 2003. Design of Water Distribution Network Under Uncertain Demands. *Pumps, Electromechanical Devices and Systems Applied to Urban Water Management; Proc. intern. conf. Valencia, 22–25 April 2003.* Netherlands: Balkema.

Bargiela, A. & Hainsworth, G.D. 1989. Pressure and Flow Uncertainty in Water Systems. *ASCE Journal of Water Resources Planning and Management* 115(2): 212–229.

Bhave, P.R. & Sonak, V.V. 1992. A Critical Study of the Linear Programming Gradient Method of Optimal Design of Water Supply Networks, *Water Resources Research*, 28(6), 1577–1584.

Chengchao Xu & Goulter, I.C. 1999. Reliability-Based Optimal Design of Water Distribution Network. *ASCE Journal of Water Resources Planning and Management* 125(6): 352–362.

Dandy, G.C., Simpson, A.R. & Murphy, L.J. 1996. An improved genetic algorithm for pipe network optimization. *Water Resources Research* 32(2): 449–458.

Duan, N., Mays, L.W. & Lansey, K.E. 1990. Optimal Reliability-Based Design of Pumping and Distribution Systems. *ASCE Journal of Hydraulic Engineering.* 116(2): 249–268.

Gessler, J. 1985. Pipe Network Optimization by Enumeration. *Proc. Speciality Conference on Computer Applications/Water Resources*, ASCE, New York.

Lansey, K.E., Ning Duan, Mays L.W. & Yeou-Kung Tung. 1989. Water Distribution System Design under Uncertainties. *ASCE Journal of Water Resources Planning and Management* 115(5): 630–645.

Lansey, K.E. 2000. Optimal Design of Water Distribution Systems, in Mays, L.W. (eds), *Water Distribution System Handbook*: McGraw-Hill, 2000.

Michalewicz, Z. 1992. *Genetic Algorithms + Data Structures = Evolutionary Programs*. Springer-Verlag.

Morgan, G.R. & Goulter, I.C. 1985. Optimal Urban Water Distribution Design, *Water Resources Research.* 21(5): 642–652.

Murphy, L.J., Simpson, A.R. & Dandy, G.C. 1993. Pipe Network Optimization Using an Improved Genetic Algorithm, *Research Report No. R109*, Department of Civil and Environmental Engineering, University of Adelaide, Australia.

Obradovic, D. & Lonsdale, P. 1998. *Public Water Supply*. London: E&FN Spon.

Quindry, G.E., Brill, E.D. & Liebman, J.C. 1981. Optimization of Looped Water Distribution Systems, *ASCE Journal of Environmental Engineering*, 107(4), 665–679.

Ross S.M. 1987. *Introduction to Probability and Statistics for Engineers and Sceintists*. John Wiley & Sons.

Savic, D.A. & G.A. Walters. 1997. Genetic Algorithms for the Least-cost Design of Water Distribution Networks. *ASCE Journal of Water Resources Planning and Management*. 123(2): 67–77.

Walski, T.M. 1985. State of the Art: Pipe Network Optimization, *Computer Applications in Water Resources*, ASCE.

Robust least cost design of water distribution systems using GAs

Z. Kapelan, D. Savic & G.A. Walters
Centre for Water Systems, University of Exeter, Exeter, UK

ABSTRACT: Performance of water distribution systems (WDS) deteriorates with time. As a consequence, these systems should be rehabilitated periodically. Current state-of-the-art methodologies tend to solve the problem of least cost WDS design/rehabilitation using evolutionary optimisation methods, genetic algorithms (GAs) in particular. However, in all these approaches, a GA is linked to a deterministic WDS simulation model neglecting a fundamental fact that a number of uncertainties exist in the decision making process. To reflect this, a stochastic least cost design problem is formulated and solved here. The objective is to minimise total design costs subject to a pre-specified level of design reliability. The optimisation problem is solved by linking a GA to a stochastic WDS hydraulic model. Each uncertain nodal demand modelled is assigned a probability density function (PDF) in the problem formulation phase. The corresponding PDFs of the computed nodal heads are calculated using the Latin Hypercube sampling technique. However, rather than using a large number of samples for each fitness evaluation throughout the search process, an alternative approach with a significantly reduced level of sampling is developed. The robust optimisation methodology is tested on a case study. The results obtained indicate that the new methodology is capable of identifying robust (near) optimal solutions despite significantly reduced computational effort compared to fitness evaluation using full sampling.

1 INTRODUCTION

UK water distribution systems have developed over several centuries growing both in number and in capacity. Unfortunately, there has been insufficient investment in renewal and maintenance of these systems, particularly during the period of public ownership. Many of the systems are now old and are experiencing problems, e.g. frequent service disruptions, inadequate capacity, etc. In addition to this, water consumption is likely to increase in the future. Therefore, the WDS need to be rehabilitated (i.e. re-designed) periodically. Unfortunately, rehabilitation budgets available are usually quite limited. To make things more difficult, a number of uncertainties are involved in the decision making process, e.g. uncertain level of present/future water consumption, incomplete underground asset data, uncertain economic environment, etc. Given the above, the need for considering the design/rehabilitation of WDS under uncertainty within an optimisation framework is obvious. The objective of this paper is to develop such optimisation methodology.

This paper is organised as follows: after a brief background, a robust least cost design optimisation model is presented. This is followed by application of the robust design methodology to a case study. At the end, conclusions are drawn.

2 BACKGROUND

Optimal (e.g. least cost) design of WDS is a topic that has captured a lot of interest in the past from both researchers and practitioners. A typical optimal design problem solved so far minimises the total design (i.e. rehabilitation) cost subject to several constraints: (1) implicit constraints (equations describing the WDS physics), (2) design reliability constraints (e.g. head at each node should be greater than or equal to the minimum required) and (3) explicit constraints in the form of decision variable (intervention options) bounds.

The least cost design problem was initially solved using non-evolutionary optimisation methods (Schaake and Lai 1969; Alperovits and Shamir 1977). Later on, after identifying limitations of local search algorithms, a number of authors solved the least cost design problem using evolutionary algorithms, and Genetic Algorithms (GAs) in particular (Dandy et al. 1993; Simpson et al. 1994). After initial successes, the GA-based WDS design was pushed to the next level by applying and modifying GAs to solve real-life WDS design problems (Halhal et al. 1997; Savic and Walters 1997; Walters et al. 1997). Due to its success, the GA-based least cost design methodology was encapsulated into various commercial software packages, e.g. the GANet software (Atkinson et al. 1998).

However, all aforementioned least cost design approaches share one serious limitation: the WDS simulation model used in the optimisation process is of a deterministic type. This means that all hydraulic simulation model input variables (e.g. demands, roughness coefficients, etc.) are assumed to be known with 100% certainty. Obviously, this is not true in the case of real-life systems and may lead to serious under-design of the WDS. This was noted first by Lansey et al. (1989) who developed a least cost design methodology which takes into account uncertainty of both hydraulic simulation model inputs and outputs. The optimal design problem was formulated as a single-objective, chance constrained minimisation problem and solved using the Generalised Reduced Gradient (GRG2) technique (Lasdon and Waren 1984).

Later on, Xu and Goulter (1999) developed another approach in which a probabilistic WDS hydraulic model was used for the first time in design optimisation. The WDS hydraulic model uncertainties were quantified using the analytical technique known as the first-order reliability method (FORM). To calculate the uncertainties, the FORM method requires repetitive calculation of the first-order derivatives and matrix inversions, which is computationally very demanding even in the case of small networks and may lead to a number of numerical problems. The least cost design problem was, again, solved using the GRG2 optimisation method. Being a local search method, GRG2 can easily be trapped in the local minimum in the complex, discrete, multimodal search spaces of real-life design problems (Savic and Walters 1997). In addition to this, to be able to use the GRG2 optimisation method, decision variables (pipe diameters) must be modelled as continuous decision variables, which is unrealistic. The facts mentioned above were the main reasons why gradient type optimisation methods were replaced by GAs in the mid 1990s.

To overcome the limitations of current WDS design approaches, a new approach called *robust GA optimisation* is developed and presented here.

3 ROBUST LEAST COST DESIGN

3.1 *Problem formulation*

The objective of the robust least cost design model presented here is to minimise total design (i.e. rehabilitation) costs subject to a minimum required level of design reliability. More specifically, the optimisation problem is formulated as follows:

$$\text{Minimise} \quad Cost\,(D_1, D_2, \ldots, D_{Nd}) \tag{1}$$

subject to:

(a) Set of reliability (i.e. probabilistic performance) constraints:

$$P_i(H_i \geq H_{i,min}) \geq P_{i,min} \quad (i=1,..,N_{nr}) \tag{2}$$

(b) Set of implicit constraints (valid for each random sample):

$$\sum_{k=1}^{N_i} Q_k - Q_{d,i} = 0 \quad (i=1,..,N_n) \tag{3}$$

$$H_{i,u} - H_{i,d} - \Delta H_i = 0 \quad (i=1,..,N_l) \tag{4}$$

(c) Set of explicit constraints:

$$D_{i,min} \le D_i \le D_{i,max} \quad (i=1,\dots,N_d) \tag{5}$$

where: *Cost* – total design/rehabilitation cost; D_i – value of the *i*-th decision variable (in general, rehabilitation option index, in the case study here duplication pipe diameter) limited by lower ($D_{i,min}$) and upper ($D_{i,max}$) search bounds; P_i – probability that head at *i*-th network node is equal to or above the minimum head required for that node; $P_{i,min}$ – minimum required value for P_i; Q_k – flows in all N_i pipes connected to *i*-th network node; $Q_{d,i}$ – demand at *i*-th node; $H_{i,u}$ – head at upstream node of the *i*-th pipe; $H_{i,d}$ – head at downstream node of the *i*-th pipe; ΔH_i – difference between *i*-th pipe's total headloss and pumping head (if any); N_d – number of decision variables; N_l – number of network links; N_n – number of network nodes; N_{nr} – number of network nodes where probabilities (i.e. reliabilities) are evaluated.

In the model presented here it is assumed that nodal demands are the only source of modelling uncertainty, i.e. it is assumed that all other WDS simulation model inputs are deterministic variables and that the WDS hydraulic model is accurately estimating the system's response (no modelling error). In addition to this, uncertain nodal demands are assumed to be independent random variables following some pre-specified PDF. Probabilities P_i in (2) are estimated from N_s random samples by calculating the relative frequency of samples where $H_i \ge H_{i,min}$.

Note that in the model presented here *Cost* is a deterministic variable. However, in general case, *Cost* can be a stochastic variable as well (e.g. due to uncertain pipe cost model parameters). If *Cost* is a stochastic variable, then some statistic of it (e.g. expected value) should be minimised.

Implicit constraints (3)–(4) are automatically satisfied by linking GA to the deterministic WDS solver, i.e. by using WDS solver to calculate nodal heads for each demand sample. Explicit constraints (5) can also be automatically satisfied by using the appropriate GA coding. Therefore, the optimisation problem (1)–(5) can be transformed into the following equivalent problem (*pr* is the penalty rate):

$$\text{Minimise} \quad Cost{+}Penalty \tag{6}$$

$$Penalty = pr \cdot \sum_{i=1}^{N_n} \max(0, P_{i,\min} - P_i) \tag{7}$$

3.2 *Stochastic optimisation framework*

The robust WDS least cost design model developed and presented here fits in the general stochastic optimisation problem framework (see Figure 1).

As can be seen from the figure, a stochastic search process is fundamentally a double loop process where sampling loop is located within the optimisation loop. In the optimisation loop, an optimiser (e.g. GA) is linked to a stochastic simulator (e.g. stochastic WDS solver). The stochastic simulator is used to calculate the statistics of the deterministic simulation model outputs, which are then used to calculate the stochastic optimisation objective value. Calculation of the statistics is done using the data from the sampling loop where the deterministic simulator (e.g. steady-state WDS solver) is run for a number of times. Details of the sampling and optimisation loops for the

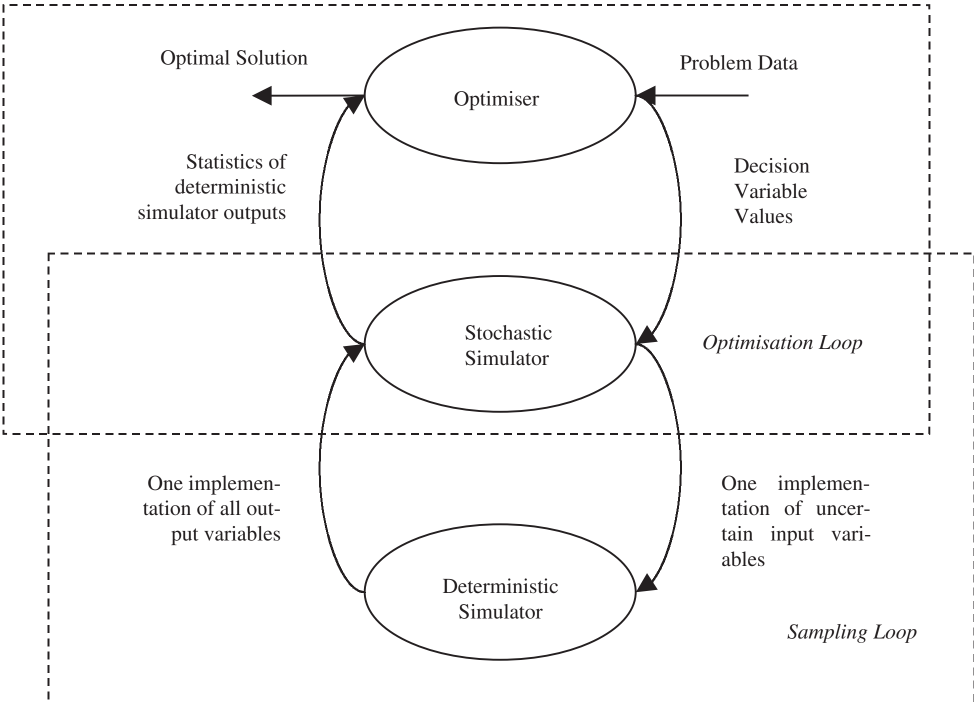

Figure 1. Stochastic optimisation framework.

robust WDS least cost design model developed and presented here are given in the next two sections.

3.3 *Uncertainty quantification*

A conventional sampling type approach to quantify WDS modelling (or any other) uncertainties is to use a (typically large) number of Monte Carlo (MC) simulations. In the MC sampling technique, N_s samples are generated at random with each sample representing one possible implementation of uncertain problem input variables (nodal demands here). Empirical PDFs and/or relevant statistics (e.g. mean, percentile, relative frequency, etc.) of WDS simulation model outputs (nodal heads here) are then calculated by running the deterministic WDS simulation model (steady-state hydraulic solver here) for N_s times. Once the uncertainty in WDS simulation model outputs is quantified, the value of the optimisation problem objective function (6) can be calculated as well. Unfortunately, the MC sampling approach usually yields reasonably accurate estimates only if a relatively large number of samples are used.

One possible alternative to the MC sampling technique is to use the Latin Hypercube (LH) sampling technique (McKay et al. 1979). In the LH sampling technique, values of stochastic input variables (nodal demands) are generated in a random yet constrained way. First, the range of each stochastic variable is divided into N_s non-overlapping intervals on the basis of equal probability. After that, a single random value is selected from each interval. This process is repeated for all stochastic variables. Once that is done, the N_s values obtained for the first stochastic variable are paired in a random manner with N_s values obtained for the second stochastic variable and so on. The drawback associated with the use of LH sampling technique when compared to the MC sampling technique is the increased computational effort required to generate the same number of samples.

However, when the number of samples is small (e.g. less than 100) this difference is almost negligible (less than 5%).

3.4 *Robust GA*

In this paper a GA capable of performing efficient search under uncertainty is presented. The GA is named robust GA (rGA). The main idea behind the rGA is to breed solutions, re-evaluate them during the search process using some small number of samples and finally, identify the best solution as the fittest that has survived over multiple generations.

Robust GA performs search in a way similar to a standard GA (Goldberg 1989). This means that the same search operators (selection, crossover, mutation) that are used in standard GA are used in rGA as well. However, several important differences exist: (1) When creating new generation population in rGA, a number of highly fit solutions from the existing generation are transferred directly (extended elitism). The rest of the population is filled-in using standard GA operators. However, unlike in a standard GA, the fitness of all directly transferred chromosomes is re-evaluated (necessary due to the presence of uncertainty); (2) Chromosome fitness in rGA is calculated as average of all past (i.e. since creation) chromosome objective function values defined by (6). Each time the objective function value is evaluated, a WDS solver is run N_s times. However, rather than using a large number of samples necessary to achieve the required level of accuracy, a significantly reduced number of samples is used; (3) In rGA, the solution (i.e. chromosome) is considered to be the best identified up to that generation if: (a) its fitness value is better than the best chromosome's fitness value identified up to that generation and (b) if it has survived for a minimum required number of generations. Once the rGA run is finished, the best solution identified during the search process is re-evaluated using a very large number of samples (100,000 MC samples in the case study here).

The rGA is effectively exploiting the fact that GA search process is of stochastic nature with a population of solutions evaluated at each generation. It is a well known fact that GA determines (near) optimal solution by combining highly fit building blocks of population chromosomes (Goldberg 1989). As the search progresses, the population is likely to have more and more chromosomes containing highly fit building blocks. As a consequence, a relatively large number of samples and, therefore, indirect evaluations of these building blocks are likely to be found in the population even if a small number of samples is used to evaluate each chromosome's fitness.

4 CASE STUDY

4.1 *Problem description*

The robust least cost design methodology presented in this paper is tested on the well-known literature problem of New York tunnels (Schaake and Lai 1969) (see Figure 2). The original objective of the problem was to identify the least cost network rehabilitation solution which satisfies minimum head requirements at all nodes for a set of fixed nodal demands. The only means of rehabilitation allowed is to duplicate existing pipes with new ones. A total of 16 solutions are possible for each pipe in the network: do not duplicate that pipe or duplicate the pipe with a new pipe having one of 15 available diameters (range 36–204 inches). Therefore, even though New York tunnels network is fairly small, the optimisation problem is quite large (total number of possible solutions is $16^{21} = 1.9 * 10^{24}$).

So far, a number of authors have solved the problem as a deterministic one. A review of these approaches can be found in Savic and Walters (1997). Unlike in these approaches, it is assumed here that nodal demands are uncertain variables following Gaussian PDF with mean equal to the deterministic demand value and standard deviation equal to 10% of the mean value. The deterministic demands and other network data used here are taken from Murphy et al. (1993). The minimum required head reliability ($P_{i,min}$ in equation (2)) is set equal to 0.90 for all network nodes. The EPANET

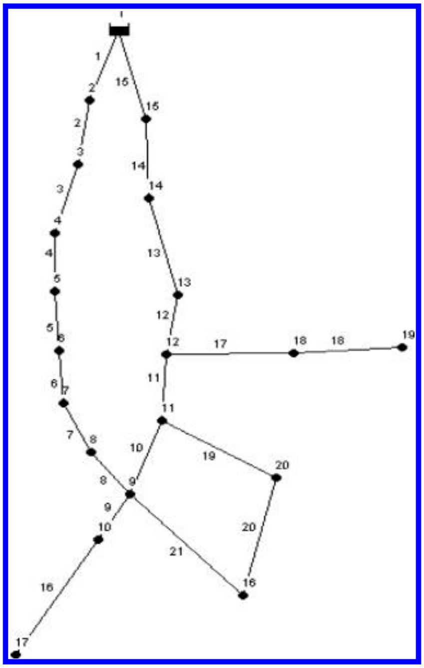

Figure 2. Layout of the pipe network for New York tunnels strengthening problem.

Table 1. Optimal deterministic solution identified by Murphy et al. (1993).

Pipe ID	Duplication diameter D_i (in)
1–14	–
15	120
16	84
17	96
18	84
19	72
20	–
21	72
Node ID	**$P_i(H_i > H_{i,min})$***
16	0.56
17	0.52
19	0.58
All other	0.99–1.00

*Based on 100,000 MC samples.

WDS hydraulic solver (Rossman 2000) is used to calculate unknown heads and flows for each demand sample.

4.2 *Uncertainty quantification results*

Results of the uncertainty quantification analysis are summarised in Figure 3. This figure shows accuracy of the stochastic part of the least cost design objective function value (i.e. the sum of the

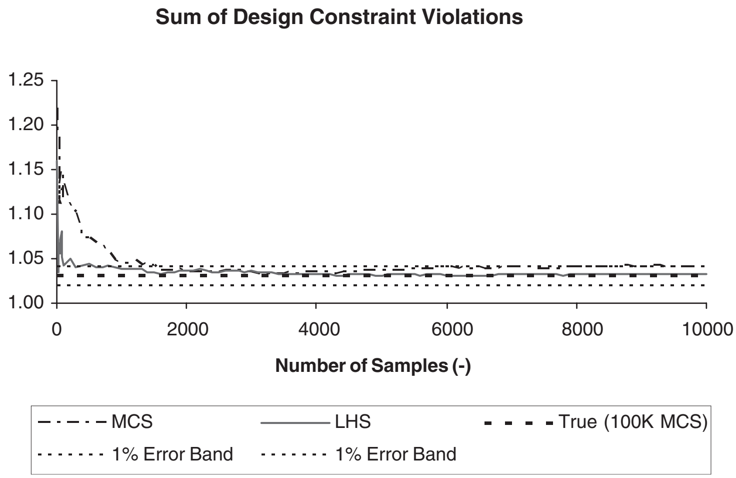

Figure 3. Uncertainty quantification for the stochastic part of the objective function.

design constraint violations in (7)) as a function of number of random demand samples N_s. Please note that both curves shown on the figure represent averages of corresponding curves generated for ten different random seeds. Two sampling techniques were analysed: MC and LH.

It can be noted from Figure 3 that, generally speaking, as the number of samples increases, accuracy of the objective function value increases too. Obviously, the increase in accuracy is faster and more stable in the LH case than in the MC case. Therefore, when compared to MC, a smaller number of LH samples are required to estimate objective function value with the same accuracy. This can be explained by the fact that samples generated by LH technique are more stratified than samples generated by the MC technique and, therefore, lead to more accurate calculation of probabilities P_i. In this case study, the minimum required number of LH samples to estimate the objective function value with 1% accuracy is approximately 1,000. Finally, note that when the number of samples is small (e.g. less than 100), 1% accuracy cannot be achieved regardless of the sampling technique used.

4.3 *Optimisation results*

A number of GA and rGA-based design optimisation runs were performed to compare performances of the two optimisation models and to identify the (near) optimal least cost design solution(s) under demand uncertainty. The following GA settings were used in all runs: binary coding, population size of 200, binary tournament selection with probability of 0.5, random-by-chromosome mutation with probability of 0.9. All GA runs were stopped after 200 generations. Note that each time GA (standard or rGA) was used to solve the optimisation problem, five runs with different random seeds (i.e. different initial populations) were performed.

Results of the GA runs performed are summarised in Table 2, from which the following can be noted: (1) The best robust solution identified by standard GA with 1,000 samples has a total design cost larger than the optimal deterministic solution identified by Murphy et al. (1993) (see Table 1). The increase in design cost from \$M 38.8 (optimal deterministic solution) to \$M 47.1 is due to increased system reliability (e.g. increase in reliability of node 17 from 0.52 to 0.90); (2) The best solution identified by standard GA with 10 samples is similar to the best solution identified by standard GA with 1,000 samples. However, two out of five solutions identified by standard GA with 10 samples are not feasible (both have node 17 reliability of $0.86 < 0.90$). Therefore, even though standard GA with small number of samples is capable of identifying near optimal solutions under uncertainty, it may lead to under-designed solutions; (3) The best solution identified by rGA with

Table 2. Optimal solutions identified by different GA models (analysis based on five GA runs with different random seeds).

	Pipe/ Node ID	GA $N_s = 1000$ Best	GA $N_s = 1000$ Average	GA $N_s = 10$ Best	GA $N_s = 10$ Average	rGA $N_s = 10$ Best	rGA $N_s = 10$ Average
Cost ($M)		47.1	50.1	46.7	49.0	46.7	49.5
Diameter D_i (in)	1–14	–		–		–	
	15	156		168		168	
	16	108		96		96	
	17	120		120		120	
	18	72		72		72	
	19	72		72		72	
	20	–		–		–	
	21	84		84		84	
Reliability P_i	16	0.97	0.94	0.97	0.95	0.97	0.97
	17	0.90	0.90	0.90	0.89(1)	0.90	0.91
	19	0.95	0.94	0.96	0.96	0.96	0.97
	Other	1.00	1.00	1.00	1.00	1.00	1.00
Relative CPU Time(2)		100	100	1	1	1	1

(1) Two out of five GA runs identified "optimal" solutions with 0.86 reliability at node 17.
(2) GA search process-related computational effort neglected.

10 samples is, again, similar to the best solution identified by standard GA with 1,000 samples and identical to the best solution identified by standard GA with 10 samples. However, unlike standard GA with small number of samples, none of the five rGA runs identified an under-designed solution. Therefore, performance of the rGA is both computationally efficient and reliable.

5 CONCLUSIONS

The following conclusions can be drawn from the case study presented here: (1) The rGA least cost design model is capable of identifying (near) optimal solutions under uncertainty while achieving significant computational savings when compared to full sampling techniques; (2) When quantifying uncertainty in the model presented here, the LH sampling technique is preferred to the MC sampling technique because it produces more accurate objective function value estimates with a smaller number of samples; (3) Neglecting uncertainty in WDS least cost design problems may lead to serious under-design of such systems.

ACKNOWLEDGEMENT

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REFERENCES

Alperovits, E. and Shamir, U. (1977). Design of Optimal Water Distribution Systems. *Water Resources Research* 13(6): 885–900.

Atkinson, R.M., Morley, M.S., Walters, G.A. and Savic, D. (1998). GANET: The integration of GIS, Network Analysis and Genetic Algorithm Optimization Software for Water Network Analysis, Hydroinformatics 98. V. Babovic and L.C. Larsen, eds, Rotterdam, *Balkema*, 347–362.

Dandy, G.C., Simpson, A.R. and Murphy, L.J. (1993). A Review of Pipe Network Optimisation Techniques, *Proc. WATERCOMP, Melbourne, Australia*, 373–383.

Goldberg, D.E. (1989). *Genetic Algorithms in Search, Optimisation and Machine Learning*. Addison-Wesley.

Halhal, D., Walters, G.A., Ouazar, D. and Savic, D.A. (1997). Water Network Rehabilitation with Structured Messy Genetic Algorithm. *Journal of Water Resources Planning and Management, ASCE*, 123(3): 137–146.

Lansey, K.E., Duan, N., Mays, L.W. and Tung, Y.K. (1989). Water Distribution System Design Under Uncertainties. *Journal of Water Resources Planning and Management, ASCE*, 115(5): 630–645.

Lasdon, L.S. and Waren, A.D. (1984). GRG2 User's Guide, Department of General Business Administration, University of Texas and Austin.

McKay, M.D., Conover, W.J. and Beckman, R.J. (1979). A Comparison of Three Methods for Selecting Values of Input Variables in the Analysis of Output from a Computer Code. *Technometrics*, 211, 239–245.

Murphy, L., Simpson, A. and Dandy, G. (1993). Pipe Network Optimization Using an Improved Genetic Algorithm. *Report No. R109*, Department of Civil and Environmental Engineering, University of Adelaide.

Rossman, L.A. (2000). Epanet2 Users Manual, US EPA.

Savic, D. and Walters, G. (1997). Genetic Algorithms for the Least-Cost Design of Water Distribution Networks. *Journal of Water Resources Planning and Management, ASCE*, 123(2): 67–77.

Schaake, J. and Lai, D. (1969). Linear Programming and Dynamic Programming Aplications to Water Distribution Network Design. *Report No. 116*, Dept. of Civil Engineering, MIT.

Simpson, A.R., Dandy, G.C. and Murphy, L.J. (1994). Genetic Algorithms Compared to Other Techniques for Pipe Optimisation. *Journal of Water Resources Planning and Management Division*, *ASCE*, 120(4): 1589–1603.

Walters, G.A., Savic, D.A., Thurley, R.W.F., Halhal, D., Kapelan, Z. and Atkinson, R. (1997). Optimal Design of Water Systems Using Genetic Algorithms: Some Recent Developments, *Proc. International Conference on the Computer Control for the Water Industry,* London, UK, 337–344.

Xu, C. and Goulter, C. (1999). Reliability-based Optimal Design of Water Distribution Networks. *Journal of Water Resources Planning and Management*, 125(6): 352–362.

Pipe sizing using a hybrid optimisation algorithm

J. Kahler & B. Ulanicki
Water Software Systems, DeMontfort University, Leicester, England

K. Pein
Technische Universitaet Ilmenau, Ilmenau, Germany

ABSTRACT: Pipe sizing is the most common task in the design of water networks carried out for enforcing water distribution systems, e.g. by adding parallel pipes, adding new connections or replacing old pipes. A typical objective is to minimise capital cost whilst satisfying hydraulic performance of the network. Pipes are manufactured in discrete diameters, where available sizes depend on the pipe material. Approaches to the pipe-sizing problem can be broadly grouped into the three categories. The trial and error method using a hydraulic simulator, the optimal design using nonlinear programming and the optimal design using discrete search methods such as genetic algorithms. The trial and error method is time consuming and requires a significant experience from the user. The nonlinear programming approach may lead to finding a local rather than the global solution. The discrete search methods can potentially find the global optimum but they require long calculation time. The paper proposes a hybrid approach by combining the nonlinear programming and the evolutionary approaches. The method is efficient in finding the global optimum in a comparatively short time and can facilitate significantly the interactive design procedure.

1 INTRODUCTION

The first approaches to the problem of optimal pipe sizing have been made back in the early 70's. In that time several researchers were trying to solve the problem using linear and nonlinear programming solvers. There are two drawbacks of these methods, they may lead to a local rather than a global optimum and they produce a continuous solution. To obtain a feasible discrete solution the continuous pipe diameters had to be rounded up to the nearest commercially available pipe diameter.

With the rise of the genetic algorithms in the late 1980's to early 1990's more researches were switching to the idea of using a genetic algorithm to solve the pipe sizing problems. Genetic algorithms can directly generate a feasible discrete solution that is likely to be close to a global optimum. The major drawback of this method is a high number of function evaluations required to reach a solution and a highly skilled user. Even with the modern high-speed computers, gaining a feasible solution can take long time depending on the size of the network.

The aim of this paper is to propose a new hybrid method combining the advantages of gradient-based nonlinear programming with advantages of evolutionary algorithms.

For problems with many local minima, gradient-based algorithms give good results when combined with a multi-start method. This means that a nonlinear solver is run many times starting from different initial points. The generation of these initial starting points as well as the modification to them for a new optimisation run can be performed by an evolutionary algorithm.

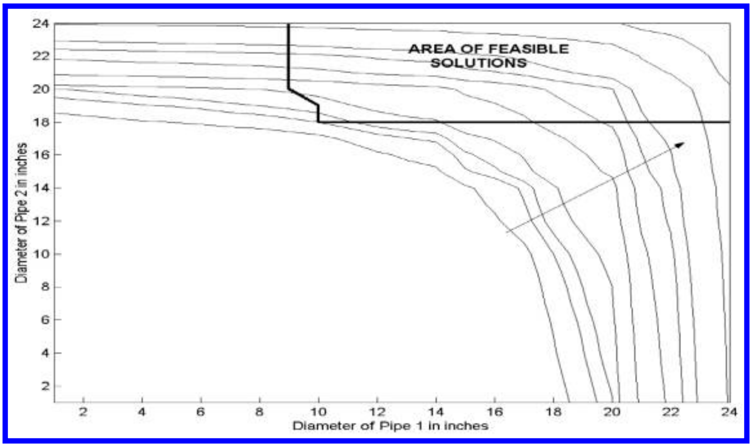

Figure 1. Smooth cost function with border of feasibility.

2 THE HYBRID ALGORITHM

The aim of each pipe sizing problem is to minimise the actual cost, while maintaining operational constraints like pressure limits at the selected nodes. The cost f increases with increasing pipe diameters D. Typically, the cost function is a smooth and monotonic and relationship (1) is true for all segments of same length.

$$D_1 > D_2 \Rightarrow f(D_1) > f(D_2) \tag{1}$$

The cost function and the feasible set shown in Figure 1 are taken from the two loop network example considered later in this paper where all but two pipe diameters were fixed. The objective function is smooth and convex, however the feasible set is non-convex and has quite complicated shape. This is typical for the pipe sizing problem, local minima exist due to complicated shape of the feasible set. Moreover, optimal solutions are always found on boundaries of the feasible set. Each existing local minimum has an "area of attraction": When the initial point for a nonlinear solver is within this "area of attraction" the algorithm should converge to this local minimum.

This observation is key for formulating the hybrid method. Each area of attraction can be represented by at least one point. If the initial starting points for a nonlinear solver are manipulated by another search algorithm (e.g. evolutionary algorithm) the search space for this algorithm is reduced to a small number of areas of attraction. For example in Figure 1, there is only one area of attraction. It can also be observed that the optimal solution is found at the corner of the feasible area which is touching the iso-cost line with the cost of 420,000 units.

The hybrid algorithm proposed here is based on a multi-start method where the initial conditions are manipulated using a differential evolution algorithm (DE) developed by Price and Storm (1998). The aim of the multi-start approach is to produce at least one initial starting point within the area of attraction of the global optimum. Subsequently, a discrete feasible solution is found in the neighbourhood of the continuous global solution by using a local branch and bound algorithm. The bounds on the pipe diameters are set to the next available diameter above and below of the continuous solution as seen in equation (2).

$$D_{discrete,below} \geq D_{continuous} \geq D_{discrete,above} \tag{2}$$

The approach can also be represented as a flowchart as shown in Figure 2.

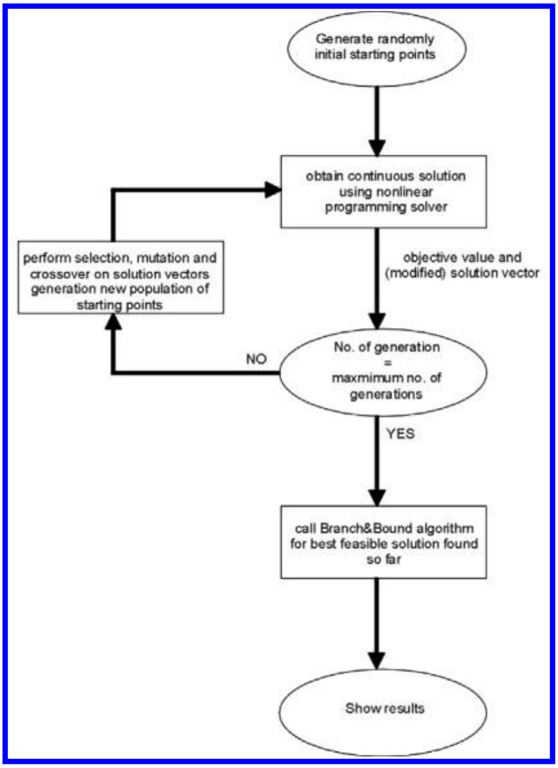

Figure 2. Flow chart of the hybrid algorithm.

Steps of the hybrid algorithm:

1. Generate a population of initial starting points
2. Use nonlinear programming algorithm to evaluate each initial starting point and obtain a solution
3. Check no of generations
4. a) NO;
   4.1 Pass the objective (cost) value as well as the solution to the DE algorithm and generate new population of starting points
   4.2 Go back to step 2
   b) YES;
   4.1 Pass best continuous solution to the Branch&Bound algorithm to obtain feasible discrete solution
5. Exit algorithm and show results

3 CALCULATION RESULTS

The search for the best discrete solution in the neighbourhood of the continuous solution has rather academic than practical value. In practice all diameters would be rounded up to the nearest available diameter to have a safety margin. The discrete solution has been evaluated for comparison with the solutions known from the literature in this area.

To illustrate the performance of the hybrid algorithm three different case studies, well known from literature, have been solved. The comparison focuses mainly on computational efficiency, some of the achieved results are similar to those already known but new solutions have also been found.

The pipe equation was represented by the following Hazen–Williams formula:

$$\boxed{\Delta H = 1.21216 * 10^{10} * L * \left(\frac{Q}{C}\right)^{1.85} * D^{-4.87}} \quad (3)$$

where L = length of the pipe in meter; C = C-Value of the pipe; Q = flow through the pipe in litre/second, and D = diameter of the pipe in millimetres.

As stated by Savic and Walters (1996), different authors have used different forms of this equation, this may affect feasibility of a solution and make the comparison of solutions from different authors difficult

Equation (3) can also be written as

$$\Delta H = \omega * \frac{L}{\left(C^{a} * D^{b}\right)} * Q^{a} \quad (4)$$

where a = 1/0.54; b = 2.63/0.54; L and D in meter and Q in m^3/second.

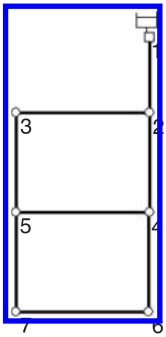

Figure 3. Two-loop network.

The advantage of equation 4 is that by changing ω, the formula can be used with different units. In this paper $\omega = 10.4516$ was used with D and L in metres and Q in m^3/s for all head loss calculations.

The data for the presented case studies can be found in the existing literature and are not repeated here.

3.1 *Two loop network*

The two-loop network, as shown in Figure 3, has been solved first by Alperovits and Shamir (1977). Other authors have followed with the latest discrete solution obtained by Savic and Walters (1996) and a split-pipe solution by Submaranian (1999). Savic and Walters used two different genetic algorithms GA1 and GA2 to solve this problem. The two-loop network is a small network with 7 nodes and 8 pipes, diameters which can be chosen from a set containing 14 values expressed in inches S = {1, 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24}.

The best solution found by Savic and Walters without violating any constraints cost 419,000 units, which can be considered as a benchmark. The solution was reached after 100 generations, each generation having 50 individuals, each individual representing one possible network design.

The cost function for this problem had a tabular form and for the purpose of the hybrid algorithm it was approximated by a sixth degree polynomial.

As shown in Table 1 the solution reached by the hybrid algorithm is the same as the solution obtained by GA2. The hybrid algorithm cannot find the solution obtained by GA1 due to the assumed continuous approximation of the cost function which is not very accurate especially for small diameters. Clearly, by inspecting Table 1 it can be observed that both solutions are very close. The continuous solution has an objective (pseudo) cost of 319,870 units and has been reached in less than 10 generations with a population size of 100 possible network designs. From this continuous solution a local search by a branch and bound algorithm has found the discrete solution. In this case, the computational effort made by the hybrid algorithm and the genetic algorithms are comparable.

3.2 *Hanoi network*

This case study considers the Hanoi Network, Vietnam. In comparison to the two-loop network, the Hanoi Network consists of 32 nodes, 34 pipes and is shown in Figure 4. The set of commercially available diameters in this case is restricted to six different diameters expressed in inches S = {12, 16, 20, 24, 30, 40}.

The smooth cost function is given by the following formula

$$Cost\,(i) = 1.1 * L\,(i) * D\,(i)^{1.5} \tag{5}$$

where Cost(i) = cost of pipe i; L(i) = length of pipe i; and D(i) = diameter of pipe i.

Table 1. Results for the two-loop network

Pipe	Eiger et al (1994)		Savic/Walters GA1 (1996)	Savic/Walters GA2 (1996)	Hybrid method. (2003)
	D (inches)	Length (meters)	D (inches)	D (inches)	D (inches)
1	18	1000	18	20	20
2	12	238.02	10	10	10
	10	761.98			
3	16	1000	16	16	16
4	1	1000	4	1	1
5	16	628.86	16	14	14
	14	371.14			
6	10	989.05	10	10	10
	8	10.95			
7	10	921.86	10	10	10
	8	78.14			
8	1	1000	1	1	1
Cost (units)	402,352		419,000	420,000	420,00

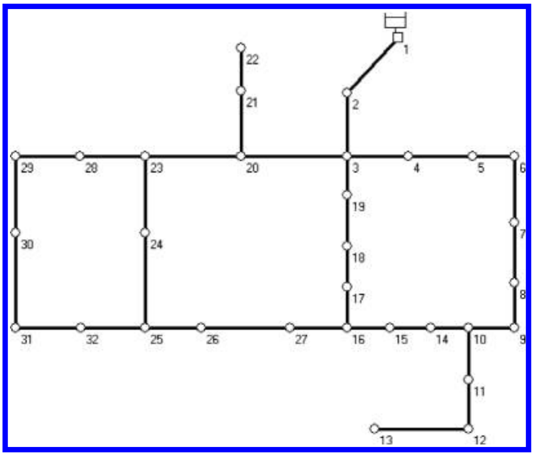

Figure 4. Hanoi network.

The network was first solved by Fujiwara and Khang (1990), they have obtained one split-pipe solution using the set of available diameters and one continuous solution. The latest discrete solution for this network has been obtained by two different GA's from Savic and Walters (1996).

The final discrete solution reported from the hybrid algorithm has the cost of \$6.177 millions, this is a new solution which has not been reported before. The price of this solution is lower than the solution generated by GA2. The solution generated by GA1 with the cost of \$6.073 millions was infeasible for the solver used by the authors and could not be reproduced. The best continuous solution of \$5.948 millions has been achieved after evaluating less than six generations with the population size of 100. The feasible discrete solution was then found by a branch and bound algorithm. The computational effort of the hybrid algorithm is significantly smaller for this problem compared to 20 runs with 10000 generation each for GAs reported in Savic and Walters (1996).

Table 2. Results for the Hanoi network.

	Fujiwara/ Khang (1990)	Savic/ Walters (1996)	Hybrid method (2003)		Fujiwara/ Khang (1990)	Savic/ Walters (1996)	Hybrid method (2003)
Pipe	D (inches)	D (inches)	D (inches)	Pipe	D (inches)	D (inches)	D (inches)
1	40	40	40	18	26.6	24	24
2	40	40	40	19	26.8	24	24
3	38.8	40	40	20	35.2	40	40
4	38.7	40	40	21	16.4	20	20
5	37.8	40	40	22	12	12	12
6	36.3	40	40	23	29.5	40	40
7	33.8	40	30	24	19.3	30	30
8	32.8	40	40	25	16.4	30	30
9	31.5	30	40	26	12	20	24
10	25.0	30	30	27	20	12	16
11	23.0	30	30	28	22	12	16
12	20.2	24	24	29	18.9	16	20
13	19	16	16	30	17.1	16	16
14	14.5	16	12	31	14.6	12	12
15	12	12	12	32	12	12	12
16	19.9	16	12	33	12	16	16
17	23.1	20	16	34	19.5	20	24
Cost ($ millions)					5.354	6.195	6.177

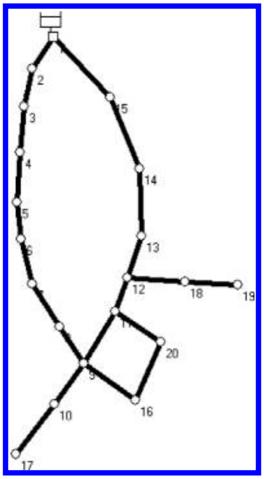

Figure 5. The New York network.

3.3 *New York tunnels network*

The New York Tunnels problem is different from the two previous studies and relies on adding parallel pipes in order to maintain pressure and demand limits on given nodes.

The network consists of 20 nodes and 21 original pipes and is shown Figure 5. The cost again is a smooth function given by (6)

$$Cost = \sum^{Pipes} 1.1 * L(Pipe) * D(Pipe)^{1.24} \tag{6}$$

where L(Pipe) = length of a pipe in feet; and D(Pipe) = diameter of pipe in inch.

Table 3. Results for the New York tunnels problem.

	Fujiwara/Khang (1990)	Murphy et al. (1993)	Savic/Walters (1996) GA1	Savic/Walters (1996) GA2	Hybrid method (2003)
Pipe	D (inches)	D (inches)	D (inches)	D (inches)	D (inches)
1	0	0	0	0	0
2	0	0	0	0	0
3	0	0	0	0	0
4	0	0	0	0	0
5	0	0	0	0	0
6	0	0	0	0	0
7	73.62	0	108	0	120
8	0	0	0	0	0
9	0	0	0	0	0
10	0	0	0	0	0
11	0	0	0	0	0
12	0	0	0	0	0
13	0	0	0	0	0
14	0	0	0	0	0
15	0	120	0	144	0
16	99.01	84	96	84	96
17	98.75	96	96	96	96
18	78.97	84	84	84	84
19	83.82	72	72	72	72
20	0	0	0	0	0
21	66.59	72	72	72	72
Cost ($ millions)	36.10	38.80	37.13	40.42	37.634

The best continuous solution found so far has been reported by Fujiwara and Khang (1990), Savic and Walters (1996) presented two discrete solutions, one with the cost of $37.13 million and another with the higher cost of $40.42 million. Finally, Murphy et al. (1993) reported a discrete solution with the cost of $38.80 million.

The differences between the solutions can be observed in Table 3. Murphy et al. suggested to add parallel pipes to pipes nos 15, 16, 17, 18, 19 and 21 whilst GA1 developed by Savic and Walters suggests to add parallel pipes to pipes nos 7, 16, 17, 18, 19 and 21. Finally, the solution generated by GA2 was to add parallel pipes to pipes nos 15, 16, 17, 18, 19 and 21. The GA algorithms were run with the population size of 100 for over 10,000 generations.

The discrete solution for the New York tunnel found by the hybrid algorithm costs $37.634 millions and is slightly more expensive than the GA1 solution. Both solutions are identical with the exception of pipe no. 7 which has a bigger diameter in the hybrid algorithm solution. However, the GA1 solution is infeasible according to the hybrid solver. This can again be attributed to small differences in the pipe equation and sensitivity of the optimal solution which lies on a boundary of the feasible set. The best continuous solution was found to cost $37.289 millions after less then 10 generations with the population size of 100. The discrete solution is again has been found by the local branch and bound algorithm.

The summary of the obtained solutions for all case studies is given in Table 4.

The major step in the pipe sizing problem is finding a global continuous solution. The search for the best discrete solution in the neighbourhood of the continuous solution has rather academic than practical value. In practice all diameters would be rounded up to the nearest available diameter to make sure that solution is inside the feasible set. The approach presented here can be applied to a variety of design problems especially with mixed continuous and integer decision variables including pump and tank sizing.

Table 4. Comparison of solutions.

Network	Best continuous (hybrid algorithm)	Best discrete (hybrid algorithm)	Best discrete (literature)	Difference
Two-loop	319.87 units	420,000 units	419,000 units	+0.23%
Hanoi	5.9485 mill $	6.177 mill $	6.073 mill $*	+1.7%
			6.195 mill $	−0.29%
New York	37.289 mill $	37.634 mill $	37.17 mill $*	+1.3%
			38.8 mill $	−3.1%

*Infeasible for nonlinear programming solver used by the authors.

4 CONCLUSIONS

The aim of the paper was to propose a hybrid method for pipe sizing which would be faster than the existing GA's approach and thus would support an interactive design. The approach uses a multi-start non-linear solver where initial starting points are manipulated by a differential evolutionary algorithm. The rationale of the approach is to search a small number of attraction areas where each attraction area contains a local minimum. After a best continuous solution is found a discrete feasible solution can be found by rounding up the continuous solution or performing a local search using, for example, branch and bound method. The solved case studies confirmed the advantages of the hybrid approach. The method in some instances found solutions known from the literature and in some instances new solutions (Hanoi network). The method requires significantly less computational effort than standard genetic algorithms especially for the more complex Hanoi and New York cases. The method can be easily extended to solve complex design tasks including pump and tank sizing. It has been confirmed that the achieved solutions are very sensitive to the head loss equation which make comparison between different solutions difficult. The head loss equation can affect feasibility of the solutions which are on a boundary of the feasible set, hence over sizing the design is a good practice as it moves the solution inside the feasible set. The described nonlinear solver is easy to use as it accepts the simulation model as its input.

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REFERENCES

Wu, Z.Y. & Simpson, A.R. 2001 Competent Genetic-Evolutionary Optimization of Water Distribution Systems in *Journal of computing in civil engineering* April, 2001.

Subramanian, S. 1999 Optimization Models and Analysis of Routing, Location, Distribution, and Design Problems on Networks, Dissertation, Virginia Polytechnic Institute, USA.

Stangner, A. & Ulanicki, B. 1998 Genetic algorithms for water network optimization, Water Software Systems Research Report 25.

Rosenthal, R.E. 1998 GAMS – A Users Guide, GAMS Development Cooperation, Washington D.C., USA.

Savic, A. & Walters, G.A. 1997 Genetic Algorithm for Least-cost Design of Water Networks Distribution Networks in *Journal of water resources planning and management* March/April, 1997.

Price, K. & Storn, R. 1997 Differential Evolution, *Dr. Dobb's Journal.*

Diurnal dynamic analysis of water distribution networks

L. Ainola, T. Koppel & N. Kändler
Tallinn Technical University, Estonia

ABSTRACT: A simple mathematical model for analysis of the diurnal dynamics of water flow in mains is applied. The description of a typical diurnal profile of domestic water consumption by means of the first members of the Fourier series is given. The influence of daily changeability of water consumption on leakage profiles is determined and diurnal leakage profiles are obtained. The efficiency of energy transmission by water flow in a main is considered and influence of diurnal time variability on it discussed.

1 INTRODUCTION

In the operation of water distribution networks we can distinguish three principal cycles with diurnal, weekly and yearly periods. The most characteristic of them is the diurnal cycle. Therefore, when designing or rehabilitating water distribution networks it is useful to be aware of the effects resulting from this cycle. Changes in daily water consumption bring about changeability of pressure in the network. Since there is a relationship between pressure and leakage in the network (Germanopoulos 1989 & 1995, Tucciarelly 1999, Obradovic 2002), the diurnal cycle has an effect on leakage.

The energy efficiency of the water distribution network also changes during the day. The performance of the water distribution network depends on the ability of the network to meet the demand for volume and pressure of flow. To evaluate the network simultaneously on the basis of both flow and pressure, the concepts of hydraulic power and energy transmission by the flow are useful (Park et al. 1998, Todini 2000, Ainola et al. 2003). In the present paper a simple mathematical model is used to analyse the diurnal dynamics of leakage and energy transmission efficiency in water mains. For typical diurnal profile of domestic water consumption a mathematical description with the first members of Fourier series is given. The influence of diurnal changeability of water demand on leakage dynamics is determined. The daily variability of energy transmission efficiency is also discussed.

2 DIURNAL WATER DEMAND

The typical data of the domestic water demand, which are given in the literature, indicate that the daily diagrams of demand have two peaks in the morning and in the evening respectively and the minimum at night (Obradovic 2002, Race et al. 2001). We can describe this daily diagram approximately by function

$$d(t) = \beta_0 + \beta_1 \cos\frac{\pi}{12}(t-3) + \beta_2 \cos\frac{\pi}{6}(t-3), \tag{1}$$

where t is time in hours. For each specific problem the coefficients here – β_0, β_1 and β_2 can be determined by the Fourier analysis or by use of the mean value – γ_0 and the maximal values γ_1 and γ_2 of the demand data (Fig. 1). In the latter case the coefficients are determined through equations

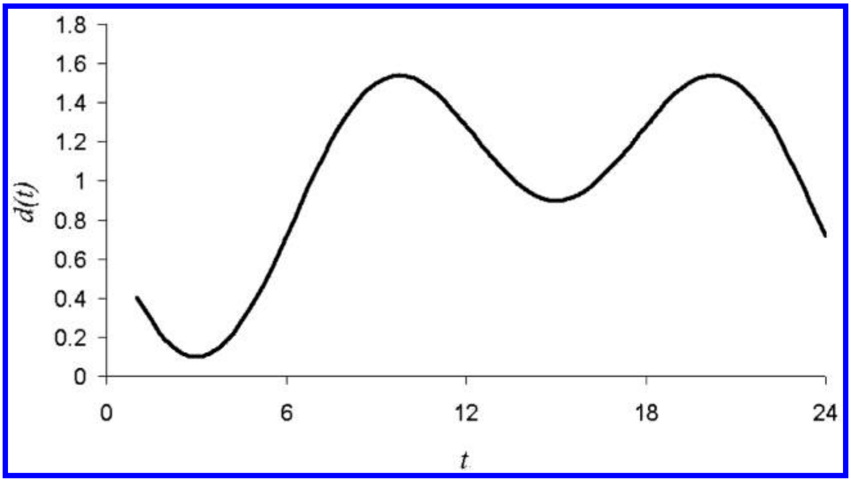

Figure 1. Daily diagram of domestic water demand.

$$\begin{aligned} &\beta_0 = \gamma_0, \\ &\beta_0 + \beta_1 + \beta_2 = \gamma_1, \\ &\beta_0 - \frac{1}{8}\frac{\beta_1^2}{\beta_2} - \beta_2 = \gamma_2. \end{aligned} \tag{2}$$

3 DIURNAL LEAKAGE DYNAMICS

In the calibration process of water network model the assumption that leakages are proportional to the controlled flow rates is often used. At the same time the leakages are dependent on the pressure in the network. But the pressure is higher at night and smaller in the daytime according to water consumption in network. Let us analyse this process in the simple mathematical model of a main. Consider the main with continuously distributed water consumption – $q(x,t)$, leakage – $\tilde{q}(x,t)$ and head – $H(x,t)$ (Fig. 2).

Some methods are proposed for evaluation of the distribution of leakage in the network. Vela et al. (1995) simulate leakage with fictitious discharge valves and determine the number of defects by statistical criteria. Germanopoulos (1995) gives a formula for determination of leakage losses from the pipe connecting two nodes as

$$S = c_1 L p^{1.18}, \tag{3}$$

where c_1 is a constant depending on the characteristics of the particular network, L is the pipe length, and p is the average pressure along the pipe. The leakage is distributed equally between the linking nodes. Tucciarelly et al. (1999) use for water losses close to node the formula

$$Q = \sum_{i=1}^{M} f_i (H - z)^e, \tag{4}$$

where coefficients f_i are determined through leak's surface, H is the total water head at node, z is the topographical elevation, e is the loss exponent, and M is the number of pipes linked to node.

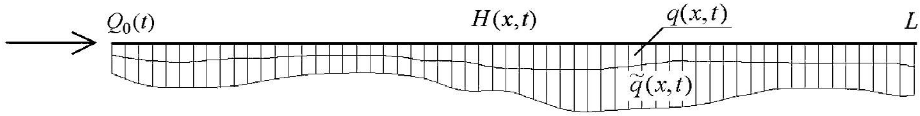

Figure 2. Main with water consumption.

For local leakage at the point x of the main in a time moment $t - q(x,t)$ the formulas 3 and 4 can be generalized and written as

$$\tilde{q}(x,t) = k\left[H(x,t)\right]^{\alpha} . \tag{5}$$

Here k is a constant which characterize the pipe leakage quality and α is the loss exponent.

Denote by $H_0(t)$ the head at the entrance of the pipe and by $h(x,t)$ the head loss due to the pipe friction on the interval $(0, x)$ of the pipe. We have

$$\boxed{h(x,t) = c\int_0^x \left\{ \int_x^L \left[q(x,t) + \tilde{q}(x,t)\right] dx \right\}^a dx ,} \tag{6}$$

where c is the resistant coefficient and a is the flow exponent.

The head at the point x is

$$H(x,t) = H_0(t) - h(x,t) . \tag{7}$$

From Equations 5 and 7 follows

$$\boxed{H(x,t) = H_0(t) - c\int_0^x \left\langle \int_x^L \left\{q(x,t) + k\left[H(x,t)\right]^{\alpha} \right\} dx \right\rangle^a dx} \tag{8}$$

Therefore

$$\frac{dH}{dx} = -c\left\{ \int_x^L \left[q(x,t) + kH^{\alpha}\right] dx \right\}^a . \tag{9}$$

and

$$\frac{d^2H}{dx^2} = ac\left[q(x,t) + kH^{\alpha}\right] \tag{10}$$

or

$$\frac{d^2H}{dx^2} - ackH^{\alpha} = acq(x,t) . \tag{11}$$

For obtaining head in pipe Equation 11 must be solved with boundary conditions

$$h(0,t) = H_0(t), \ \left.\frac{dH}{dx}\right|_{x=L} = 0 . \tag{12}$$

In general if $\alpha \neq 1$, a solution of Equations 11 and 12 is not expressible by elementary functions.
Let us consider the special case if

$$\alpha = 1, \quad q(x,t) = q_0 d(t), \quad H_0(t) = H_0 = const\,. \tag{13}$$

Then the solution of Equations 11 and 12 can be written as

$$H(x,t) = \frac{ch\, s(L-x)}{ch\, s\, L}\left[H_0 + \frac{q_0}{k}d(t)\right] - \frac{q_0}{k}d(t)\,, \tag{14}$$

where $s = \sqrt{ack}$.
From Equations 5, 13 and 14 we have

$$\tilde{q}(x,t) = \frac{ch\, s(L-x)}{ch\, sL}\left[kH_0 + q_0 d(t)\right] - q_0 d(t) \tag{15}$$

or

$$\boxed{\tilde{q}(x,t) = k\frac{ch\, s(L-x)}{ch\, s\, L}H_0 - q_0\left[1 - \frac{ch\, s(L-x)}{ch\, s\, L}\right]d(t)} \tag{16}$$

By $0 < x < L$ is valid

$$\frac{ch\, s(L-x)}{ch\, s\, L} < 1\,. \tag{17}$$

Therefore from Equation 16 follows that if the consumption $q(x,t)$ increases, then the leakage $\tilde{q}(x,t)$ decreases and vice versa. In Figure 3 the distribution of leakage in the main for two moments of time – t^* and t^{**} is given, when the consumption of water has maximal and minimal values. Here the following values of parameters are used:

$$sL = 1\,, \quad kH_0 = 10, \quad q_0 = 10, \quad \beta_0 = 1, \quad \beta_1 = -0.5, \quad \beta_2 = -0.4. \tag{18}$$

In case if leakage and other unaccounted for water flows form 10–15% of summary consumption, it is recommended to use the assumption of proportionality between the controlled and uncontrolled flows (Martinez et al. 1993, Vela et al. 1995). Let us check up this assumption in the present case.
If $k << 1$, then from Equation 16 we obtain

$$\tilde{q}(x,t) = k\left[H_0 - a\,c\,q_0\, x(L - \frac{x}{2})d(t)\right] \tag{19}$$

From Equation 19 it follows that here leakage $\tilde{q}(x,t)$ depends in addition to consumption q_0 still on head H_0 and coordinate x, i.e. the assumption of proportionality is not valid.
Denote the volume of total demand flow and total leakage flow as $Q(t)$ and $\tilde{Q}(t)$, respectively. Then

$$\boxed{Q(t) = \int_0^L q(x,t)dx, \quad \tilde{Q}(t) = \int_0^L \tilde{q}(x,t)dx} \tag{20}$$

From Equations 13, 16 and 20 we obtain

$$Q(t) = q_0 L d(t) \tag{21}$$

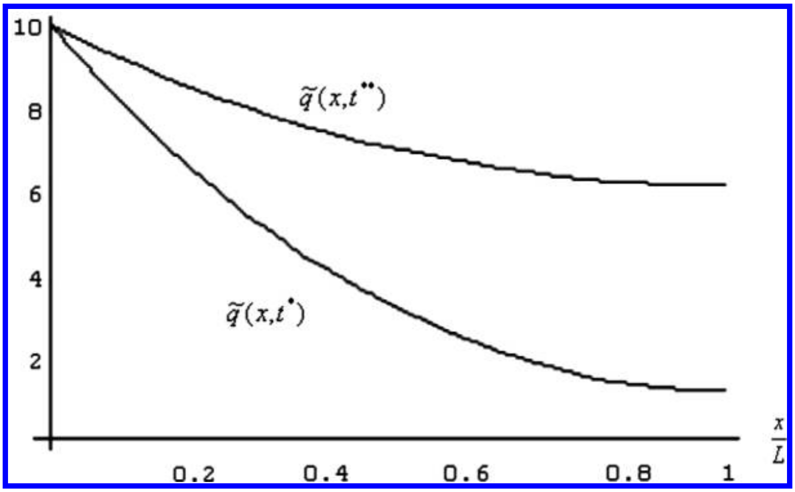

Figure 3. Leakage distribution in the main.

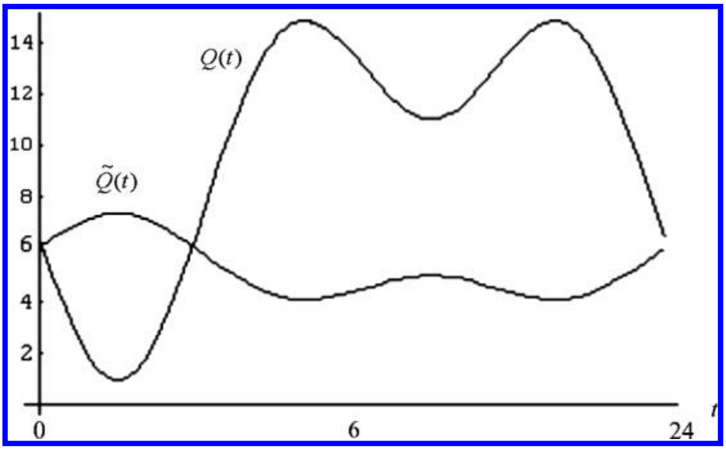

Figure 4. Diagrams of daily domestic demand and relevant leakage.

$$\tilde{Q}(t) = \frac{th\, s\, L}{s}\left[kH_0 + q_0 d(t)\right] - q_0\, L d(t)\,. \tag{22}$$

In Figure 4 diagrams of daily water demand and relevant leakage respectively to Equations 19, 21 and 22 are given.

Define the flow efficiency the main at the moment t as the volume of demanded water over the input volume, i.e.

$$g(t) = \frac{Q(t)}{Q(t) + \tilde{Q}(t)} \tag{23}$$

From Equations 21 and 22 we have

$$g(t) = \frac{s\, q_0\, L d(t)}{th\, s L\left[kH_0 + q_0 d(t)\right]}\,. \tag{24}$$

The average flow efficiency of the main is

$$G = \frac{\int_0^{24} Q(t)}{\int_0^{24} \left[Q(t) + \tilde{Q}(t) dt \right]} \tag{25}$$

or

$$G = \frac{s q_0 L \int_0^{24} d(t) dt}{th\ s\ L \int_0^{24} \left[k H_0 - q_0 d(t) \right] dt}. \tag{26}$$

For parameter values given by Equation 19 the average flow efficiency becomes G = 0.71.

4 DIURNAL VARIABILITY OF ENERGY EFFICIENCY

Let us consider energy transmission by water flow in the main. The hydraulic power in a pipe in location x at a time t is given as

$$P(x,t) = \gamma\ q(x,t)\ H(x,t), \tag{27}$$

where γ is the specific weight of water. The power which is applied to the consumers can be expressed as

$$P(t) = \gamma \int_0^L q(x,t) H(x,t) dx. \tag{28}$$

Similarly, the power which is connected with leakages is

$$\tilde{P}(t) = \gamma \int_0^L \tilde{q}(x,t) H(x,t) dx. \tag{29}$$

Let respective energy deliveries during the day be

$$E = \int_0^{24} P(t) dt \quad , \tilde{E} = \int_0^{24} \tilde{P}(t) dt. \tag{30}$$

So, we have

$$E = \gamma \int_0^{24} \int_0^L q(x,t) H(x,t) dx dt,$$

$$\tilde{E} = \gamma \int_0^{24} \int_0^L \tilde{q}(x,t) H(x,t) dx dt. \tag{31}$$

Define the average energy efficiency as

$$e = \frac{E}{E + \widetilde{E}} \quad . \tag{32}$$

From Equations 31 and 32 we have

$$\boxed{e = \frac{\int_0^{24}\int_0^L q(x,t)H(x,t)dxdt}{\int_0^{24}\int_0^L [q(x,t) + \widetilde{q}(x,t)]H(x,t)dxdt}} \,. \tag{33}$$

For special case determined through Equation 13 and parameter values given by Equation 18 the energies and energy efficiency become $E = 1583\,\gamma/k$, $\widetilde{E} = 1338\,\gamma/k$ and $e = 0.542$.

5 CONCLUSIONS

Using a simple mathematical model, the influence of daily changeability of water consumption on leakage dynamics can be demonstrated. To obtain diurnal leakage profiles the typical diurnal profile of domestic water consumption can be described by the first members of the Fourier series. This approximation enable also to determine the changes in the efficiency of energy transmission due to diurnal dynamics of water flow in the mains.

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REFERENCES

Ainola, L., Koppel, T. & Kändler, N. 2003. Energetic optimization by rehabilitation of oversized water distribution networks. In E. Cabrera & E. Cabrera Jr. (eds), *Pumps, electromechanical devices and system. Applied to urban water management. I.*: 3–9. Lisse, Abington, Exton (Pa), Tokyo: Balkema.

Germanopoulos, G. & Jowitt, P.W. 1989. Leakage reduction by excess pressure minimisation in a water supply network. *Proceedings of Institution of Civil Engineers*, UK, 2, **87**: 195–214.

Germanopoulos, G. 1995. Valve control regulation for reducing leakage. In E. Cabrera & A.F. Vela (eds), *Improving efficiency and reliability in water distribution systems*: 165–188. Dordrecht, Boston, Boston: Kluwer.

Martinez, F. & Gracia-serra, Y. 1993. Mathematical modelling of water distribution systemservice. In E. Cabrera & Martinez F. (eds), *Water supply systems. State of the art and future trends*: 141–174. Southampton, UK: Universidad International Menendezy Pelayo, Valencia.

Obradovic, D. 2000. Modelling of demand and losses in real-life water distribution systems. *Urban Water* **2** (2): 131–139.

Race, Y. & Burnell, D. 2001. Making the most of distric metering modelling patterns in water use. In B. Ulanicki, B. Coulbeck & Y. Race (eds), *Water software systems: theory and applications*: 327–336. Baldock, Hertfordshire: Research Studies Press.

Tucciarelly, T., Criminisi, A. & Tarmini, O. 1999. Leak analysis in pipeline systems by means of optimal valve regulation. *Journal of Hydraulic Engineering,* ASCE **125** (3): 277–285.

Vela, A.F., Espert V. & Fuertes, V.S. 1995. General overview of unaccounted for water in water distribution systems. In E. Cabrera & A.F. Vela (eds), *Improving efficiency and reliability in water distribution systems*: 64–95. Dordrecht, Boston, London: Kluwer.

A more realistic approach to the “extended period simulation” of water distribution networks

E. Todini
Department of Earth and Geo-Environmental Sciences, University of Bologna, Bologna, Italy

ABSTRACT: This paper deals with the problem of extending several readily available packages, originally developed for the analysis of looped water distribution systems, such as for instance EPANET2, to perform more realistic “extended period simulations” (EPS).

The EPS problem can be solved by introducing a pressure–demand relationship at the nodes, as proposed by several authors, but this tends to reduce convergence.

The purpose of this paper is to show that there is no need of defining and introducing such a pressure–demand relationship. Based upon the minimization properties of constrained convex functions and some logical reasoning, a three steps technique is here introduced as a simple alternative to allow most of the available packages to perform a more realistic EPS. The performances of the new approach will be demonstrated with a numerical example for which a clearly incorrect solution is obtained using EPANET2 with “emitters”.

1 INTRODUCTION

Several packages, originally developed for steady state analysis of looped water distribution systems, such as for instance EPANET2 (Rossman, 1994), have been recently extended to include the possibility of “extended period simulations” (EPS), namely the possibility of simulating long periods of time by means of a succession of steady states, only accounting for the change in storage of reservoirs occurring from one time step to the next.

This approach is reasonable since the actual transients in water distribution systems, tend to have high frequencies and therefore time periodicities of seconds or minutes, by far smaller than the time increments of interest, generally of hours.

Unfortunately the original steady state algorithms were thought and developed for the analysis of “design solutions”, which means that (a) the boundary conditions, which assume contemporary “maximum” or “nearly maximum” nodal demands in order to stress the network, are unrealistic for an EPS and (b) the analysis is only interested in assessing whether the head is higher than a specified lower value at all the nodes, in order to ascertain weather the design, in terms of pipe diameters, is appropriate.

In principle, the first point can be overcome by providing more realistic boundary conditions; unfortunately the second problem is more serious, since available packages are “demand driven”, which means that they tend to keep the same nodal demand even when, during the simulation in time, the head at some of the nodes falls beyond the lower limiting values.

The layman perfectly knows that, if the head at a node is insufficient, a reduction in the water flowing from the tap is expected and, in the worst case, the discharge that can be drafted will be zero, regardless to the actual demand.

Several solutions have been proposed in the literature (Ackley et al., 2001) for converting the mathematical problem from demand driven to “head driven”: this generally requires the definition of a head-discharge loss function, emulating the losses due to the generally unknown pipe network going from the water distribution system to the taps (Reddy & Elango, 1989; Salgado et al., 1993; Aoki, 1998; Udo & Ozawa, 2001). Section 2, shows how this curve can be directly introduced in the GGA as well as in EPANET2.

Unfortunately, on the one hand it is practically impossible to develop a realistic curve and, on the other hand, as pointed out by Ackley et al. (2001), all the proposed functions do not satisfy the requirements of being hydraulically meaningful and at the same time smooth and differentiable, giving rise to reduced convergence in Newton-Raphson based algorithms. To overcome these problems, a three steps technique is introduced in Section 3.

The proposed technique, which is currently under test for assessing possible instabilities, requires at most three classical analysis solutions of a network, with noticeable computational advantages, since each previous step solution is an excellent starting point for the following one.

2 EXTENSION OF THE GGA TO INCLUDE HEAD DRIVEN DEMAND

Following the formulation of the Global Gradient Algorithm (GGA) given in Todini & Pilati (1988), Todini (1999), the looped water distribution network problem can be posed as follows:

$$\boxed{\begin{bmatrix} \mathbf{A}_{11} & \vdots & \mathbf{A}_{12} \\ \cdots & \cdots & \cdots \\ \mathbf{A}_{21} & \vdots & \mathbf{0} \end{bmatrix} \begin{bmatrix} \mathbf{Q} \\ \cdots \\ \mathbf{H} \end{bmatrix} = \begin{bmatrix} -\mathbf{A}_{10}\mathbf{H}_0 \\ \cdots \\ -\mathbf{q} \end{bmatrix}} \tag{1}$$

with

$\mathbf{Q}^\mathbf{T} = [Q_1, Q_2, \ldots, Q_{\mathrm{np}}]$	the $[1, \mathrm{n_p}]$ unknown pipe discharges
$\mathbf{H}^\mathbf{T} = [H_1, H_2, \ldots, H_{\mathrm{nn}}]$	the $[1, \mathrm{n_n}]$ unknown nodal heads
$\mathbf{H}_0^\mathbf{T} = [H_{\mathrm{nn}+1}, H_{\mathrm{nn}+1}, \ldots H_{\mathrm{nt}}]$	the $[1, \mathrm{n_t} - \mathrm{n_n}]$ known nodal heads
$\mathbf{q}^\mathbf{T} = [q_1, q_2, \ldots, q_{\mathrm{nn}}]$	the $[1, \mathrm{n_n}]$ known nodal demands

where $\mathrm{n_p}$ is the number of pipes; $\mathrm{n_n}$ is the number of nodes; $\mathrm{n_t}$ is the total number of nodes in the network; $\mathrm{n_t} - \mathrm{n_n}$ is the number of nodes with known head.

In equation (1) $\mathbf{A}_{11}$ is a diagonal matrix which elements, including minor losses, are defined for $k \in 1, \mathrm{n_p}$; $i \in 1, \mathrm{n_t}$; $j \in 1, \mathrm{n_t}$ as:

$$\mathbf{A}_{11}(k,k) = r\left|Q_{ij}\right|^{n-1} + m\left|Q_{ij}\right| \tag{2}$$

for pipes and:

$$\mathbf{A}_{11}(k,k) = -\omega^2\left(h_0 - r\left(Q_{ij}/\omega\right)^n\right)/Q_{ij} \quad \text{or} \quad \mathbf{A}_{11}(k,k) = -\left(a_0\omega^2/Q_{ij} + b_0\omega + c_0 Q_{ij}\right) \tag{3}$$

(or other similar equations) for pumps. Note that in reality all the coefficients, $r, m, n, \omega, a_0, b_0, c_0$ are relevant to specific pipes or pumps; the i, j indexes have been omitted to more closely follow the representation given in EPANET2.

The actual network topology is then described by means of a topological "incidence matrix" $\overline{\mathrm{A}}_{\mathbf{12}}$ defined as follows:

$$\boxed{\overline{\mathbf{A}}_{12}(i,j) = \begin{cases} -1 & \text{if the flow of pipe } j \text{ leaves node } i \\ 0 & \text{if pipe } j \text{ is not connected to node } i \\ +1 & \text{if the flow of pipe } j \text{ enters node } i \end{cases}} \tag{4}$$

The uniqueness of the solution of equation (1) requires at least one node with known head. The overall incidence matrix, which is an $[n_p, n_t]$ matrix, can thus be partitioned into the two matrices,

$$\overline{\mathbf{A}}_{12} = \left[\mathbf{A}_{12} \vdots \mathbf{A}_{10}\right] \tag{5}$$

relating the pipes to the nodes with unknown head ($\mathbf{A}_{12}$) and to the nodes with known head ($\mathbf{A}_{10}$). For the sake of clarity the following notation is used $\mathbf{A}_{10} = \mathbf{A}^{\mathbf{T}}_{01}$ and $\mathbf{A}_{12} = \mathbf{A}^{\mathbf{T}}_{21}$.

A more general formulation, including head driven nodes and/or leakages is the following:

$$\boxed{\begin{bmatrix} \mathbf{A}_{11} & \vdots & \mathbf{A}_{12} \\ \cdots & \cdots & \cdots \\ \mathbf{A}_{21} & \vdots & \mathbf{A}_{22} \end{bmatrix} \begin{bmatrix} \mathbf{Q} \\ \cdots \\ \mathbf{H} \end{bmatrix} = \begin{bmatrix} -\mathbf{A}_{10}\mathbf{H}_0 \\ \cdots \\ -\mathbf{q}^* \end{bmatrix}} \tag{6}$$

where matrix $\mathbf{A}_{22}$ is an $[n_n, n_n]$ diagonal matrix whose generic element is either zero, if the demand of the relevant node is not "head driven", or is a non-linear function of the pressure at the relevant node (namely the difference between the actual head H_i and the terrain elevation Z_i). Following the formulation of Aoki (1998), this function can be for instance defined as:

$$\boxed{\mathbf{A}_{22}(i,i) = \begin{cases} 0 & H_i < Z_i \\ \dfrac{q_i}{H_i} \dfrac{(H_i - Z_i)^{1/2}}{(H_i^* - Z_i)^{1/2}} & Z_i \le H_i < H_i^* \\ \dfrac{q_i}{H_i} & H_i \ge H_i^* \end{cases}} \tag{7}$$

where H_i* is the required nodal head.

Alternatively, one could use similar expressions proposed by several authors: Reddy & Elango, (1989), Salgado et al. (1993), Udo & Ozawa (2001) and Ackley et al. (2001). Vector $\mathbf{q}^*$ is a $[1, n_n]$ vector whose generic element is defined as q_i, the actual demand, if the relevant node is not head driven, or is equal zero in the case of head driven node. Note that the following GGA derivation remains formally the same when solving the leakage problem, if the leakage is considered dependent upon the nodal head, as in Martinez et al. (1999). It is only a matter of appropriately define matrix $\mathbf{A}_{22}$ and vector $\mathbf{q}^*$.

The derivation of the GGA for the case of head driven demand (or leakages) closely follows the original derivation by Todini & Pilati (1988):

$$\boxed{\begin{bmatrix} \mathbf{D}_{11} & \vdots & \mathbf{A}_{12} \\ \cdots & \cdots & \cdots \\ \mathbf{A}_{21} & \vdots & \mathbf{D}_{22} \end{bmatrix} \begin{bmatrix} \mathbf{dQ} \\ \cdots \\ \mathbf{dH} \end{bmatrix} = \begin{bmatrix} \mathbf{dE} \\ \cdots \\ \mathbf{dq}^* \end{bmatrix}} \tag{8}$$

where $\mathbf{D}_{11}$ is a diagonal matrix which elements are defined for $k \in 1, n_p$; $i \in 1, n_t$; $j \in 1, n_t$ as:

$$\mathbf{D}_{11}(k,k) = nr\left|Q_{ij}\right|^{n-1} + 2m\left|Q_{ij}\right| \tag{9}$$

for pipes and:

$$\mathbf{D}_{11}(k,k) = nr\omega^{2-n}\left|Q_{ij}\right|^{n-1} \quad \text{or} \quad \mathbf{D}_{11}(k,k) = -\left(b_0\omega + 2c_0 Q_{ij}\right) \tag{10}$$

for pumps according to the chosen model.

In order to include the head driven demand, a new diagonal matrix $\mathbf{D_{22}}$ of size $[n_n, n_n]$ is introduced. The elements of matrix $\mathbf{D_{22}}$ are either zero, if the demand at the node is not assumed to be head driven, or a function of the expression chosen to represent the relationship between pressure and withdrawals. If one uses the expression given by equation (7), the corresponding expression for $\mathbf{D_{22}}$ becomes:

$$\mathbf{D}_{22}(i,i)=\begin{cases} 0 & H_i < Z_i \\ q_i \dfrac{(H_i - Z_i)^{-1/2}}{2\left(H_i^* - Z_i\right)^{1/2}} & Z_i \le H_i < H_i^* \\ 0 & H_i \ge H_i^* \end{cases} \tag{11}$$

Equation (8) can be written, assuming a local linearisation between the solution at iteration τ and that at iteration $\tau + 1$, by defining:

$$\begin{aligned} &\mathbf{dQ} = \mathbf{Q}^{\tau} - \mathbf{Q}^{\tau+1} \\ &\mathbf{dH} = \mathbf{H}^{\tau} - \mathbf{H}^{\tau+1} \\ &\mathbf{dE} = \mathbf{A}_{11}\mathbf{Q}^{\tau} + \mathbf{A}_{12}\mathbf{H}^{\tau} + \mathbf{A}_{10}\mathbf{H}_0 \\ &\mathbf{dq}^* = \mathbf{A}_{21}\mathbf{Q}^{\tau} + \mathbf{A}_{22}\mathbf{H}^{\tau} + \mathbf{q}^* \end{aligned} \tag{12}$$

Substituting for equations (12) into equation (8) and analytically solving the system of equations, the iterative formulation of the GGA algorithm can be found:

$$\begin{cases} \mathbf{H}^{\tau+1} = \mathbf{A}^{-1}\mathbf{F} \\ \mathbf{Q}^{\tau+1} = \mathbf{Q}^{\tau} - \mathbf{D}_{11}^{-1}\left(\mathbf{A}_{11}\mathbf{Q}^{\tau} + \mathbf{A}_{12}\mathbf{H}^{\tau+1} + \mathbf{A}_{10}\mathbf{H}_0\right) \end{cases} \tag{13}$$

where:

$$\mathbf{A} = \mathbf{A}_{21}\mathbf{D}_{11}^{-1}\mathbf{A}_{12} - \mathbf{D}_{22} \tag{14}$$

and

$$\mathbf{F} = \mathbf{A}_{21}\mathbf{Q}^{\tau} + \mathbf{q}^* - \mathbf{A}_{21}\mathbf{D}_{11}^{-1}\mathbf{A}_{11}\mathbf{Q}^{\tau} - \mathbf{A}_{21}\mathbf{D}_{11}^{-1}\mathbf{A}_{10}\mathbf{H}_0 \tag{15}$$

As can be noticed, similarly to the original GGA, the problem is reduced to the iterated solution of a symmetrical and sparse matrix of size $[n_n, n_n]$; the only difference with the original GGA lays in the presence of the new diagonal matrix $\mathbf{D_{22}}$ and in the different definition of vector $\mathbf{q}^*$.

A scalar formulation of this algorithm can also be provided, by defining the following quantities in accordance to the notation used by Rossman (1994) in the development of EPANET, namely

$$p_{ij} = \frac{1}{nr\left|Q_{ij}^{\tau}\right|^{n-1} + 2m\left|Q_{ij}^{\tau}\right|} \quad \text{and} \quad y_{ij} = p_{ij}\left(r\left|Q_{ij}^{\tau}\right|^{n-1} + m\left|Q_{ij}^{\tau}\right|\right) \tag{16}$$

for pipes and:

$$p_{ij} = \frac{1}{nr\omega^{2-n}\left|Q_{ij}^{\tau}\right|^{n-1}} \quad \text{and} \quad y_{ij} = -p_{ij}\,\omega^2\left(h_0 - r\left(Q_{ij}^{\tau}/\omega\right)^n\right)\Big/Q_{ij}^{\tau} \tag{17}$$

or

$$p_{ij} = -\frac{1}{b_0\omega + 2c_0 Q_{ij}^{\tau}} \qquad \text{and} \qquad y_{ij} = -\,p_{ij}\left(a_0\omega^2 / Q_{ij}^{\tau} + b_0\omega + c_0 Q_{ij}^{\tau}\right) \tag{18}$$

for pumps, according to the chosen model given by equations (3).

Note that, in order to avoid problems with the sign a slightly different definition of y_{ij} is used in this paper, namely:

$$y_{ij} = \frac{\hat{y}_{ij}}{Q_{ij}^{\tau}} \tag{19}$$

where y_{ij} is the expression defined in EPANET2 manual.

In order to solve the head driven problem, a new variable, η_i must be introduced, which is defined as:

$$\eta_i = \mathbf{D}_{22}(i,i) \tag{20}$$

η_i is computed using equation (11) or alternative ones depending upon the original choice.

With the given notations it is possible to fill the matrix **A** and vector **F** as follows:

$$\mathbf{A}(i,i) = \sum_j p_{ij} - \eta_i \qquad \forall\ i \cap j \neq \emptyset, i \in 1,\ \mathrm{n}_\mathrm{n}, j \in 1,\ \mathrm{n}_\mathrm{t} \tag{21}$$

$$\mathbf{A}(i,j) = -p_{ij} \qquad \forall\ i \cap j \neq \emptyset,\ i, j \in 1,\ \mathrm{n}_\mathrm{n} \tag{22}$$

$$\boxed{\mathbf{F}(i) = \sum_j \left(1 - y_{ij}\right) Q_{ij}^{\tau} + q_i + \sum_f p_{if} H_f^{\tau} \qquad \forall \begin{cases} i \in 1,\ \mathrm{n}_\mathrm{n} \\ i \cap j \neq \emptyset, j \in 1,\ \mathrm{n}_\mathrm{t} \\ i \cap f \neq \emptyset, f \in \mathrm{n}_\mathrm{n} + 1,\ \mathrm{n}_\mathrm{t} \end{cases}} \tag{23}$$

The solution of equation (5) is thus obtainable by repeated solutions of equations (13) until a sufficient degree of accuracy is reached.

3 THE PROPOSED THREE STEPS APPROACH

The algorithm represented by equations (13) could be used to directly solve the head driven demand problem, bearing in mind the reservations expressed in the introduction. Several commercial or freely available packages (for instance EPANET2) do not include such an algorithm, but solve the problem in a sub-optimal way, for instance by taking part of the equations (7) to the right hand side:

$$\boxed{\begin{bmatrix} \mathbf{A}_{11} & \vdots & \mathbf{A}_{12} \\ \cdots & \cdots & \cdots \\ \mathbf{A}_{21} & \vdots & \mathbf{0} \end{bmatrix} \begin{bmatrix} \mathbf{Q} \\ \cdots \\ \mathbf{H} \end{bmatrix} = \begin{bmatrix} -\mathbf{A}_{10}\mathbf{H}_0 \\ \cdots \\ -\mathbf{A}_{22}\mathbf{H} - \mathbf{q}^* \end{bmatrix}} \tag{24}$$

and by differentiating the left hand side only, which inevitably increases the number of iterations.

Nevertheless, even if this algorithm is used, particularly when it is applied to all the nodes, it may turn sour: the functions used to represent the pressure-demand relation show either discontinuities in the first derivative (such as the one due to Aoki and used as an example in this paper) or a square root type of shape (Udo & Ozawa, 2001), which can substantially increase the number of iterations required to converge when a Newton-Raphson approach is being used, as discussed by Ackley et al. (2001). According to Udo & Ozawa (2001), there is also the difficulty of choosing a curve that will actually reproduce the losses from the water distribution network to the tap(s), not knowing in detail what the real situation is.

The purpose of this paper is to show that in reality there is no such need for introducing this curve and it is possible to correctly solving the water distribution network problem when the head is insufficient, by using a three steps technique.

The proposed approach stems from three basic considerations.

The first one is that both equations (1) and (7) derive from the minimisation of a convex functional known in the literature as the "content" (Collins et al., 1978; Todini, 1999) with linear constraints at the nodes.

The second one is that if the head is really insufficient with a negative pressure, no water will be drafted from the tap(s).

Finally, the third one is that if the head is small but the pressure is non-negative, thus insufficient to allow drafting the actual demand, a reduced demand may be satisfied. The logic says that what will be drafted from the tap(s) is the possible maximum (given that the actual demand is higher) that will make the pressure drop to zero at the node.

With these simple considerations in mind, and taking into account optimisation methods available for constrained convex functions, such as the projected gradient due to Theil & van de Panne (1960), who introduced it for solving quadratic programming problems, it is possible to develop an extremely simple procedure that can allow most of the commercially available packages to deal with the problem of finding correct solutions.

In simple terms, the projected gradient approach says that, when minimising a convex functional, if the free solution violates a number of constraints, the constrained solution can be found by projecting the problem on the constraints space. This can be done by converting inequalities into equality constraints and introducing them into the function to be minimised by means of Lagrange multipliers.

Applying these ideas to the solution of the hydraulic networks, one must:

1. Start by solving the water distribution network in the conventional manner with fixed demands. If all the constraints are satisfied, namely all the $H_i \geqslant H_i^*$, the found solution is the correct one.
2. If some of the nodes show an insufficient head, $H_i < H_i^*$, solve a new network problem setting $H_i = H_i^*$, in non-satisfied nodes and compute $\hat{q}_i$, the maximum demand compatible with this constraint. Three possibilities inevitably descend from this solution. Either $\hat{q}_i \geqslant q_i$, or $0 \leqslant \hat{q}_i < q_i$, or $\hat{q}_i < 0$.
3. At this point a third step is needed which will requires replacing a number of constraints.

- At the nodes where $\hat{q}_i \geqslant q_i$, meaning that there is enough power at that node to allow for the entire demand, the original constraint is set back in terms of demand, where the demand will be again equal to q_i.
- At the nodes where $0 \leqslant \hat{q}_i < q_i$, meaning that the power is insufficient to deliver the entire demand, the head constraint $H_i = H_i^*$, is retained, since the users will inevitably try to draw as much water as possible.
- At the nodes where $\hat{q}_i < 0$, meaning that the power is insufficient to provide any water, the original constraint is set back in terms of demand but with demand q_i equal to zero.

As it will be shown using the simple numerical example proposed in this paper, this procedure converges to the right solution, while EPANET2, using the "emitters" provides clearly incorrect solutions.

4 A NUMERICAL EXAMPLE

In order to demonstrate the advantages of the proposed approach, the simplified water distribution network shown in Figure 1(a), was used. For the sake of simplicity, the same Hazen-Williams roughness coefficient C = 130 was assumed for all the 14 pipes of identical length of 1000 m, while no minor losses have been added. The following diameters have been used in the example: 500 mm (P-2); 400 mm (P-1); 300 mm (P-4, P-7); 250 mm (P-10); 200 mm (P-3, P-5, P-6, P-13); 150 mm (P-8, P-9, P-11, P-12, P-14). The nodal demands are listed in the following tables together with the ground elevation Z_i. Without loss of generality, in this example, the minimum head requirement H_i has been assumed equal to the ground elevation Z_i.

The results of the first step are given in Table 1, while the pressure field is plotted in Figure 2(a). As it can be noticed, the resulting pressure at nodes 5, 6, 7 and 8 is negative, thus showing that the demand cannot be entirely satisfied.

A second step is then formulated, by changing the fixed demand condition to fixed head condition at the non-satisfied nodes. Four reservoirs are introduced, with elevation fixed at H_i (Figure 1(b)) at nodes 5, 6, 7 and 8.

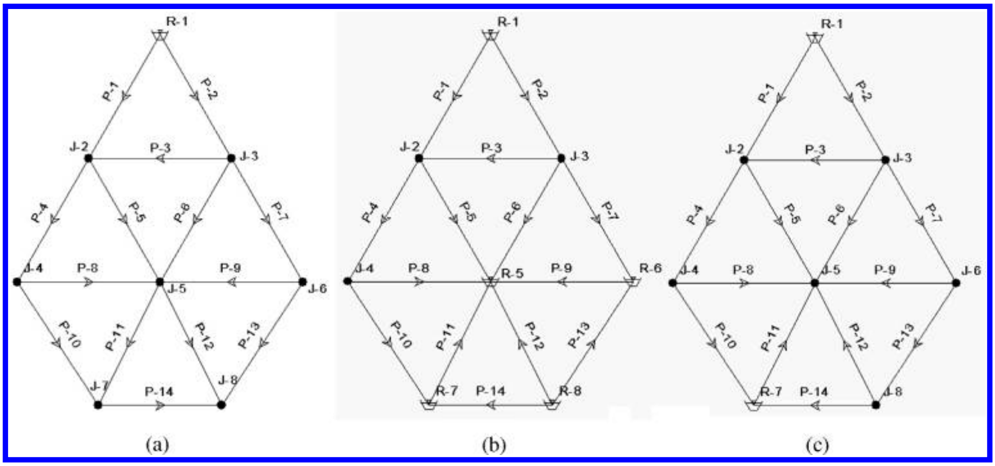

Figure 1. The simplified water distribution networks successively used for the numerical example. (a) demand driven; (b) setting to zero the negative pressures; (c) imposing either the head or the flow according to the results of (b).

Table 1. First step: demand driven solution.

Node label	Elevation (m)	Actual demand (m^3/min)	Calculated demand (m^3/min)	Calculated hydraulic head (m)	Pressure (m)
R-1	140.00	−49.00	−49.00	140.00	0.00
J-2	80.00	1.00	1.00	125.56	45.56
J-3	90.00	1.00	1.00	130.00	40.00
J-4	70.00	2.00	2.00	96.16	26.16
J-5	80.00	15.00	15.00	66.31	−13.69
J-6	90.00	15.00	15.00	73.62	−16.38
J-7	90.00	10.00	10.00	58.91	−41.09
J-8	100.00	5.00	5.00	96.16	−3.84

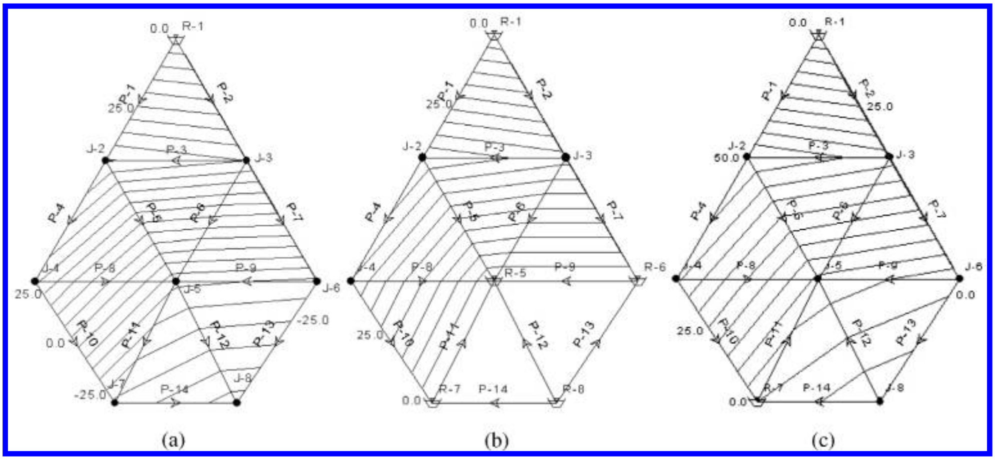

Figure 2. Pressure fields resulting from the three steps. (a) demand driven; (b) setting to zero the negative pressure; (c) imposing either the head or the flow according to the results of (b).

Table 2. Second step: fixed hydraulic head at nodes 5, 6, 7, 8.

Node label	Elevation (m)	Actual demand (m^3/min)	Calculated demand (m^3/min)	Calculated hydraulic head (m)	Pressure (m)
R-1	140.00	−49.00	−42.14	140.00	0.00
J-2	80.00	1.00	1.00	129.36	49.36
J-3	90.00	1.00	1.00	132.30	42.30
J-4	70.00	2.00	2.00	109.86	39.86
R-5	80.00	15.00	19.14	80.00	0.00
R-6	90.00	15.00	17.94	90.00	0.00
R-7	90.00	10.00	6.78	90.00	0.00
R-8	100.00	5.00	−5.72	100.00	0.00

The results of this second step are given in Table 2 while the relevant pressure field is drawn in Figure 2(b). It can be noticed that, under these constraint conditions, nodes 5 and 6 can deliver more water than the requested, node 7 can still deliver water, although less than the demanded, while node 8 shows a negative computed demand, which means that there are no possibilities of delivering water.

According to the second step results, the third step formulation sets back the actual demand at nodes 5 and 6, imposes zero demand at node 8 and retains one fixed head condition at node 7. The results of this third step are illustrated in Table 3, while the relevant pressure field is shown in Figure 2(c).

As can be noticed all the nodal demands and nodal heads are now satisfied except at node 7, where the calculated demand (6.70 m^3/min) is smaller than the actual (10.00 m^3/min) and at node 8, where the pressure is negative (−9.57 m), which implies zero flow.

An interesting comparison can also be made with the results provided by using emitters in EPANET2. A first run was tried with the emitters applied to the four unsatisfied nodes (5, 6, 7, 8). The results, given in Table 4 and Figure 3(a), show several inconsistencies. A negative flow results at node 8, while nodes 5 and 6 deliver more water than the demanded and moreover nodes 6 and 7 still deliver water, although the computed pressure is negative.

A second run was made to reduce the impact of emitters, which were only introduced at the only two really unsatisfied nodes 7 and 8. The results shown in Table 5 and Figure 3(b) are still incorrect. At node 6 the demand is entirely fulfilled, but the pressure is negative; negative pressure is

Table 3. Third step: actual demand fixed at nodes 5 and 6, hydraulic head fixed at node 7, zero demand imposed at node 8.

Node label	Elevation (m)	Actual demand (m^3/min)	Calculated demand (m^3/min)	Calculated hydraulic head (m)	Pressure (m)
R-1	140.00	−49.00	−40.70	140.00	0.00
J-2	80.00	1.00	1.00	130.07	50.07
J-3	90.00	1.00	1.00	132.76	42.76
J-4	70.00	2.00	2.00	110.96	40.96
J-5	80.00	15.00	15.00	88.54	8.54
J-6	90.00	15.00	15.00	91.45	1.45
R-7	90.00	10.00	6.70	90.00	0.00
J-8	100.00	5.00	0.00	90.43	−9.57

Table 4. EPANET2 solution with emitters set at nodes 5, 6, 7, 8.

Node label	Elevation (m)	Actual demand (m^3/min)	Calculated demand (m^3/min)	Calculated hydraulic head (m)	Pressure (m)
R-1	140.00	−49.00	−42.24	140.00	0.00
J-2	80.00	1.00	1.00	129.19	49.19
J-3	90.00	1.00	1.00	132.33	42.33
J-4	70.00	2.00	2.00	108.53	38.53
J-5	80.00	15.00	17.16	82.05	2.05
J-6	90.00	15.00	14.42	89.85	−0.15
J-7	90.00	10.00	6.96	85.92	−4.08
J-8	100.00	5.00	−0.30	87.66	−12.34

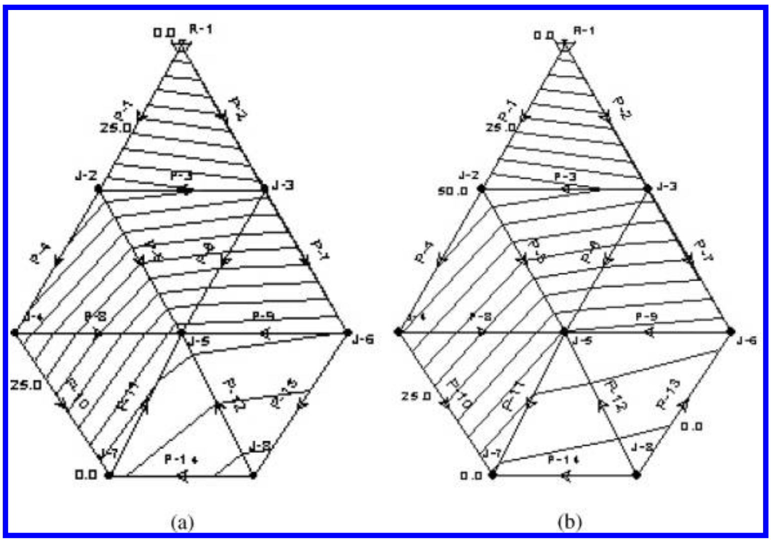

Figure 3. The EPANET2 resulting pressure filed. (a) emitters set at nodes 5, 6, 7, 8; (b) emitters set only at nodes 7, 8.

Table 5. EPANET2 solution with emitters set at nodes 7, 8.

Node label	Elevation (m)	Actual demand (m^3/min)	Calculated demand (m^3/min)	Calculated hydraulic head (m)	Pressure (m)
R-1	140.00	−49.00	−39.75	140.00	0.00
J-2	80.00	1.00	1.00	130.16	50.16
J-3	90.00	1.00	1.00	133.33	43.33
J-4	70.00	2.00	2.00	110.76	40.76
J-5	80.00	15.00	15.00	89.57	9.57
J-6	90.00	15.00	15.00	96.18	−3.82
J-7	90.00	10.00	8.48	88.98	−1.02
J-8	100.00	5.00	−2.73	96.71	−3.29

also computed at nodes 7 and 8 but at node 7 a positive flow is delivered and at node 8 the resulting flow is not zero as it should.

5 CONCLUSIONS

After showing how the "head driven" problem can be formulated in terms of the GGA, this paper has introduced an extremely simple approach to solving the cases of insufficient head that may occur in extended period simulations.

It was demonstrated, with a simple example, that there is no real need of introducing a hypothetical function that describes the relation between flow and pressure at the nodes. This is due to the fact that when a feasible demand driven solution cannot be found, the solution will inevitably occur on one of the limiting constraints: either a null pressure constraint or a null delivered flow.

In addition, the paper shows that the use of emitters for solving the low head problem, may lead to incorrect solutions and should therefore be avoided.

The importance of the proposed approach lays mainly in the possibility of extending to several existing codes the correct handling of low head conditions during EPS, by simply modifying their logic and not the actual algorithm.

Although presently all the examples treated show a very stable behaviour, with converge in three steps, several test cases are underway to exclude the possibility of eventual instabilities.

REFERENCES

Ackley, J.R.L., Tanyimboh, T.T., Tahar, B. & Templeman, A.B. 2001. Head Driven Analysis of Water Distribution Systems. In B. Ulanicki, B. Coulbeck & J.P.Rance (eds) *Water Software Systems: Theory and Applications*. Vol. 1, pp. 183–192.

Aoki, Y. 1998. Flow analysis considering pressures. 49th National Meeting on Waterworks, pp. 262–263 (in Japanese).

Collins, M., Cooper, L., Helgason, R., Kenningston, J. & LeBlanc, L. 1978. Solving the pipe network analysis problem using optimization techniques, Management Science, Vol. 24.

Martìnez, F., Conejos, P. & Vercher, J. 1999. Developing an integrated model for water distribution systems considering both distributed leakage and pressure-dependent demands. Proc. 26th ASCE Water Resources Planning and Management Division Conference. Tempe Arizona.

Reddy, L.S. & Elango, K. 1989. Analysis of water distribution networks with head dependent outlets. Civil Engineering Systems 6(3):102–110.

Rossman, L.A. 1994. *EPANET Users Manual* Drinking Water Research Division, Risk Reduction Engineering Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnaty, Ohio.

Salgado, R., Rojo, J., Zepeda, S. 1993. Extended gradient method for fully non-linear head and flow analysis in pipe networks. In B. Coulbeck (ed) *Integrated Computer Applications in Water Supply*. Vol. 1, pp. 49–60.

Theil, H. & van de Panne, C. 1960. Quadratic programming as an extension of conventional quadratic maximisation. Manage. Sci., 7:1–20.

Todini, E. & Pilati, S. 1988. A gradient algorithm for the analysis of pipe networks. In Coulbeck B. and Chun-Hou O. (eds). *Computer Applications in Water Supply, Vol.1 – System Analysis and Simulation*, John Wiley & Sons, London, pp.1–20.

Todini, E. 1999. A Unifying View on the Different Looped Pipe Network Analysis Algorithms. In Powell R. and Hindi K.S (eds) *Computing and Control for the water Industry*. Research Studies Press Ltd, pp. 63–80.

Udo, A. & Ozawa, T. 2001. Steady-State Flow Analysis of Pipe Networks Considering Reduction of Outflow in the case of Low Water Pressures. In B. Ulanicki, B. Coulbeck and J.P. Rance (eds) *Water Software Systems: Theory and Applications*. Vol. 1, pp. 173–182.

Reliability assessment of the Bekasi distribution network by the method of Cullinane

N. Trifunović
UNESCO-IHE Institute for Water Education, Delft, The Netherlands

D. Umar
Ministry of Public Works, Jakarta, Indonesia

ABSTRACT: Bekasi is a rapidly growing satellite city on the fringe of the Indonesia's capital, Jakarta. The study examines the reliability of its water distribution system according to the method of Cullinane focusing on Rawa Tembaga and Kota, two of the five branches serving the municipality. A computer model built in the EPANET programme was used for pressure analysis conducted for event of single pipe failure in the network. The results show rather low reliability of the two branches: 0.587 and 0.753, respectively. To test the sensitivity of the method, a limited rehabilitation of the network was suggested, which improves the two factors to 0.665 and 0.821. Moreover, rather minor investment in reconstruction of the Rawa Tembaga branch further raises its reliability to 0.961. Despite some deficiencies of the applied method, the results lead to conclusions matching the initial perception about hydraulic performance of the two networks. Minor modification of the source code of EPANET was recommended, which would drastically reduce the number of simulations required in this kind of analyses.

1 BACKGROUND

1.1 *Bekasi water supply system*

A reliable distribution network is not a high priority in developing countries due to the lack of funds. Still, the customer awareness regarding the service levels rises significantly in present days in Indonesia. This motivates the water authorities, despite the limitations, to start considering reliability improvement of their system as an issue that contributes significantly to the acceptable service levels.

Bekasi is a satellite city growing rapidly on the eastern fringe of capital Jakarta. The city area originates from the Bekasi regency but is, due to its rapid development, governed by the separate local government. The population of the Bekasi municipality is around 1.7 million while the Bekasi regency has additional 1.6 million inhabitants. The annual population growth is around 5% (Bekasi water authority, 2001).

Both areas are supplied by an integral system consisting of five branches with the total production capacity close to 2.2 million m^3/month. This capacity is supplied from 17 small-scale treatment installations and eight deep wells. The coverage is very low: 17% in the Bekasi municipality and just 4% in the Bekasi regency. The rest of the population is supplied from individual wells and those who can afford, buy the water for drinking and cooking from vendors. Table 1 shows the unit water consumption in each of the branches, based on the customer bills. The structure of the water demand is presented in Table 2.

In many ways, the Bekasi water supply system is a typical (peri-) urban case of a developing country, with low water supply service level that is reflected in high UFW percentage (around 35%),

Table 1. Water consumption in Bekasi water supply system (Bekasi water authority, 2001).

Branch	Number of service connections	Served population	Consumption (m^3/month)	Unit consumption lpcpd
Rawa Tembaga	13,670	68,350	266,595	130
Kota	18,180	90,900	388,389	142
Pondok Ungu	23,840	119,200	479,188	134
Rawa Lumbu	5320	26,600	85,353	107
Cikarang	7570	37,850	206,197	181
Total	68,580	342,900	1,425,722	138

Table 2. Demand structure (Bekasi water authority, 2001).

Category	Number of service connections	Consumption (m^3/month)
Domestic	66,510	1,312,112
Institutions	400	22,687
Government	80	5,058
Commercial	1,570	63,286
Industry	20	22,579
Total	68,580	1,425,722

illegal water use, lots of customer complaints resulting from poor quality of water, insufficient pressures and frequent interruptions in some areas.

1.2 *Study area*

Two branches, Rawa Tembaga and Kota, were selected for the reliability analysis in this study based on their location in the system. Within the total distribution system coverage that is for the Bekasi municipality around 20%, the two branches take large portion of the network. The area is with mild topography ranging between 9–22 m above sea level. Both branches are supplied by direct pumping and the surplus in production of some 300,000 m^3/month is transferred from Rawa Tembaga to Kota branch through the only pipe linking the two networks. The network layouts are shown in Figure 1.

Several pipe materials, such as GI, Steel, AC and PVC, are used in the network, ranging from 40 to 600 mm in diameter. The oldest pipes in the system are 21 years of age. Despite the fact that the system is relatively new, many pipes are in bad condition creating lots of leakage, predominantly in small diameters. This happens as a result of a low level of workmanship (e.g. poor jointing of the pipes), as well as from excessive external loadings, roots of trees, etc. The total length of pipe diameters and materials present in the system is given in Table 3.

The company registers calamities on distribution pipes. The frequency of the pipe bursts in period 1999–2001 is shown in Table 4. The average repair time of these pipes is 1–2 days.

Leakage accounts approximately 60% of the total UFW. Additional contributors to the high UFW level are faulty meters and illegal connections. Some 30% of the meters at service connections are older than 7 years. Those are normally replaced after significant discrepancy in the monthly records of the same customer is noticed, but in reality lots of measuring equipment in the system, including those in the pumping stations, is malfunctioning.

The main objective of this study was to analyse hydraulic performance of the two branches, in particular their vulnerability towards pipe failures, and to propose modifications that could enhance the system reliability. This was an interesting challenge given the complexity of the problems in the system and yet, relatively detailed records available in the company. Moreover, lots of additional information about the operation could have been collected from the discussions with the field staff.

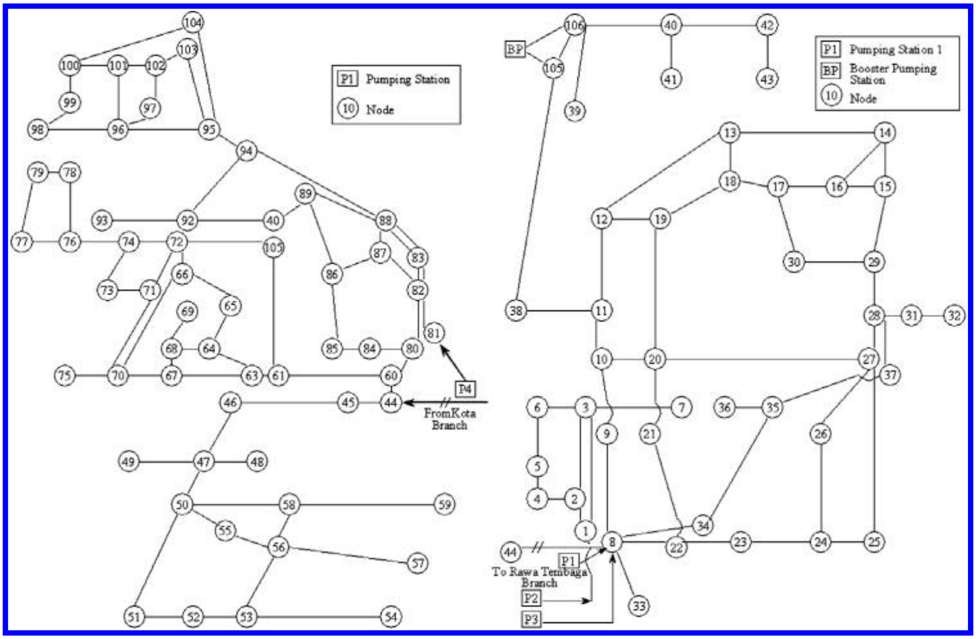

Figure 1. Schematic layout of the Rawa Tembaga and Kota branches.

Table 3. Pipe lengths (m) in Rawa Tembaga and Kota branches (Bekasi water authority, 2001).

	Rawa Tembaga branch					Kota branch				
Diameter	PVC	GI	Steel	AC	Total	PVC	GI	Steel	AC	Total
600	–	–	–	–	–	–	–	125	–	125
500	2475	–	–	–	2475	1450	–	–	–	1450
400	3175	–	50	–	3225	–	–	685	–	685
300	3895	–	140	–	4035	7135	–	–	–	7135
250	7420	–	95	325	7840	11,610	210	210	–	12,030
200	10,570	30	90	650	11,340	4085	200	80	4195	8560
150	24,750	175	120	760	25,805	14,230	170	–	1765	16,165
100	16,950	215	–	–	17,165	25,500	1285	–	3000	29,785
75	31,100	280	–	–	31,380	50,500	130	–	–	50,630
50	74,000	150	–	–	74,150	107,800	100	–	–	107,900
40	–	–	–	–	–	435	–	–	–	435
Total	174,335	850	495	1735	177,415	222,745	2095	1100	8960	234,900

Table 4. Recorded pipe bursts in period 1999–2001 (Bekasi water authority, 2001).

	Rawa Tembaga branch				Kota branch			
Diameter	1999	2000	2001	Total	1999	2000	2001	Total
500	–	1	–	1	–	–	–	–
400	1	1	2	4	–	–	–	–
300	3	2	3	8	2	3	1	6
250	2	4	7	13	2	3	16	21
200	7	8	10	25	6	2	1	9
150	9	13	27	49	8	12	4	24
Total	22	29	49	100	18	20	22	60

2 NETWORK RELIABILITY BY THE METHOD OF CULLINANE

2.1 *Principle*

The method of Cullinane (1989) was tested in this study, using the hydraulic model created in the EPANET software (US Environmental Protection Agency, Version 2). This method analyses the nodal reliability, which is judged by the length of period when the nodal pressure is above a threshold value for variety of demand scenarios and failure of different system components.

Relative simplicity of the method combined with user-friendliness of EPANET and reasonable availability of the field data, gave optimism that workable results could be reached in case of the Bekasi network. Other methods studied in the literature were either too simplified, e.g. the method of Goulter and Coals (1986), or too complex, like the method of Gupta and Bhave (1994).

Cullinane distinguishes two types of reliability in the reliability assessment of water distribution systems:

– Mechanical reliability, which is the probability that or sub-component performs its task within specified limits for a given period of time in specified environment.
– Hydraulic reliability, which is the ability of the system to provide service flows and pressures with an acceptable level of interruption in spite of irregular conditions.

2.1.1 *Mechanical reliability*

The mechanical reliability has to do with the lifetime of water distribution system components. Mathematically, the reliability R(t) of a component is defined as the probability that the component experiences no failures during an interval from time 0 to time t. In other words, the reliability is the probability that the time to the failure T exceeds t. The formula for R(t) is:

$$R(t) = \int_{t}^{\infty} f(t)dt \tag{1}$$

where *R(t)* = the reliability factor, which takes values ranging between 0 and 1; and *f(t)* = the probability density function of the time to the failure, which can be developed from the failure records.

This concept of reliability is meant for so called non-repairable components, in which the component has to be replaced after it fails. Nevertheless, many of the components in water distribution systems, such as pumps and pipes, are generally repairable and can be put back into operation. It is therefore more appropriate to use the concept of "component availability". Whereas the reliability is the probability that the component experiences no failures during the time interval 0–t, the availability of a component is the probability that the component is functional at time t, assuming that the component is as good as new at time 0. For example, after a segment of broken pipe is replaced, this pipe will function again as one of the system components. Evaluating the pipe itself, it would be considered as a non-repairable component as it has to be replaced by a new one, while in the evaluation of the pipe as a component of the water distribution system, it is repairable i.e. available component.

The availability of a component can be expressed as a percentage of the time during which the component is in operational state. The remaining is the time when the component is not functional or is scheduled for repair or maintenance. There are two basic categories of maintenance events:

– Corrective maintenance, which includes repair after a breakdown or unscheduled maintenance resulting from the equipment failure.
– Preventive maintenance, which includes regular activities to prevent breakdowns before they occur.

On annual basis, the component availability can be calculated from the following equation:

$$A = \frac{8760 - CMT - PMT}{8760} \tag{2}$$

where A = component availability; CMT = annual corrective maintenance time in hours; and PMT = annual preventive maintenance time in hours.

2.1.2 *Hydraulic reliability*

The hydraulic reliability relates directly to the basic objective of water distribution systems, which is to deliver the required water quantity at particular moment and under the desired pressure. The hydraulic reliability depends greatly on the availability of the mechanical components therefore, the mechanical availability must be explicitly considered in it.

Implicit in determination of the availability is the necessity to define a failure. The hydraulic failure happens when the system cannot satisfy the above objective. Depending on the safety requirements, the specified demand requirement can be based on the average or peak hour demand on the maximum consumption day, combined with the needed fire flows.

The nodal reliability can be defined as a percentage of the time in which the pressure at the node is above defined threshold. Cullinane states it as follows:

$$R_j = \sum_{i=1}^{m} \frac{r_{ij} t_i}{T} \tag{3}$$

where R_j = hydraulic reliability of node j; r_{ij} = hydraulic reliability of node j during time step i; t_i = duration of time step i; m = total number of the time steps; and T = length of the simulation period.

r_{ij} = 1 for the nodal pressure p_{ij} equal or above the threshold pressure $p_{\min}$, and $r_{ij} = 0$ in the remaining case of $p_{ij} < p_{\min}$. For equal time intervals, $t_i = T/m$.

Finally, the reliability of the entire system consisting of n nodes can be defined as the average of all nodal reliabilities:

$$R = \sum_{j=1}^{n} \frac{R_j}{n} \tag{4}$$

The above equations assume that the components and sub-components are fully functional, i.e. 100% available, which is rarely the case. Applying so called expected value of the nodal reliability includes impacts of the mechanical availability on the hydraulic performance. This value can be determined as follows:

$$RE_{jm} = A_m R_{jm} + U_m R_j \tag{5}$$

where RE_{jm} = expected value of the nodal reliability while considering link m; A_m = availability of link m i.e. the probability that this link is operational; U_m = unavailability of link m i.e. the probability that it is non-operational; R_{jm} = reliability of node j with link m available i.e. operational; and R_j = reliability of node j with link m unavailable i.e. non-operational.

Availability A_m is determined by Equation 2, while $U_m = 1 - A_m$. The values for R_{jm} and R_j are calculated by Equation 3, running the network simulation once with link m operational, and than again, by excluding it from the layout.

With such correction of the nodal reliability, the overall system reliability can be calculated as the mean value, by Formula 4.

2.2 *Sample calculation*

The method is illustrated on a simple system of two loops, shown in Figure 2. The data about the nodes and pipes are given in Tables 5 & 6, respectively. An arbitrary demand pattern with the morning and afternoon peaks is applied.

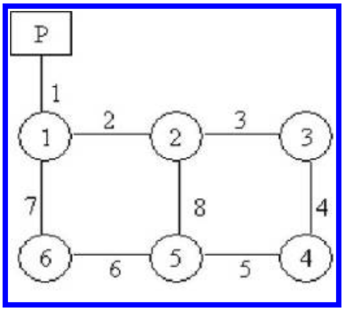

Figure 2. Sample case network.

Table 5. Nodal elevations & demands.

Node	Elevation (msl)	Demand (l/s)
1	16.8	3
2	22.2	3
3	17.5	3
4	20.2	3
5	14.6	3
6	14.3	4
P1	14.0	–
P2	15.0	–

Table 6. Pipe properties.

Pipe	Diameter (mm)	Length (m)	Roughness (mm)	Burst rate* ($km^{-1} yr^{-1}$)	Burst frequency (year/burst)
1	200	300	0.5	.71	4.69
2	200	300	0.5	.71	4.69
3	200	300	0.5	.71	4.69
4	150	400	0.5	1.04	2.40
5	100	300	0.5	1.04	3.21
6	100	300	0.5	1.04	3.21
7	100	400	0.5	1.04	2.40
8	150	400	0.5	1.04	2.40

* Assumed based on Gupta & Bhave (1994).

The pumping station at P consists of 2 units with following characteristics:

1. P1 – duty head = 25 mwc, duty flow = 40 l/s (operated constantly).
2. P2 – duty head = 40 mwc, duty flow = 40 l/s (operated during peak supply).

Operation of the system should meet the following requirements:

– The minimum pressure for every hour in all nodes is 20 mwc.
– Node 6 must have minimum 30 mwc of pressure during the maximum peak hour.

Furthermore, the irregular condition is simulated by the event of single pipe failure. Assuming that only the corrective maintenance is applied, Table 7 displays the information about the pipe availabilities based on the information from Table 6 and applying Equation 2. The average repair time of 2 days has been adopted for each burst.

Nine simulation runs were conducted by the EPANET programme, one with the complete system and eight by disconnecting each of the pipes. Based on the pressure variation in the system, the

Table 7. Pipe availability in the network.

Pipe	Burst rate (yr^{-1})	CMT (days/year)	A_m	U_m
1	0.213	0.426	0.9988	0.0012
2	0.213	0.426	0.9988	0.0012
3	0.213	0.426	0.9988	0.0012
4	0.416	0.832	0.9977	0.0023
5	0.312	0.624	0.9983	0.0017
6	0.312	0.624	0.9983	0.0017
7	0.416	0.832	0.9977	0.0023
8	0.416	0.832	0.9977	0.0023

Table 8. Sample of the reliability calculation.

Node		Pipe 1	2	3	4	5	6	7	8	RE_{jm} average
	R_{jm}	1	1	1	1	1	1	1	1	
1	R_j	0	1	1	1	1	1	1	1	
	RE_{jm}	0.9988	1	1	1	1	1	1	1	0.9999
	R_{jm}	0.5417	0.5417	0.5417	0.5417	0.5417	0.5417	0.5417	0.5417	
2	R_j	0	0	0.5417	0.5417	0.5417	0.5417	0.5417	0.5417	
	RE_{jm}	0.5410	0.5410	0.5417	0.5417	0.5417	0.5417	0.5417	0.5417	0.5415
	R_{jm}	0.8750	0.8750	0.8750	0.8750	0.8750	0.8750	0.8750	0.8750	
3	R_j	0	0	0.4167	0.8750	0.8750	0.8750	0.8333	0.8750	
	RE_{jm}	0.8740	0.8740	0.8745	0.8750	0.8750	0.8750	0.8749	0.8750	0.8747
	R_{jm}	0.5833	0.5833	**0.5833**	0.8533	0.5833	0.5833	0.5833	0.5833	
4	R_j	0	0	**0.2917**	0.5417	0.5833	0.6250	0.5833	0.5833	
	RE_{jm}	0.5827	0.5827	**0.5830**	0.5832	0.5833	0.5834	0.5833	0.5833	0.5831
	R_{jm}	1	1	1	1	1	1	1	1	
5	R_j	0	0	0.8750	0.9167	1	1	0.9167	0.8333	
	RE_{jm}	0.9988	0.9988	0.9999	0.9998	1	1	0.9998	0.9996	0.9996
	R_{jm}	1	1	1	1	1	1	1	1	
6	R_j	0	0	0.8750	0.9167	1	0.8333	0.6667	0.8333	
	RE_{jm}	0.9988	0.9988	0.9999	0.9998	1	0.9997	0.9992	0.9996	0.9995

nodal reliabilities were calculated by Equation 5, with the results shown in Table 8. As an example, the hydraulic calculation with all pipes available shows that the pressure in node 4 is above the threshold during 14 out of 24 hours, whereas the calculation with pipe 3 out of operation shows the sufficient pressure in this node during 7 hours, only. Hence, $R_{jm} = 14/24 = 0.5833$, $R_j = 7/24 = 0.2917$ and $RE_{jm} = 0.5833 \times 0.9988 + 0.2917 \times 0.0012 = 0.5830$. The overall reliability factor of the whole system is 0.8330.

3 NETWORK MODELLING

3.1 *Model building*

The fieldwork data collection in the Rawa Tembaga and Kota branches took place in period October–December 2001. The data collected included:

– Population figures and water supply coverage.
– Maps with the layout of the existing network.

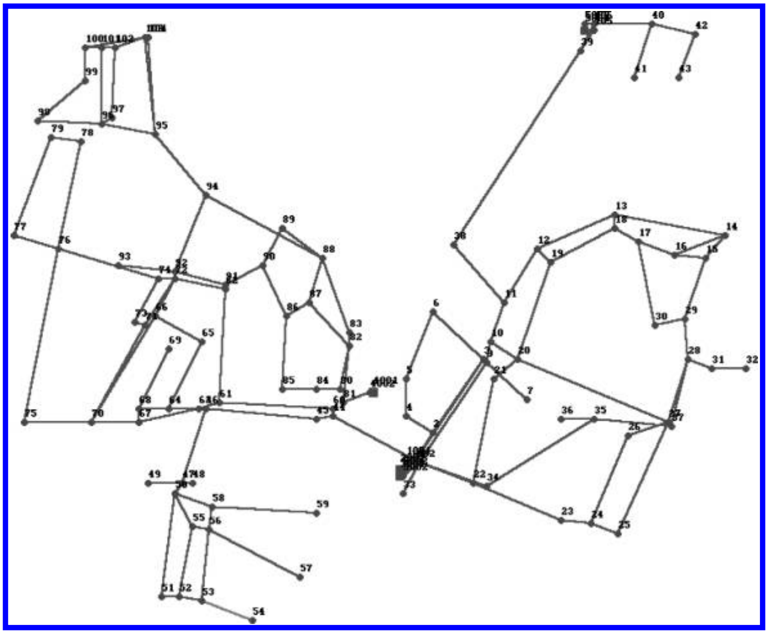

Figure 3. EPANET layout of the Rawa Tembaga and Kota branch.

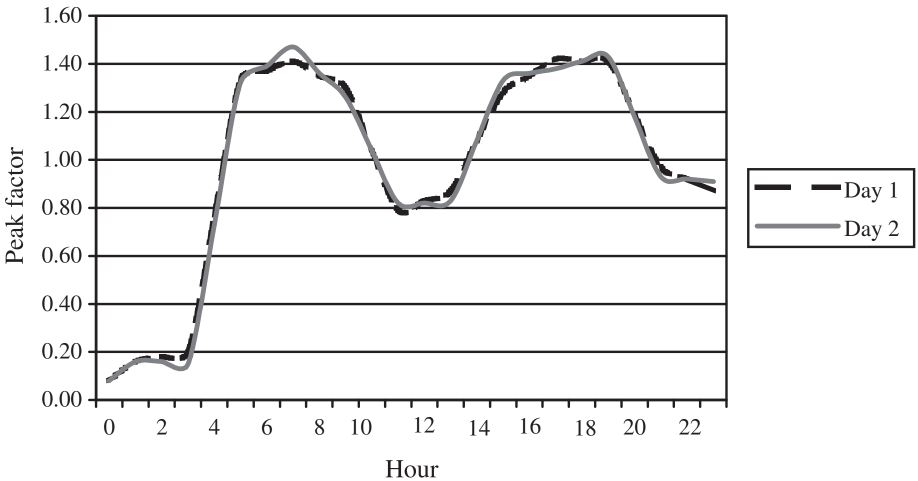

Figure 4. Sample of domestic diurnal demand pattern in the Kota branch.

– Information about the pumping units at the sources.
– Water demand information studied from the consumers' bills.
– Diurnal demand patterns measured at the supply points.
– Pressure measurements for the model calibration.
– Pipe burst records for period 1999–2001.
– Common reconstruction costs for possible investment into the reliability improvement.

An integral model of the schematised layouts of the Rawa Tembaga and Kota branches was created in the EPANET programme, consisting of approximately 140 and 100 links, respectively (Figure 3). The absolute roughness adopted for the pipes was classified in 2 groups: the pipes younger than

10 years, mainly made of PVC, took the k-values between 0.02–0.05 mm. The older pipes were considered as in worse condition and k = 1.0 mm was generally accepted.

The diurnal demand patterns were established during a two-day measurement conducted at the supplying points. An example for domestic demand in the Kota branch is shown in Figure 4. The variation of the daily demands during the week is typically in the range of 0.80–1.05 while the range between the monthly demands is between 0.80–1.15 from the average values.

The source pumping stations in the system consist of units of variable size ranging in duty heads between 8 and 40 mwc and duty flows of 12 to 240 l/s. The pumps are operated manually and the typical schedule has been applied in the simulations resulting from the information collected by the field operators.

3.2 *Simulation runs and reliability calculations*

Computer simulations were run for period of 24 hours at the time step of 1 hour. The first run was done for the entire system with all components available and then the pipes were eliminated, one at a time, for each new simulation. After each calculation, the pressure in the system was evaluated and the nodal reliabilities were calculated from spreadsheet tables, as explained in Paragraph 2.2. The threshold pressure in the system was set at 10 mwc.

The fieldwork information shown in Tables 3 & 4 was used to determine the pipe availabilities. Due to the lack of information, the study considered the pipe availability based on the corrective maintenance only.

Finally, a uniform burst rate was assumed for diameters 400–600 mm in both branches, although the failure of these pipe diameters in the Kota branch was not registered during period 1999–2001.

4 DISCUSSION OF THE RESULTS

4.1 *Hydraulic performance and system reliability*

The hydraulic calculations showed low pressures in both branches during the peak supply hours. Specifically critical area in the Rawa Tembaga branch was the area of nodes 95–104; this part of the system can be served only over night. The Kota branch performed even worse with the majority of the nodes frequently with the pressure below the threshold. Nevertheless, the part of the network close to the pumping station P2 had sufficient pressures, as well as the area of nodes 39–43, supported from the local booster station during the peak hours.

The calculation of the reliability factors showed rather low figures: 0.753 in case of Rawa Tembaga and 0.587 in case of Kota. Analyses of the head losses and velocities in Kota give strong impression that such a low factor is to large extent the result of insufficient pumping capacity. It can be said that the results fit the predictions based on the existing knowledge about the system, its layout and operational problems, fairly well.

4.2 *Proposed modifications*

Strengthening of the loped structure in the Rawa Tembaga branch was proposed by connecting nodes 79 & 98 and 72 & 92. In addition, to improve the pressure in the area of nodes 95–104 further, an additional pipe in parallel was proposed along route 94–95. Repeated calculation of the reliability for the new layout yielded a significant increase of the reliability factor, from 0.753 to 0.961. A simple economic analysis conducted based on the local prices suggests an investment in the order of US$ 43,000 for this kind of action.

Two alternatives were proposed in case of the Kota branch. The first one, similar to the alternative in the Rawa Tembaga branch, includes connection between nodes 7 & 36, laying of a few parallel pipes and rehabilitation of the connections in the vicinity of the supply points. The investment for total 7 new pipes would amount to about US$ 76,000 and the reliability factor would grow from 0.587 to 0.665, as a result.

The second alternative looked into possible overhaul of the pumping station P3. This action seems to be quite effective, as with the installation of the new, stronger pumps it is expected that the reliability factor would grow to 0.821 at relatively low investment costs of US$ 15,000. However, the operational costs would grow to US$ 48,000 per annum, in this case.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 *The system*

– The reliability of the Rawa Tembaga and Kota branches is rather low. The Rawa Tembaga branch shows somewhat better performance, with the overall reliability factor established at 0.753, whereas the same factor for the Kota branch was 0.587. This is to large extent a consequence of poor operation resulting in low pressures, even with all pipes available.
– Additional pipes in the Rawa Tembaga branch improve significantly the reliability to 0.961 at annual cost of US$ 8708. With similar action in the Kota branch, the reliability factor will increase to 0.665 at total annual cost of US$ 15,446. Additional pumping capacity would raise the reliability factor to 0.821 but the high operational costs boost the total annual costs to US$ 51,245, which is rendered unacceptable.
– The quality of input data for the computer model should further be scrutinised; more solid calibration of the model is required with more measurements in the system.
– The longer history of bursts is to be recorded for more accurate determination of pipe availabilities. In addition, the failure records of individual pumping units and production facilities should also be included in the analysis.
– It is also recommended to consider building of balancing storage in the system as a possible measure for reliability improvement and obviously, to try to reduce the leakage levels.

5.2 *The method*

– In distribution systems like Bekasi, where full and accurate information is lacking, the Cullinane method appears to be practical, transparent and easy applicable method that does not demand too abstract or highly complex input data.
– While they give fairly straight forward conclusions in terms of "higher/lower i.e. better/worse than…" when comparing several alternatives or scenarios, the reliability factor values from 0 to 1 are still a bit unclear description of the magnitude of the problem when evaluated independently. It is not clear in advance which values/ranges of this factor can be considered as high/medium/low reliability, in other words acceptable or unacceptable. Moreover, once this factor has been improved to certain degree, the question is how this translates into improvement of the service level and are the costs involved really justified.
– In some aspects, namely the determination of the nodal reliability (and consequently the overall reliability) as an average value, this method seems to be too simple. Averaged figures potentially hide some calamities that can be tolerated in no case, due to their bad consequences for the system. Those cases should be studied separately in the analysis.
– Generalising the pipe availability per diameter is not entirely accurate as it neglects the implications of pipe location in the system. The same diameter pipe has more chance to brake if located in the high pressure area (e.g. closer to the source), or is exposed to more aggressive soil conditions, excessive surface loading, etc. than if it is located in the area where all these risks are smaller; the pipe age and material may also differ. These entire factors should also be taken into consideration while determining the pipe availability.

5.3 *The programme*

– The computer simulations to be executed by the Cullinane method are simple but numerous; the analysis of the results and manual calculation of the nodal reliabilities takes lots of time. This could specifically be a problem in case of larger number of pipes.

– An additional routine in the computer programme that would run one integral simulation by excluding pipes one by one, compare the pressures in each case with the threshold value and code them accordingly, would greatly enhance the speed of the analysis. In fact, the whole procedure for determination of the overall reliability factor can potentially be programmed without too much of difficulty. It would be interesting attempt to do this in EPANET as its source code can be requested by the US EPA.

REFERENCES

Bekasi Water Authority (1999–2001). Monthly Report, Indonesia.

Bekasi Water Authority (2001). General Description of Bekasi Water Supply System, Indonesia.

Cullinane, M.J. (1989). "Determining availability and reliability of water distribution system, in L.W. Mays, ed., Reliability analysis of water distribution system, ASCE, New York, N.Y., 190–224.

Cullinane, M.J., J. Goodrich and I.C. Goulter (1989). "Water distribution system evaluation," in L.W. Mays, ed., Reliability analysis of water distribution system, ASCE, New York, N.Y., 85–101.

Goulter, I.C. (1995). "Analytical and Simulation Models for Reliability Analysis in Water Distribution System," in E. Caberra and A. Vela, eds., Improving Efficiency and Reliability in Water Distribution System, Kluwer Academic Publisher, Dordrecht, The Netherlands.

Goulter, I.C., Thomas M. Walski, Larry W. Mays, A. Burcu Altan Sakarya, Francious Bouchart, Y.K. Tung (2000). "Reliability Analysis for Design," in L.W. Mays, ed., Water Distribution Systems Handbook, The McGraw-Hill Companies, Inc, New York, N.Y., 18.1–18.48.

Goulter, I.C. and A.V. Coals (1986). "Quantitative approaches to reliability assessment in pipe networks." J. Transp. Engrg., ASCE, 112(3), 287–301.

Gupta, R. and P. Bhave (1994). "Reliability analysis of water distribution systems," ASCE Journal of Hydraulic Engineering, 120: 447–460.

Inussha Wahana Consultant (1993). Final Report : Detailed Design and Supervision of Bekasi Water Supply System, Indonesia.

Rossman, Lewis A. (2000). EPANET 2 Users Manual, U.S. Environmental Protection Agency's, National Risk Management Research Laboratory, Cincinnati, OH 45268, USA.

Head-Driven Simulation Model (HDSM) for water distribution system calibration

A.K. Soares, L.F.R. Reis & I.B. Carrijo
São Carlos School of Engineering, University of São Paulo, São Carlos, São Paulo, Brazil

ABSTRACT: In order to reproduce the behavior of water distribution networks, calibration models have been proposed. They normally do not consider the relationship between pressure and leakage, as well as the dependence of demand on pressure in a water distribution system. In this paper, we seek to extend the calibration process through the development of a hydraulic simulation model based on leakage and pressure-dependent demands with support of the hydraulic simulator EPANET 2 (Rossman 2000). A hypothetical network was used to calibrate the model in terms of the absolute roughness of pipes and the parameters of the leakage model, simultaneously. The Simplex search method was used to improve the solutions obtained by Genetic Algorithms (GAs), successfully reproducing the nodal pressures and pipe flows values taken as reference to the calibration process.

1 INTRODUCTION

The validity of a hydraulic network model depends not only on the accuracy of its physical and geometric data but also on the accuracy of certain parameters such as pipe roughness coefficients and nodal demands. To render network simulation models useful they need to be calibrated.

Network model calibration efforts should generally encompass "basic" steps (Ormsbee & Lingireddy 1997) which includes identification of the intended use of the model, determination of initial estimates of the model parameters, collection of calibration data, evaluation of the model results, performing the macro level calibration, performing the sensitivity analysis and performing the micro level calibration. The last and most difficult step is micro level calibration, which involves the adjustment of demand loadings and pipe roughness until computed and observed field pressures and flow rates are in reasonable agreement.

Consequently, calibration of water distribution system models is, undoubtedly, an important and complex issue.

Several methods are proposed for the calibration of network models (Walski 1983, Bhave 1988, Ormsbee 1989, Boulos & Wood 1990, Lansey & Basnet 1991, Savic & Walters 1995, Lingireddy & Ormsbee 2002), but most of the existing calibration procedures focus on the computational problems related to optimization algorithms and the estimation of roughness factors. Little attention has been directed to the estimation of water loss parameters, although the systems that show major leakage problems are those that need calibration the most.

Water companies are well aware that there is a deficit between the total water produced from sources over a period of time and those recorded during the same period in consumer water meters. This difference, known as unaccounted for water, is due to different factors, such as measurement errors and leakage in pipes and connections (Martínez et al. 1999). Leakage from pipes and junctions can account for a significant proportion of the water that is fed into a distribution system, with figures usually ranging from 10% to 50% (Tucciarelli et al. 1999). Therefore, the use of existing hydraulic models to simulate hydraulic performance of the system is unrealistic, because these models, based on the demand driven simulation method (DDSM), consider fixed values for nodal demands regardless of pressure variations.

In order to build a reliable water distribution network model the dependence of both leakage and demands on the pressure has to be considered simultaneously, through a head-driven simulation method – HDSM (Tabesh & Karimzadeh 2000).

Tucciarelli et al. (1999) proposed a simulation model used in the calibration procedure according to which the water losses are computed assuming that in the pipes of each zone there is a constant leakage per unit area of the pipes surface, based on the idea that the aging effect is uniform on all pipe walls. The authors assumed that the spatial distribution of the potential demand could be estimated with some degree of confidence based on the knowledge of the consumers served by each node in the network.

The same assumption was adopted here for the development of a hydraulic simulation model based on leakage and on pressure-dependent demand, with the hydraulic simulator EPANET 2 (Rossman 2000) support. A hypothetical network was used for calibration purpose in terms of the absolute roughness of pipes and the parameters of the leakage model. A hybrid GA-Simplex method is then implemented for resolution of the problem taking the advantage of the combination of local (Simplex method – Nelder & Mead 1965) and global (Genetic Algorithms) search methods. The proposed GA-Simplex method is then applied to a hypothetical network model in order to verify its ability to determine the optimal solution and to compare its performance to that of standard GA.

2 FORMULATION OF THE CALIBRATION PROBLEM

The indirect approach to solve the inverse problem of parameters identification of a water network model is usually set as the minimization of errors between true (measured for real systems or synthetic for hypothetical systems) and calculated state variables (pressures and discharges) as follows.

$$\boxed{\min_{Z} OF = \sum_{t=1}^{n^{PD}} \left[\sum_{j=1}^{n_t^P} \frac{\left(P_{t,j} - P^*_{t,j}\right)^2}{\left(\sum_{i=1}^{n_t^P} P^*_{t,i} / n_t^P \right)^2} + \sum_{j=1}^{n_t^Q} \frac{\left(Q_{t,j} - Q^*_{t,j}\right)^2}{\left(\sum_{i=1}^{n_t^Q} Q^*_{t,i} / n_t^Q \right)^2} \right]} \tag{1}$$

where n^{PD} = number of scenarios (in this paper equal to three: maximum, medium, and minimum daily consumption); P = computed pressure vector; Q = computed flow rate vector; P^* = true pressure vector; Q^* = true flow rate vector; and Z = decision variables vector corresponding to:

$$Z = (\varepsilon_1, \ldots, \varepsilon_{n_\varepsilon}, \theta_1, \ldots, \theta_{n_\theta}, \beta_1, \ldots, \beta_{n_\beta}) \tag{2}$$

where ε = pipe roughness; n_ε = number of zones with constant roughness; θ = loss factor; n_θ = number of zones with constant loss factors; β = loss exponent; and n_β = number of zones with constant loss exponent. It can be observed that to decrease the level of parameterization of the problem it is assumed that both the pipe roughness and the leakage parameters are homogeneous within specified areas of the network, called zones.

3 HYDRAULIC SIMULATION MODEL

Nodal pressures and pipe discharges are evaluated through the hydraulic model of which the simulator EPANET 2 is an auxiliary module, which considers the dependence of both leakage and demands on pressure. Assuming that the total amount of supplied water (TS) can be divided in the total demand actually supplied to the consumers (TD) and leakage (V):

$$TS = TD + V \tag{3}$$

It can be observed that the proportion corresponding to leakage or actual demand in Equation 3 is not known.

Considering that the spatial demand factors are known and that the temporal factor (FT) refers to a unique pattern, the total demand potential (TDP) can be expressed as:

$$TDP = FT.TS^* \tag{4}$$

where TS^* is the reference total demand (monthly average).

Hence, the total demand will be:

$$TD = TDP - PNA \tag{5}$$

where PNA is the part of the potential demand not met due to restriction on pressure levels in the network.

Replacing equations (4) and (5) into (3), we have:

$$TS = FT.TS^* - PNA + V \tag{6}$$

The temporal factor can then be estimated, based on the prior estimates of PNA and V:

$$FT = \frac{TS + PNA - V}{TS^*} \tag{7}$$

where $PNA = (FT.TS^* - TD)_{\mathrm{initial}}$; and V_{initial}.

So the temporal factor in the next iteration (FT_2) will be:

$$FT_2 = \frac{TS + FT_1.TS^* - TD_1 - V_1}{TS^*} \tag{8}$$

where FT_1, TD_1 and V_1 are, respectively, the values of the temporal factor, total demand and leakage in the previous iteration. According to the scheme in Figure 1, the hydraulic simulator produces this temporal factor and, consequently, the proportion corresponding to leakage or actual demand in Equation 3.

In Figure 1 we have:

Δ_1 = maximum deviation between previous pressures and real pressures in each node.

$$TS^* = \sum_{i \in N} d_i^* \tag{9}$$

$$TD = FT_1 \sum_{i \in N} d_i^* . \rho_i \tag{10}$$

$$V = \sum_{i \in N} v_i \tag{11}$$

N = number of nodes.

ρ_i is the relationship between the demand actually supplied and the potential demand in node i.

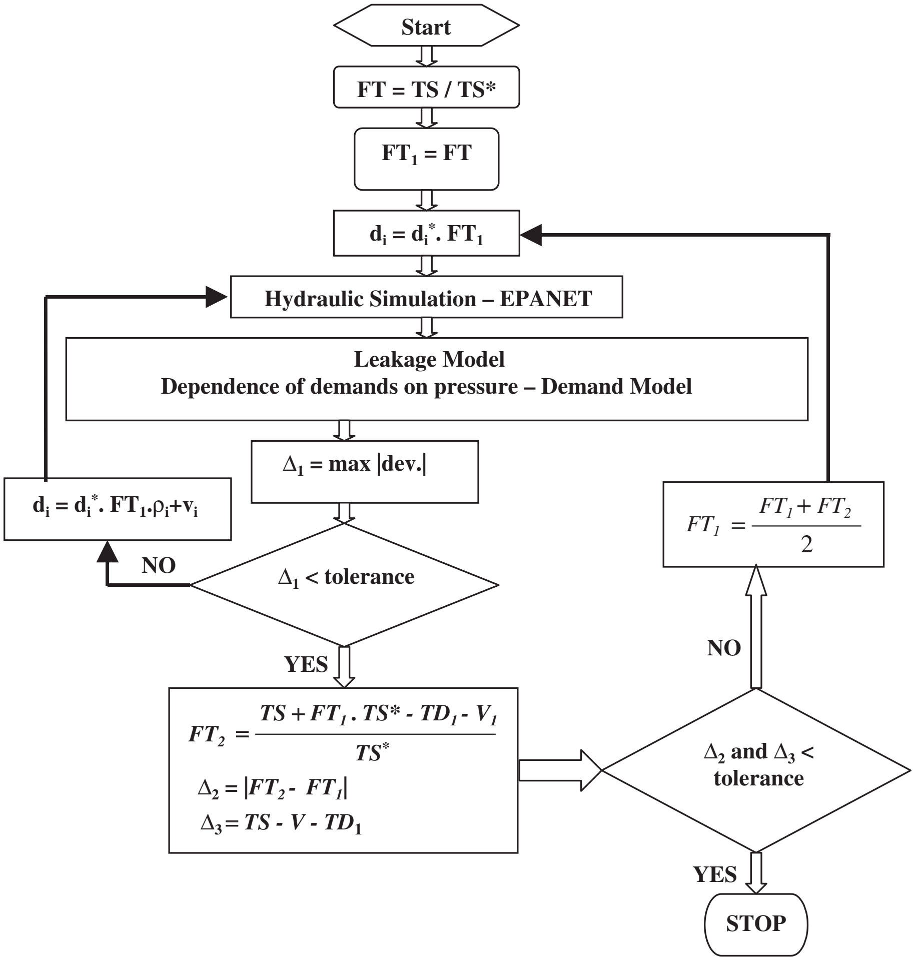

Figure 1. Flowchart of proposed hydraulic simulator.

Soares (2003) made an analysis of convergence of the head-driven models described in the literature (Fujiwara & Li 1998, Tucciarelli et al. 1999, and Tabesh & Karimzadeh 2000) concluding that the demand model proposed by Tucciarelli et al. (1999) is the most computationally efficient one. Thus, the spatial distribution factor ρ is assumed to be equal to:

$$\begin{array}{lll} \rho_i = 1 & \text{for} & P_i \geq P_i^{des} \\ \rho_i = sin^2\left(\dfrac{P_i}{2.P^{des}}.\pi\right) & \text{for} & P_i^{min} \leq P_i \leq P_i^{des} \\ \rho_i = 0 & \text{for} & P_i \leq P_i^{min} \end{array} \tag{12}$$

where P_i^{des} = minimum desired pressure to totally satisfy the demand; and P_i^{min} = absolute minimum pressure below which no flow can be discharged (in this paper it is equal to zero).

Leakage is computed based on the assumption that in each pipe there is a constant leakage per unit area of the pipe's surface (Tucciarelli et al. 1999), as expressed in the following equation:

$$v_i = \left(H_i - z_i\right)^\beta \sum_{j=1}^{M_i} \frac{\pi}{2} D_{ij} \theta_{ij} L_{ij} \tag{13}$$

where H_i = total water head at node i; z_i = topographic elevation; β = loss exponent; M_i = total number of pipes linked to node i; D_{ij} = pipe diameter; L_{ij} = pipe length; and θ_{ij} = leakage surface per unit pipe surface of the pipe linking nodes i and j.

4 HYBRID GA-SIMPLEX METHOD

In order to solve the inverse problem (1), a hybrid GA-Simplex was introduced and its performance compared to that of the standard GA. The idea was to link an effective global search method (GA) to an efficient local search method (Simplex method – Nelder & Mead 1965).

Computationally simple yet powerful search algorithms, Genetic Algorithms (GAs) are search methods that seek to reproduce mathematically the mechanisms of natural selection and population genetics, according to the biological processes of survival and adaptation (Goldberg 1989). The advantages of GAs over traditional search methods include the fact that they retain a population of well-adapted sample points, thus increasing the chance of reaching the global optimum (Reis et al. 1997). Moreover, these algorithms consider probability rules for the transition from one set of trial solutions to the next, and they have the flexibility of admitting many types of objective functions without requiring the continuity and existence of their derivatives.

GAs were first introduced into the area of water distribution network calibration by Savic & Walters (1995). Recently, Kapelan et al. (2000 and 2002) proposed two types of hybridization based on the GAs (global search method) and on Levenberg–Marquardt's method (local search method) to solve the inverse transient problem (network leak detection and roughness calibration).

In this paper the *GAlib C++* (Wall 1996) library was used to develop the GA implementation. The library has a large number of built-in features: modelling different types of GAs, admitting various coding and selection schemes, several crossover and mutation operators, optional use of elitism, etc.

Davis[1] apud Michalewicz (1994) stated:

"When I talk to the user, I explain that my plan is to hybridize the genetic algorithm technique and the current algorithm by employing the following three principles:

- Use the current encoding in the hybrid algorithm.
- Hybridize where possible. Incorporate the positive features of the current algorithm in the hybrid algorithm.
- Adapt the genetic operators. Create crossover and mutation operators for the new type of encoding. Incorporate domain-based heuristics as operators as well.

"[…] I use the term *hybrid genetic algorithm* for algorithms created by applying these three principles."

The simplest form of hybridization of GA and Simplex methods is a two-stage hybrid. The best solution found by GA, according to a certain convergence criterion, here established as the number of generations, is used as the starting point for the Simplex method (see Fig. 2). Therefore, problems associated with the GA's slow convergence (large number of objective function evaluations) and the optimization method to converge to a local optimum can be reduced.

The *downhill simplex method* was developed by Nelder & Mead (1965) and it is one of the most popular direct-search methods among those which rely only on the function values to find the location of the optimum. This method does not require first- or second-order Taylor approximations to the function for its choice of search direction and it is important when gradient techniques cannot be applied or requires great computational effort such as objective functions of inverse problem.

Subrahmanyam (1989) extended the simplex algorithm of Nelder and Mead to handle nonlinear optimization problems with constraints. To prevent the simplex from collapsing into a subspace near the constraints, a delayed reflection is introduced for those points moving into the infeasible region. Subrahmanyam's technique has been used in this study because of the constraints in the variable decision vector (minimum and maximum values).

[1] Davis, L. 1991 (Editor). *Handbook of Genetic Algorithms*, Van Nostrand Reinhold, New York.

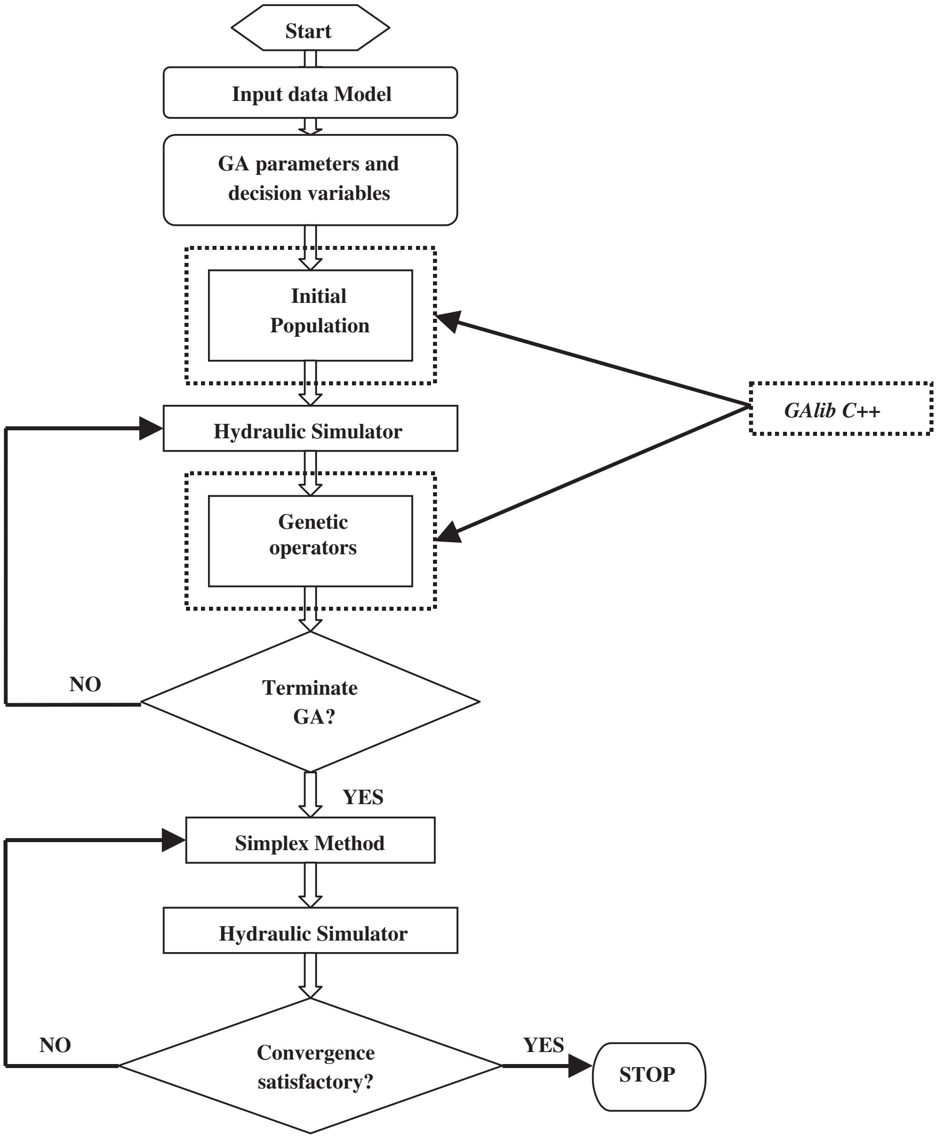

Figure 2. The hybrid GA-simplex method flowchart.

5 CASE STUDY

The proposed method was applied to the calibration of the hypothetical network model presented in Figure 3 (Tucciarelli et al. 1999).

The gravity network has been grouped into three types of sectors: one is composed by three loss factor (θ) zones, other by one pipe roughness (ε) zone and the last one by one loss exponent (β) zone, as shown in Table 1. The third loss factor zone is assumed to have a known zero value (no leakage).

The location of the nodes with nonzero demand is shown in Figure 3, each one with a spatial distribution factor equal to 0.20.

Three scenarios were built in terms of the water levels in the reservoirs and total supply demands in order to provide pressure heads and flow rates for different network behaviors, as

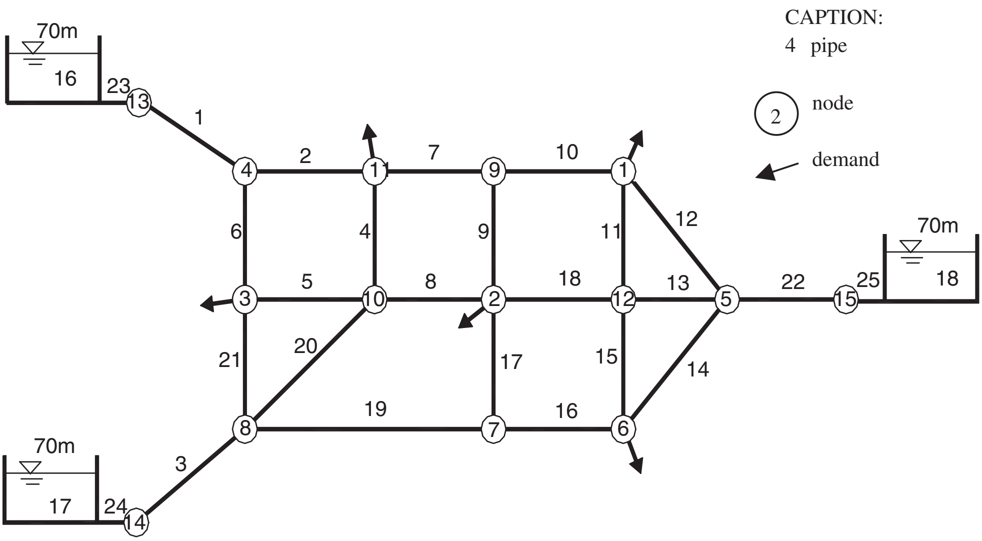

Figure 3. Case study network (Tucciarelli et al. 1999).

Table 1. Network input data (Tucciarelli et al. 1999).

Loss Exponent (β)	Reference Demand (l/s)	Roughness (mm)	Loss factor		
			θ_1	θ_2	θ_3
1.253	180	3.153	7.504E−08	1.909E−07	0

Table 2. Scenarios Performed in the Simulations.

Scenario	Tank levels (m)	Supply Demand (l/s)		
		Reservoir 16	Reservoir 17	Reservoir 18
1	65.0	49.25	54.80	83.12
2	55.0	76.37	75.73	115.14
3	45.0	103.77	97.00	146.67

reported in Table 2. In the simulations it is used minimum pressure value $P^{min} = 0$ and desired pressure value $P^{des} = 15$ m.

Synthetic pressure heads and flow rates were obtained through the application of the simulation model (Fig. 1). The set of values of flow rates and pressure heads used for calibration purpose were taken from the three pipes linking the network to the reservoirs and from five demand nodes according to Table 3.

6 RESULTS AND DISCUSSIONS

In order to compare the performances of the standard GA and the GA-Simplex optimization methods, three distinct initial population of solutions (represented by real strings) were run using an elitist

Table 3. Pressure heads in five nodes with nonzero demand.

Scenario	Pressure (m) Node 1	Node 2	Node 3	Node 6	Node 11
1	38.15	42.26	42.31	36.83	41.20
2	25.60	28.14	31.17	25.55	28.60
3	12.16	12.65	19.38	13.73	14.98

Table 4. Calibration parameter values found by GA and GA-simplex method.

Decision Variable	Search Min	Search Max	True Value	Optimum Value GA	Optimum Value GA-Simplex
ε (mm)	0.010	5.000	3.153	3.153	3.162
θ_1 (×10e−7)	0.050	3.000	0.750	0.648	0.792
θ_2 (× 10e−7)	0.050	3.000	1.909	1.971	1.993
β	0.100	1.300	1.253	1.218	1.244
CPU Time				12.6 minutes	38 seconds

(30%) generational GA, linear scaling of fitness, stochastic remainder sampling (SRS) as selection scheme, uniform arithmetic crossover with 70% probability and gene by gene Gaussian mutator with 1% probability. The number of generations used for standard GA was 500. Several tests (1, 5, 10, 30, and 50 generations) were performed to define that the best solution found by GA-Simplex only needed 10 generations.

Table 4 shows the minimum and maximum values admitted as well as the average values obtained for decision variables using both calibration procedures (GA and GA-Simplex). The average processing time spent for both procedures are also reported, showing that GA-Simplex is faster than standard GA. The GA CPU time is twenty times bigger than the hybrid and GA-Simplex presented better solutions than that found by the GA alone.

The case study demonstrates that the hybrid GA-Simplex method performs better than the standard GA in terms of both effectiveness and efficiency.

The values of pressures heads and flow rates computed are presented in Tables 5, 6, 7, and 8 for the three distinct initial population of solutions (1, 2, and 3) and for the two optimization methods (GA and GA-Simplex).

It can be seen that the pressure heads obtained for the simulations with GA alone are close to the corresponding true values for nodal pressures, as illustrated in Table 5. The largest error is 0.65 m, produced in simulation 1, node 2, and scenario 2.

Table 6 presents the computed and true flow rates for the simulations with standard GA. The errors range from 0.0% (simulation 1, pipe 1, and scenario 3) to −2.5% (simulation 2, pipe 1, and scenario 1).

Tables 7 and 8 permit the comparison between computed and true pressure heads and flow rates, respectively, when the hybrid GA-Simplex method is applied.

The largest absolute error presented between computed and true pressure heads is 0.02 m, and the largest relative error between computed and true flow rates is −0.01%. It can be concluded that computed pressure heads and flow rates by the hybrid GA-Simplex method are very close to the true values and they are better than the computed pressure heads and flow rates by the GA alone.

Table 5. Computed and True Pressure Heads (Standard GA).

Scenario	Node	Computed Pressures (m) 1	2	3	True Pressure (m)	Absolute Error (m) 1	2	3
1	1	37.88	38.07	38.18	38.15	0.27	0.08	0.03
	2	41.68	42.00	42.17	42.26	0.58	0.26	0.09
	3	42.18	42.28	42.31	42.31	0.13	0.03	0.00
	6	36.70	36.74	36.82	36.83	0.13	0.09	0.01
	11	40.96	41.18	41.28	41.20	0.24	0.02	0.08
2	1	25.27	25.54	25.83	25.60	0.33	0.06	0.23
	2	27.49	27.92	28.40	28.14	0.65	0.22	0.26
	3	31.04	31.16	31.28	31.17	0.13	0.01	0.11
	6	25.39	25.47	25.64	25.55	0.16	0.08	0.09
	11	28.31	28.59	28.87	28.60	0.29	0.01	0.27
3	1	11.89	12.17	12.64	12.16	0.27	0.01	0.48
	2	12.27	12.62	13.26	12.65	0.38	0.03	0.61
	3	19.25	19.41	19.70	19.38	0.13	0.03	0.32
	6	13.57	13.72	14.01	13.73	0.16	0.01	0.28
	11	14.71	15.01	15.53	14.98	0.27	0.03	0.55

Table 6. Computed and true flow rates (Standard GA).

Scenario	Pipe	Computed Flows (l/s) 1	2	3	True Flow (l/s)	Relative Error (%) 1	2	3
1	1	50.06	48.00	48.33	49.25	1.6	−2.5	−1.9
	3	53.86	54.89	54.69	54.80	−1.7	0.2	−0.2
	22	83.24	84.30	84.16	83.12	0.1	1.4	1.3
2	1	76.62	75.62	75.66	76.37	0.3	−1.0	−0.9
	3	74.83	75.53	75.43	75.73	−1.2	−0.3	−0.4
	22	115.79	116.09	116.15	115.14	0.6	0.8	0.9
3	1	103.73	103.38	103.08	103.77	0.0	−0.4	−0.7
	3	96.59	96.76	96.57	97.00	−0.4	−0.2	−0.4
	22	147.09	147.25	147.80	146.67	0.3	0.4	0.8

Table 7. Computed and true pressure heads (GA-Simplex).

Scenario	Node	Computed pressures (m) 1	2	3	True pressure (m)	Absolute error (m) 1	2	3
1	1	38.15	38.16	38.15	38.15	0.00	0.01	0.00
	2	42.26	42.28	42.27	42.26	0.00	0.02	0.01
	3	42.31	42.31	42.31	42.31	0.00	0.00	0.00
	6	36.83	36.83	36.83	36.83	0.00	0.00	0.00
	11	41.19	41.20	41.20	41.20	0.01	0.00	0.00
2	1	25.60	25.61	25.60	25.60	0.00	0.01	0.00
	2	28.14	28.15	28.14	28.14	0.00	0.01	0.00
	3	31.17	31.17	31.17	31.17	0.00	0.00	0.00
	6	25.55	25.55	25.55	25.55	0.00	0.00	0.00
	11	28.60	28.61	28.61	28.60	0.00	0.01	0.01
3	1	12.16	12.16	12.16	12.16	0.00	0.00	0.00
	2	12.65	12.65	12.65	12.65	0.00	0.00	0.00
	3	19.38	19.38	19.38	19.38	0.00	0.00	0.00
	6	13.73	13.73	13.73	13.73	0.00	0.00	0.00
	11	14.97	14.98	14.98	14.98	0.01	0.00	0.00

Table 8. Computed and true flow rates (GA-Simplex).

Scenario	Pipe	Computed flows 1	2	3	True flow (l/s)	Relative error (%) 1	2	3
1	1	49.26	49.21	49.21	49.25	0.0	−0.1	−0.1
	3	54.77	54.83	54.82	54.80	0.0	0.0	0.0
	22	83.14	83.15	83.14	83.12	0.0	0.0	0.0
2	1	76.31	76.33	76.35	76.37	−0.1	−0.1	0.0
	3	75.74	75.76	75.75	75.73	0.0	0.0	0.0
	22	115.19	115.13	115.13	115.14	0.0	0.0	0.0
3	1	103.69	103.75	103.77	103.77	−0.1	0.0	0.0
	3	97.04	97.03	97.01	97.00	0.0	0.0	0.0
	22	146.69	146.66	146.66	146.67	0.0	0.0	0.0

7 CONCLUSIONS

This paper focuses on the development of a calibration hydraulic model using a hydraulic simulation model with the support of EPANET, which considers both leakage and pressure-dependent demands. The method was used to the evaluation of the water losses and actual demands using the data from a hypothetical network, but further studies are required, especially with field data.

Moreover, a hybrid GA-Simplex method is presented and its advantages like run-time savings and increased efficiency on determining calibration parameters are shown.

It should be observed that, sometimes, during the calibration process, the hydraulic simulator through EPANET could not be completed for certain combinations of parameters of the leakage model. It is surmised that this difficulty can be remedied by incorporating the leakage model into the hydraulic simulation code, avoiding the leakage calculations being conducted independently.

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REFERENCES

Bhave, P.R. 1988. Calibrating Water Distribution Network Models. *Journal of Environmental Engineering*, 114(1): 120–136, Feb.

Boulos, P.F., Wood, D.J. 1990. Explicit Calculation of Pipe-Network Parameters. *Journal of Hydraulic Engineering*, 116(11): 1329–1344, Nov.

Fujiwara, O., Li, J. 1998. Reliability Analysis of Water Distribution Networks in Consideration of Equity, Redistribution, and Pressure-Dependent Demand. *Water Resources Research*, 34(7), 1843–1850, July.

Goldberg, D.E. 1989. *Genetic Algorithms in Search, Optimization and Machine Learning*, Addison-Wesley, Reading, MA.

Kapelan, Z.S., Savic, D.A., Walters, G.A. 2000. Inverse Transient Analysis in Pipe Networks for Leakage Detection and Roughness Calibration. In: Savic, D.A. & Walters, G.A. (eds): *Water Network Modelling for Optimal Design and Management*: 143–159.

Kapelan, Z.S., Savic, D.A., Walters, G.A. 2002. Hybrid GA for Calibration of Water Distribution Hydraulic Models. *Proceedings of the 2002 Conference on Water Resources Planning and Management*, Roanoke, Virginia.

Lansey, K.E., Basnet, C. 1991. Parameter Estimation for Water Distribution Networks. *Journal of Water Resources Planning and Management*, 117(1): 126–144, Jan/Feb.

Lingireddy, S., Ormsbee, L.E. 2002. Hydraulic Network Calibration Using Genetic Optimization. *Civil Engineering and Environmental Systems*, 19(1): 13–39.

Martínez, F., Conejos, P., Vercher, J. 1999. Developing an Integrated Model for Water Distribution Systems Considering both Distributed Leakage and Pressure-Dependent Demands. *Proceedings of the 26th ASCE Water Resources Planning and Management Division Conference*, Tempe, Arizona.

Michalewicz, Z. 1994. *Genetic Algorithms + Data Structures = Evolution Programs*. 2nd ed., Springer-Verlag, USA.

Nelder, J.A., Mead, R. 1965. A Simplex Method for Function Minimization. *The Computer Journal*, 7(4): 308–313.

Ormsbee, L.E. 1989. Implicit Network Calibration. *Journal of Water Resources Planning and Management*, 115(2): 243–257, Mar.

Ormsbee, L.E., Lingireddy, S. 1997. Calibrating Hydraulic Network Models. *Journal of the American Water Works Association*, 89(2): 42–50.

Reis, L.F.R., Porto, R.M., Chaudhry, F.H. 1997. Optimal Location of Control Valves in Pipe Networks by Genetic Algorithm. *Journal of Water Resources Planning and Management*, 123(6): 317–326, Nov/Dec.

Rossman, L.A. 2000. *EPANET 2 users manual*. U.S. Environmental Protection Agency, Cincinnati, Ohio.

Savic, D.A., Walters, G.A. 1995. Genetic Algorithm Techniques for Calibrating Network Models. *Report No. 95/12*, Centre for Systems and Control Engineering, University of Exeter, p. 41.

Soares, A.K. 2003. *Water Distribution Network Calibration Model Considering Leakage and Head-Driven Demands.* Master's Dissertation – São Carlos School of Engineering, University of São Paulo. (in Portuguese)

Subrahmanyam, M.B. 1989. An Extension of the Simplex Method to Constrained Nonlinear Optimization. *Journal of Optimization Theory and Applications*, 62(2): 311–319, Aug.

Tabesh, M., Karimzadeh, A. 2000. Optimum Design of Reliable Distribution Systems Considering Pressure Dependency of Outflows. In: Savic, D.A. & Walters, G.A. (eds): *Water Network Modelling for Optimal Design and Management*: 211–220.

Tucciarelli, T., Criminisi, A., Termini, D. 1999. Leak Analysis in Pipeline Systems by Means of Optimal Valve Regulation. *Journal of Hydraulic Engineering*, 125(3): 277–285, Mar.

Wall, M. 1996. *GAlib: A C++ Library of Genetic Algorithm Components*. Mechanical Engineering Department, Massachussetts Institute of Technology.

Walski, T.M. 1983. Technique for Calibrating Network Models. *Journal of Water Resources Planning and Management*, 109(4): 360–372, Oct.

Entropy-based water distribution network rehabilitation

Y.B. Wu & H. Tian
Harbin Institute of Technology, Harbin, Heilongjiang, China

ABSTRACT: A strategy for prioritizing decisions for the rehabilitation of a water distribution network is developed. The nodal and network entropy are respectively calculated based on the redundancy concepts and the maximum entropy flow distribution formulation. The network performance index decides whether rehabilitation is required or not. The nodal relative entropy values reflect the nodal performance. The contour plot based on the values determines a performance surface. At any time, this surface is used to locate low performance areas, which identify parts of the network that need rehabilitation priority. Using the nodal relative entropy and taking the pipe un-failure principle into consideration, the pipe performance indices are worked out. The indices measure the performance of the pipes, and provide a reference for ranking the different parts to be restored. The approach is demonstrated in an example network.

1 INTRODUCTION

Water distribution systems, a less visible but very important part of the public works infrastructure, are in various states of disrepair. Limited budgets for their rehabilitation require a scheme for ranking the different parts of the network to be restored to acceptable status. Rehabilitation priority may be based on performance criteria. The network performance having been simulated, when the performance goes below the acceptable level, rehabilitation will be undertaken. Comparisons of the local performance values among the components (nodes or pipes) decide on which parts of the network will be paid more attention. The implementation of the approach is the subject of the paper.

General speaking, the objective of a water utility is to satisfy all demands with sufficient water at adequate pressure and at the minimum possible cost. To what degree that objective is accomplished depends on how well the network performs. Further speaking, the performance of a water distribution network can be measured by how reliably it actually does it achieve the objective. Indeed, the performance of a water network associates closely with its reliability.

Over the last few years, various techniques have been developed for quantifying the reliability of a water distribution network. In general, they can be classed into direct reliability measures based on mechanical failure and indirect reliability measures based on hydraulic failure. Since direct measures are difficult and time consuming, some surrogate measures for reliability are applied instead of direct methods. The use of maximum entropy flows is one of the main methods for surrogate evaluation of reliability. Awumah, Bhatt and Goulter (1990) introduced firstly entropy concepts, derived from information theory, to water distribution networks. Awumah and Goulter (1992) and Tanyimboh and Templeman, (1993) proposed further maximum entropy concepts of water distribution system. Actually, redundancy reflects the diversity of paths between supply and demand, is a major contributor to reliability. Furthermore, the distribution of flows in a network, which guarantees the greatest uniformity between all the supply paths to all the nodes, minimizes the expected shortfall in case of a pipe breakdown. The uniformity corresponds to the highest possible value of an entropy function reflecting the uncertainty generated by different paths of supply to each and every node of a network (Tanyimboh and Templeman 1993); (Tanyimboh 2000). In addition, dimensionless characteristic of entropy makes itself more suitable to be a surrogate measure with the great objectivity. So the entropy-based measure is a feasible approach to the development of performance measures (Tanyimboh & Templeman 1994).

2 NODAL PERFORMANCES

2.1 *Introduction*

Nodes, pipe joints, are one of important components made up of a network. Actually, reliability of a water system emphasizes the water approachability to nodes and the availability of the demands at nodes. In the way, the nodal performance in practical operation is associated closely with the performance of the whole network. In addition, the performance values at all the nodes can provide a performance profile across the whole network. Comparisons of performance values among all the nodes indicate which parts of the network should be paid more attention.

2.2 *Calculation of the nodal actual entropy*

Actual entropy is referred to as operational entropy or working entropy for a given hydraulic pattern.

The formulation of entropy based on the nodes as the elementary level is presented by the following expression:

$$\boxed{S_n = \sum_{i \in U^n} \left(-\frac{q_{in}}{Q_n} \ln \frac{q_{in}}{Q_n} + \frac{q_{in}}{Q_n} S_i \right) \quad \forall n \in N} \tag{1}$$

where S_n is the actual entropy at any node n, i is a upstream node adjacent to node n, U^n the set of upstream nodes adjacent to n, q_{in} the flow in the pipe connecting node i with node n, Q_n the total upstream flow joining together at node n, and S_i is the entropy value at node i. q_{in} and Q_n depend on the hydraulic pattern.

Beginning from sources, the sequential method that is easily implemented in a computer program will be performed in the upstream area at a node to acquire the nodal actual entropy.

2.3 *Calculation of nodal maximum entropy*

All paths start at the sources in a water distribution network. If there is no further information about the paths, there is no reason for any path to be preferred over any other path to a demand node, since any demand node is usually served by more than one path. This accords with the consequence of the maximum entropy formalism. That is, flow to the node should be distributed equally among all the paths supplying the node. Based on the principle related above, the maximum entropy flow distribution in all paths can be obtained through a concise arithmetic offered by Tanyimboh, T.T. et al. (Tanyimboh & Templeman 1993); (Yassin-Kassab et al. 1999). Then, alike the nodal actual entropy calculation, the nodal maximum entropy can be worked out by using Equation 1 and the sequential method.

2.4 *Nodal relative entropy*

In a water distribution network, the maximum entropy at any node is considered to be itself characteristic constant; of course, all nodes are of different characteristic constant values. In this case, the relative entropy at any node is formed:

$$E_n^r = \frac{S_n}{S_n^{\max}} \qquad n \in \forall N \quad (E_n^r < 1) \tag{2}$$

where E_n^r is the relative entropy at node n; S_n the actual entropy at node n; and $S_n^{\max}$ the maximum entropy at node n.

Nodal relative entropy, which uses itself characteristic constant as a reference level, indicates the local performance at a node with reasonableness. As much more as possible does the relative entropy value approach to 1, better is the local performance at the node.

2.5 *Contour nodal relative entropy*

Nodal relative entropy values are used in conjunction with a simple interpolation algorithm to plot contour lines of equal relative entropy values. The performance surface in the network is displayed, after the contours are superimposed on a scaled layout of the distribution network. As the hydraulic pattern of the system changes, the performance surface will be varied.

Contour nodal relative entropy, which provides a straight sight on the performance surface in a network as the other type of exhibition on the nodal relative entropy values, is of a practical significance in decision on the rehabilitation priority.

3 NETWORK PERFORMANCE SIMULATIONS

3.1 *Calculation of the network actual entropy*

The flows carried by the various pipes of the network, as well as external inflows (supplies) and outflows (demands), are joined at junction nodes. Supposing that all demands congregate together to a fictitious node 0. There is no upstream path at sources, so the values of entropy at sources are respectively defined as zero (Tanyimboh & Templeman 1993); (Yassin-Kassab et al. 1999). The formulation of entropy for a network is expressed as follows:

$$S_N = S_0 = \sum_{j \in U^0} \left(-\frac{q_{j0}}{Q_0} \ln \frac{q_{j0}}{Q_0} + \frac{q_{j0}}{Q_0} S_j \right) \tag{3}$$

where j is a demand node in a network, U^0 the set of all demand nodes in the network, q_{j0} the demand at node j, Q_0 the total demands in the network, and S_j is the actual entropy at node j, which can be worked out by Equation 1. S_0 is the actual entropy at the fictitious node 0, at the same time, it is the actual global entropy of the network (S_N) as well.

3.2 *Calculation of the network maximum entropy*

Based on the principle of the maximum entropy formalism, the maximum entropy flow distribution in all paths is obtained firstly. Alike the network actual entropy calculation, the fictitious demand point generates the network maximum entropy by using Equation 3 and the sequential method.

3.3 *Performance simulation for the network*

The maximum entropy for a network is, to the great extent, conditioned by the shape and layout of the network. It represents the most possible potential reliability for the network with a given shape and layout. In other words, the maximum entropy flow distribution is an ideal or optimum hydraulic state, which provides a reference value for performance simulation. Therefore, performance of a water distribution network can be referred to as the ability to fulfill the potential reliability of the network. In this case, the maximum entropy for any network can be regarded as a characteristic constant of the network itself, and the performance function for the network is constructed as follows:

$$P_N = \frac{S_N}{S_N^{\max}} \qquad (0 < P_N \leq 1) \tag{4}$$

where S_N is the actual entropy for any network, $S_N^{\max}$ the network maximum entropy, and P_N is defined as the performance index of the network.

In fact, the performance function is the objective function of rehabilitation. The principle of rehabilitation for a water distribution network is, from the point of view of entropy flow, to maximize the objective function as possible.

4 PIPE PERFORMANCE ANALYSES

4.1 *Network rehabilitation, pipe performance and nodal relative entropy*

A pipe is one of the smallest units to be restored in a water distribution network that is about to be rehabilitated. Limited budgets should be put on the pipes at some hazard. The security of the pipe depends greatly on its reliability performance. Therefore, rehabilitation priority may be based on pipe performance evaluation.

Broadly speaking, the macroscopic effect of the reliability of a water distribution system is the water approachability to nodes or the availability of nodal demands. A pipe is a link between two nodes. Pipes build each and every path from sources to nodes. The reliability performances of the pipes define the nodal performance. In the same way, the nodal performances reflect the pipe performances. Since the nodal relative entropy can be used to measure the nodal performance, it is reasonable for it to be applied in the analyses of the pipe performances.

4.2 *Pipe performance evaluation*

Given P_{ij}, it is the pipe performance index. The value is related to the physical attributes of the pipe besides the flow distribution in the pipe must be taken into consideration.

Flow distribution-related performance index, P'_{ij}, is expressed base on the entropy by the following equation:

$$P'_{ij} = w_{ij} E_i^r + (1 - w_{ij}) E_j^r \tag{5}$$

where w_{ij} is the nodal weight, E_i^r the relative entropy at node i, and E_j^r the relative entropy at node j.

The weight reflects different contributions of two nodes to the performance of the pipe between two nodes. It depends on the original allocation proportion of two nodal demands based on the users' water consumptions on two sides of the pipe. In general, the allocation proportion is 1 in a water distribution system; therefore, the weight is 0.5.

Diameter and length, two important physical parameters of a pipe, to a certain extent, decide the failure probability of the pipe (Xu & Guercio 1997). On the un-failure negative-exponential principle consideration, a un-failure coefficient is given in the following expression:

$$\kappa_{ij} = e^{-\mu\left(\frac{L_{ij}}{D_{ij}}\right)} \tag{6}$$

where D_{ij} is the diameter of pipe ij in mm; L_{ij} is the length of pipe ij in m; and μ is a factor related to the duration of the pipe failure. In the paper, the duration is not considered, and then μ is 1.

Combination Equation 5 with Equation 6, the performance index of pipe ij is given by an improved equation:

$$P_{ij} = e^{-\frac{L_{ij}}{D_{ij}}} \left(\frac{E_i^r + E_j^r}{2}\right) \tag{7}$$

5 AN EXAMPLE APPLICATION

The example network shown in Figure 1 includes 23 nodes and 34 pipes with a single-source at node 1. The network nodes and pipes are globally numbered as shown in Figure 1. The nodal demands (q_{n0}) are shown in Table 1, and diameter (D_{ij}) and length (L_{ij}) are respectively listed in Table 2.

The network simulation was implemented by EPANET. The flows in pipes (q_{ij}) are shown in Table 2, and the supply paths are displayed in Figure 1.

The maximum entropy flows in pipes ($q_{ij}^{\max}$), shown in Table 2, were acquired on the principle of maximum entropy flow distributions.

Along the paths, using the values of q_{ij} and $q_{ij}^{\max}$, the nodal actual entropy S_n and maximum entropy $S_n^{\max}$ were calculated respectively by Equation 1, and the results are shown in Table 1. In that case, Equation 2 generated the values of the nodal relative entropy E_n^r, displayed in Table 1 as well. The contour relative nodal entropy, the performance surface in the example network, is shown in Figure 2.

Similarly, the network actual entropy S_N and maximum entropy $S_N^{\max}$ equal to 3.2577 and 3.3914 respectively by Equation 3. Then, the performance index of the example network is 0.9606 by Equation 4. Assuming the acceptable performance index is 0.97, and then the index value of the example network is below slightly the level, a rehabilitation scheme will be required. Equation 7 generated the performance indices of pipes shown in Table 2.

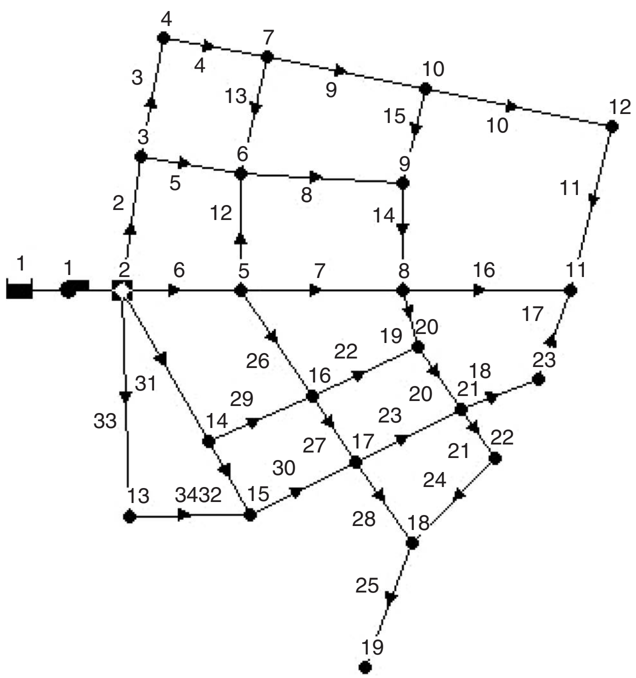

Figure 1. The example network.

Table 1. Nodal data of the example network.

Node	q_{n0} (m^3/h)	S_n	S_n^{max}	E_n^r
1	−4000.83			
2	254.75	0	0	0
3	212.1	0	0	0
4	50.85	0	0	0
5	378.3	0	0	0
6	332.7	1.011579	1.098612	0.920778
7	103.95	0	0	0
8	161.5	0.560486	1.609438	0.34825
9	160.25	1.193865	1.386294	0.861191
10	121.9	0	0	0
11	763.03	1.87963	2.833213	0.663427
12	64.68	0	0	0
13	125.93	0	0	0
14	90.9	0	0	0
15	143.8	0.6065	0.693147	0.874995
16	256.43	0.691952	0.693147	0.998276
17	253.18	1.333356	1.386294	0.961813
18	146.25	2.306532	2.70805	0.851732
19	54.25	2.306532	2.70805	0.851732
20	107.23	1.094962	1.94591	0.562699
21	116.68	1.905784	2.397895	0.794774
22	45	1.905784	2.397895	0.794774
23	57.17	1.905784	2.397895	0.794774
0*	4000.83	3.257736	3.391413	0.960584

* The fictitious demand point.

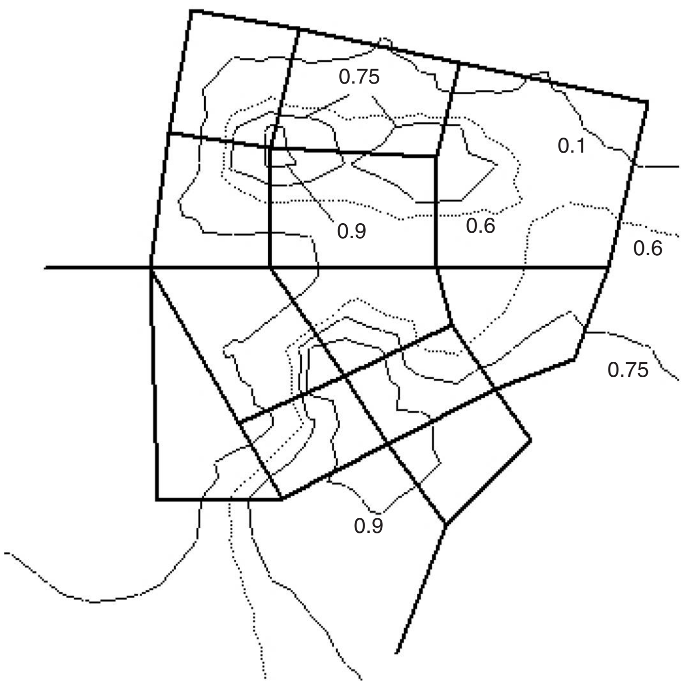

Figure 2. Contour relative nodal entropy.

Table 2. Pipe data of the example network.

Pipe	D_{ij} (mm)	L_{ij} (m)	q_{ij} (m^3/h)	$q_{ij}^{\max}$ (m^3/h)	k_{ij}	p'_{ij}	P_{ij}
2	500	427.1	1073.76	1452.3	0.31015	0.0000	0.0000
3	500	376.6	554.51	918.588	0.26509	0.0000	0.0000
4	400	436	503.66	867.738	0.39954	0.0000	0.0000
5	300	435	307.15	321.612	0.50175	0.5623	0.2310
6	500	442.9	1523.4	1247.05	0.32338	0.0000	0.0000
7	400	487.3	617.24	170.649	0.44006	0.1042	0.0766
8	300	492	239.61	632.135	0.54348	0.9014	0.4842
9	300	480.7	268.11	442.176	0.53575	0.0000	0.0000
10	200	617.4	128.27	109.564	0.72329	0.0000	0.0000
11	200	555.4	63.59	44.884	0.69761	0.6116	0.2314
12	300	380	133.55	321.612	0.45408	0.4309	0.2091
13	300	352	131.6	321.612	0.42644	0.7016	0.1963
14	300	380	97.29	682.596	0.45408	0.6037	0.2746
15	200	271.7	17.94	210.712	0.47898	0.4891	0.2062
16	300	553	432.95	224.421	0.5813	0.6084	0.2940
17	300	417	266.48	493.725	0.48703	0.6726	0.3551
18	300	287.1	323.65	550.895	0.35172	0.7948	0.2795
19	300	202.6	120.08	467.324	0.22747	0.4338	0.1036
20	300	400	295.29	547.023	0.47237	0.6836	0.3206
21	300	229	143.16	192.033	0.26981	0.7948	0.1831
22	300	356.9	282.45	186.929	0.43146	0.8698	0.3368
23	300	356.9	288.2	312.585	0.43146	0.9091	0.3790
24	300	419.7	98.16	147.033	0.48929	0.8383	0.4028
25	150	597.3	54.25	54.25	0.77792	0.8517	0.6626
26	400	427.2	394.31	376.488	0.39206	0.4033	0.1957
27	300	400	290.27	309.616	0.47237	0.9802	0.4629
28	200	450	102.34	53.467	0.64118	0.9215	0.5814
29	350	387.1	434.84	376.487	0.40488	0.7370	0.2021
30	300	467.5	353.45	309.616	0.52639	0.9304	0.4834
31	400	507.4	672.38	694.096	0.4546	0.0000	0.0000
32	300	400	146.65	226.708	0.47237	0.5361	0.2067
33	400	663	476.53	352.638	0.54699	0.0000	0.0000
34	350	442.3	350.6	226.708	0.45325	0.4665	0.1983

Table 2 shows that the performance index values of pipe 7 and 19 are much lower than values of other ones except the zero value. The contour plot reveals that pipe 7 and 19 locate in the contour of 0.6. Therefore, the two pipes should be paid more attentions.

It is worth noting that the branch pipe to node 25 has a much higher performance index. It is an exception to the approach proposed in the paper.

Some nodes in the network, to which there are not alternative supply paths, are in the state of zero nodal relative entropy. Therefore, the pipes between the nodes have respectively a zero pipe performance index. They should be paid more attentions in making a rehabilitation scheme.

6 CONCLUSIONS

The study shows how entropy-based reliability performance evaluation may be used to formulate a strategy for rehabilitating water distribution networks. The use of the pipe performance index and the topology of the distribution network to develop a performance surface can be a valuable aid in prioritizing a rehabilitation scheme.

Although the applicability of the technique has been demonstrated in an example network, it is not still applied in real-life distribution networks. The complexity of the real-life network may be requires the improvement of the technique. In addition, the pipe failure is a complicated problem with many factors involved. The study only took diameter and length into considerations, which may be has resulted in the errors in pipe performance evaluations. Finally, economic factors were not considered in ranking the parts of the network to be restored. In fact, the cost as a constrained condition should be taken into considerations on the rehabilitation priority for a real-life water distribution system. All problems, as related above, require further researches.

REFERENCES

Awumah, K. et al. 1990. Assessment of reliability in water distribution networks using entropy based measures. *Stochastic Hydraulic and Hydrology* 4(4): 309–320

Awumah, K. & Goulter, I. 1992. Maximizing entropy-defined reliability of water distribution networks. *Engineering Optimization* 20(1): 57–80

Tanyimboh, T.T & Templeman, A.B. 1993. Calculating maximum entropy flows in networks. *Journal of the Operational Research Society* 44(4): 383–396

Tanyimboh, T.T. et al. 2000. A quantified assessment of the relationship between the reliability and entropy of water distribution systems. *Engineering Optimization* 22(2): 179–199

Tanyimboh, T.T & Templeman, A.B. 1994. Discussion of redundancy-constrained minimum-cost design of water-distribution nets. *J. Water resources planning and management* 120(4): 568–571

Tanyimboh, T.T & Templeman, A.B. 1993. Calculating maximum entropy flows in single-source networks. *Engineering Optimization* 22(1): 49–63

Yassin-Kassab, A. et al. 1999. Calculating maximum entropy flows in multi-source, multi-demand networks. *Engineering Optimization* 31(1): 695–729

Xu, Z.X. & Guercio, R. 1997. The new entropy formula for optimal design of water distribution systems. *Journal of Hydrodynamics* 97(2): 51–57

Need for pressure dependent demand in analysing failure of pipe networks

M.A.M. Mansoor & K. Vairavamoorthy
WEDC, Department of Civil Engineering, Loughborough University, Loughborough, UK

ABSTRACT: Failures within Water Distribution Systems are inevitable and their effect results in reduced levels of service. The reduction in levels of service can be obtained through simulation, however it is important that when simulating these failures an appropriate analysis method is available. Traditionally failures in water distribution systems are simulated using demand driven analysis, that is not appropriate for extreme events in the network. A more appropriate analysis approach is pressure dependent demand one. The paper highlights the way a demand driven analysis distorts the performance of the network during failure and hence predicts the levels of service incorrectly.

1 INTRODUCTION

Water distribution systems (WDS) are very important lifeline infrastructure systems, where failures are inevitable. One of the consequences of failure of WDS is the reduced level of service (LOS). In times of peak demand parts of the network may experience inadequate flows and pressures. In addition, major disruptions in supply may occur due to failures, which may come in the form of pipe bursts, pump outages and reservoir failures. Understanding the events and behavior of WDS during a failure will enable us to accurately assess the deviations in LOS and to develop appropriate responses to counter the consequences of failure events. Hence it is essential to perform failure analysis of WDS.

Conventional demand driven (DD) analysis assumes that nodal demands are met irrespective of pressure conditions in the network. This type of analysis will show that the demand is supplied even when there is negative pressure. Hence the applicability of this method to extreme circumstances is questionable. On the other hand pressure dependent demand (PDD) analysis is sensitive to the variations of pressure in the system. When the system pressures fall below the minimum allowable, flows to consumers would be significantly reduced.

In general the motivation for including PDD functions in network analysis has been either to simulate intermittent systems or to simulate failure scenarios. Reddy (1990), Vairavamoorthy (1994) and Akinpelu (2001) all incorporated PDD functions to network analysis to simulate the behavior of intermittent systems. All the approaches used a PDD function based on the orifice equation. They argued that in water starved system outlets are rarely controlled and hence behave as an orifice. Akinpelu (2001) coupled the PDD function with a queuing process to simulate activities of the outlet where there is some control due to increased quantities of water. However his study found that the system was most vulnerable when all outlets were discharging and hence the assumption of the orifice equation for the analysis of the network was still appropriate. Application of orifice equation to controlled flow situations would predict unrealistic demands therefore it is not appropriate to incorporate the orifice relationship into a continuous system since the outlets are always controlled.

Gemanopoulos (1988) who primarily dealt with simulations of failed water distribution systems presented an exponential relationship for head dependent outflow. This empirical expression requires certain key parameters that are difficult to establish, hence its practicality is questionable. Wagner et al. (1988) recognized the Pressure dependent outflow relationship but proposed to modify outflow retrospectively after obtaining pressures from a network analysis. Clearly this method is inappropriate as the pressure and flow should simultaneously satisfy the head dependent relationship. In addition their method only modifies outflows at certain failed nodes and not all demand nodes. Hence global variations and flow redistributions in the network are not accounted for.

From the authors work it was found that both orifice equation and that proposed by Germonopoulos do not correctly simulate the pressure dependent nature of outflow during sudden drops in pressure in a network. In this paper the authors attempt to develop a micro level model, which simulates the behavior of individual consumers during a failure. It is anticipated that this work will enable the development of more appropriate PDD function than those previously suggested.

In this paper a comparison is given between the performance of the network during a failure when analysed with a demand driven (DD) approach and pressure dependent (PDD) approach. The results of this analysis indicate that in the DD approach all the nodes have their demands satisfied although pressures are greatly reduced. However in PDD approach the results indicate that several nodes become isolated and receive no water due to the reduction in pressure. In addition the PDD approach highlights that the flows get redistributed due to overall reduction in the flows in the network. Hence it would be inappropriate to use DD analysis to failure analysis.

2 METHODOLOGY

The purpose of this paper is to demonstrate the need for a PDD approach when simulating failures in water distribution systems. The methodology applied to any network is as follows:

- Replace primary nodes in the network with level controlled reservoirs.
- Calculate primary node water usage (consumption) profiles.
- Perform extended period simulation under normal operating conditions using both a DD and PDD approach (to ensure good fit).
- Perform extended period simulations of failure scenarios using both a DD and PDD approach and compare results.

2.1 *Primary node level controlled reservoirs*

In this paper explicit PDD functions are not used but PDD is modelled using water tanks(WT). Based on the number of consumers connected to a primary node, the reservoir dimensions are calculated. Action of the ball valve (as is commonly found in WT's in UK (Field,1978)) is modelled using level control switches coupled with parallel inlet pipes of varying hydraulic carrying capacities (as shown in Figure 1a). Parallel pipes are included to simulate the varying cross sectional area of the ball valve inlet. Using control switches triggered by the level in the WT, it was made possible to ensure a single pipe from the set of parallel pipes to be opened at any particular time. Each pipe allows a particular flow based on its carrying capacity, which is related to the tank level. The ball valve characteristic is matched by using different pipes at different trigger levels. Here 4 pipes have been used to simulate the scenario. Table 1 gives the trigger levels, pipe capacity the pipes used.

Figure 1b shows the above relationship and also the characteristic curve of a normal ball valve which is being used in household tanks.

2.2 *Water usage calculation*

The outflows from the reservoirs are the demands of the consumers connected to particular primary node. To calculate outflow, individual outflow profiles per household were established and coupled with a probability distribution to calculate the spread of the number of household using water at any particular time. See Figure 2

The water usage profile used is based on data obtained from Field (1978). However, Field's data set covers only morning usage and hence evening usage was added based on the authors' own

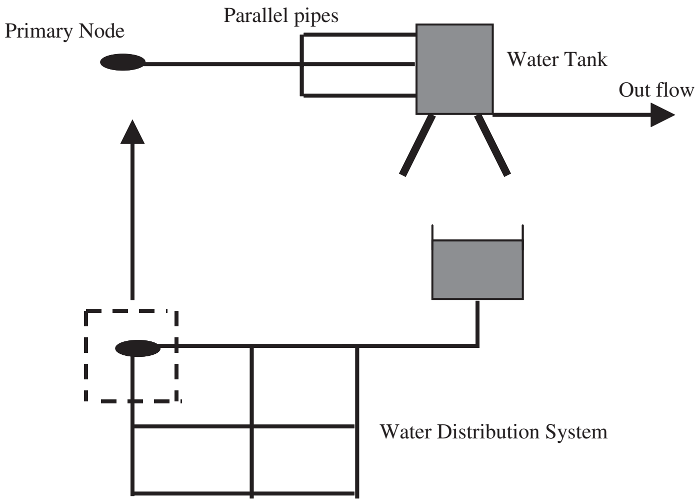

Figure 1a. Primary node model.

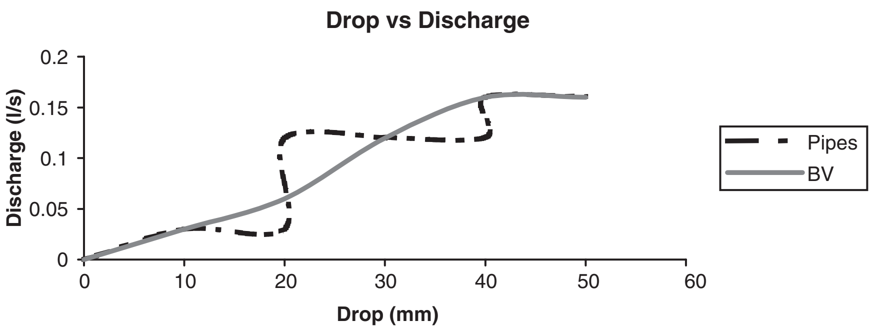

Figure 1b. Pipe trigger levels and ball valve characteristics.

Table 1. Pipe trigger levels and carrying capacities.

Pipe	Tank level(m)	Trigger level(mm)	Hazen Williams K value
1	0.74	0–10	0.026
1	0.73	10–20	0.026
2	0.72	20–30	0.053
3	0.71	30–40	0.11
4	0.70	40–50	0.14

observations. Typical values of individual usage and corresponding time durations are given in Table 2 and Figure 3. To generate a household usage profile from the individual usage profile, it was assumed that all households consisted of 4 individuals whose activities between individuals took place in 5 minute intervals. Although this assumption may be simplistic the purpose of this paper is to only demonstrate an approach.

The next step was to develop usage profiles for individual primary nodes that may represent hundreds of households. This was achieved by combing the household usage profile with a normal

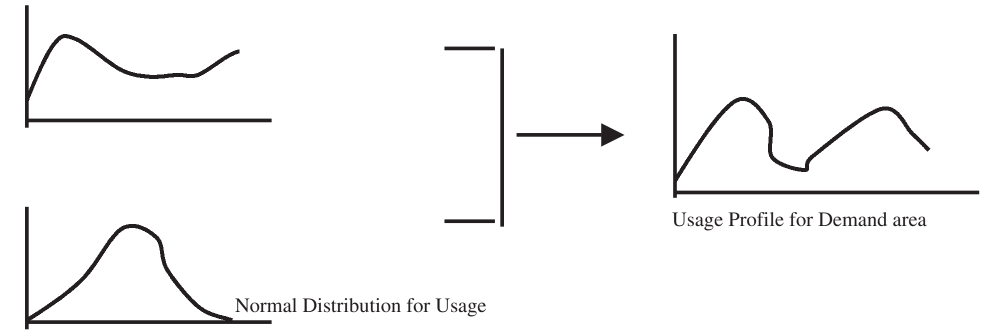

Figure 2. Water usage profile.

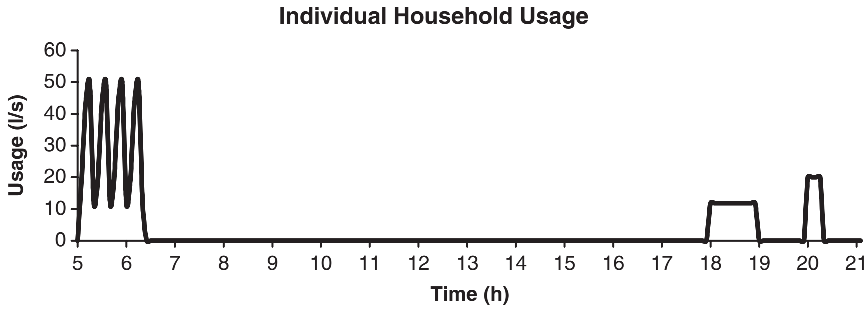

Figure 3. Individual household usage.

Table 2. Water usage in households.

Water use	Flow rate (lps)	Duration(s)
Water closet	0.145	70
Wash	0.167	50
Clean teeth	0.09	40
Kettle	0.167	10
Shower	0.1	600

distribution Figure 4. It is assumed that the normal distribution is distributed about 7.00 am with a standard deviation of 0.9. This would produce initial demand activities for a primary node at 5.00 am with the maximum activities at 7.00 am. and the last activities begin at 9.00 am. Figure 2 indicates this process. Therefore by combining Figures 3 & 4 we are able to generate Figure 5. A similar distribution was applied for the evening water usage. The outcome from this step is therefore a 24 hour primary node usage profile, which then assign to the primary network model. Figure 5 shows the primary node usage profile for an individual node.

2.3 *Normal operating conditions analysis (using DD and PDD)*

The first step in the analysis is to develop lumped demands that can be used for the DD analysis. This is achieved by performing an extended period simulation for the entire network using the water usage profiles generated above. This extended period simulation is performed in 5 minute intervals and the results are used to generate lumped hourly demand using standard averaging procedures. The diurnal lumped demands calculated are then assigned to demand nodes then used for DD analysis.

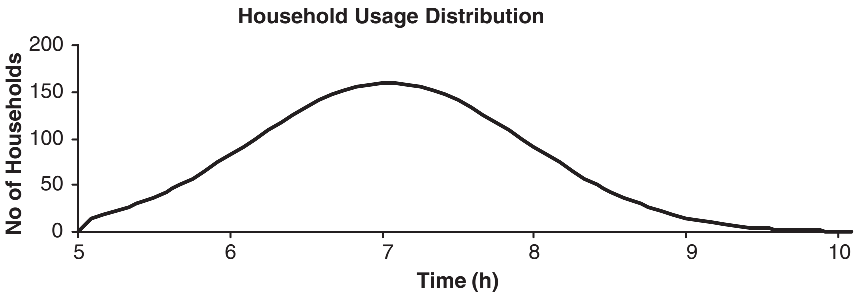

Figure 4. Household usage distribution.

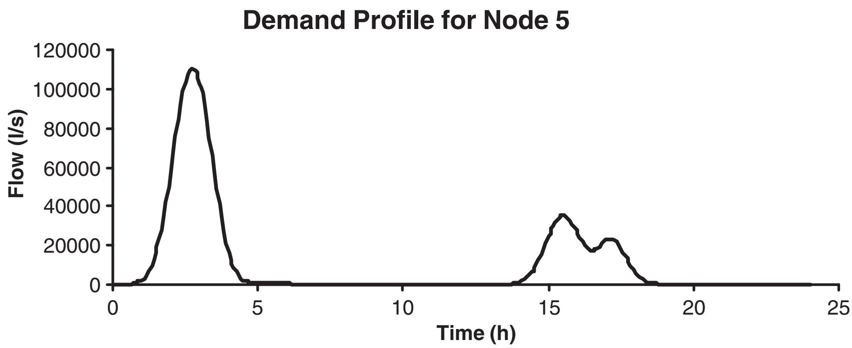

Figure 5. Usage demand profile.

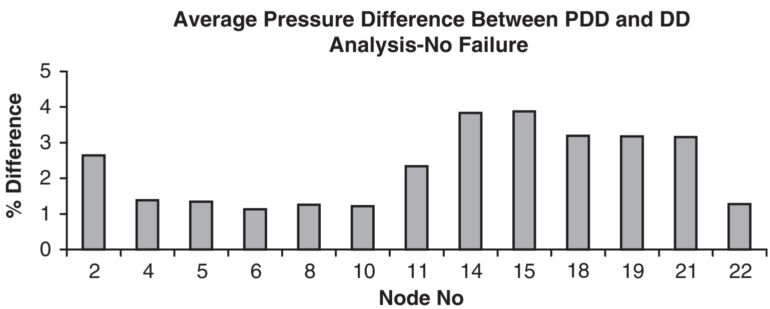

Figure 6. Average pressure differences for PDD and DD formulations.

To show that both DD and PDD analysis are predicting similar results EPS was performed. Figure 6 shows the difference (percentage error) in pressure for an example network in all the demand nodes when analysed using DD and PDD approaches. From this Figure it can be seen that the variations in results are very minimal and hence can be concluded that under normal conditions PDD and DD approaches behave similarly.

2.4 *Failure analysis (using DD and PDD)*

At this stage of the analysis, two models exist a DD and a PDD one. The next stage is to simulate failures to compare the performance of both these models to failure scenarios. Several failures can be simulated in the network including pump outages, pipe bursts etc.

For example, a pump outage is simulated by simply isolating the pump from the system. This can be achieved in two ways; by giving controls to make the pump switch off at certain times or by giving a pump pattern where the pattern will indicate the times at which pump stops.

Pipe bursts are simulated in the same way as described by Germanopolus (1998), and are shown in Figure 7. A short pipe is connected to the main pipe at the point where the burst is to occur. A constant head reservoir is attached to the end of the short pipe. The water level in the reservoir is equal to the elevation of the main pipe at the point where the short pipe connects with it. The burst is simulated by setting controls that open and shut pipes as given in Figure 7.

3 APPLICATION

To demonstrate the above method an example application will be given to a 25 node network, Figure 8. This network has been widely used in the literature (Vairavamoorthy and Lumbers 1998), and the physical data for this network can be obtained there. The primary node reservoir demands were calculated for this network as described above.

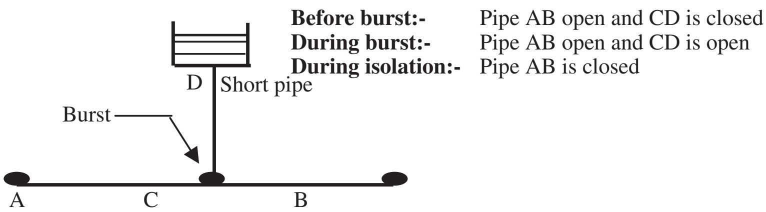

Figure 7. Representation of a burst.

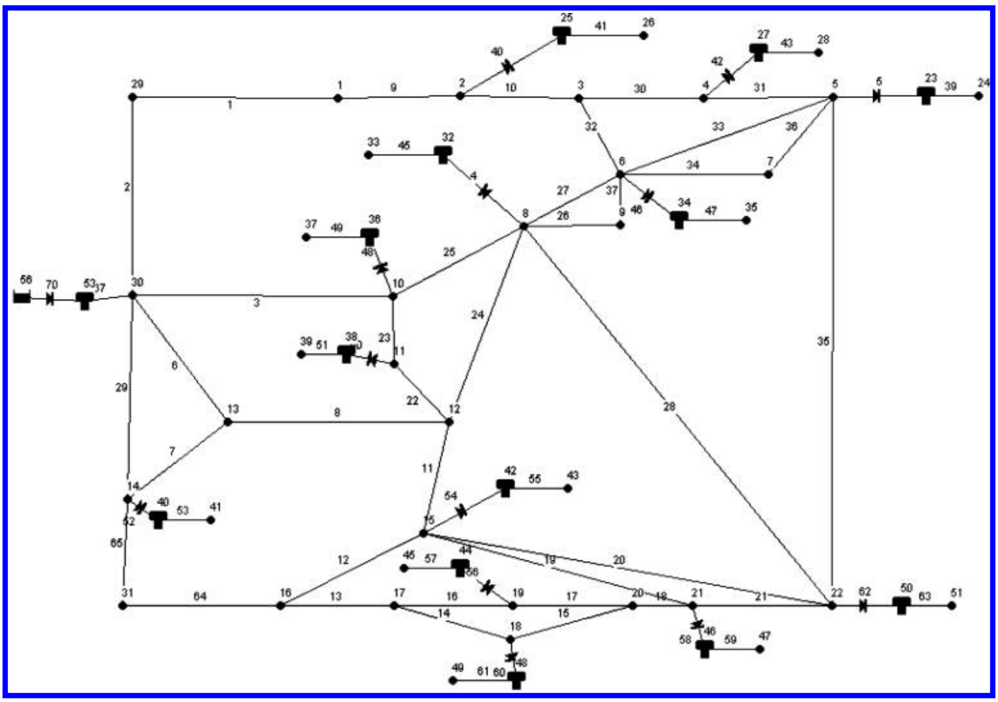

Figure 8. Network schematic.

3.1 *Results*

To verify that the DD and PDD approach give similar results when the network is operating under normal operating conditions, an extend period simulation is performed. Figures 9,10 & Figure 11 shows that there is a good fit between the DD and PDD approach during normal operating condition. The PDD analysis is carried out in 5 minutes time step whereas the DD simulation is done with a 1 hour time step. Both approaches gave similar results with very little deviations.

The next step is to demonstrate the difference in the results of DD and PDD analysis when applied to a failure. In this example a simple pipe burst is simulated in pipe 8 (between nodes 12 & 13), to occur at 6.30 am. Clearly the consequence of this failure is a sudden drop in pressure and the redistribution of flows. It is unlikely that the DD model will be able to respond appropriately as it will continue to show demand being satisfied irrespective of the drop in pressure. However, we would expect the PDD analysis to show variations in demand due to the sudden drop in pressure.

Figures 12–13 show the pressures and outflows at selected nodes during the simulated failure event. From these figures it is clear that when the failure is simulated using DD analysis the pressures have dropped drastically but that demand has not been affected (as one would expect with a DD analysis). In some parts the pressure have fallen below 1 m and still the nodes are being satisfied. Clearly this is a very distorted picture of what would happen in the network

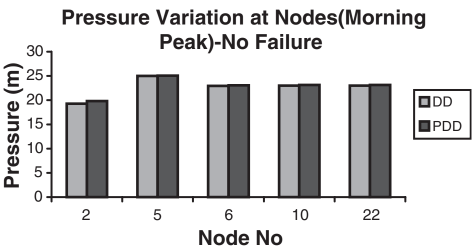

Figure 9. Pressure variation at nodes.

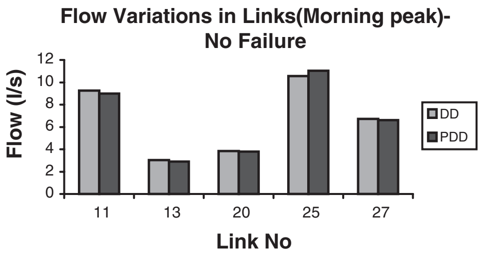

Figure 10. Flow variations.

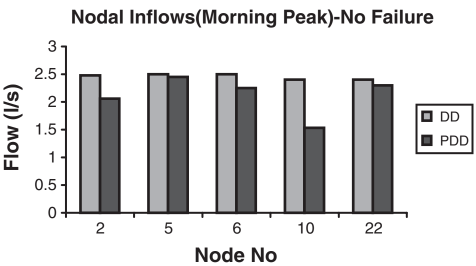

Figure 11. Nodal inflows.

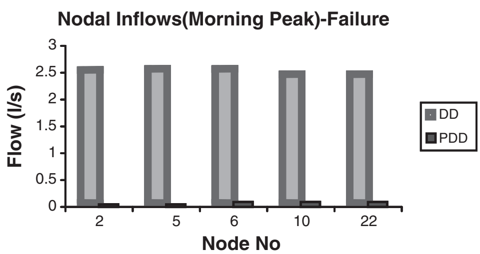

Figure 12. Flow to nodes.

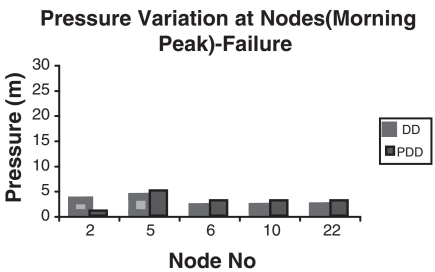

Figure 13. Pressures at node.

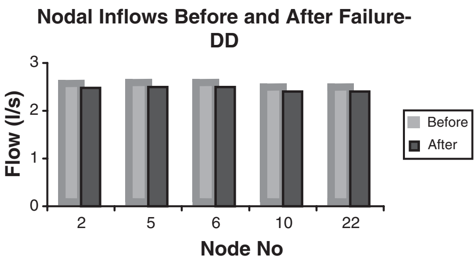

Figure 14. Flow to nodes.

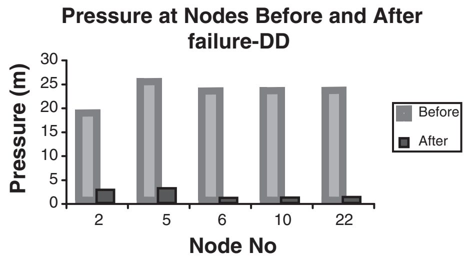

Figure 15. Pressures at node.

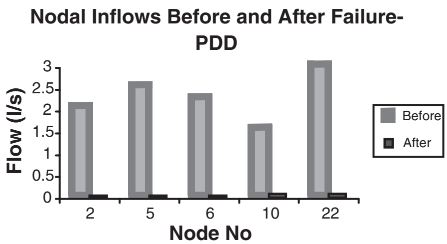

Figure 16. Inflows at nodes-PDD.

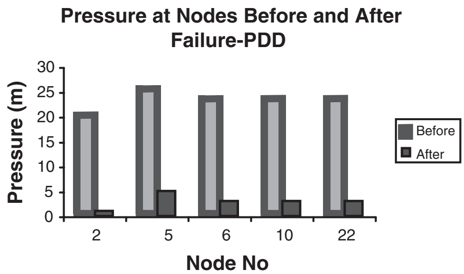

Figure 17. Pressures at nodes-DD.

However when the same failure event was simulated using the PDD analysis, the picture is very different. Figures 12–14 indicated the pressures and demands for the same selection of nodes as given in the DD analysis. It is apparent that there would be drastic drop in pressure but this drop results in many nodes not receiving water (nodes in a failed mode). Figures 14–17 shows the flow and pressure to selected nodes before and immediately after a failure.

Figures 14 & 15 shows the flows and pressures to selected nodes of DD analysis before and immediately after failure. Figures 16 & 17 are similar charts but for PDD analysis. It is clear from these charts that although the DD analysis indicates that pressure drop is not translated into reduced nodal outflows. It is also interesting to note that the pressure in the DD analysis are lower (after failure) than in the PDD case. Clearly this is due to higher flows occurring in the DD case. Although not shown in the paper, the analysis indicated that 5 demand nodes became isolated after 3 hours and more beyond this. The DD analysis indicate that all the nodes continue receiving water.

From the results of the simple example, it is clear that the DD approach gives a distorted picture of the performance of the network during failures. It is demonstrated here that while using the DD approach certain nodes are shown to be still receiving their allocated quantum of water during the failure but the PDD approach indicates that these nodes are in a failed state. Clearly using a DD approach would give misleading results as far as reduced level of service predictions were concerned.

4 CONCLUSION

In this paper it was shown that during normal operating conditions both PDD and DD analysis of WDS give similar results. However when there is an extreme events like pipe burst or failure, predictions of conventional DD analysis and PDD analysis differ considerably. Results from the analysis show that a burst on pipe creates pressure deficiency at demand nodes. The effect of the pressure deficiency on the demand (flow) is in DD formulation where the results indicate that the demand at nodes are satisfied even at very low pressures. However pressure dependent analysis indicates that 39% of the demand nodes experience failure and reduced flow. Further more the remaining demand nodes operate in a reduced mode due to the reduced flows in to the nodes. Hence the demands at the nodes are not being satisfied. Therefore in extreme (low) pressure situations specially during failure/risk analysis pressure dependent formulation gives more realistic results.

REFERENCES

Akinpelu, E.O. 2001. Development of Appropriate Demand Modeling Procedure for Water Distribution Systems in Developing Countries. Thesis Presented to the South Bank University, at London, UK in partial fulfillment of the requirement for the degree of Doctor of Philosophy.

Bhave, P.R. 1981. Node Flow Analysis of Water Distribution Systems. *Journal of Transportation Engineering (ASCE)*, **107**(TE4): 457–467.

Field, D.B. 1978. Effects of Domestic Storage on Demand Patterns. *WRC Technical Report*, Stevenage Laboratory.

Germanopoulos, 1988. Modelling and Operational Control of Water Supply Networks. Thesis Presented to the University of London, at London, UK in partial fulfillment of the requirement for the degree of Doctor of Philosophy.

Gupta, R. and Bhave, P.R. 1994. Reliability Analysis of Water Distribution Systems. *Journal of Environmental Engineering(ASCE)*, **120**(2): 447–460.

Gupta, R. and Bhave, P.R. 1996. Comparison of Methods for Predicting Deficient Network Performance. *Journal of Water Resources Planning and Management(ASCE*), **122**(3): 214–217.

Vairavamoorthy, K. and Lumbers J. 1998. Leakage Reduction in Water Distribution Systems: Optimal Valve Control. *Journal of Hydr Engineering (ASCE)*, 112(11):1146–1153.

Reddy, S.L. 1990. Analysis and Design of Pipe Network with Head Dependent Outflows. Thesis Presented to the Indian Institute of Technology, Madras, at Madras, India in partial fulfillment of the requirement for the degree of Doctor of Philosophy.

Tanyimboh, T.T., Tahar, B. and Templeman, A.B. 2002. Pressure-Driven Modelling of water distribution systems. *IWA 3rd World Water Congress*, Melbourne, Australia.

Tanyimboh, T.T., Tabesh, M. et al. 2001. Appraisal of Source Head Methods for Calculating Reliability of Water Distribution Networks. *Journal of Water Resources Planning and Management(ASCE)*, **127**(4).

Tahar, B., Taniyimboh, T. and Templeman, A.B. 2002. Pressure-Dependent Modelling of water distribution systems. *DMinUCE 3rd International Conference*, London.

Vairavamoorthy, K. 1994. Water Distribution Networks: Design and Control of Intermittent Supply. Thesis Presented to the University of London, at London, UK in partial fulfillment of the requirement for the degree of Doctor of Philosophy.

Wagner, J.M. and Shamir, U. et al. 1988. Water Distribution Reliability. *Journal of Water Resources Planning and Management(ASCE)*, **114**(3): 276–294.

Deterioration evaluation model of pipelines based on field examination survey in water distribution system

Ju-Hwan Kim, Hyoungmin Woo, Hyo-Won Ahn & Woo-Gu Kim
Water Resources Research Institute, Korea Water Resources Corporation, Taejon, South Korea

ABSTRACT: The field investigation was performed to improve the evaluation process of pipe deterioration degree, which can be used as the criteria index of pipelines' replacement and rehabilitation in water distribution system. To evaluate the deterioration degree of pipelines, accompanied by the field investigation, the data of pipe-related facilities and maintenance history, pipe material, soil types for installation and water qualities etc. are surveyed and collected. The field survey results cover thirty-one local sites under the inspection of 10 Korea water resources corporation (KOWACO) local water works branches. All the variables which is used by conventional models were considered and selected to find out the determination stage of influential factors on the deterioration of water pipelines. The way to find those factors can be divided into two categories as direct and indirect method. Direct method for the evaluation of the deterioration degree of water pipeline requires excavation at laying sites to collect pre-determined factors. While, deterioration degree can be guessed or predicted roughly by indirect method using pipe history, soil condition data, and so on. Factors can be modified and corrected by the field inspectors. Also, experts in the area of pipeline management works can judge the deterioration degree of pipeline on their old experiences and accumulated know-how. In this paper, four kinds of models are developed and compared with the previous models as on the field survey data. It shows that the developed model can predict well the deterioration degree of pipelines and good results with the improvement and consistency within tolerable acceptance limits. The proposed model can give user reasonable data to make a decision of rehabilitate/replace or not.

1 INTRODUCTION

The deterioration of water pipe can lead to increased leaks, increased maintenance and repair costs, and undesirable water quality changes in supply systems. Pipe accident by those reasons can lead to not only a suspension of water supply but also losses of water and finance. Decisions on whether to rehabilitate or replace a segment of water main must be based on the most accurate information on the condition of the system, cost, and anticipated benefits. Many researches have been performed to reduce the pipe breakage by the evaluation or prediction of pipe's physical condition, installation circumstances and water quality. However, it is very difficult to confirm and verify the degree of pipe deterioration since most pipes are installed underground.

Many factors, which influence on pipe deterioration, have been considered to evaluate the pipe condition indirectly without excavation of installation sites. This is a reason why models should be developed to access the pipe deterioration of water supply systems by indirect factors. Especially, inner and outer corrosion of pipe is one of important factors to the deterioration. The corrosiveness water quality in pipe and the characteristics of soil around pipe installation site are generally included in addition to internal and external condition of buried pipes. Also, hydraulic condition, maintenance/operation history including leakage and breakage, have to be considered. This is a complex task, requiring knowledge of past performance and future forecasts. Based on the evaluation of old pipe by the model, decision can be made whether the pipes have to be replaced or rehabilitated. Timing of rehabilitation or replacement also is all important, as financial implications.

Shamir and Howard (1979) were the first to develop pipe replacement criteria based on economic analysis of pipe replacement cost versus pipe break costs. A procedure proposed to determine the optimal replacement time for pipes in water distribution systems based on the prediction of break ratio and the failure rate of pipe. Similar evaluation methods were applied by the US Army Corps of engineering District (1981), Walski and Pelliccia (1982), O'Day (1982) and the City of Philadelphia (Weiss et al 1985). Based on the results, AWWA Research Foundation (AWWARF) and the U.S. Environmental Protection Agency identify the factors that are important to water main failure and rehabilitation/replacement decisions in 1986. Also, the correlations between internal corrosion levels and water quality deterioration are surveyed and a rehabilitation planning system is developed in the report. Evaluation of leak detection and repair programs were made by Boyle Engineering (1982) and Moyer et al (1983). Walski (1982, 1984) proposed methods for making pipe cleaning and lining decisions. In a manual prepared for the US Department of Housing and Urban Development (1984), Maddaus et al provided overall guidance for making water distribution system rehabilitation decisions. In 1995, Korea Water resources Corporation (KOWACO) reported that a numerical weighting model was developed, which proportionally score the effect of each factor to the deterioration rate depending on the condition of pipes. The model sets a hundred point for a new pipe and adds some points for each affecting factor according to current condition. Decision can be made that pipes scored totally less than 60 should be replaced in the application of the model.

2 METHODS FOR EVALUATION OF DETERIORATION

It is very difficult to determine the underlying causes of water pipe failure due to the complex interrelationship of the many factors involved in the deterioration process. The condition of water distribution system can vary dramatically by pipe materials, or geographic region. Even the condition of an individual pipe segment may vary due to localized corrosion, bedding or casting problems. The following data on pipe deterioration can be summarized.

- Pipe characteristics
- Environmental data
- Leak/break and damage data
- Corrosion
- Repair data
- Excavation and paving data
- Other

3 IMPROVED PROCEDURE OF EVALUATION

The objective of this study is to improve the evaluation method of water pipe installed underground. If the conditions of buried pipe are investigated directly, suspension of water service and excavation is accompanied. However, it costs much time, money and causes customer's complaint. Accordingly, three steps are proposed to evaluation of deteriorated pipe. In the first step, indirect factors are selected which can be accessed by surveying circumstances and conditions in pipe-buried region without excavation. Secondly, direct factors are classified, which can be obtained by direct measure of inner and exterior conditions of the pipe body and buried circumstances through a excavation in pipe buried site, water service suspension and cutting a pipe body. Detailed conditions can be estimated in this step. There are complex causes and effects to deteriorate buried pipes. More detailed investigation is carried out in the third step. It performs to clarify the causes

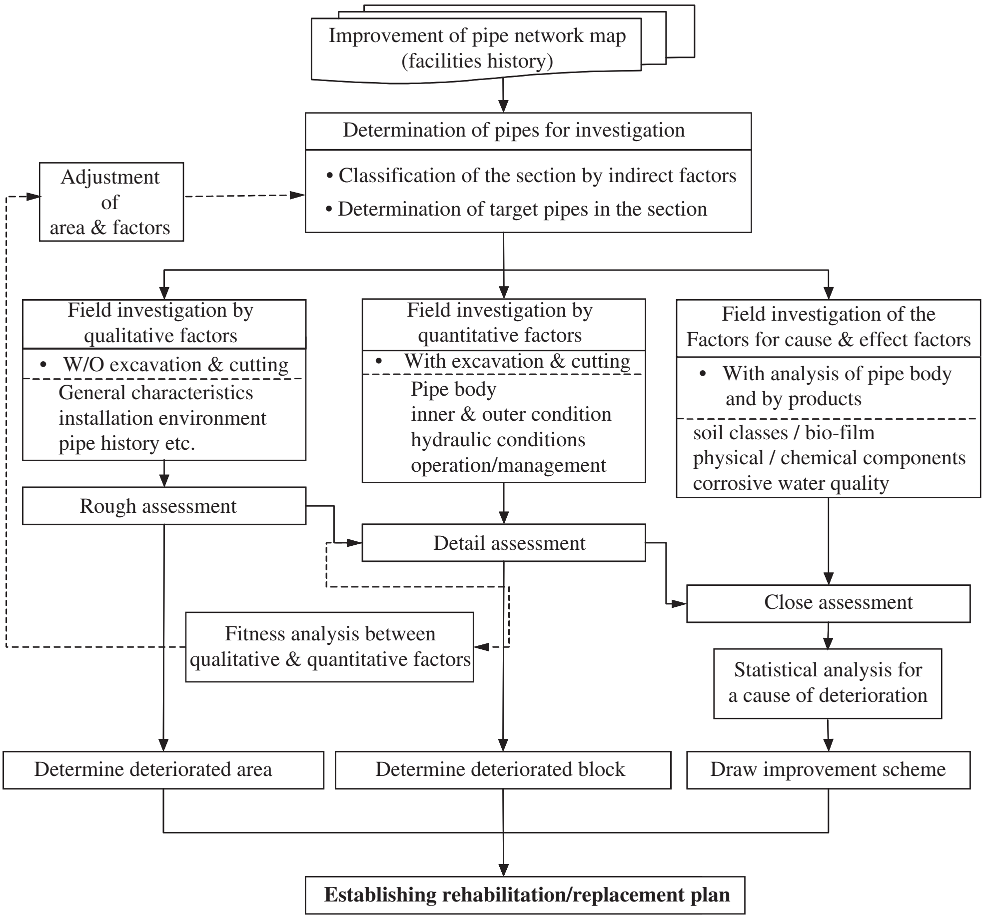

Figure 1. Evaluation process of deteriorated pipes.

Table 1. Deterioration weight of indirect factors for site01–site05.

No.	Detail factor	Category	Weight	Site01	Site02	Site03	Site04	Site05
1	Transferred water	Purified water	1.00	1.00	1.00	1.00	1.00	1.00
		Raw water	0.75					
		Distribution line	0.75					
		Service line	0.50					
2	Pipe material	STS, PFP	1.00	0.00	0.00	0.75	0.75	0.75
		DCIP	0.75					
		SP, PC, PCC	0.50					
		PVC, PE	0.25					
		CIP, GSP	0.00					
3	Diameter	Over 2000 mm	1.00	0.60	0.40	0.60	0.40	0.40
		2000–1000 mm	0.80					
		1000–600 mm	0.60					
		600–350 mm	0.40					
		350–150 mm	0.20					
		below 150 mm	0.00					

(*continued*)

Table 1. (*continued*).

No.	Detail factor	Category	Weight	Site01	Site02	Site03	Site04	Site05
4	Inner clothing	Cement mortar	1.00	0.00	0.00	0.00	0.00	0.00
		Asphalt clothing	0.75					
		Coal–tar enamel	0.75					
		Epoxy	0.50					
		No clothing	0.00					
5	Outer clothing	Asphalt clothing	1.00	–	–	–	–	0.00
		Coal–tar enamel	0.75					
		No clothing	0.00					
6	Age	Over 25 year	0.00	0.00	0.50	0.25	0.00	0.25
		25–20 year	0.25					
		20–15 year	0.50					
		15–10 year	0.75					
		Below 10 year	1.00					
7	Installation depth	Over 2 m	0.50	1.00	0.50	1.00	1.00	1.00
		2–1 m	1.00					
		Below 1 m	0.50					
8	Soil type	Sand	1.00	0.50	0.50	0.50	1.00	–
		Sand + gravel, loam,	0.50					
		Clay + gravel, slit	0.25					
		Clay	0.00					
9	Road environment	Express, high way	0.00	0.25	0.25	0.25	0.25	0.25
		Over 4-lane	0.25					
		2-lane	0.50					
		Alley	0.75					
		Footway	1.00					
10	Foundation	Compacted bedding	1.00	1.00	1.00	1.00	0.75	–
		Sand bedding	0.75					
		Concrete bedding	0.50					
		Pile foundation	0.50					
		No foundation	0.00					
11	Electric corrosion	No	1.00	–	–	–	–	–
		Yes	0.00					
12	Joint type	Clothing after welding	1.00	1.00	1.00	1.00	1.00	1.00
		Mechnical, Push-on J.	1.00					
		Thermal fusion	0.50					
		Flange, soclet	0.25					
		Sleeve coupling	0.25					
13	Number of valve, junction pipe	Over average	1.00	1.00	1.00	1.00	1.00	1.00
		Average	0.50					
		Below average	0.00					
14	Joint	No	1.00	1.00	1.00	1.00	1.00	1.00
		Yes	0.00					
15	Leakage/ breakage history	No	1.00	0.50	0.50	0.50	0.50	0.50
		1–3 times/5 yr-50 km	0.50					
		3–5times/5 yr-50 km	0.25					
		Over 5times/5 yr-50 km	0.00					
16	Customer's complaint	No	1.00	–	–	–	–	–
		1–3 times/5 yr	0.50					
		3–5 times/5 yr	0.25					
		Over 5times/5 yr	0.00					

Table 2. Deterioration weight of direct factors for site01–site05.

No.	Detail factor	Range	Weight	Site01	Site02	Site03	Site04	Site05
1	Inner diameter	Over 4%	0.00	–	–	–	–	1.00
		4–3%	0.25					
		3–2%	0.50					
		2–1%	0.75					
		Below 1%	1.00					
2	Thickness	Over 10%	0.00	0.0	1.00	1.00	0.25	0.25
		10–7.5%	0.25					
		7.5–5%	0.50					
		5–2.5%	0.75					
		Below 2.5%	1.00					
3	Clothing thickness	Over 10%	0.00	–	–	–	–	–
		10–7.5%	0.25					
		7.5–5%	0.50					
		5–2.5%	0.75					
		Below 2.5%	1.00					
4	Area of exterior corrosion	Over 80%	0.00	0.50	0.50	1.00	1.00	1.00
		80– 60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
5	Depth of exterior corrosion	Over 20%	0.00	0.00	1.00	0.75	0.50	1.00
		20–15%	0.25					
		15–10%	0.50					
		10–5%	0.75					
		Below 5%	1.00					
6	Circumference of exeterior corrosion	Over 80%	0.00	1.00	1.00	1.00	1.00	1.00
		80–60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
7	Fissure size	Over 0.0 mm	0.00	1.00	1.00	1.00	1.00	1.00
		0.0 mm	1.00					
8	Depth of pin-hole	Over 20%	0.00	1.00	1.00	1.00	1.00	1.00
		20–15%	0.25					
		15–10%	0.50					
		10–5%	0.75					
		Below 5%	1.00					
9	Exfoliation of outer clothing	Over 80%	0.00	–	–	–	–	–
		80–60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
10	Scale area	Over 80%	0.00	0.75	0.25	0.00	0.25	0.00
		80–60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
11	Scale thickness	Over 20 mm	0.00	0.00	0.00	0.00	0.00	0.00
		20–15 mm	0.25					
		15–10 mm	0.50					
		10–5 mm	0.75					
		Below 5 mm	1.00					
12	Sludge area	100%	1.00	1.00	1.00	1.00	1.00	1.00
		100–75%	0.25					

(*continued*)

Table 2. (*continued*).

No.	Detail factor	Range	Weight	Site01	Site02	Site03	Site04	Site05
		75–50%	0.50					
		50–25%	0.75					
		Below 25%	1.00					
13	Sludge thickness	Over 4 mm	0.00	1.00	1.00	1.00	1.00	1.00
		4–3 mm	0.25					
		3–2 mm	0.50					
		2–1 mm	0.75					
		Below1 mm	1.00					
14	Depth of inner corrosion	Over 20%	0.00	0.00	0.25	0.00	0.00	0.25
		20–15%	0.25					
		15–10%	0.50					
		10–5%	0.75					
		Below 5%	1.00					
15	Circumference of inner corrosion	Over 80%	0.00	0.75	1.00	0.25	0.75	1.00
		80–60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
16	CML neutralization (concrete neutralization)	Over 80%	0.00	–	–	–	–	–
		80–60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
17	CML damage	Over 80%	0.00	–	–	–	–	–
		80–60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
18	Exfoliation of inner clothing	Over 80%	0.00	–	–	–	–	–
		80–60%	0.25					
		60–40%	0.50					
		40–20%	0.75					
		Below 20%	1.00					
19	H-W C value (roughness)	Below 60	0.00	0.25	0.50	0.50	0.50	0.25
		60–80	0.25					
		80–100	0.50					
		100–120	0.75					
		Over 120	1.00					
20	Hydraulic pressure	Over 7.0 kg	0.00	1.00	–	–	–	1.00
		7.0–6.0 kg	0.25					
		6.0–5.0 kg	0.50					
		5.0–4.0 kg	0.75					
		Below 4.0 kg	1.00					
21	Inhabitation of clam	Group	0.00	1.00	1.00	1.00	1.00	1.00
		Separate	0.50					
		No	1.00					
22	Mud accumulation on pipe floor	Over 3 cm	0.00	1.00	1.00	1.00	1.00	1.00
		3.0–0.5 cm	0.50					
		Below 0.5 cm	1.00					
23	Operative condition of valves	No work	0.00	1.00	1.00	1.00	–	–
		Not enough	0.50					
		Good work	1.00					
24	Penetration of ground water	Yes	0.00	1.00	1.00	1.00	1.00	1.00
		No	1.00					

of deterioration of pipes with physical and chemical analysis of the samples which is collected in the field where the rehabilitation and replacement are worked.

4 MODEL DEVELOPMENT

Four models are developed to estimate the pipe deterioration degree. The models can be used when rehabilitation/replcament plan should be established without any detail information of objective region, such as inner state of the pipes since they are installed underground. The relationship between indirect factor and direct factors make it possible to estimate the overall state of pipes just using indirect factors. Although it is desirable to estimate deterioration degree by direct factors, field situation is not often allowed to carry out direct investigation and experiment. Direct factors can be measured through excavation and cutting pipes during the water service are suspended.

Thirty one sites are examined and the data are collected the observations of 16 indirect factors and 24 direct factors. Based on the observations, data of sixteen sites are selected. Multi-regression models are assumed as follows;

$$\text{Model} \;: y = a_1 \cdot x_1 + a_2 \cdot x_2 + \cdots + a_8 \cdot x_8 + a_9 \tag{1}$$

where y is deterioration degree(weight) by direct factors and x_1, x_2, …, x_7, x_8 express the weight scores for each variable, transferred water, pipe material, diameter, inner clothing, age, soil class, road environment and leak/breakage record. Independent variables can be collected easily without excavation, pipe cutting and water service suspension.

$$\text{Model} \;: y = a_1 \cdot x_1 + a_2 \cdot x_2 + \cdots + a_7 \cdot x_7 + a_9 \tag{2}$$

where x_1, x_2, …, x_7 express the weight scores for each variable, transferred water, pipe material, inner clothing, age, soil class, road environment and leak/breakage record.

$$\text{Model} \;: y = a_1 \cdot x_1 + a_2 \cdot x_2 + \cdots + a_7 \cdot x_7 + a_9 \tag{3}$$

where x_1, x_2, …, x_7 express the weight scores for each variable, transferred water, pipe material, diameter, age, soil class, road environment and leak/breakage record.

$$\text{Model} \;: y = a_1 \cdot x_1 + a_2 \cdot x_2 + \cdots + a_6 \cdot x_6 + a_9 \tag{4}$$

where x_1, x_2, …, x_6 express the weight scores for each variable, transferred water, pipe material, age, soil class, road environment and leak/breakage record.

The results of parameter estimation for each model can be shown in Table 3. The models are compared with used data in Fig. 2 through Fig. 5. Also verifications are performed with these models using data surveyed in different years. The results can be seen in Table 4.

5 CONCLUSIONS

The purpose of this study is to improve the evaluation process of deteriorated water pipes in for planning the replacement and rehabilitation in Korea. Final results is an annual rehabilitation plan which is not produced by a black-box decision support system, but by a decision help with the arguments of the decision maker.

Criteria and factors are established and three steps are proposed to evaluate the condition of deteriorated pipes by the classifying the factors, which are the indirect, indirect methods and cause

Table 3. Estimated parameters of multi-regression models.

Variables	Parameter coefficients Model I	Model II	Model III	Model IV
Transferred water	−0.1718	−0.1187	−0.1187	−0.1027
Pipe material	0.0341	0.0361	0.0361	0.0271
Diameter	−0.0458	–	−0.0184	–
Inner clothing	−0.0154	−0.0184	–	–
Age	0.2580	0.2620	0.2620	0.2490
Soil class	0.0514	0.0582	0.0582	0.0602
Road environment	0.0206	0.0133	0.0133	0.0058
Leak/breakage History	−0.0848	−0.1006	−0.1006	−0.0940
Constant	0.8575	0.7866	0.7866	0.7710
Determination Coeff.	0.7747	0.7661	0.7661	0.7633

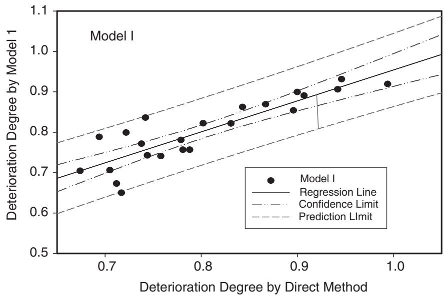

Figure 2. Estimation results by model I.

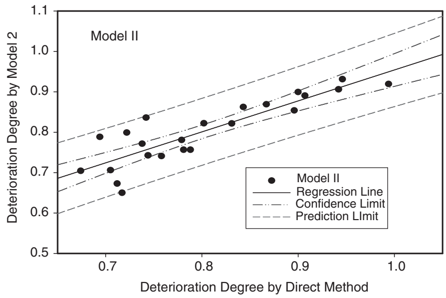

Figure 3. Estimation results by model II.

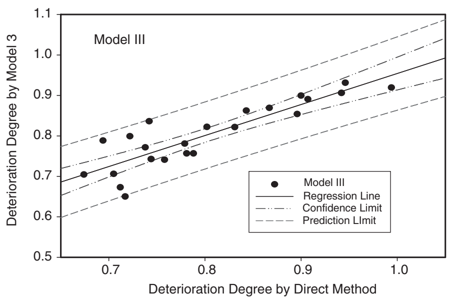

Figure 4. Estimation results by model III.

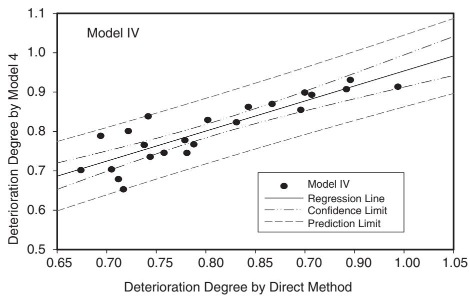

Figure 5. Estimation results by model IV.

analysis method. Four models are developed by considering the relationship between indirect and direct factors, which are obtained by the field investigation at the 31 sites where old pipes are buried. The proposed model and variables can be modified by field practitioners or experts by considering the contribution to the deterioration state of each pipe in the system and field characteristics. However, it is certain that the proposed process and model can play a role to set up the planning the replacement and rehabilitation of deteriorated water pipe by quantifying the factors, which must be an influence on the deterioration of water pipes.

Table 4. Verification results for each model.

Index	Result	Model I	Model II	Model III	Model IV
Site 1	Replace or rehabilitate	0.647	0.650	0.639	0.653
Site 2	Rehabilitate	0.785	0.781	0.774	0.792
Site 3	Replace or rehabilitate	0.737	0.743	0.732	0.743
Site 4	Replace or rehabilitate	0.707	0.706	0.699	0.703
Site 5	Replace or rehabilitate	0.772	0.772	0.764	0.773
Site 6	Near good	0.850	0.841	0.837	0.844
Site 7	Replace or rehabilitate	0.777	0.775	0.768	0.774
Site 8	Good	0.782	0.789	0.784	0.796
Site 9	Good	0.864	0.869	0.868	0.900
Site 10	Replace or rehabilitate	0.759	0.757	0.749	0.753
Site 11	Near good	0.678	0.673	0.672	0.678
Site 12	Good	0.861	0.863	0.862	0.885
Site 13	Good	0.851	0.854	0.857	0.877
Site 14	Good	0.820	0.822	0.824	0.846
Site 15	Good	0.920	0.906	0.921	0.937
Site 16	Review	0.724	0.741	0.740	0.753
Site 17	Good	0.942	0.931	0.946	0.961
Site 18	Good	0.888	0.900	0.894	0.929
Site 19	Good	0.921	0.920	0.919	0.936
Site 20	Replace or rehabilitate	0.722	0.704	0.701	0.709
Site 21	Good	0.833	0.836	0.839	0.861
Site 22	Good	0.886	0.891	0.898	0.915
Site 23	Good	0.786	0.799	0.798	0.816

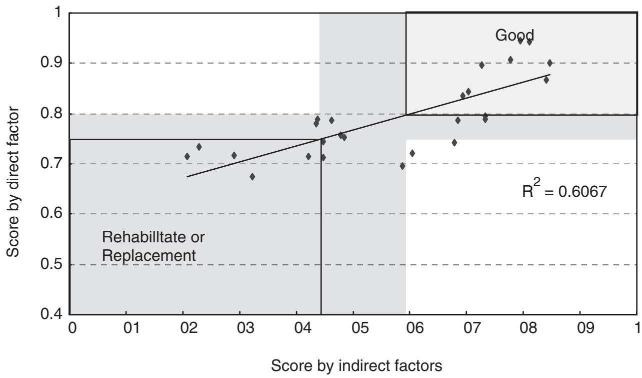

Figure 6. Relationship between direct and indirect deterioration.

REFERENCES

Deb, A.K., Snyder, J.K., Chelius, J.J., Urie, J. and O'Day D.K. "Assessment of existing and developing water main rehabilitation practices." AWWARF, 1990.

O'Day, D. Kelly, Guidance Manual: Water Main Evaluation for Rehabilitation/Replacement. AWWA Research Foundation, No. CR810988, 55–86, 1986.

Water Research Centre, Planning the Rehabilitation of Water Distribution System, 1989.

Prioritizing water mains rehabilitation under uncertainty

J.M. Yan & K. Vairavamoorthy
WEDC, Department of Civil and Building Engineering, Loughborough University, UK

ABSTRACT: A pro-active water mains rehabilitation approach is more cost-effective than a reactive one. However the pro-active rehabilitation requires a effective decision making tool to allow it to be cost-effective. Therefore, there is a need for decision making tool to prioritize water mains rehabilitation scheme. This paper presents a decision making tool for ranking the condition of pipe with available, inexpensive, but vague or imprecise data. Possibility approach has been used to account for the uncertainties inherent in pipe condition indicators, and distance based multiple-criteria decision making approach has been employed to combine the pipe condition indicators according to their similarities. The decision making tools developed that ranks the pipes in terms of their condition. Sensitivity of the model to weights and balance factors has been studied and it has shown that the proposed model is robust. It is anticipated that the developed model will assist engineers in prioritizing pipe rehabilitation scheme.

1 INTRODUCTION

Water distribution system (WDS) is an important component of water supply system, which conveys water to the consumer from the sources. These systems constitute a substantial proportion of the cost of a water supply system, in some cases as much as half the overall cost of the system. Meanwhile, water mains deteriorate both structurally and functionally over time. Aging of WDS is a global problem. For example, the water networks serving the utilities in Western Europe and North America are up to 150 years old (Sægrov et al. 1999). And it is estimated that 50% of all large diameter water mains in the 50 largest US cities are more than 50 years old (Summers 2001). USEPA emphasis that water pipes corrosion and ageing is one of most concerns related to water distribution networks that may pose public health (AWWSC 2002).

It is important to maintain the integrity of WDS to ensure the level of services. WDS rehabilitation (e.g. repairs, replacement) can mitigate the risk of pipe break, hydraulic capacity decrease, and water quality degradation. However, WDS rehabilitation is very costly, for example, it is estimated that it will cost US water utilities more than $325 billion to rehabilitate water pipes over next decade (Mcneill & Edwards 2001, Summers 2001). Therefore, the rehabilitation of WDS is often restricted by the scare capital resources of water utilities.

Water utilities have found from experience that pro-active approach is more cost-effective than a reactive one in pipe rehabilitation since that the reactive approach performs the rule "do nothing until a system component fails", which increases cost and leads to customer dissatisfaction and potential environmental problems (Loganathan et al. 2002). However, pro-active rehabilitation requires the assessment of current pipe condition and predication of future pipe break rates.

Water pipe condition is affected by pipe deterioration process that are complex because there are many factors which interactively contribute to such deterioration. These factors can be broadly categorized into three groups, i.e. physical factors (e.g. pipe age, diameter, length, material, etc.), environmental factors (e.g. soil corrosivity, internal and external loads, pipe location, etc.), and operational factors (e.g. break history, leak records, operation pressure, etc.) (AWWSC 2002). The condition of water pipe is the cumulative effects of these attributing factors. Some of these factors, such as the physical factors, are usually available in inventory database, which are express in crisp form except pipe material. However, there are many factors, such as environmental factors, are difficult to be quantified in a crisp way, but can be dealt with the use of possibility approach to account for the uncertainties.

In this paper, a pipe condition ranking model based on a fuzzy approach is presented. By using this model, the pipe condition can be evaluated with basic pipe condition indictors such as pipe age, pipe material, pipe diameter, soil condition, traffic loads, etc. The uncertainties inherent in these pipes condition indictors are described with fuzzy theory (Zadeh 1965). The first-level indictors are aggregated into two groups based on their similarities to form the second-level indictors. Similarly, the second-level pipe condition indictors are grouped to form the final indicator. Based on the hierarchical pipe condition structure established from the above aggregation process, fuzzy composite programming is used to compute an "indicator distance metric" for each indicator, and finally an "overall distance metric" was obtained. This final distance metric is used to evaluate and rank the condition of pipes. To demonstrate the robustness of the model to weights and balance factors required in this approach, a sensitivity analysis is performed.

2 PROPOSED METHODOLOGY

The proposed methodology uses multiple-criteria decision making (MCDM) techniques to combine the available, often completely different, pipe condition indictors into a final overall pipe condition indictor. The selected MCDM technique is fuzzy composite programming (FCP) which incorporates both fuzzy set theory and its arithmetic corollaries (Dubois & Prade 1988, Kaufmann & Gupta 1991). Application of FCP in MCDM to water resource and environment engineering problems includes those of Lee et al. (1991, 1992), Bardossy & Duckstein (1992), and Hagemeister et al. (1996).

More recently, Yan & Vairavamoorthy (2003) have applied FCP method for pipe condition assessment. However, it was stated in their paper that the application of FCP to pipe condition assessment may be sensitive to weights and balance factors used in the process. In this paper, a sensitive analysis is performed to demonstrated the impact of weights and balance factors on the output generated by the FCP method.

2.1 *Fuzzy composite programming*

Composite programming is a normalized multi-level based methodology that deals with problems of a hierarchical nature (i.e., when certain criteria contain a number of sub-criteria), was developed by Bardossy et al. (1985) from compromise programming. The latter technique, first developed by Zeleny (1973) is a mathematical programming technique that employs single level non-normalized distance based methodology to rank a discrete set of solution according to their distance from an ideal solution.

Composite programming applies Equation (1) to each sub-criterion within the same group, and then combines the compromise distance metrics of each sub-criterion to form a single composite distance metric. Then the process iterates with the successive levels until final level composite distance metric is reached (one composite distance metric for each alternative).

$$L_j = \left\{ \sum_{i=1}^{n_j} \left[w_{j,i}^{p_j} \left(\frac{f_{j,i} - f_{j,i}^{w}}{f_{j,i}^{b} - f_{j,i}^{w}} \right)^{p_j} \right] \right\}^{\frac{1}{p_j}} \tag{1}$$

Where L_j = distance metric of alternative j; $w_{j,i}$ = weight of indictor i in group j; p_j = balance factors among indicators for group j; $f_{j,i}^{b}$ = best value for indictor i in group j; $f_{j,i}^{w}$ = worst value for indictor i in group j; $f_{j,i}$ = actual value for indictor i in group j.

The addition of fuzzy set theory (Zadeh 1965) to compromise programming to represent uncertainties of indicators forms fuzzy compromise programming. Similar to normalized multilevel composite programming, fuzzy compromise programming can also be extended to normalized multi-level distance based methodology to account for uncertainties, which also known as fuzzy composite programming (FCP).

The uncertainties inherent in the indicators are accounted for with the use of possibilities approach. Fuzzy compromise programming is extended to a normalized multi-level distance based methodology with the use of best and worst first-level indictor values (Hagemeister et al. 1996)

$$S_{j,i} = \frac{f_{j,i} - f_{j,i}^{w}}{f_{j,i}^{b} - f_{j,i}^{w}} \tag{2}$$

where $S_{j,i}$ = normalized ith fuzzy indictor of group j. The normalization formula given above can have different form depend on whether the maximum is the "best" or "worst" value.

It should be noted that this normalization process will result in the coordinate (1, 1) to be the ideal point. Substitution of Equation (2) into Equation (1), and applied to n_j indicators of group j, yields the following composite distance for jth group of indictors. The composite distance, L_j, is the distance between the actual point of indicators and the ideal one (Woldt & Bogardi 1992):

$$L_j = \left[\sum_{i=1}^{n_j} w_{j,i}^{p_j} S_{j,i}^{p_j} \right]^{1/p_j} \tag{3}$$

Therefore Equation (3) combines the normalized first-level indictors to obtain their respective second-level composite distance. The process of computing successive levels of composite distances is repeated with previous level composite distances, L_j, being substituted in place of variable $S_{j,i}$ until the final composite distance is reached for the system. In the case of pipe condition, this final-level indicator is the combination between physical factor and environment factor.

2.2 *Weights and balance factors*

Prior to examining alternatives, the decision maker (DM) must assign weights to indicate their preferences to the relative importance of indicators in the same group. The method of assigning weights to indicator is not typically defined or thoroughly documented. Most of the applications of FCP method mentioned above use crisp numbers to express weights according to the judgment of DM except that of Lee et al. (1991, 1992), who used the Analytic Hierarchy Process (AHP).

The DM is also required to determine balance factors in order to evaluate alternatives using FCP. Balance factor determine the degree of compromise between indicators of the same group. Low balance factors are used for a high level of allowable compromise among indicators of the same group. Balance factor of 1 suggests that there is a perfect compromise between indicators of the group. If the level of compromise between indicators is moderate, a balance factor of 2 will be sufficient. A balance factor of 3 or bigger indicates that there is minimal compromise between indicators (Jones & Barnes 2000).

Clearly the FCP approach is depended on weights and balance factors used. However, if it can be shown that while the magnitude of the outputs change with changing factors, the ranking order of the alternatives (i.e. pipes in this case) does not, then the impact of weights and balance factors on decision making will be minimized. To observe the influence of different weights and balance factors on final fuzzy outputs, five weights and balance factors scenarios are used in this paper. These five groups of weights and balance factors are applied to perform sensitivity analysis on final indicator fuzzy result.

3 APPLICATION TO WATER MAINS REHABILITATION

The FCP approach presented herein can be applied effectively for prioritizing water mains rehabilitation scheme. To demonstrate the application of FCP to prioritize water mains rehabilitation, ten pipe condition indicators, as shown in Table 1 are selected (there is no limitation to the number of indicators used). We use the composite structure shown in Figure 1 to combine the different

Table 1. First-level indicators and the definitions.

Level 1indicators (1)	Indicators description (2)
Physical indicators	
Pipe diameter	Crisp value, the design diameter
Pipe age	Crisp value, year that the pipe was laid
Pipe material	Fuzzy number combining maximum pressure and corrosion resistance
Pipe length	Crisp value, the design length
Pipe joint	Fuzzy number representing the impact of joint type to condition of pipe
Environmental indicators	
Road loading	Linguistic variable used to describe traffic density
Soil condition	Fuzzy number expressing the corrosivity of soil
Location	Linguistic variable represent pipe surroundings conditions
Bedding condition	Fuzzy number describes the magnitude of impact of soil to condition of pipe
Buried depth	Crisp value, the depth from ground to upper pipe wall

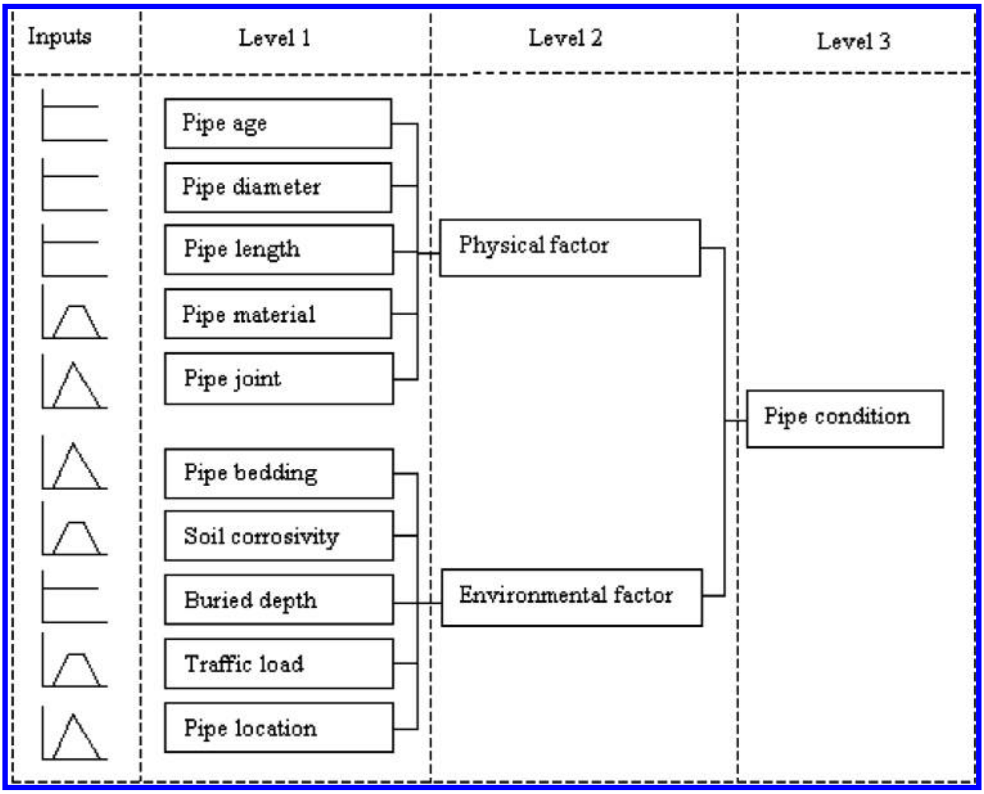

Figure 1. Pipe condition assessment composite structure.

level indicators successively to produce the final level indicators. Both crisp and fuzzy numbers have been used to account for the respective properties of different indicators, as shown in column 1 of Figure 1. In the following sections the composite structure used in this paper will be first presented. Followed by the uncertainties inherent in basic pipe condition indicators. Then a sensitivity analysis to weights and balance factors on the final fuzzy result will be presented. Finally the results and comments on sensitivity analysis will be given.

3.1 *Composite structure*

The hierarchical structure of FCP provides a process of integrating different types of information into a single indictor that can gain deeper understanding of the interrelationships between numerous pipe condition indictors. Figure 1 gives the composite hierarchical structure used in pipe condition ranking system in this paper. This structure is developed in a way so that known or relatively easily obtained information can be used to produce the first-level indicators.

The FCP hierarchical structure is used to combine first-level indicators into second-level fuzzy indicators etc. The aggregation process continues until the final-level fuzzy indicator is achieved. The finial indicator is a fuzzy number due to imprecise first-level indictors. There are many factors that attribute to the deterioration. To illustrative this, in this paper, we select ten first-level indicators (Table 1), which can be broadly divided into two groups, i.e., physical indicators and environmental indicators. It should be noted that more indicators could be added into this composite structure if more information is available (e.g. operational factor, such as breakage history, operation pressure, leakage recorder, water quality, etc.).

3.2 *Uncertainties of basic indicators*

Among the ten selected pipe condition indicators, many of them are difficult to be expressed in crisp form, for example soil corrosivity, pipe material, pipe bedding condition, pipe joint method, which involve vague and imprecise information. In addition to vagueness, some information such as traffic loads and pipe location are expressed linguistically. Such vague or imprecise and linguistic information can be dealt with possibility theory (Zadeh 1965). The possibility theory uses fuzzy numbers to represent parameter uncertainty. A fuzzy number describes the relationship between an uncertainty quantity X and a membership function μ. The function, which comprises between zero and one, describes the possibility that the quantity X may take on a certain value x. In this paper, both triangular fuzzy number (Fig. 2(a)) and trapezoidal fuzzy number (Fig. 2(b)) have been used to account for the uncertainties inherent in pipe condition indicators.

To represent the importance of pipe material, two surrogate measures are used, namely, maximum pressure and corrosion resistance, to indicate the difference impact of pipe material on pipe condition. The comparisons of pipe material in terms of these two properties are given in Table 2 (Mays 2000). The maximum pressure reflects the strength of pipe material and express in crisp form, while the corrosion resistance that implies the capacity of pipe material to internal and external loads is given in linguistic form whose fuzzy description as shown in Figure 3 (left). The two pipe material indicators are combined using appropriate weights to derive a single pipe material indicator, as given in Figure 3 (right).

Equation (2) is used to normalize pipe condition indicators. The maximum and minimum values for normalization can be obtained from the design criteria or can simply be obtained by comparing the values of all alternatives (i.e. pipes in this case) for each indicator. In this paper, the maximum and minimum values of each indicator are obtained by comparing all the value sets of each indicator, as shown in Table 3. For example, the maximum pressure of pipe material is normalized with the maximum and minimum values shown in Table 3, and the normalized maximum pressure is combined with corrosion resistance to produce a single pipe material indicator.

Fuzzy numbers were used to convert the linguistic variable to numerical form. Fuzzy numbers representing pipe location and traffic loads are shown in Figure 4. The values of the ten indicators for selected 5 pipes are given in Table 4.

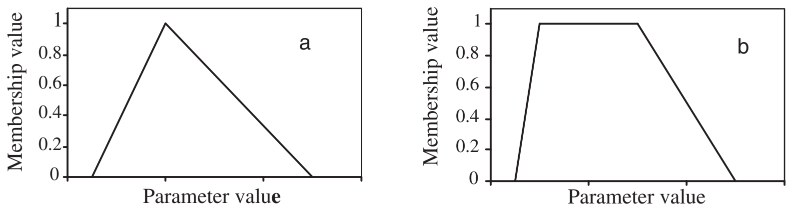

Figure 2. Two representations of fuzzy number (a) Triangular (b) Trapezoidal.

Table 2. Comparison of pipe material.

Type of pipe of different material (1)	Maximum pressure (kPa) (2)	Corrosion resistance (3)
Steel pipe (ST)	17000	Medium
Ductile iron pipe (DI)	2400	Good
Cast iron pipe (CI)	1300	Medium
Polyvinyl chloride (PVC)	2400	Strong
High density polyethylene (HDP)	1750	
Reinforced concrete pressure pipe (RCPP)	1380	Weak
Asbestos cement pipe (AC)	1380	

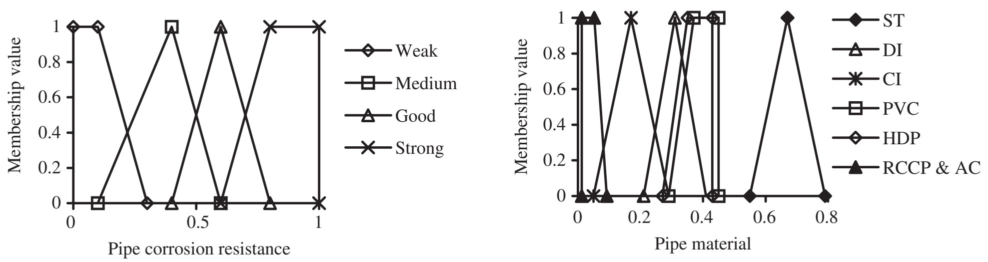

Figure 3. Fuzzy description of the pipe material. Left: corrosion resistance; right: pipe material indicator. (Maximum pressure weights 0.6, corrosion resistance weights 0.4).

Table 3. Best and worst indicators value.

Indicators (1)	Best values (2)	Worst values (3)
Pipe age (year of construction)	2000	1900
Pipe diameter (mm)	2000	50
Pipe length (m)	50	2000
Pipe material	1	0
Pipe joint	1	0
Road Loading (cars/min)	0	100
Soil condition	50000	0
Location	1	0
Pipe bedding	1	0
Buried depth (m)	1	10
Maximum pressure (kPa)	20000	1000
Corrosion resistance	1	0

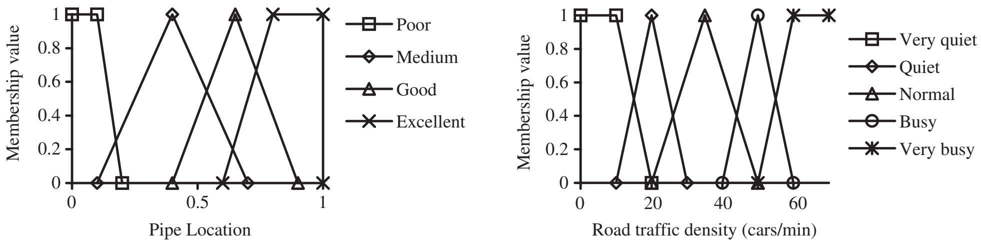

Figure 4. Fuzzy description of linguistic variable. Left: pipe location; right: road loads.

Table 4. Values of first-level indicators for illustrative case study.

	Values of pipe condition indicator				
First level indicators	Pipe 1	Pipe 2	Pipe 3	Pipe 4	Pipe 5
Pipe diameter (mm)	400	300	300	600	500
Pipe age	1953	1964	1978	1988	1992
Pipe material	CI	CI	DI	ST	PVC
($\times 10^{-1}$)	(0.5, 1.7, 2.9)	(0.5, 1.7, 2.9)	(2.1, 3.1, 4.1)	(5.5, 6.7, 7.9)	(2.9, 3.7, 4.5, 4.5)
Pipe length (m)	600	400	800	400	300
Pipe joint	lead	leadite	rubber	rubber	rubber
($\times 10^{-1}$)	(3, 5, 7)	(0, 0, 2, 4)	(6, 8,10, 10)	(6, 8, 10, 10)	(6, 8, 10, 10)
Traffic loads	very quite	very busy	busy	normal	very busy
Soil condition	clay	gravel	caly	sand	sand
($\times 10^{3}$) (ohm $\cdot$ cm)	(0, 0, 3, 5)	(7, 10, 15)	(0, 0, 3, 5)	(7, 10, 15)	(3, 6, 10)
Location	poor	medium	excellent	excellent	good
Bedding condition	clay	gravel	clay	sand	sand
($\times 10^{-1}$)	(6, 8, 10, 10)	(3, 5, 7)	(6, 8, 10, 10)	(0, 0, 2, 4)	(0, 0, 2, 4)
Buried depth (m)	2.5	2.0	1.8	1.2	1.5

3.3 *Sensitivity analysis of weights and balance factors*

This section is designed to demonstrate the robustness of the FCP approach in prioritizing water mains rehabilitation. Five weights and balance factors scenarios were chosen to analyze the sensitivity of final fuzzy result and the ranking order of pipe condition to different perception of DM.

The five weights and balance factors scenarios shown in Table 5 were generated using a combination of engineering judgment and consultation with practitioners. The five weight and balance factor scenarios were applied to the pipes condition ranking system iteratively. Furthermore, the ten indicators' hierarchical structure, as shown in Figure 1, was applied iteratively to each pipe to be ranked. It should notice that all the pipes to be ranked have identical weights and balance factors for each scenario.

3.4 *Results and comments on sensitivity analysis*

The results of sensitivity analysis on five scenarios of weights and balance factors were reported both for each pipe (Figs 5& 6) and for each scenario (Table 7). As an example, the results of sensitivity analysis for pipe no. 4 are shown in Table 6 and Figure 5. Table 6 gives the most likely ranges (fuzzy number membership value equal to 1) and largest likely ranges (fuzzy number membership

Table 5. Weights and balance factor for 5 trials.

Pipe condition indicators	Weights					Balance factors				
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Level 1 indicators										
Pipe diameter	0.2	0.3	0.2	0.25	0.2	1	2	1	2	1
Pipe material	0.3	0.2	0.4	0.2	0.25	1	2	1	2	1
Pipe age	0.3	0.3	0.2	0.4	0.3	1	2	1	2	1
Pipe length	0.1	0.1	0.1	0.1	0.15	1	2	1	2	1
Pipe joint	0.1	0.1	0.1	0.15	1	1	2	1	2	1
Traffic Loads	0.2	0.2	0.15	0.2	0.15	1	1	2	2	1
Soil condition	0.3	0.2	0.2	0.15	0.3	1	1	2	2	1
Pipe location	0.1	0.3	0.3	0.2	0.2	1	1	2	2	1
Bed condition	0.2	0.2	0.2	0.15	0.25	1	1	2	2	1
Buried depth	0.2	0.1	0.15	0.3	0.1	1	1	2	2	1
Level 2 indicators										
Physical	0.5	0.6	0.8	0.6	0.7	1	2	3	4	2
Environmental	0.5	0.4	0.2	0.4	0.3	1	2	3	4	2

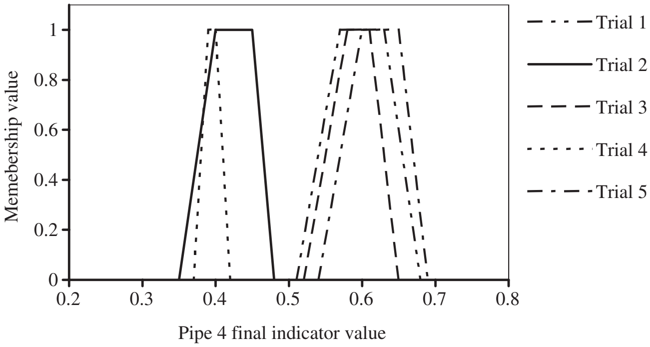

Figure 5. Final fuzzy results of pipe no. 4 for five weights and balance factors scenarios.

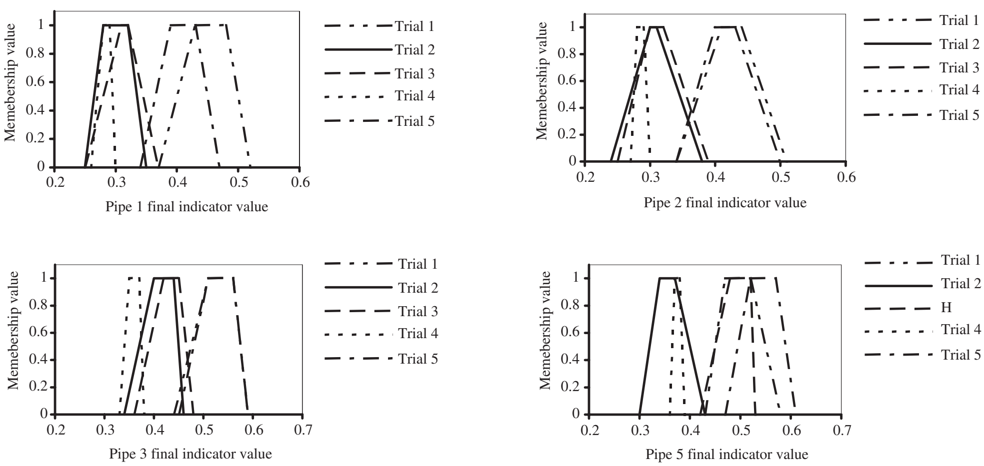

Figure 6. Final fuzzy results of pipes no. 1–3 and 5 for five weights and balance factors scenarios.

Table 6. Fuzzy result of pipe no. 4 for five weights and balance factors scenarios.

Trial No.	Pipe No. 4	
	Most likely ranges	Largest likely ranges
1	0.57–0.63	0.51–0.68
2	0.40–0.45	0.35–0.48
3	0.58–0.61	0.52–0.65
4	0.39–0.40	0.37–0.42
5	0.60–0.65	0.54–0.69

Table 7. Pipe condition ranking for five weights and balance factors scenarios.

Ascending order	Trial No.				
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
1	2	1	1	1	1
2	1	2	2	2	2
3	5	5	3	3	3
4	3	3	5	5	5
5	4	4	4	4	4

value equal to 0) of the fuzzy result for different scenarios. Figure 5 shows the range of fuzzy result for different weights and balance factors scenarios. Figure 6 gives the similar analysis to other pipes (pipe no. 1–3, and 5).

From Table 6 and Figure 5, we can see that five weights and balance factors scenarios produce similar result. The most likely fuzzy interval difference varied from 0.01 (trial 4) to 0.06 (trial 1). The largest likely fuzzy interval differences were similar with a range from 0.05 (trial 4) to 0.17 (trial 1). The most likely and largest likely fuzzy intervals were narrow for trial 4 due to the balance factors were larger (see trial 4 of Table 5). Similarly, the most likely and largest likely fuzzy intervals were wide for trial 1 because of the smaller balance factors. The same reasons can explain the fuzzy numbers for trial 4 is smaller (close to 0 on x direction), and for trial 1 is larger (close to 1 on x direction), as shown in Figure 5. Similar conclusions can be drawn for other pipes, as shown in Figure 6.

By using FCP hierarchical aggregation process, a fuzzy number was obtained to evaluate pipe condition (i.e. for a pipe network with n pipes, n fuzzy numbers were obtained). The next step is to rank the fuzzy numbers, corresponding to rank the condition of pipes. In this paper a fuzzy number ranking method developed by Chen (1985) is used.

Table 7 gives the ranking order of pipe condition in terms of the five weights and balance factors scenarios. Trials 3–5 produces the same ranking orders, however, trials 1–2 are slightly different. This partially due to trial 1 shows no preference to physical indicators (identical weights for second level physical and environmental indicators), comparing with trials 3–5. Trial 2 assigns larger balance factors for first level physical indicators, which allows for little compromise among indicators so as to produce relative smaller fuzzy result for second level physical indicator.

The sensitivity analysis has shown that the ranking order of pipes condition does not get affected greatly by weights and balance factors although the magnitude of fuzzy outputs of each pipe does. The pipe condition ranking order is what DM uses to develop water mains rehabilitation scheme, thus, the present model shall therefore be robust for this purpose.

4 SUMMARIES AND CONCLUSIONS

In this paper a pipe condition ranking model has been presented that can assist in prioritizing water mains rehabilitation scheme. Fuzzy set theory has been employed to account for the uncertainties inherent in pipe condition indicators. Composite programming techniques has been applied to combine pipe condition indicators successively until final fuzzy indicator is reached.

The method of FCP (developed using distance based MCDM techniques and fuzzy set theory) enable the analysis of vague and imprecise information in MCDM systems. A major advantage of the FCP assessment procedure is its ability to refine and improve the FCP hierarchical structure when more data becomes available. Besides, the FCP assessment procedure can utilizes relatively easy available, imprecise, and/or subjective data. The model results include the uncertainties associated with the first-level indicators. Additionally, individual perceptions are included in the analysis by assigning weights and balancing factors. The major disadvantage of FCP is the final result quantification. This can be achieved by either performing defuzzification or applying fuzzy ranking method.

The model is demonstrated by means of a simple example, and this example involves ten first-level indicators including crisp, fuzzy, and linguistic variables. A sensitivity analysis was performed based on five weights and balance factors scenarios. The results were reported both for each pipe with five trials, and for each trial with ranking order of five pipes. The results reveal that the final fuzzy indicator values of each pipe varies with trial, and the degree of variation depends mainly on the balance factors. However, the overall ranking of the five pipes are not affected greatly by the trail scenarios. As it is the ranking order that influences the decision making process, the result of the sensitivity analysis indicates that the method to be robust when applied to water mains rehabilitation.

It is anticipated that the model presented in this paper will assist engineers in prioritizing water mains inspection and rehabilitation scheme.

REFERENCES

AWWSC 2002. Deteriorating buried infrastructure management challenges and strategies. *White Paper Prepared for US Environmental Protection Agency workshop on Distribution System Issues, March 7–8.* Voorhees, N.J.

Bardossy, A., Bogardi, I. and Duckstein, L. 1985. Composite programming as an extension of compromise programming. In: P. Serfini (ed.), *Mathematics of multiple objective optimization*. Vienna, Austria: Springer Verlag. pp. 375–408.

Bardossy, A. and Duckstein, L. 1992. Analysis of a karstic aquifer management problem by fuzzy composite programming. *Water Resources Bulletin* 28(1): 63–73.

Chen, S.H. 1985. Ranking fuzzy numbers with maximizing set and minimizing set. *Fuzzy sets and systems* 17: 113–129.

Dubois, D. and Prade, H. 1988. *Possibility theory – An approach to computerized processing of uncertainty*. New York: Plenum Press.

Hagemeister, M.E., Jones, D.D. and Woldt, W.E. 1996. Hazard ranking of landfills using fuzzy composite programming. *Journal of Environmental Engineering, ASCE* 122(4): 248–258.

Jones, D. and Barnes, E.M. 2000. Fuzzy composite programming to combine remote sensing and crop models for decision support in precision crop management. *Agricultural Systems* 65: 137–158.

Kaufmann, A. and Gupta, M.M. 1991. *Introduction to fuzzy arithmetic: Theory and applications*. New York: Van Nostrand Reinhold.

Lee, Y.W., Bogardi, I. and Stansbury, J. 1991. Fuzzy decision making in dredged-material management. *Journal of Environmental Engineering, ASCE* 117(5): 614–629.

Lee, Y.W., Dahab, M.F. and Bogardi, I. 1992. Nitrate risk management under uncertainty. *Journal of Water Resources Planning and Management, ASCE* 118(2): 151–165.

Loganathan, G.V., Park, S. and Sherali, H.D. 2002. Threshold break rate for pipeline replacement in water distribution systems. *Journal of Water Resources Planning and Management, ASCE* 128(4): 271–279.

Mays, L.W. 2000. *Water distribution systems handbook*. New York: McGraw-Hill.

Mcneill, L.S. and Edwards, M. 2001. Iron pipe corrosion in distribution systems. *Journal American Water Works Association* 93(7): 88–100.

Sægrov, S. et al. 1999. Rehabilitation of water networks – Survey of research needs and on-going efforts. *Urban Water* 1(1): 15–22.

Summers, P. 2001. Finding your way. *Journal American Water Works Association* 93(11): 58–61.

Woldt, W. and Bogardi, I. 1992. Groundwater monitoring network design using multiple criteria decision making and geostatistics. *Water Resources Bulletin* 28(1): 45–61.

Yan, J.M. and Vairavamoorthy, K. 2003. Fuzzy approach for pipe condition assessment. *Proc. ASCE International Conference on Pipeline Engineering and Construction, July 13–16, 2003.* Baltimore, Maryland, U.S.A.

Zadeh, L.A. 1965. Fuzzy sets. *Information and Control* 8(3): 338–353.

Zeleny, M. 1973. Compromise programming. In: J.L. Cochrane and M. Zeleny (ed.), *Multiple criteria decision making*. Columbia, South Carolina: University of South Carolina Press. pp. 263–301.

Chapter 4 – Network optimisation

Benchmark problems for design and optimisation of water distribution systems

R. Farmani, D.A. Savic & G.A. Walters
School of Engineering and Computer Science, Department of Engineering, University of Exeter, Exeter, UK

ABSTRACT: Water distribution system design optimisation is one of the most heavily researched areas in the hydraulics profession. Hundreds of papers and reports on approaches have been developed over the past few decades. However, there are only a few known examples of such systems from literature that can be used for testing and evaluating new design procedures. This paper introduces a benchmark library of water distribution models, which has been set up to ensure that practitioners and researchers have access to water system data, ranging from simple to very complex distribution systems. In particular, special emphasis is paid to data allowing users to test and compare their design and operation modelling algorithms and tools with others. The aim is to clear any inconsistency in the information related to the existing benchmark systems considering both design and optimisation data. Addition of new benchmark water systems is actively encouraged in order to have a diverse set of systems that compare well to real water systems. Most importantly, we encourage the water community to become active users of and contributors to the benchmark library. The motivation and the advantages of building such a library is discussed. The format of a typical entry to the library is illustrated by an example. The current version of the library (which is available from http://www.ex.ac.uk/cws/benchmarks) is described.

1 INTRODUCTION

Benchmarking is a tool for improving performance by learning from best practices and understanding the processes by which they are achieved. When the lessons learnt from a benchmarking exercise are applied appropriately, they facilitate improved performance in critical functions.

Application of benchmarking involves four basic steps:

- understand in detail your own processes,
- analyse the processes of others,
- compare your own performance with the others,
- implement the steps necessary to close the performance gap.

Benchmarking is above all a practical tool. It is constantly evolving in the light of ever increasing experience applying in different organisational and design settings. To be effective, it must become an integral part of an ongoing improvement process. Benchmarking was originally developed by companies operating in an industrial environment. It has therefore been applied most widely at the level of the business enterprise. However, benchmarking is also a common way of evaluating quality of a computational model.

2 BENCHMARKING

Benchmarking can be done based on two criteria, process benchmarking or design benchmarking.

2.1 *Process benchmarking*

Process benchmarking is usually a systematic mechanism of comparing different utilities based on improvement of their performance, lowering costs and increasing the provided service level. The four steps of benchmarking will be considered in process benchmarking (Cabrera et al., 2003), as follows:

- Planning the project: It includes the identification of the processes to be studied and the project partners "i.e. all those participating in the benchmarking study".
- Information collection: As well as indicator values, observation of the partners' processes is very important.
- Information analysis: Determination of the current performance and the performance gap.
- Adaptation for improvement: Collecting and analysing the information will result in actual changes in the company. It is necessary to implement changes, measure the final performance obtained and recalibrate objectives to proceed with a continuous improvement program.

Process benchmarking has been done in the past for the water industry, for sewerage systems and wastewater treatment plant (Parena and Smeets, 2001; Starkl et al., 2002; Stemplewski et al., 2001).

Parena and Smeets (2001) provided a framework for the most interesting benchmarking experience in the water sector and described in detail both the final results of the survey and the methodology focused on identification of possible improvement areas. The benchmarking task force of the IWA statistics and economics committee provided a valid tool to compare all over the world the utilities' performance and identify and isolate areas for possible improvement. Water quality, services, environment, finance and efficiency were identified as the five main areas to be considered for evaluation of the utility's performance.

Starkl et al. (2002) presented benchmarking of sewerage systems with a special focus on investment costs of 34 sewerage system projects. The emphasis of the development of benchmarks was to allow the investment, operation and maintenance costs to be directly compared and in turn to identify the most cost-effective service provider.

Stemplewski et al. (2001) presented benchmarking for efficiency enhancement in planning, construction and operation of wastewater treatment plants. The method was developed within a pilot project, in which four wastewater treatment plants (WWTP) were involved. The method was applied to 100 WWTPs and specific technical and economic parameters were determined for the whole treatment plant. They observed differences between the examined plants and the respective benchmarks. They concluded that because of external influences not all the plants can reach the benchmarks.

2.2 *Design benchmarking*

Academia have made a lot of effort in developing methods and optimisation models, however, there is a limited usage in practice. Optimisation methods have been criticised for ignoring the uncertainty in defining design parameters, including cost coefficients, future roughness coefficients and most significantly demands (Lansey, 1999). Walski (2001) recently criticized most previous optimisation approaches for several practical reasons. The main ones being that:

- It is difficult for practitioners to define objective functions and constraints,
- There should not be a single set of demands for which the system is designed,
- Optimisation fails to account for the fact that a total distribution system is not built all at once, and
- Optimisation tends to reduce costs by reducing the diameter of or completely eliminating pipes thus leaving the system with insufficient capacity to respond to pipe breaks or demands that exceed design values without failing to achieve required performance levels.

Recent advances in tools for simulation and optimisation of hydraulic techniques, made it possible to tackle most of these criticisms. However, there is need for a good set of test cases to assess the performance of these fast growing techniques.

Design benchmarking provides a systematic mechanism for comparing computational tools for hydraulic simulation or optimisation techniques. The main aim in design benchmarking is to improve search and decision-making using different algorithms. The four steps of benchmarking will be considered in design benchmarking as well, however they are used slightly differently as follows:

- Planning the project: It includes identification of the performance indicators, definition of objective functions and constraints.
- Information collection: Information collection for design benchmarking includes detailed information on how the design problem been set up such as boundaries for design variables, number of loading conditions, etc.
- Information analysis: Analysing your own design and comparing the results with the others.
- Adaptation for improvement: Collecting and analysing the information will result in new initiatives for future models and will give decision makers choice among competing methods.

Hydraulic systems have been benchmarked previously by researchers for open-channel flows and for design, modelling and operation of water systems (MacDonald et al., 1997 and Ulanicka et al., 1998).

MacDonald et al. (1997) provided benchmark solutions for a wide range of cases for open-channel flows. These open-channels may have a nonprismatic cross section, nonuniform bed slope, and transitions between subcritical and supercritical flow. This makes it possible to assess the underlying quality of computational algorithms in more difficult cases including those with hydraulic jumps. The test cases may also be used as benchmarks for both steady flow models and unsteady flow models in the steady limit.

Ulanicka et al. (1998) presented benchmarks for algorithms that are used in water applications (http://www.eng.dmu.ac.uk/~wss/welcome.htm). They covered the ontology of water network models and discussed numerical algorithms for extended period simulations. In their work, benchmark models have been prepared for hydraulic simulation, optimal scheduling, network simplification, state and parameter estimation, meter placement and demand prediction. There are three networks in the hydraulic simulation models ranging from relatively simple networks to a medium level of difficulty. They consist of different elements such as pumps, valves, reservoirs and sources with or without control loops. In the network simplification models section there is a typical real distribution network with three sources of water and a complicated structure with many parallel connections and loops. The optimal scheduling model section has a network that is a supply zone for a big town.

The main objectives of their project were:

- To unify problem formulations for different classes of algorithm,
- To define performance measures which enable algorithm evaluation,
- To produce and publish benchmark models together with performance measures, for different classes of algorithm,
- To provide and maintain these resources to other researchers and industry.

The motivation behind setting up the benchmark library described in the present paper is, in principle, very similar to that of Ulanicka et al. 1998. However, the main differences are in type of networks and purpose of them being in the library. The benchmark networks in their library are for hydraulic simulation models, network simplification models and optimal scheduling models. However, the networks in our library are mainly for design and optimisation of water distribution networks. It is evident that, despite pursing similar goals, not only is there no overlap in general but also the libraries complement each other by covering the whole range of problems that can be faced in water engineering. Therefore we strongly recommend the combined use of these libraries for completeness of the idea of benchmarking.

3 MOTIVATION

The motivation for developing the benchmark library for water systems is as follows:

Benchmarking: One of the main applications of the library is to provide a common set of problems on which different groups can quickly benchmark their algorithms.

Modelling: The library can be used by researchers on important issues concerning design, modelling and problem reformulation.
Real Networks: Current benchmark networks can be criticized for being too simple and not representative of real networks. A benchmark library can contain problems that are both simple and complex.
New Applications: A library can provide rapid entry into new domains.
Challenges: It can be a forum for open problems and other challenges that can push water technology to new heights.
Showcase: The existence of a problem in the library may suggest that modelling is a suitable technology for this type of problem; the entry may then provide clues to help a user model a related problem.
Bridging the gap: The gap between development and application resulted from perceived lack of a market by researchers and a lack of interest among practicing engineers. Recently, several commercially available optimisation programs based on genetic algorithms have been produced. The models are more intuitive and hopefully begin a broader application of technology that has been developing over the last 30 years (Lansey, 1999).
Clear data inconsistency: The main pitfall of fast growing optimisation techniques is the lack of test cases that could stand up to the challenge (i.e., could be used to assess the performance of each algorithm). Consequently, researchers and software developers are faced with simple networks with powerful tools in their hand. In recent years, change in the way design problems have been set, for example addition of new pipe diameters to available pipe size set, has become common. A benchmark library can be used to clear any inconsistency in the information related to the existing benchmark systems considering both design and optimisation data.

The New York Tunnel system has been used as a benchmark network since 1969 to compare various optimisation procedures. It would be nice to compare methods using a set of other benchmark problems including more complex ones. Walski et al. (1987) set up the hypothetical Anytown water distribution system as a realistic benchmark on which to compare and test network optimisation software, and has features and problems typical of those found in many real systems. Expansion and rehabilitation decisions were necessary for a realistic network in Anytown, USA. Pipe costs were varied by location. Additional tanks and pumps could be added over time to meet demands. Energy costs were computed for daily demands, and a large set of peak and fire conditions were to be satisfied. The Anytown problem was originally tackled by participants at the Battle of the Network Models workshop. All participants in the original workshop used optimisation models to size the piping system while manually choosing the location and size of tanks. The feasibility of solutions was checked using a simulation of the system operation over 24 hours. The difference between the various methods used lies essentially in the pipe optimisation models, which were based on linear programming, partial enumeration or non-linear programming techniques. No attempt was made to optimise the provision of tanks or pumps, except by use of expert judgment and experience. The Anytown water distribution system has since been examined only by Murphy et al. (1994) and Walters et al. (1999). The main reason for lack of interest is that the design and operation of this network is very complicated. As an example, complications of tank design can be mentioned which are due to water stagnation in storage tanks oversized due to fireflow, emergency storage requirements and a sequence of demands describing daily operations, which increase computational effort. However, the use of the computer model has greatly enhanced the design and operation of water distribution systems. Recent advances in optimisation techniques, especially those based on evolutionary algorithms, make it possible to overcome limited usage in practice. The main reasons are that optimisation problems can be considered as multi-objective (reflecting real world problems) rather than single objective. Also evolutionary methods can handle both discrete and continuous design variables.

4 A MODEL LIBRARY

Benchmarking is a well established trend in assessment of performance. Design benchmarking is a very common practice in optimisation problems such as constraint handling techniques, multi-objective optimisation, combinatorial optimisation etc. The success of these types of benchmarking are due to:

- **Diversity:** diversity among the data sets that are representative of those that could arise for corresponding optimisation problems.
- **Ease of use and completeness:** They are easy to use and as comprehensive as possible. So users should not need to look anywhere else for further information.
- **Regularly updated:** They are updated regularly to prevent duplication of effort.

Having taken on board lessons from the benchmarking in other research areas, the benchmark for design and optimisation of water systems has been set up (Figure 1). The benchmark library is available from http://www.ex.ac.uk/cws/benchmarks.

There are links to other pages such as benchmark samples, table of results, publications and contact information. The benchmark samples page shown in Figure 2 contains different networks. In the first version there are two groups, new networks and expansion or rehabilitation of existing networks, with three benchmark networks. Two of these networks (Hanoi and New York Tunnels) are the most researched networks in simulation and optimisation of water systems, and presence of them in the library and the corresponding publications is just to have a complete set of data in one place. Presence of the third benchmark network (Anytown network) adds some difficulty and challenges the computational tools. Hopefully this gradual growth in problem difficulty will result in use of these networks. The ultimate goal is to have a selection of groups with a variety of networks belonging to each group.

Completeness of the library is another important factor in success of benchmarking. Therefore, there are links to full information on how the problems have been set up, the input files in Epanet format, publications related to each network and results in tabulated format.

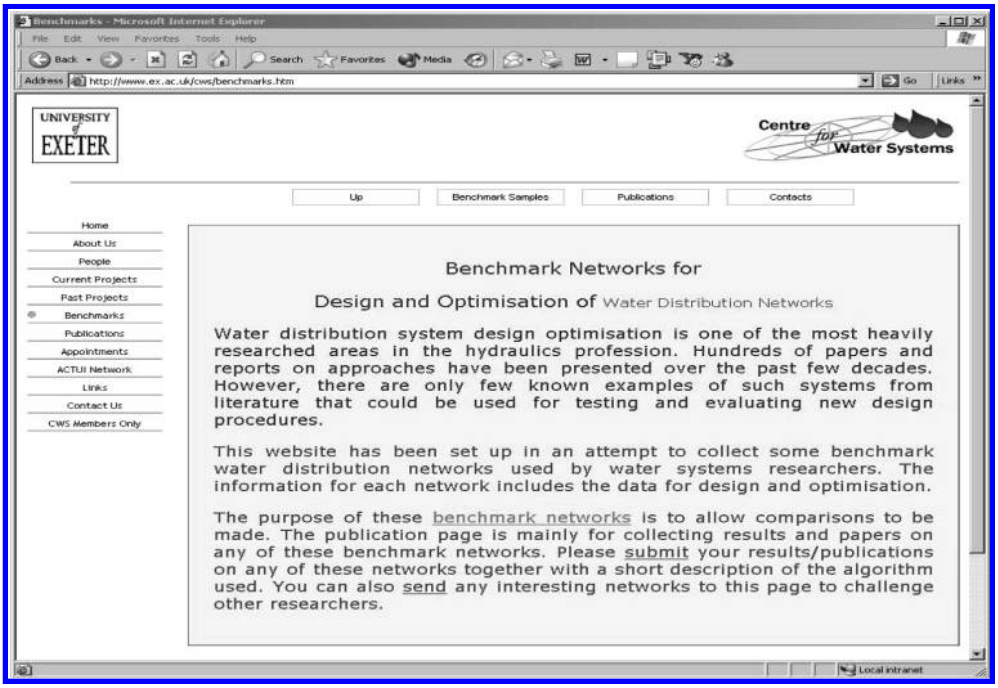

Figure 1. Benchmark Library.

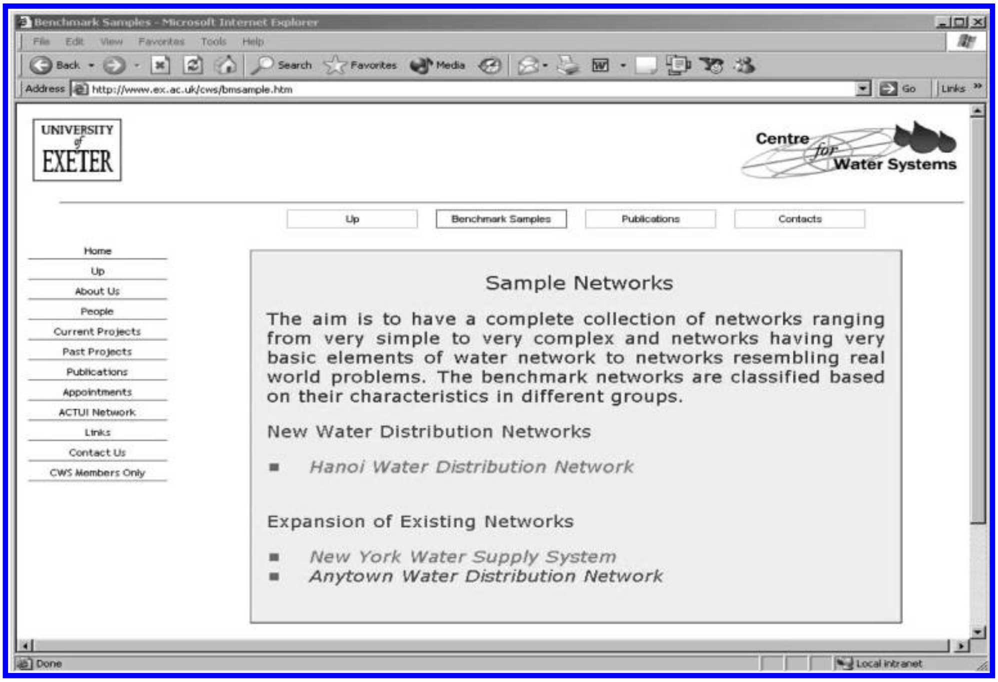

Figure 2. Sample Networks.

5 SUBMISSION GUIDELINES

To help users submit benchmark problems to the library, there are simple guidelines as follows. Users may submit:

- A brief description of the benchmark problem, covering the network elements such as number of pipes, nodes, tanks etc;
- The boundary conditions, minimum pressure at nodes, commercially available diameters;
- The cost functions for pipes, tanks and pumps;
- The hydraulic input file in Epanet network solver format.

To make comparison with previous works easier, a link is provided to the papers that use these benchmarks. If you use a problem in the library, please send us the details of your papers so that we can add them to the reference section. To make it easy to compare your results with others, we will provide a record of results. Please help us keep these records up to date by sending in your results promptly.

6 AN EXAMPLE

Entries in the benchmark library consist of a sample networks page (Figure 2), a description page (Figure 3), an input file, network configuration, publications, results and software tool. The sample networks page separates benchmark problems based on hydraulic characteristics of the network such as new network, rehabilitation of existing networks etc. (Figure 2).

The description page simply summarizes problem characteristics. There are links to input file, configuration and table of results for the network from the description page. Figure 3 shows the description page for New York Tunnels.

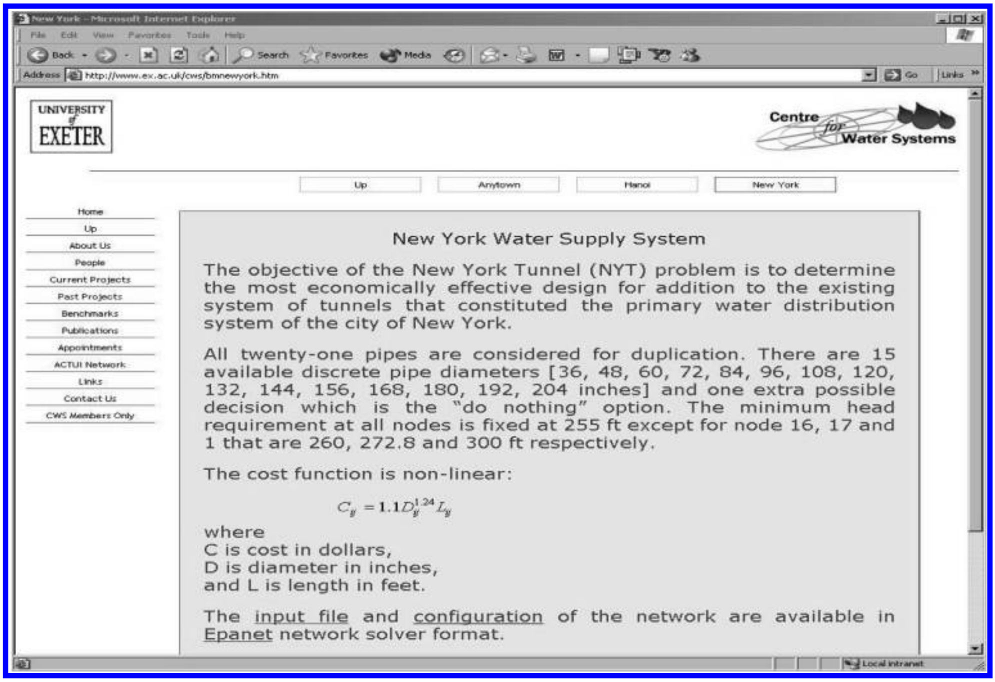

Figure 3. Description page for New York Water Supply System.

Input file is in Epanet network solver format (Rossman, 1993). Configuration of the problem is in bmp format which also could be extracted from the Epanet interface, if coordinates of nodes have been given in the input file. List of papers using these benchmark problems are in the publication page. Results and underlying algorithms are summarized in the result page in tabulated format. There is also a link to similar web pages such as http://www.eng.dmu.ac.uk/~wss/welcome.htm that have some interesting benchmark problems which match real world networks.

7 CONCLUSIONS

A benchmark library has been introduced for design and optimisation of water systems. The sample networks in the library range from simple to very complex distribution systems. Addition of the input file and related publications and tabulated results allows users to test and compare their design and operation algorithms with others. Having a collection of a large number of algorithms for simulation and optimisation of water systems and their performance on the same set of benchmark networks will allow users to assess the underlying quality of algorithms in more difficult cases. Hopefully this can be starting point for the broader application of technologies that have been developing over the last 30 years to real world networks. The success or failure of the benchmark web page depends on its uptake by the water community. Therefore, we encourage you to benchmark your algorithms on the problems contained within the benchmark library, to send us your results and any other data of interest that can be included in the web page.

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REFERENCES

Cabrera Jr. E., Jordan R. C. and Alvaro M. H., 2003. Benchmarking in the water industry. Basic principal, applicability and methodology. Pumps, Electromechanical Devices and systems: Applied to urban water Management, Eds. E. Cabrera and E. Cabrera Jr., ISBN 9058095606, (1), 37–44.

Lansey K.E., 1999. Optimal design of water distribution systems. Water distribution systems handbook, Ed. L.W. Mays, McGraw-Hill 7.1–7.15.

MacDonald I., Baines M.J., Nichols N.K. and Samuels P.G., 1997. Analytic benchmark solutions for open-channel flows. Journal of Hydraulic Engineering, 123(11), 1041–1045.

Murphy L.J., Dandy G.C. and Simpson A.R., 1994. Optimum design and operation of pumped water distribution systems. Proceedings of the Conference on Hydraulics in Civil Engineering, Brisbane, Australia: Institution of Engineers, Australia, 149–155.

Parena R. and Smeets E., 2001. Benchmarking initiatives in the water industry. Water science & technology, 1st World Water Congress: Operation and Management, Landfill and Sludge Management, 44(2–3): 103–110.

Rossman L.A., 1993. EPANET, users manual. U.S. Envir. Protection Agency, Cincinnati, Ohio, 1993.

Starkl M., Ertl T. and Haberl R., 2002. Experiences with benchmarking of sewerage system with a special focus on investment costs. Proceedings of the International Conference on Sewer Operation and Maintenance, 26–28 November 2002, Bradford, UK, on CDROM.

Stemplewski J., Schulz A. and Schon J., 2001. Benchmarking – an approach to efficiency enhancement in planning, construction and operation of wastewater treatment plants. Water science & technology, 1st World Water Congress: Operation and Management, Landfill and Sludge Management, 44(2–3), 111–117.

Ulanicka K., Ulanicki B., Rance J.P. and Coulbeck B., 1998. Benchmarks for water network modeling. Hydroinformatics, Ed. V. Babovic and L.C. Larsen, Balkema, Rotterdam, ISBN 9054109831, 2: 1469–1477.

Walski T.M., Brill E.D., Gessler J., Goulter I.C., Jeppson R.M., Lansey K., Han-Lin Lee, Liebman J.C., Mays L., Morgan D.R. and Ormsbee L., 1987. Battle of the network models: Epilogue. Journal of Water Resources Planning and Management, ASCE, 113(2), 191–203.

Walski T.M., 2001. The wrong paradigm – Why water distribution optimization doesn't work. Journal of Water Resources planning and Management, ASCE, 127(4), 203–225.

Walters G.A., Halhal D., Savic D. and Ouazar D., 1999. Improved design of Anytown distribution network using structured messy genetic algorithms. Urban Water, (1), 23–38.

More choices in water system design through hybrid optimisation

E.C. Keedwell & S.T. Khu
Centre for Water Systems, University of Exeter, Exeter, UK

ABSTRACT: In recent years there has been an increased interest in using genetic algorithm techniques for the rehabilitation of water distribution networks. In particular, multi-objective (MOGA) techniques have found application in this domain and the results have been promising. MOGA techniques can be used to obtain Pareto optimal solutions which represent a set of compromises between two or more conflicting objectives. A desirable Pareto-curve has well-spaced points along the optimum trade-off between the two or more objectives. Pursuing this optimal trade-off though can be computationally expensive and often the optimal curve is not known and cannot be readily computed. Therefore a technique which can provide near optimal solutions which are well spread on the Pareto surface will ensure that this technique will be readily taken up by the water industry. This work investigates the possibility of combining traditional MOGA techniques and methods of local search to both improve solutions and reduce running times. Encouraging results from experiments were gained using this hybrid MOGA, applied to the New York City Tunnels water supply network rehabilitation problem.

KEYWORDS: Water distribution network; modelling; multi-objective genetic algorithm; hybrid; Pareto

1 INTRODUCTION

1.1 *Water distribution network (WDN) modelling*

Water distribution network models or model simulations are used for a variety of purposes such as: strategic and master planning; system design; energy management and daily operations; fire protection studies and emergency responses; water quality investigations; and many others. The computer modelling of water distribution networks continues apace in the water industry as computers become increasingly powerful, more complex systems are available to be modelled. Open source software such as EPANET (Rossman, 1993), which is the water distribution modelling software used in this paper, allows easy simulation of these complex systems. In order to simulate the WDN successfully, it is necessary to perform some kind of network optimisation, usually based on the performance of the model with respect to the chosen model parameters. More recently this work has been extended to optimise these models for both performance and cost by using various computational methods, such as non-linear programming, dynamic programming and search techniques.

1.2 *Network optimisation*

The optimisation of WDN models can primarily be used for three purposes: the design of networks for new supply areas (design problems); modifying existing designs to meet new demands or other factors (rehabilitation problems) and modifying network parameters to ensure that they are accurate with respect to the real world (calibration problems). This paper is primarily concerned with the problem of rehabilitation although the methods described herein can theoretically be used for any of these problems. A standard rehabilitation problem is concerned with taking an existing network and modifying its parameters or components to satisfy new constraints in the performance of the network. Therefore any process designed to optimise a rehabilitation problem must consider both the performance of the network (to be maximised) and the cost of the proposed solution (to be minimised).

1.3 *Optimisation algorithms*

As described in 1.2, a rehabilitation problem is concerned with taking an existing network and modifying its components, however, in the actual WDN there exist a large number of components which act in combination. This combinatorial effect means that even a network with a modest number of components will contain a huge number of potential solutions and therefore that it is not possible to evaluate every possible solution within the timeframe of the project. Therefore there exist a number of algorithms which are designed to find near optimal solutions from the overall set in real-time. Notably genetic algorithms have found considerable success in this domain (Savic and Walters, 1997) and are the subject of much of the rest of this paper.

Genetic algorithms (GAs) make use of the principles of evolution to efficiently search a large space of solutions. GAs derive their strength from the use of multiple starting points for optimisation (known as population-based search) and operators to find new points mimicking the process of evolution. By using a special set of operators-selection, crossover and mutation, a GA can make a progress towards a near-optimal solution. The basis of the algorithm is that a population of solutions is created and run for a number of generations. Each time a new solution is created, it is assigned a fitness based on the solutions' ability to solve the problem at hand. The algorithm then uses selection to preserve the genetic material of those "highly fit" individuals for progression into the next generation, whilst crossover and mutation ensure that new solutions are also created. (For a more in-depth discussion of genetic algorithms please see Goldberg, 1989). Formulated as a global population-based search technique, GAs are inherently suited for multiple objectives optimisation. In recent years, a large number of multi-objective genetic algorithms (MOGA) have emerged which are capable of dealing with two or more objective functions (Zitzler et al., 2000). Amongst them, the elitist non-dominated sorted genetic algorithm (NSGA-II) appears to be one of the best MOGAs (Deb and Goel, 2001).

Many engineering problems involve conflicting objectives, and the optimisation of WDNs is no exception. For instance in a rehabilitation problem, the optimal monetary solution is to leave the system as it is with no modifications, equally though investing a huge amount of money in the system will give superior network performance. Clearly there is a trade-off here between investment and network performance. It is this trade-off, which, by using the same evolutionary processes as a standard GA, can be optimised. The goal of a MOGA therefore is to yield a set of solutions which represent the optimal trade-off between two or more conflicting aspects of a system. In this paper we describe the use of MOGAs on a standard rehabilitation problem.

1.4 *Hybrid algorithms*

The above scenario shows that GAs, and in particular, MOGAs can be used to present realistic solutions to the problems of optimising WDN performance. However, these population-based techniques suffer from difficulties when applied on today's complex models. The problem exists because WDN modelling, especially for large networks and extended period simulation can incur large computation time. Typically, GAs and MOGAs use a population size of 50–200 and 100–10,000 generations. This can therefore lead to anywhere between 5,000 and 2 million model evaluations for an optimisation run. This is evidently not feasible even if each model simulation requires 1 minute to run on a standard machine – $1 \times 2{,}000{,}000 = 2$ million minutes $= 3.8$ years. Even for the shortest runs (5,000 evaluations), if the objective function takes 1 minute to evaluate, then running times of 1/2 a week can be expected. Thus, it is imperative that for simulation of large WDNs that the number of evalutions for any optimisation algorithm to be reduced to a manageable number of evaluations.

The main work in this paper therefore is to investigate the possibility that a MOGA-Local Search hybrid algorithm can exploit the behaviour of the GA whilst reducing the overall number of evaluations required to obtain near-optimal solutions. In particular we explore a combined method of neighbourhood search and NSGA-II, the current best performer in WDN MOGA optimisation.

2 PROPOSED METHOD

2.1 *Trade off surfaces for multiple objectives*

In this paper, the version of MOGA used is the Non-Dominated Sorting Algorithm-II (Deb et al., 2001), which is currently considered to be one of the best algorithm for water distribution system optimisation (Farmani et al., 2003). The hybrid approach uses NSGA-II unchanged except for the fact that the optimisation is periodically stopped, the local search element is run, and the results from the local search are returned to the population and the optimisation continues. Both the local search and all MOGAs (including NSGA-II) rely on the concept of domination. With single objective GAs, solutions can be compared with respect to their fitness and a selection made. However, MOGAs need to find a trade-off of solutions which is, ideally, both optimal and well-spread over the objective space. Figure 1 shows a comparison between trade-off surfaces.

In Figure 1, three sets of Pareto optimal solutions are represented on the same plot for comparison. The circles represent well spaced solution sets and are the most preferred solutions. The crosses are less optimal (further away from the ideal position, in a minimisation problem, the origin) compared to the circles. When comparing the squares and the crosses, the squares may be more optimal that the crosses, but they are not well spaced.

The concept of domination and non-domination is a convenient way to analyse the situation in Figure 1. It ensures that multi-objective algorithms can be entirely independent with respect to the objective ranges and values.

Domination
One solution is said to dominate another if it is better (what constitutes better is determined by the problem, maximisation or minimisation) or equal in every objective.

Non-domination
One solution can be said to be non-dominated if there is no other solution in the solution set which dominates it.

Figure 2 shows the concepts of domination and non-domination. Each of the solution sets marked by a "circle" are said to be non-dominated as they have at least one better objective value. Comparing

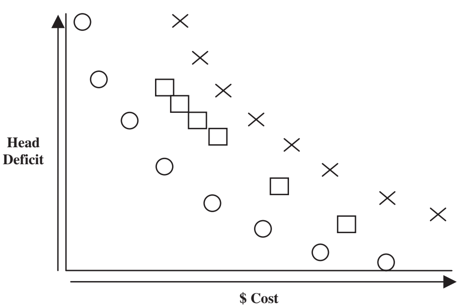

Figure 1. Comparison of trade-offs between cost and head deficit in a minimisation problem.

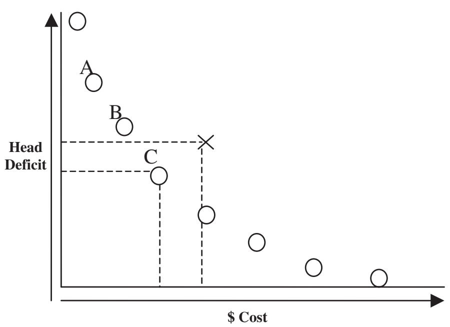

Figure 2. Dominated and non-dominated solutions.

point "A" and point "B", one can see that even though the cost function for "A" is less than that of "B", the head deficit of "B" is less than that of "A". The solution set marked by a "cross" is dominated by the point "C" because "C" is better in both objectives. Extending the above analysis to all the solution sets, one can note that all the circles are all non-dominated with respect to each other.

2.2 *Proposed hybrid algorithm*

The hybrid algorithm is used to drive the MOGA solutions towards the optimum during the optimisation to reduce the need for extended runs which require more model simulations. It uses a simplistic local search for a set number of iterations to find new, dominant solutions which are then entered back into the population of the MOGA. The local search is a neighbourhood search, it increments and decrements two pipe diameters at a time until it finds a solution which dominates the current solution. This is then selected and the process is repeated until some predetermined number of iterations has been completed. Figure 3 describes the operations of the algorithm in a flowchart.

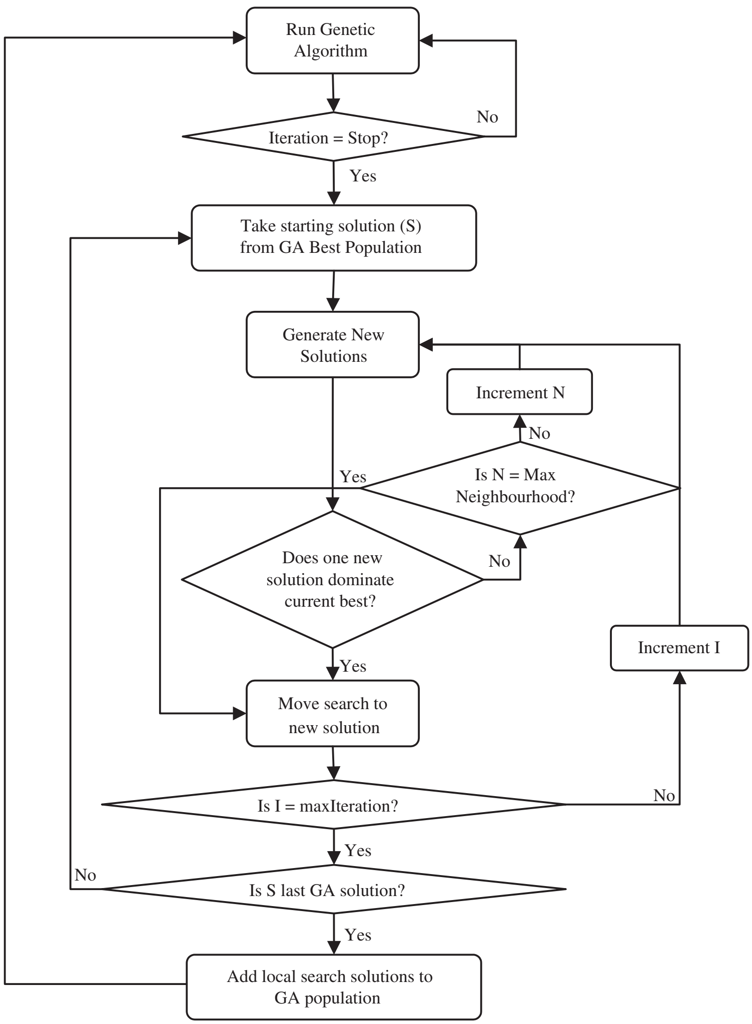

Figure 3. Flowchart depicting the overall execution of the hybrid algorithm. (S is the current solution selected from the GA population, N is the neighbourhood currently being searched and I is the local search iteration.)

This domination search ensures that only solutions which dominate the current best set are included in the search and therefore the insertion into the MOGA population. Once the solutions have been discovered, they are added back into the population, replacing the lowest ranked (and therefore most-optimal) solutions in the MOGA. Therefore the algorithm drives the MOGA to a more optimal solution set, but potentially at the price of some diversity in the population.

3 APPLICATION

3.1 *New York City Tunnels problem*

The work carried out in the following sections is based on the New York City Tunnel (NYT) rehabilitation problem (Schaake and Lai, 1969). This is a well known problem which has been studied extensively in the literature (Bhave and Sonak, 1985; Murphy et al., 1993; Savic and Walters, 1997; Farmani, 2003 etc.) and involves replacing old trunk pipes with new pipes of larger diameters or putting in new trunk pipes alongside old ones within the network to meet new demands for the City of New York. Figure 4 shows the existing network configuration. There are a maximum of 21 new pipes to be laid, with the option of doing nothing. The new pipes vary in diameters, ranging between 60–204 inches. There is one reservoir which supplies the water to the network which contains 20 nodes which have various demands.

From an optimisation perspective, the objective of the NYT problem is to modify the rehabilitated pipe diameters to meet the demands at the nodes. The current optimal solution for this is 37.09 million dollars and no pressure deficit (Vairavamoorthy and Ali, 2000) although this can vary depending on the modelling software and parameters used.

3.2 *Parameters*

During experimentation with the algorithm we have discovered a number of parameters that gives the algorithm flexibility to discover the near-optimal sets. They include the following:

- *Local search frequency* – determines the number of times the local search is used throughout the optimisation.

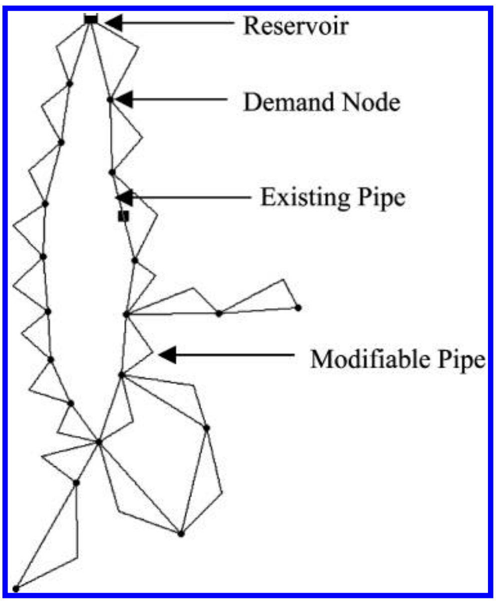

Figure 4. Network representation of the New York City Tunnels problem.

- *Local search iterations* – determines the computational effort given to the local search when it is used.
- *Replacement strategy* – determines which individuals in the GA population to replace, the best? Or the worst?
- *MOGA Parameters* – Those which can effect hybrid optimisation, such as population size.

Generally speaking we have found that low frequency, low iteration searches have produced the best results in conjunction with an optimal-replacement strategy which replaces the best-ranked solutions in the current GA population.

4 RESULTS AND DISCUSSIONS

Each of the runs below is optimising the New York Tunnels problem, minimising Total Head Deficit and Dollar Cost. These runs used the local search once, around 1/2 way through the optimisation, with 2 iterations and neighbourhood of 1. Each of the new solutions replaces the current best in the population. The MOGA uses crossover rate = 0.9, mutation rate = 0.9 and a population of 100. The surfaces below are showing these optimisations.

The following figures show a direct comparison of the hybrid algorithm and NSGA-II for a variable number of evaluations. The random seed is different for each run, but the same for NSGA-II and the hybrid. In all the runs, the number of model evaluations is approximately equal.

To demonstrate the effectiveness of the proposed hybrid algorithm compared with NSGA-II, the performance of these two algorithms were compared based on the same number of model evaluations. Figures 5, 6 and 7 show the resultant Pareto fronts for these runs with 5,000, 10,000 and 15,000 models evaluations respectively. It is evident from these figures that the proposed hybrid algorithm consistently out performed NSGA-II. In every run the proposed algorithm has either discovered more optimal solutions, or a better spread of objective values for a given number of model evaluations.

In Figure 8, the hybrid and NSGA-II results are shown in comparison to a run of NSGA-II that gave the best report Pareto optimal solution using NSGA-II for the NYT problem (Farmani et al., 2003). The population size used in generating all the fronts in Figure 8 is 200. Thus, the near-optimal result was achieved with 200,000 model evaluations. By comparison, it can be seen that the hybrid run often approaches this curve and is closer to this than the standard NSGA-II run. This is a remarkable result given that the hybrid has only used 15,000 model evaluations, or 7.5% of those used in this "optimal" NSGA-II run.

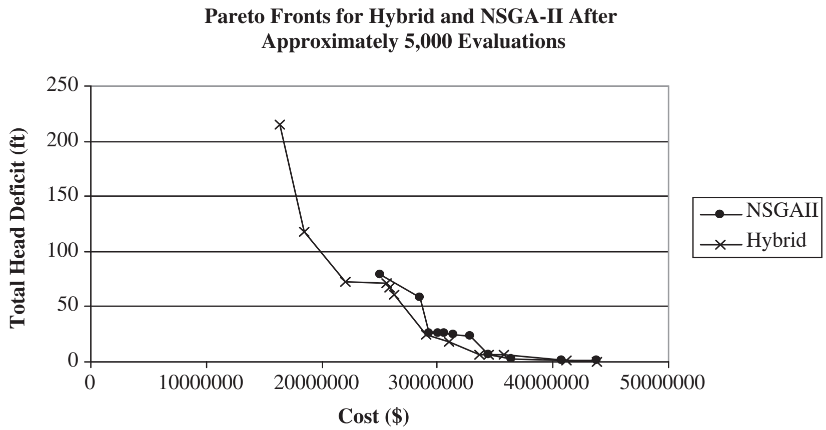

Figure 5. Comparison of Pareto fronts after approximately 5,000 evaluations.

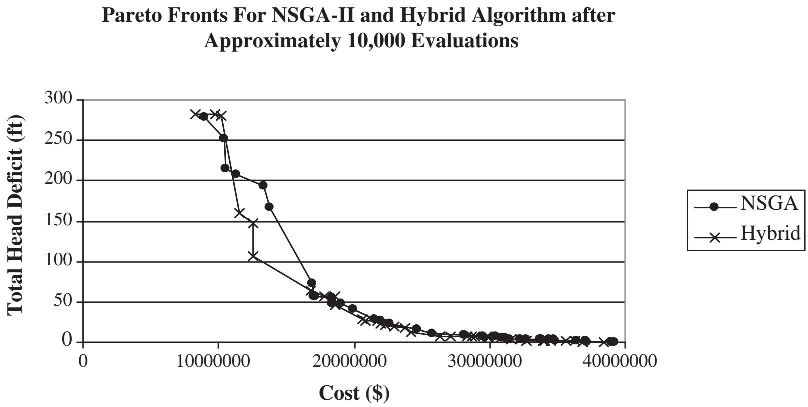

Figure 6. Comparison of Pareto fronts after approximately 10,000 evaluations.

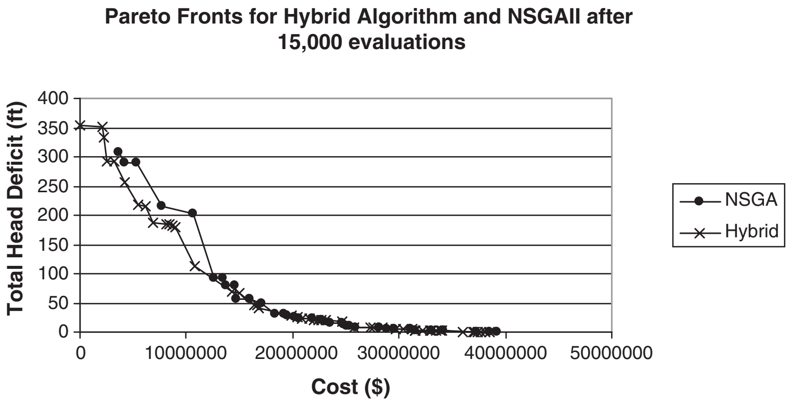

Figure 7. Comparison of Pareto fronts after approximately 15,000 evaluations.

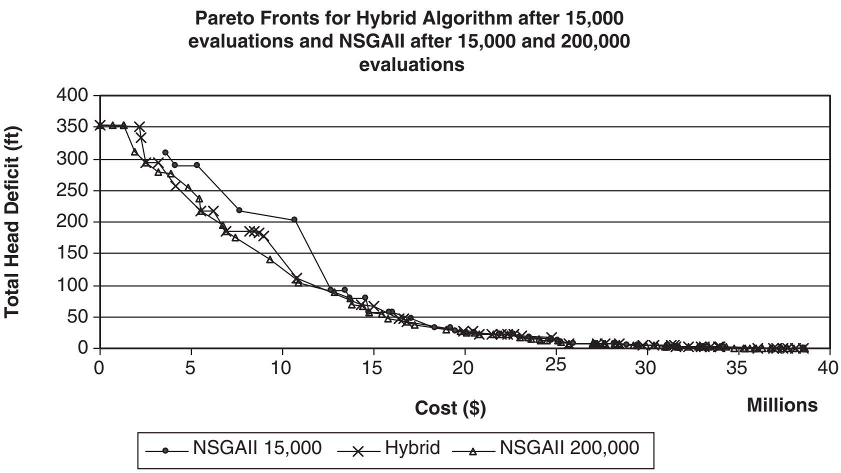

Figure 8. Comparison of Pareto fronts from 15,000 evaluations and a 200,000 evaluation run.

5 CONCLUSIONS

This paper has shown that a hybrid approach to the optimisation of water distribution networks can go some way towards mitigating some of the problems associated with running genetic algorithms on these networks. The algorithm runs are deliberately short in GA terms because many objective function evaluations in WDN optimisation can take a considerable length of time. Even for the shortest runs (5,000 evaluations), if the objective function takes 1 minute to evaluate then running times of 1/2 a week can be expected.

Whilst the New York Tunnels problem is not particularly complex or time consuming, the approach shows the improvements that can be found given a number of model evaluations, or the reduction in evaluations which is possible by using such an algorithm. This hybrid could increase the appeal of population based approaches such as GAs in fields where the simulation of the system is computationally expensive. Also, as the optimisation techniques improve, then increasingly complex and more importantly, realistic models of WDNs can be optimised such as those using extended period simulation.

REFERENCES

Bhave P.R. and Sonak V.V. (1992). A critical study of the linear programming gradient method of optimal design of water supply networks. *Water Resour. Res.*, *28(6)*, 1577–1584.

Deb K. and Goel T. (2001) "Controlled elitist non-dominated sorting genetic algorithms for better convergence," *Proceedings of the First International Conference on Evolutionary Multi-Criterion Optimization (EMO-2001)*, 7–9 March. (Zurich, Switzerland), (pp. 67–81).

Deb K., Agrawal S., Pratap A. and Meyarivan, T. (2000). A Fast Elitist Non-dominated sorting genetic algorithm for multi-objective optimization: NSGA-II. *Proceedings of the Parallel Problem Solving from Nature VI Conference*, 16–20 September. (Paris, France), (pp. 849–858).

Farmani R., Savic D. and Walters G. (2003) "Multi-objective optimisation of water systems: A comparative study" *Pumps Electromagnetic Devices and Systems Applied to Urban Water Management* (Cabrera, Cabrera Jr. Eds). AA Balkema Publishers. (pp. 247–255).

Goldberg D.E. (1989) *Genetic Algorithms in Search, Optimisation and Machine Learning*. Addison Wesley Publishing Company Inc.

Murphy L.J., Simpson A.R. and Dandy G.C. (1993). Pipe network optimisation using an improved genetic algorithm. *Res. Rep. No. R109. Dept. of Civil and Envir. Engrg.*, Univ. of Adelaide, Australia.

Rossman L. (1993). "EPANET", U.S. Environment Protection Agency.

Savic D. and Walters G. (1997) "Genetic Algorithms for Least-Cost Design of Water Distribution Networks" *J. Water Resources Planning and Mgt.*, *123(2)*, 67–75.

Schaake J. and Lai D. (1969). Linear programming and dynamic programming applications to water distribution network design. *Rep. 116*, *Dept. of Civil Engrg.*, *Massachusetts Inst. of Technol.*, Cambridge, Mass.

Vairavamoorthy K. and Ali M. (2000). Optimal design of water distribution systems using genetic algorithms. *Computer Aided Civil and Infrastructure Engineering*, *15*, 374–382.

Zitzler E., Deb K. and Thiele L. (2000). Comparison of multiobjective evolutionary algorithms: Empirical results. *Evolutionary Computation*, *8(2)*, 173–195.

Water distribution systems optimization by Metaheuristic Approach

S. Liberatore, G.M. Sechi & P. Zuddas
Dip. di Ingegneria del Territorio – Sez. di Ingegneria Idraulica – Università degli Studi di Cagliari, Italy

ABSTRACT: Three different metaheuristic optimization techniques are proposed for designing and maintaining water distribution systems. Since the design and maintenance of pipe networks for water supply distribution require high costs, it is mandatory to achieve the highest level of performance of the existing networks at minimum costs. The domain of variables is discrete in nature, due to the fact that pipes having unified dimensions are available. Furthermore, the objective function to be minimized, i.e., the total cost of the plant, is non linear, non differentiable, highly ill-conditioned, and presents a large amount of local minima. Genetic algorithms (GA), scatter search (SS), and tabù-search (TS) have been applied to the problem. The results show good performance of the GA in terms of objective function values, but high computation time. More promising approaches to combinatorial optimization problems are the SS and TS metaheuristics, which showed flexibility and effectiveness in applications.

1 INTRODUCTION

Water distribution networks design (WDND has been extensively investigated in many research studies in past decades (Alperovits & Shamir, 1977; Kessler & Shamir, 1989; Fujiwara & Khang, 1990; Eigher et al., 1994). Many investigative methods and solution techniques have been proposed and tested for the WDND problem. Nevertheless, it seems generally accepted that using general optimization tools it is not possible to solve the problem with real-size systems complexity. An adequate approach to the design and maintenance optimization of pipe networks should consider non-linear relations between head-losses in each pipe, the pipe diameter, its length, and hydraulic property. The literature on optimization procedures of pipe networks is extensive. The mathematical models describing this kind of problem are typically characterized by difficulties in handling features, such as discrete variables, large dimensions, and often the absence of mathematical features that allow the development and convergence of most classical resolution techniques, such as descent algorithms or gradient projection methods. Several resolution methods have been developed to solve this kind of non-linear, non-smooth problems, if sufficient adherence is required (Fujiwara & Khang, 1990). The non-convexity of the problem reduces the convergence properties at the local level, and its non-differentiability requires adopting non-differentiable optimization techniques, such as subgradient or e-gradient ones. When used in decomposition techniques, the latter are very efficient computationally if the mathematical model shows a special structure (Eigher et al., 1994). Interior point methods are particularly efficient in large-scale problems, but mostly in linear constraint models. On the other hand, using techniques based on duality, it is difficult to evaluate the gap between the obtained solution and the global solution of the original problem (Ben-Tal et al., 1994).

Over the past years metaheuristic approaches, mainly based on the genetic algorithm (GA) and tabù search (TS) technique methodology has been developed for pipe-network optimization (Murphy & Simpson, 1992; Dandy et al., 1993; Simpson et al., 1994; Murphy et al., 1993; Dandy et al., 1996; Liberatore et al., 1998; Fanni et al., 1999; Lippai et al., 1999). In this paper, comparisons between three different metaheuristic optimization approaches based on GA, reactive TS, and a combined scatter search (SS) – tabù search technique, has been considered. These techniques have been implemented and tested on a well-known test-problem given by Gessler (1985), while introducing some extensions.

In what follows, the graph terminology will be adopted referring to a network $G = (N, R)$, where N is the set of nodes and R the set of arcs. Nodes can represent sources and demand centers as well as transshipments in the network, arcs represent connection pipes between nodes.

2 THE BASIC PROBLEM

The goal of the WDND optimization problem is to find the lowest cost network that can satisfy demands under the hydraulic and continuity equation restrictions and under the penalties that have been imposed to ensure predefined hydraulic heads in node subsets.

In this paper we refer to a simplified problem of pipe network design for a pressure system (without pumping-stations) with the following main features:

- The network demands are known and configured as node-outflows;
- Different demand patterns are considered;
- The continuity of flow must be maintained at all nodes in the pipe-network;
- The head loss in each pipe-arc is a known function of the flow in the pipe, its diameter, length, and hydraulic properties:
- At each node minimum and maximum pressure head limitations must be satisfied;
- At each arc minimum and maximum velocity limitations can be imposed;
- Diameter constraints may be applied to the pipes;
- Different possible pipe-arc states and design options can be considered.

In the network $G = (N, R)$ the existing pipes (that have known diameters) as well as new pipes can be taken into account. For each pipe-arc, different options are examined, i.e. leaving exactly the same pipe-arc (LEAVE), cleaning the existing pipe (CLEAN), adding a new pipe-arc to the existing one (DUPLICATE), and installing a new pipe-arc (NEW).

The general constraint equations considered for a given demand pattern are as follows:

Continuity at each node:

$$\sum_{j \in R_i} Q_j + q_i = 0 \quad i \in N \tag{1}$$

where Q_j represents the flow in each of the set of pipes R_i, R connected to node i , and q_i, is the demand at node i.

Head-loss equation:

$$\boxed{H_{i_1} - H_{i_2} = \frac{k\, L_j (Q_j)^{\beta_j}}{(C_j)^{\alpha} (D_j)^{n_j}} \quad j=(i_1\ ,\ i_2 \in R)} \tag{2}$$

where H_i is the node-head, L_j the length of pipe j from node i_1 to node i_2, C_j the roughness coefficient and D_j the pipe diameter.

Minimum pressure head constraint:

$$H_i \geq H_i^* \quad i \in N \tag{3}$$

where H_i^* is the node hydraulic-head that must be guaranteed.

Bounds on water velocity in the pipe:

$$V_{j,\min} \leq V_j \leq V_{j,\max} \quad j \in R \tag{4}$$

Bounds on pipe diameters:

$$D_{j,\min} \le D_j \le D_{j,\max} \quad j \in R \tag{5}$$

where the minimum diameter refers to the existing diameter in the event of duplication.

Considering s different demand patterns ($s = 1, S$), the purpose is to optimize a non-linear objective function (O.F.), i.e. the total cost needed to construct new pipes, or clean or duplicate the existing ones. For the last two cases, we used the "equivalent diameter" approach (Simpson et al., 1994; Liberatore et al., 1998). Moreover, implementing the optimization procedures, equation (3) is relaxed on the O.F. as penalty components depending on whether the network satisfies the minimum pressure constraints at the nodes, and equation (4) can be treated as flow bounds for defined diameters. After generating an initial network configuration, the procedure performs a hydraulic analysis of the pipe network, resolving the non-linear system given by equations (1), (2), and (4). Node pressure differences from target values are then used in the O.F. to compute penalty costs.

The O.F. assumes the general form:

$$\boxed{\begin{aligned}\min \quad & \sum_{j\in R_1} C_{1j}\, L_j + \sum_{j\in R_2} C_{2j}\, L_j + \sum_{j\in R_3} C_{3j}\, L_j + \\ & + \sum_{j\in R_4} C_{4j}\, Lj + \sum_{s=1,S} \left(\sum_{i\in N^*} C_{5i}\, (H_i^* - H_i)^{\gamma} \right)\end{aligned}} \tag{6}$$

where the first term refers to the maintenance of old pipes, the second to cleaned pipes, the third to the duplicate set, the fourth to new pipes and the fifth to hydraulic head differences.

3 METAHEURISTIC OPTIMIZATION

As mentioned previously, the design must consider the discrete elements of the system, and an adherent formulation of the problem generates non-linear and non-convex models. Traditional design methods, though still in use today, are based on trial and error, and are guided by single iteration results and by user-experience. Recent works based on the application of metaheuristics may consider design of new pipes as well as duplication and maintenance rehabilitation. For the WDND problem, moreover, metaheuristic algorithms have several advantages compared to other mathematical programming techniques, as they can be implemented without heavy a-priori requirements, such as convexity and differentiability in O.F. and constraints. Thanks to their ability to manage discrete variables, they can deal directly with the alternatives available (commercial diameters, cleaning, duplication, etc.). Each alternative consists of a set of discrete, organized strings that are usually coded using predefined rules. Starting from initial pipe-network configurations, the proposed metaheuristic procedures only use O.F. cost values or other fitness information, (i.e.: the hydraulic-head constraint violation to be penalized) to allow the algorithm to reach a feasible solution as a final optimum.

As extensively referred in Liberatore et al. (1998), the utilization of metaheuristic optimization procedures to the WDND problem has been summarized in the following steps:

1. Initialization procedure: The sets of possible network element configurations (i.e. diameters, pipe-states, etc.) are proposed to the algorithm; an initial set of values is thus adopted.
2. Hydraulic verification procedure: Continuity (1), head-loss (2), and velocity constraints (4) equations are solved retrieving pressure-heads at the nodes.
3. O.F. evaluation: For the chosen system configuration, the economic O.F. evaluations are added to the penalty evaluation caused by target node pressure violation.
4. Design variables values replacement: Using the fitness information, and thanks also to suitable strategic decisions, a new set is generated with the metaheuristic optimizer.
5. Optimization cycle closure: The cycle is closed and the procedure is returned to step 2.

The stop criteria can be related to the number of cycles and to improvements in a fixed number of iterations. Even though the tests indicate that optimal or near-optimal solutions are almost always obtained using well-calibrated procedure parameters related to the design variables replacement, metaheuristics do not guarantee that the global optimum will be reached. Moreover, the computational time needed to reach near-optimal configurations should be tested to check the ability of these approaches to fit real WDND problems.

In the following some general remarks on GA, TS, and SS will be given, as well as information on the applied codes.

GA algorithms are search algorithms based on the mechanism of natural selection and natural genetics. The primary monograph on the topic is by Holland (1975), and extended applications of this approach have been made by Goldberg (1989). As stated in the opening paragraph, many applications of this technique are available in the recent literature on WDND problems. This study uses the PGA-Pack Library optimization module (Argonne National Laboratories, 1996). One of the main advantages of the genetic algorithm is that situations that cannot be adequately described with only one numeric parameter can be represented synthetically. This is possible thanks to the fact that the symbolic code system used by the GA is related to each variable configuration. The tests carried out on the GA algorithm have shown that a correct calibration of the model parameters must be reached. Using the PGA-Pack Library, the following aspects should be kept under control:

– Initial population size;
– Definition procedure and string initialization;
– Population replacement parameters.

The O.F. evaluation number needed to reach optimality remains the most limiting problem when applying GAs to real water systems. This is mainly due to the large number of system configurations in each GA population.

The tabù-search approach main concepts are collected in Glover (1994), and Glover & Laguna (1997). As a matter of fact, TS is usually defined as a meta-strategy (Glover & Laguna, 1997; Hertz & De Werra, 1990) that guides several subordinated heuristics to produce solutions beyond those normally generated by the search for a local optimum.

The employed methodologies were essentially two: an adaptive memory and a "sensitive" exploration, both of which typify the method. The system practically exploits its memories in an attempt to avoid being trapped in attraction basins, or better, in order to direct the search towards domain areas believed to be more promising. Limitations on search space generally operate by direct exclusion of search alternatives that are classified as prohibited, hence tabù.

We have implemented a so-called reactive kind of TS scheme: RTS (Battiti & Tecchiolli, 1994). An extended illustration of the RTS variant to WDND problems has been reported in Fanni et al. (1999). In this variant, the RTS memory structure of the past solutions is dynamical, and its optimal value is estimated automatically from the algorithm by means of a retroactive assessment of the search history. The reaction mechanisms are related to the transition frequency of the current solution. The RTS algorithm has been implemented using a general purpose tabù search tool called Universal Tabù Search (UTS) (Fanni et al., 1998).

Scatter search metaheuristics, as well as GAs, is designed to operate on a set of solutions maintained from iteration to iteration, while TS typically maintains only one solution by applying specific mechanisms to update solutions from one iteration to the next. A description of the SS can be found in Glover & Laguna (1997). In the present paper, we have developed an interface for WDND problems using the OptQuest general-purpose optimizer as a resolution module. OptQuest was developed by Glover et al. (1996) using scatter search methodology. This optimizer uses the SS framework associated with tabù search strategies to obtain enhanced solutions for problems defined using complex settings. The optimization process is organized in such a way as to utilize auxiliary solutions in evaluating the combination obtained from the previous solutions, and in generating new solution vectors actively. A significant difference between classical GA implementation and SS is that, while the former heavily relies on randomization and somewhat limiting operations to create new solutions, the latter employs strategic choices and memory along with a combination of solutions to generate new solutions. Moreover OptQuest exploits a neural network accelerator trained on the historical data collected during search. Even though OptQuest can be used to take into account the linear and non-linear constraints of the model, in the interface we have implemented the OptQuest module as the GA and TS above, only in design variables replacement using fitness information.

4 TEST CASES

In order to test metaheuristic optimization techniques, we have examined a case-study presented by Gessler (1985) and considered also by other authors (Simpson et al., 1994; Liberatore et al., 1998, Fanni et al., 1999). The system is shown in Figure 1; the solid lines represent existing pipes, while the dashed lines are the new pipes to be dimensioned. The given possibilities are to plan five new pipes ex-novo, as well as to choose among options that include cleaning, duplication, or maintaining three existing pipes. The Gessler problem considers eight different pipe sizes (commercial diameters) available for new pipes, while existing pipes may be left as they are, cleaned, or duplicated with new pipes. Moreover, two supply resources are available, and three demand patterns together with their associated minimum pressure heads must be satisfied. Elevation, pipe length, roughness, and demand patterns are given as well as pipe cost, and available diameters. The Gessler problem can be used to compare solutions obtained with different approaches. With non-linear optimization software, only near-optimal solutions were obtained. Simpson et al. (1994) perform an exhaustive search of all possible combinations for the Gessler scheme. Besides the complete enumeration, the same problem solved with the optimization package GINO (Liebman et al., 1986) but relaxing a few constraints, such as the discrete nature of the variables, only leads to poorly accurate solutions especially for small size pipes.

Starting from the Gessler scheme, two design problems, described in the following, will be used as benchmarks to test the performance of the proposed TS algorithm. Following Liberatore et al. (1998), the hydraulic scheme is used as a benchmark case with the same constraints on the flow rate and the hydraulic head at the nodes. But, besides providing for a reliability procedure of the system, the search space is extended to include, among the options for the existing arcs, replacing the pipeline completely, and doubling the alternatives for the available pipes. In the latter case, the

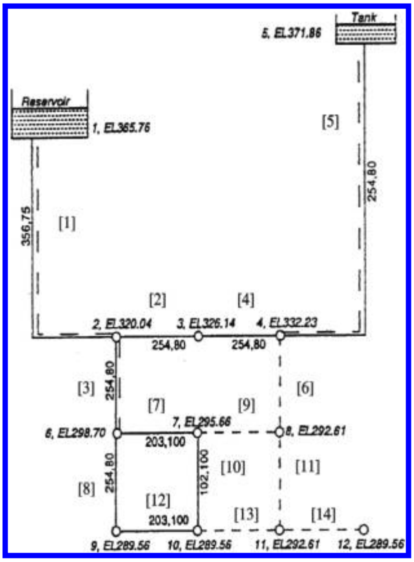

Figure 1. Gessler (1985) network scheme.

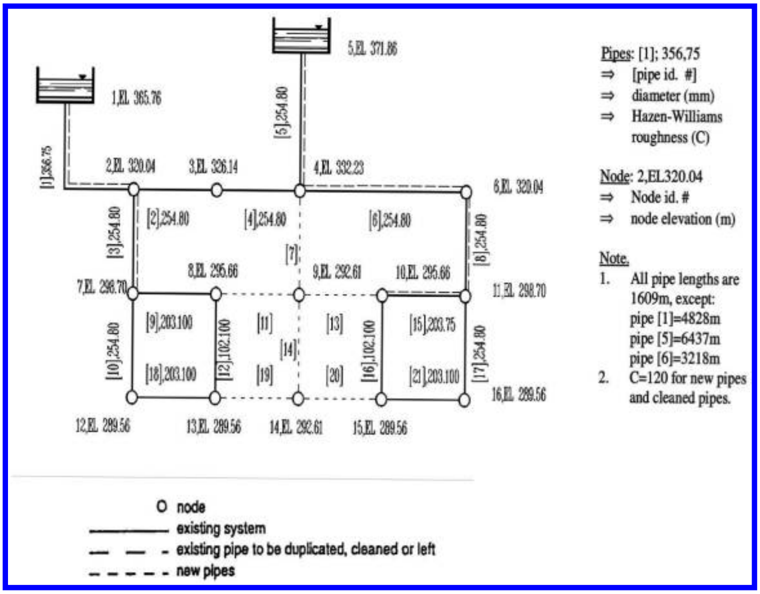

Figure 2. Extended Gessler network scheme.

Table 1. Demand patterns associated with the Gessler network.

	Demand pattern 1		Demand pattern 2		Demand pattern 3	
Node	Demand l/s	Min. head m	Demand l/s	Min. head m	Demand l/s	Min. head m
2	12.62	28.18	12.62	14.09	12.62	14.09
3	12.62	17.61	12.62	14.09	12.62	14.09
4	0	17.61	0	14.09	0	14.09
6	18.93	35.22	18.93	14.09	18.93	14.09
7	18.93	35.22	82.03	10.57	18.93	14.09
8	18.93	35.22	18.93	14.09	18.93	14.09
9	12.62	35.22	12.62	14.09	12.62	14.09
10	18.93	35.22	18.93	14.09	18.93	14.09
11	18.93	35.22	18.93	14.09	18.93	14.09
12	12.62	35.22	12.62	14.09	50.48	10.57

total search space is $3.436 \cdot 10^{10}$ possible configurations (32 alternatives for existing arcs and 16 for new arcs). In the following we refer to this problem as Problem 1.

The need to check the performances of the implemented algorithms led us to the study of a further scheme that introduces an increment in the number of variables, increasing the arcs to 21 and obtaining a search space of $1.801 \cdot 10^{16}$ possible configurations. We refer to this problem as Problem 2. This expanded scheme is reported in Figure 2.

5 RESULTS

In this section we report a brief summary of the results obtained applying metaheuristic approaches to the benchmark cases, and comparing them. In Table 2 and 3, we report statistics over 100 run results obtained using the previously described resolution modules implementing the GA, TS, and SS approaches. Each run starts from randomly generated initial configurations.

Table 2. Average results (100 runs) obtained from Problem 1.

Optimization technique	Average cost ($)	Success %	Average iteration	Average O.F. evaluations
GA	1,818,756	80%	205	20,790
TS	1,750,300	100%	172	2632
SS	1,750,300	100%	124	1110

Table 3. Average results (100 runs) obtained for Problem 2.

Optimization. technique.	Average cost ($)	Success %	Average iteration	Average O.F. evaluations
GA	2,436,279	60%	498	50,500
TS	2,256,013	100%	698	11,753
SS	2,256,013	100%	264	6818

The presented results report the following indications:

– The average of obtained minimum cost functions;
– The relative occurrence frequency of the optimum;
– The average number of iterations needed to assess the corresponding optimum;
– The number of O.F. evaluations needed to reach the optimum.

As can be observed in Table 2 for Problem 1, TS and SS find absolute optima in 100% of the cases, with a relatively low number of function calculations.

The performance of the applied techniques is remarkably better than any other previously described method. For example, the cost of the optimum solution supplied by Gessler (1985) with selective enumeration is 4.8% higher than the absolute optimum.

Though according to correct guidelines, the search space may drop to the 2632 checks by TS (negligible, considering the total search space) and even further using SS, on average 1110 evaluations. Moreover, a comparison of the results for Problem 1 shows that the success percentage rises from 80% to 100% using TS and SS instead of GA. Besides this clear difference in success percentage, it should be observed that the function calculations are remarkably more numerous in GA.

Table 3 shows the results obtained by metaheuristics for Problem 2. The Table shows that also for this problem, SS results are significantly better than those obtained using TS, and even more so than those obtained using GA. Using a PC Pentium, the CPU time required to solve the extended scheme is never longer than a few minutes in any run.

The global optimum of the water network considered in Problem 2 is unknown, since a complete enumeration is clearly prohibitive. Nevertheless, a reliability check of the obtained optimal solution has been carried out by the water system solver WaterCAD (Haestad Methods, 1998).

6 CONCLUSIONS

Three different metaheuristic approaches are proposed to design water distribution systems. The complexity of the real water distribution network problem grows as it becomes necessary to consider non-smooth non-convex large-size problems and discrete variables. Metaheuristic techniques seem to overcome many of these difficulties. The resolution interface developed for pipe-network optimization problems using GA, TS, and SS modules can easily manage this class of models. The computation time needed to reach optimal configuration is frequently the limiting problem to the use of this approach in real water distribution systems, and remains extremely limited in the considered test systems, even when common PCs are used.

Metaheuristic optimization seems to be a viable method for WDND problems, even though the computational effort of the proposed resolution algorithms in solving more complex problems normally arising in real water system design and management is still to be checked.

REFERENCES

Argonne National Laboratory. 1996. *PGA – Pack Library. Parallel Genetic Algorithm.* Argonne (Il).

Alperovits E. & Shamir U. 1977. Design of optimal water distribution system, *Water Resour. Res.*, 13(6): 885–900.

Battiti R. & Tecchiolli G. 1994. The Reactive Tabu-Search, *ORSA Journal on Computing*, 6 (2): 126–140.

Ben-Tal A., Eiger G. & Gershovitz Z. 1994. Global minimization by reducing the duality gap. *Math. Programming*. 63: 193–212.

Dandy G.C., Simpson A.R. & Murphy L.J. 1993. A review of pipe network optimization techniques, *Proceedings of Watercomp '93*. Melbourne.

Dandy G.C., Simpson A.R. & Murphy L.J. 1996. An improved genetic algorithm for pipe network optimization, *Water Resour. Res.*, 32 (2): 449–458.

Eiger G., and Shamir U. & Ben-Tal A. 1994. Optimal design of water distribution networks, *Water Resour. Res.* 30 (9): 2637–2646.

Fanni A., Bibbo' S., Giua A. & Matta A. 1998. A general purpose Tabu Search code: an application to Digital filters design, *IEEE Int. Conf. on Systems, Man, and Cybernetics*. San Diego.

Fanni A., Liberatore S., Sechi G.M., Soro M. & Zuddas P. 1999. Optimization of Water Distribution Systems by a Tabù Search Metaheuristic. In Laguna and J.L.Gonzales-Velarde (eds), *Computing Tools for Optimization and Simulation*. Boston: Kluwer Academic Publ.

Fujiwara O. & Khang D.B. 1990. A two phase decomposition method for optimal design of looped water distribution networks. *Water Resour. Res.* 26 (4): 539–549.

Gessler J. 1985. Pipe network optimization by enumeration, Proc. *Computer applications for water resources*: 572–581.

Glover F. & Laguna M. 1997. *Tabu Search*. Boston: Kluwer Academic Publishers.

Glover F. 1994. Tabu Search fundamentals and uses. University of Colorado, Boulder.

Glover F., Kelly J.P. & Laguna M. 1996. New Advances and Applications of Combining Simulation and Optimization. In J.M. Charnes, D.J. Morrice, D.T.Brunner and J.J. Swain (eds). *Proceedings of the 1996 Winter Simulation Conference*: 144–152.

Goldberg D.E. 1989. Genetic Algorithms in Search. *Optimization and Machine Learning*. Reading: Addison-Wesley Publ. Co. Inc.

Haestad Methods. 1998. *WaterCAD*. Waterbury, CT.

Hertz A. & De Werra D. 1990. The Tabu Search metaheuristic: how we used it. *Annals of Mathematics and Artificial Intelligence.* 1/1990: 111–121.

Holland J.H. 1975, Adaptation in natural and artificial systems. Ann Arbor: The University of Michigan Press.

Kessler A. & Shamir U. 1989. Analysis of the linear programming gradient method for optimal design of water supply networks. *Water Resour. Res*. 25 (7): 1469–1480.

Liberatore S., Sechi G.M. & Zuddas P. 1998. A Genetic Algorithm approach for water system optimization. *AICE Annual Conference*. Milano.

Liebman J.S., Lasdon L., Schrage L. & Waren A. 1986. Modelling and optimization with GINO. Palo Alto: The Scientific Press.

Lippai I., Heaney J.P. & Laguna M. 1999. Robust water system design with commercial intelligent search optimizers. Accepted by *J.Computing in Civil Eng*.

Murphy L.J. & Simpson A.R. 1992. Pipe optimization using genetic algorithms, *Res. Rep*. 93: 95, Dept. of Civ. Eng., Univ. of Adelaide.

Murphy L.J., Simpson A.R. & Dandy G. 1993. Design of a network using genetic algorithms, *Water*, 20: 40–42.

Simpson A.R., Dandy G.C. & Murphy L.J. 1994. Genetic algorithms compared to other techniques for pipe optimization. *Journal of Water Resources Planning and Management*, ASCE 120 (4): 423–443.

Optimisation methods for the calibration of water network systems

Ramon Pérez & Joseba Quevedo
Departament d'ESAII/UPC, Rambla Sant Nebridi, Terrasssa, Spain

ABSTRACT: The model of a water distribution network is a set of equations with parameters used in many tasks in water management. Such model needs to be calibrated to update the parameter values. Ormsbee and Lingireddy describe the calibration task (Omersbee 1997). The water distribution system must be represented by node-link database (links represent individual pipe sections, and nodes represent points in the system where two or more pipes join or where water is being input or withdrawn). The measurements are not ideal, the estimation-calibration problem is then formulated as a problem of minimising a suitably chosen measure of the inconsistencies in the microcalibration stage. In this paper the microcalibration stage is treated. The optimisation problem is characterised. A classification of optimisation problems and algorithms help to fix the kind of algorithms that can assure a good result. Three Global Optimisation Algorithms have been used. All three based on deterministic search using branch and bound. One takes advantage of the signomial formulation of the problem (Falk 1973). The second is the algorithm DIRECT based on lipsichtz properties of the function (Holmström 1998), the one that has given best results in time consume and robustness. The third uses intervalar arithmetic to bind the function (Kearfott 1996), in this case the software problems seem to be the limitation for its application. Small examples help to understand the different phases of the process.

1 INTRODUCTION

In the calibration process two types of modelling errors are detected and solved (Omersbee 1997). The macrocalibration process can detect some errors in the model as have been shown in the previous chapter. These are big errors, the ones that are not solved by tuning some parameters, as their existence is unexpected. But there are other errors in the model. Generally they are less dramatic but more widespread and abundant. These little mismatches with the reality are treated in identification problems as the variables of an optimisation problem because of the redundancy in the equations. The problem tries to find the set of parameters that give a simulation closer to reality. The reality provided by the measurements of levels, heads and flows together with some parameters that are assumed to be correct as the area of reservoirs, demand curves, length and diameter of the pipes are the inputs of the problem. The formulation comes from the known topology of the network and the laws of physics. The outputs of the optimisation are variables such as pipe roughness and demand factors. This idea is shown by figure 1.

Several researchers have proposed algorithms for system calibration. Walski (1983) suggested lumping groups of pipes to equivalent pipes and determining the lumped coefficients using analytic equations for single demand patterns. When the number of unknown parameters equals the number of nodes or pipes for single load patterns, it is possible to rework the equations of continuity and energy to consider the parameters as unknown, which can be solved in an iterative manner (Donachie 1974; Rahal et al., 1980; Gofman and Rodeh 1981). Shamir (1974) presented a method to calibrate the network using an optimisation algorithm, which could analyse single

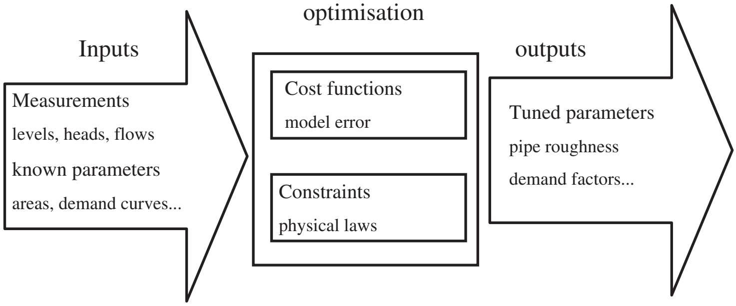

Figure 1. Flux diagram of the microcalibration.

demand patterns. Later, Coulbeck (1985) linearised the network equations and used lumped parameters in an optimisation procedure for single loading conditions. Finally, Ormsbee (1989) combined a simulation model and a modified Box Complex Direct Search method to consider sets of independent loads or extended time period simulations.

In next section the background of optimisation is presented. In section 3 some algorithms that have been tested in this application are described and illustrated with examples.

2 BACKGROUND

The optimisation problems can be classified depending on the mathematical characteristics of the objective function and constraints (Puig 1999). Such a classification is presented in figure 2 with special attention to the problem that is examined in this paper.

Optimisation problems are classified as *continuous* if the unknown variables take real and continuous values, that means that *D* is the domain of the real numbers. A problem is *discrete* if its variables take discrete values, normally integers and that is why they are known sometimes by *integer problems*. There are some problems with both types of variables that are called *mixed*. This is the first classification that appears in the tree. The problem examined in this paper belongs to the first group. All variables (heads, flows, demand factors, roughness, etc.) take real continuous values.

Constraints are another concept used for the classification. Problems can be *constrained* if they have constraints otherwise they are *unconstrained*. Depending on the form of these constraints it can be classified in *linear* or *non-linear*. If the constraints that define the domain *D* are linear the problem is relatively easy to solve. Included in this type of problems there are two classes that have been exhaustively studied and well solved: *lineal program* and *quadratic program***.** Problems with non-linear constraints are difficult to solve as they determine feasibility zones difficult to find and a non-linear objective function can have local minima. A problem is *convex* if the function *f* and domain *D* are convex, these problems have only a global minimum. All problems that don't satisfy this condition are *non-convex*. In this last group are included most of the real problems.

$$J = \sum_{l=1}^{t} \left[\sum_{i=1}^{r+n} \left(h_{0il} - h_{il} \right)^2 + \rho \sum_{i=1}^{m} \left(q_{0il} - q_{il} \right)^2 \right] \qquad (1)$$

where
h_{il} is head at the node (junction or reservoir) *i* in time step *l* (variable)
h_{0il} is head at the node (junction or reservoir) *i* in time step *l* (measurement)
q_{il} is flow in element *i* in time step *l* (variable)

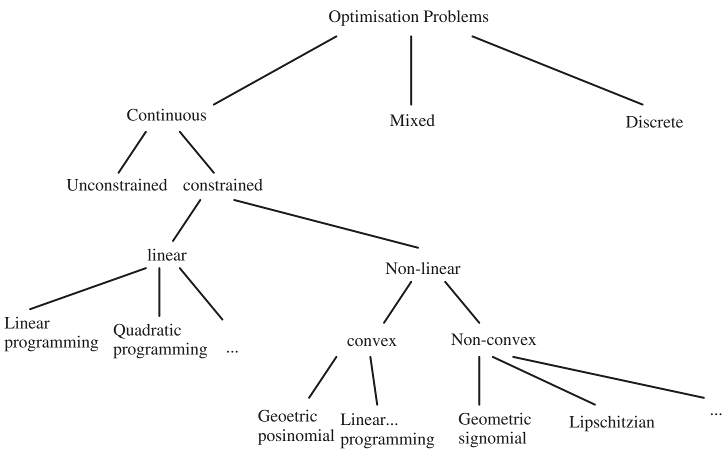

Figure 2. Optimisation problem classification.

q_{0il} is flow in element *i* in time step *l* (measurement)
ρ is the weight factor
r is number of reservoirs
n is number of junctions
m is number of elements
t is number of time steps

$$A_i h_{il+1} - A_i h_{il} - \sum_{j=1}^{p} q_{jl} = 0 \ \mathit{for}\ \forall \ \mathrm{i} \le \mathrm{r} \text{ and } \mathrm{l} \le \mathrm{t} \tag{2}$$

h_{il} is the level of water in reservoir *i* at time step *l*
q_{jl} are the flows that arrive to reservoir *i* in time step *l*
p is number of flows that arrive to the reservoir *1* in time step *l*
A_i is the area of reservoir *i*

$$\sum_{i=1}^{m} \Lambda_{ji} q_{il} - d_{jl} = 0 \ \mathit{for}\ \forall \ \mathrm{j} \le \mathrm{n} \text{ and } \mathrm{l} \le \mathrm{t} \tag{3}$$

d_{jl} are the demands at junction *j* in time step *l*

$$h_{1il} - h_{2il} - R_i q_{il}^{1,85} = 0 \ \mathit{for}\ \forall \ \mathrm{i} \le \mathrm{m} \text{ and } \mathrm{l} \le \mathrm{t} \tag{4}$$

The problem treated here (equations 1–4) has constraints that are the physics of the network. The region determined by constraint 4 is non-convex as it is a non-linear equality constraint.

When the problem is non-convex a new classification is needed from the algorithm point of view. This is showed in figure 3.

Now the problem is classified depending on the aim of the optimisation. Here the global minimiser is searched as this is the one that has the parameters of the network. *Deterministic* optimisation methods do an exhaustive search of the feasibility domain *D* of the optimisation problem. In order to do this they use different strategies. *Branch and Bound* methods find and keep areas that may contain minima while eliminating the ones that cannot contain any. *Covering* based

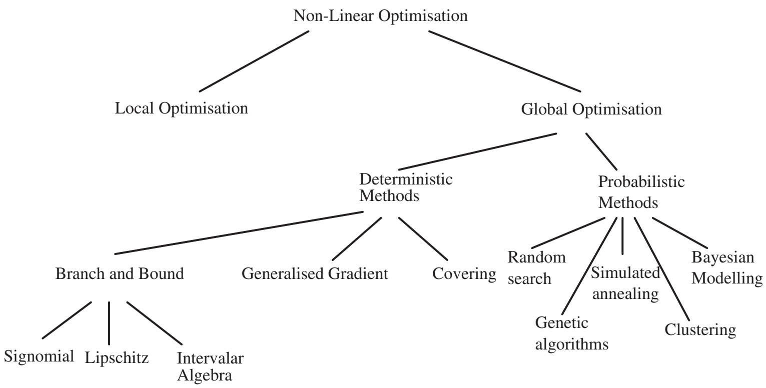

Figure 3. Optimisation algorithms classification.

methods approach the solution using tighter bounds each time. *Generalised Gradient* methods avoid being trapped in a local optimum changing the shape of the function.

3 PROBLEM SOLUTION

The problem presented in equations 1–4 corresponds to a optimisation problem with equality and inequality (variables bounds) constraints. The box $X = (x_1, \ldots, x_n)$ defines an artificial limitation on the search region, while $\underline{x}_i \leqslant x_i \leqslant \bar{x}_i$ represent actual bound constraints for a subset of the coordinates. In fact, if actual bound constraints exist, it can be advantageous to treat bound constraints separatedly from the inequality and equality constraints.

Deterministic algorithms used to find the global minimum of a function, in a region, do an exhaustive search of all such regions. In this paper three different algorithms have been used. All of them have in common that they use the branch and bound algorithm for the search. The basic branch and bound is presented:

Algorithm 1: Abstract Branch-and-Bound Pattern for Optimisation
INPUT: an initial box X_0.
OUTPUT: a list of boxes that have been proven to contain critical points and a list U of boxes with small objective function values, but which could not otherwise be resolved.

1. Initialise a list of boxes L by placing the initial search region $\mathbf{X_0}$ in L.
2. DO WHILE L ≠ Ø.
   a) *Remove the first box **X** from L.*
   b) *(Process **X**) Do one of the following:*
      - *reject **X**;*
      - *reduce the size of **X**;*
      - *determine that **X** contains a unique critical point, then find the critical point to high accuracy;*
      - *subdivide **X** to make it more likely to succeed at rejecting, reducing, or verifying uniqueness.*
   c) *Insert one or more boxes derived from **X** onto L, U or C, depending on the size of the resulting box(es) from Step 2b and whether the (possible) computational existence test in that step determined an unique critical point.*

   END DO

Algorithm 1 represents a general description, and many details such as stopping criteria and tolerances, are absent. Such details differ in particular actual algorithms. The three algorithms used in this paper used different methods for both branching (subdivide the boxes or regions) and

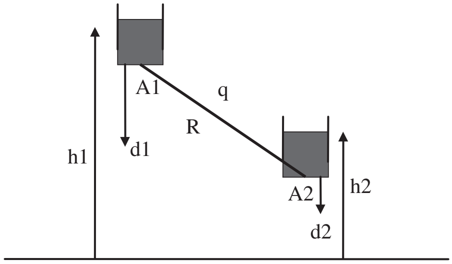

Figure 4. Simple network.

bounding (finding lower and upper bounds for the function in each region in order to reject boxes). Reasons for selecting these algorithms were different in each one.

3.1 *Signomial programming*

In the characterisation of the problem, it was formulated as a signomial problem. It is a non-convex problem but taking advantage of the form and using a special way in the branching process the bounding can be done based on convex optimisation. This algorithm has been used to utilise the geometry of the problem.

There have been some investigations of methods for finding global minima for signomial problems. Falk (1973) proposes a branch-and bound algorithm that converges to global minima for (nonconvex) signomial programs.

Algorithm 2:

1. Initialisation. Set up and solve Problem Q^1 to obtain $\mathbf{z^1}$, v^1, V^1, U^1. Set $s = 1$, $t = 1$, $I(1)$
2. Check for solution. If $V_b(s) = v_b(s)$ then a global solution of Problem Q is $\mathbf{z}^* = \mathbf{z}^T$ where $v^T = V^s$. Otherwise go to step 3.
3. Choose a Branching Node T. Select $T \in I(s)$ such that $v^T = v_b(s)$. Go to Step 4.
4. Select a Branching Term $(\mathbf{a_i}, \mathbf{z})$
   a) Select a $K \in \{0,1,\ldots q\}$ such that $L_k^T(z^T)$-$H_k(z^T)$ is maximal.
   b) Select an $I \in IH(K)$ so that $l_i^T(z^T)$-$e^{(ai,zT)}$ is maximal.
   c) Go to Step 5
5. Generate Problem Q^{2s}. Set $m_i^{2s} = m_i^T$; $M_i^{2s} = M_i^T$; $\forall i \in IH;\ i \neq I.$ $M_I^{2s} = (a_I, \mathbf{z}^T)$; $m_I^{2s} = m_I^T$.
6. Generate Problem Q^{2s+1}. Set $m_i^{2s+1} = m_i^T$; $M_i^{2s+1} = M_i^T$; $\forall i \in IH;\ i \neq I.$ $m_I^{2s+1} = (a_I, \mathbf{z}^T)$; $M_I^{2s+1} = m_I^T$.
7. Solve Problems Q^{2s} and Q^{2s+1}
   a) Solve (or check feasibility) Problems Q^{2s} and Q^{2s+1} to obtain: $\mathbf{z}^{2s}$, $\mathbf{z}^{2s+1}$, $\mathbf{v}^{2s}$, $\mathbf{v}^{2s+1}$, u^{2s}, u^{2s+1}.
   b) Compute V^{s+1} and U^{s+1}.
   c) Update the set $I(s+1)$ from $I(s)$: delete the node T, add two nodes $2s$ and $2s+1$. Set $s := s+1$.
   d) Go to Step 3.

Example 1: Figure 4 shows the simplest system used to illustrate this algorithm. Heads are known, so variables are flow and roughness. Constraint 4 is expressed in residual form. The problem appears this way. There are no constraints and only two variables in order to make it understandable.

$$\begin{aligned} &min\ f_0(x) = (hr1 - h1 - R_1 q_1^{1.852})^2 = \\ &= (hr1 - h1)^2 + R_1{}^2 q_1^{3.7} - 2(hr1 - h1) R_1 q_1^{1.852} = g_0(x) - h_0(x) \end{aligned} \tag{5}$$

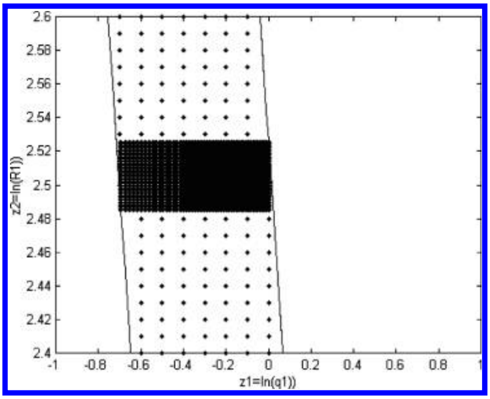

Figure 5. Boundary region.

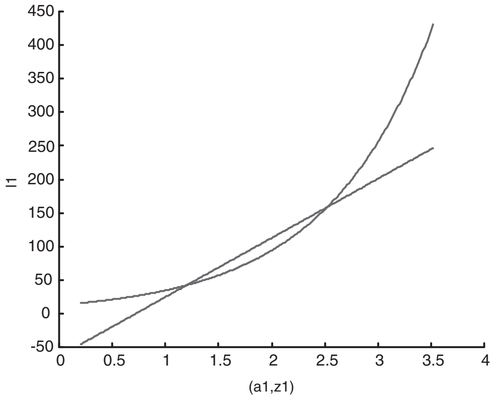

Figure 6. Overestimation of the H terms.

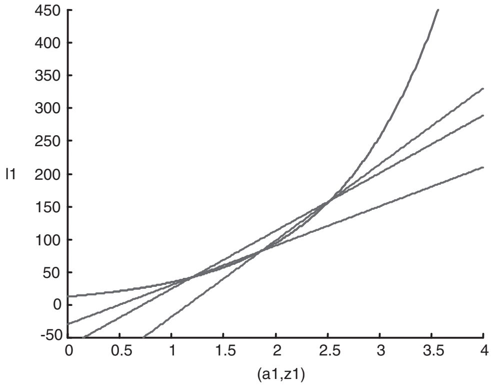

Figure 7. Overestimation in second step.

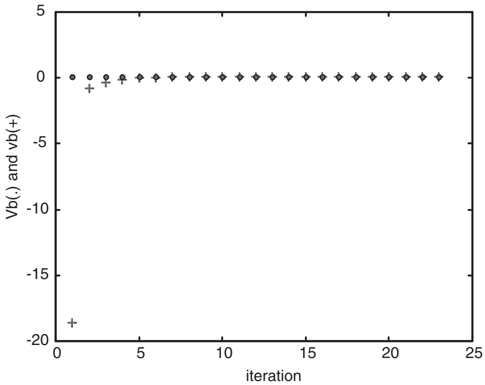

Figure 8. Convergence of the boundaries.

In figure 5 both regions R (original boundary region) and S^1 for the first problem is shown. In order to get this region all these values have been calculated:

The overestimation of the only H term appears in figure 6. These are the values for this special case:

In figure 7 appears the new overestimation for the two new problems Q^2 and Q^3. Such overestimations are convex problems that are solved in step 7 of the algorithm.

This algorithm assures the global optimal. In figure 8 the boundaries of the cost function appear and it can be seen that it converges and solution is found. It generates a huge number of optimisation problems that become unbearable when the number of variables increases as occurs with problems in the real network.

3.2 *Lipschitzian optimisation*

Some branch-and-bound algorithms use the Lipschitz constant of the function for the bounding process. The research group works mainly with Matlab software and there is a collaboration agreement with the distribution company. In Matlab environment there is commercial software for global optimisation based in these algorithms and has been tested for the calibration problem. DIRECT (dividing rectangles) is an algorithm for finding the minimum of a Lipschitz continuous function. DIRECT is designed to solve problems subject to box constraints.

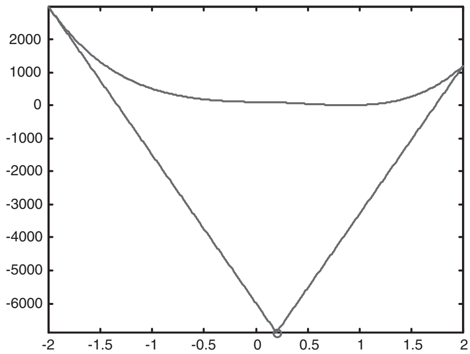

Figure 9. Cover of the non-convex function by a linear approximation.

Example 2: Figure 9 shows how the algorithm covers the non-convex function with convex approximations using the same cost function as in example 1. This is the constraint that makes the problem non-convex. Here it appears one-dimensional in order to show the way this algorithm works as simply as possible.

The idea is now to divide the search area into two intervals $I_1 = [a, x(a, b)]$ and $I_2 = [x(a, b), b]$, calculate the new values of x and B for each of these two intervals and choose a new interval to divide. Obviously the interval with the lowest value of B will be chosen. Following these ideas is Piyavskii's [Piyavskii 1972] algorithm, that obtains a piecewise linear function, this produces a better approximation of the real function in every iteration.

Algorithm 3

1. $n = 1$, $sample = 1$, $l_{sample} = a$, $u_{sample} = b$
2. *Calculate* B_1, x_1
3. *Do while* $n < numit$
   a) $n = n + 1$
   b) $l_n = x_{sample}$, $u_n = u_{sample}$, $u_{sample} = x_{sample}$
   c) *Calculate* $B_n, x_n, B_{sample}, x_{sample}$
   d) *Choose new interval to sample*
4. *Substitute* x_{sample}, l_{sample}, u_{sample} and B_{sample} by x_n, x_{sample}, l_n, l_{sample}, u_n, u_{sample}, B_n, B_{sample}

In this algorithm the first two steps are the initialisation, where n is a counter-variable, *sample* is the interval in which the algorithm is sampling and l_{sample} and u_{sample} are the lower and upper bounds of the sampling area. In the inner loop the counter-variable n is increased, after which the active interval is divided into two new intervals. In the next step the algorithm recalculates the values of x and B and then chooses a new interval to sample. This new interval will be the interval with the lowest value of B. This is done until a given number of function evaluation is done. In figure 10 an example of the first 3 steps iterations is shown and it can be seen that the piecewise linear function $\hat{f}$ becomes a better approximation of the real function in every iteration. Because of the appearance, $\hat{f}$ is often called saw-tooth cover.

The Tomlab optimisation environment for MATLAB is a flexible and reliable environment for the solution of most types of applied optimisation problems. Tomlab/SOL was written by Professor Kenneth Holmström, in cooperation with the Systems Optimisation Laboratory (SOL) at Stanford and UC San Diego. It runs on a matlab 5 or later version.

The toolbox includes two solvers for costly global optimisation, which are possible to mix together in the solution process. The routine *glcSolve* implements an extended version of DIRECT that handles problems with both nonlinear and integer constraints. Since no Lipschitz is used there is no natural way of defining convergence (except when the optimal function value is known). Therefore *glcSolve* is run for a predefined number of function evaluations and considers the best function value found as the optimal one. It is possible for the user to restart *glcSolve* with the final

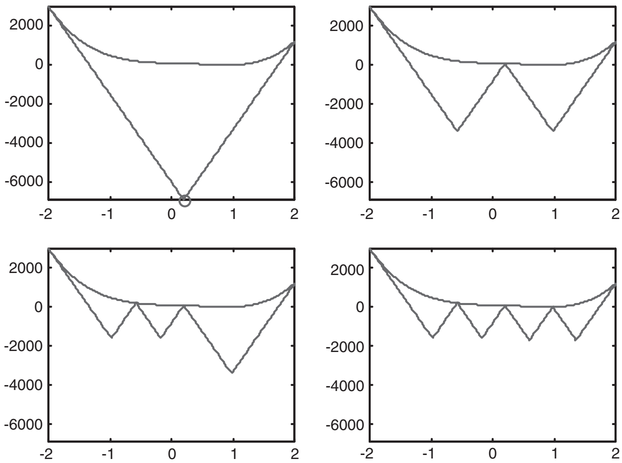

Figure 10. Cover in the first three steps.

status of all parameters from the previous run. DIRECT does not exactly handle equality constraints but the inequality constraint can be used equivalently.

The example of two-reservoirs has been identified using 39 iterations and 6.65 CPU time. Barcelona simplified network woith 8 reservoirs and 11 pipes has been identified, as the results seemed to be promising another more complex network has tried to be identified. No better results than those obtained by the local optimisers were found.

3.3 *Interval arithmetic*

The research group has worked since 1998 in interval arithmetic with different purposes such as fault detection, robust control, identification.… The knowledge on this subject included the optimisation algorithms that use interval arithmetic for the bounding of the function and interval boxes for the branching. A tool based on these algorithms has been tested presenting the best results.

Description of a global optimisation problem requires specification of an objective function and constraints (Kearfott 1996). However, algorithms for verified global optimisation may require both floating point and interval values of the objective function, gradient and Hessian matrices, as well as, possibly, interval values of the intermediate quantities obtained in computing a natural interval extension. With floating point and interval values of the constraints and their gradients, this makes at least ten different routines that would be required for each problem, an unacceptable burden if many problems are to be solved. The early approaches used numerical differentiation, others pre-processed using symbolic packages such as Maple. Each of these approaches has failings. Numerical differentiation can be over-whelmed by both truncation and roundoff error. Symbolic differentiation requires substantial separate machinery, and can also result in expressions for the derivatives that are many times larger than the optimal. An alternate technique is *automatic differentiation*.

A combination of several techniques is used in state-of-the-art interval global optimisation codes for step 2b of algorithm 4. Some of these techniques are outlined as the following algorithm.

Algorithm 4(Range check and Critical Point Verification)

INPUT: *a box* $\boldsymbol{X}$ *and a current rigorous upper bound* $\bar{\phi}$ *on the global minimum.*

OUTPUT: *one or more boxes derived from* $\boldsymbol{X}$*, or the information that* $\boldsymbol{X}$ *cannot contain a global minimum, or information that* $\boldsymbol{X}$ *contains a critical point.*

1. *(Feasibility check; for constrained problems only)*
   a. *(Exist if infeasibility is proven)* DO *for* $i = 1$ *to* m*:*
      i. *Compute an enclosure* $c_i(\boldsymbol{X})$ *for the range of* c_i over $\boldsymbol{X}$.

ii. IF $0 \notin c_i(\boldsymbol{X})$ THEN *discard* ***X*** *and* EXIT
b. Verify, if possible, that there exists at least one feasible point in ***X****.*

2. *(Range check or "midpoint test")*
   a. Compute a lower bound $\underline{\phi}(\mathbf{X})$ *on the range of* ϕ *over* ***X****.*
   b. IF $\underline{\phi}(\mathbf{X}) > \bar{\phi}$ THEN *discard* ***X*** *and* EXIT
3. *(Update the upper bound on the minimum.) If the problem is unconstrained or feasibility was proven is Step 1b,* THEN
   a. *Use interval arithmetic to compute an upper bound* $\bar{\phi}(\mathbf{X})$ *of the objective function* ϕ *over* ***X****.*
   b. $\bar{\phi} \leftarrow min\{\bar{\phi}, \bar{\phi}(\mathbf{X})\}$.
4 *("monotonicity test")*
   a. Compute ab enclosure $\nabla\phi(\mathbf{X})$ *of the range of* $\nabla\phi$ *over* ***X****.. (Note: if* ***X*** *is "thin", i.e. some bound constraints are active over* ***X****, then a reduced gradient can be used)*
   b. IF $0 \notin \nabla\phi(\mathbf{X})$ THEN *discard* ***X*** *and* EXIT.
5 *("concavity test") If the Hessian matrix*[2] $\nabla^2\phi$ *cannot be positive definite any-where in* ***X*** THEN *discard* ***X*** *and* EXIT.
6 *Quadratic convergence and computational existence/uniqueness) Use an interval Newton method (with Fritz John equations in the constrained case) to possibly do one or more of the following:*
   reduce the size of ***X****;*
   discard ***X****;*
   verify that a unique critical point exists in ***X****.*

(Bisection or geometric tessellation) If Step 6 did not result in a sufficient change in ***X****, then bisect* ***X*** *along a coordinate direction)(or otherwise tessellate* ***X****), returning all resulting boxes for subsequent processing.*

End Algorithm 4

The Global Solution initiative of sun Microsystems is intended to develop R. Baker Kearfott's INOPT_90 software for solving non-linear systems and global optimisation problems into a commercial quality package. It has been used for the networks treated so far. The results didn't seem to guarantee the solution of any network. The cause of this problem may be more in the software than in the algorithm. The development of this software is on-going and it is difficult to say whether the improvement of such tools will overcome these difficulties.

4 CONCLUSIONS

The microcalibration process is not understood as an error detection method but one of tuning of parameters and is formulated as an optimisation problem. This formulation and its theoretical study confirm the need of global optimisers due to the non-convexity of the problem.

The comparative study of three methods has been done. A small example has been used in order to illustrate this application and to understand how these methods apply to water networks. The algorithms used for the study and the solvers are presented.

Signomial algorithms appeared overwhelmed by the simplest network. Interval arithmetics optimiser seemed to give promising results but the software available failed and however the improved versions of it could open a field of application. Lipschitz optimisation gives results that cannot be guaranteed as global minima with a similar limitation to that of the local optimisers.

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REFERENCES

Coulbeck B. *An application of hierarchical optimisation in calibration of large-scale water networks.* Optimal Control applications and Methods. June 1985. 6, 31–42.

Donachie R.P. *Digital program for water network analysis.* Journal of the Hydraulics Division, ASCE 1974. 100(3), 393–403.

Falk E. (1973) *Global Solutions of Signomial Programs, Technical Report T-274*. George Washington University, Washington DC, 1973.

Floudas C.A., Pardalos P.M. *Encyclopaedia of Optimization*. Kluwer Academic Publishers 2001 Netherlands.

Gofman E., Rodeh M. (1981) *Loop equations with Unknown pipe characteristics.* Jour. ASCE. – Hydraulics Div, ASCE September 1981. 107: 9: 1047.

Holmström K. (1998) *TOMLAB- A General Purpose, Open Matlab Environnment for Research and teaching Optimization*. Research Report in Mathematics. Technical report Ima-TOM-1997-3.

Kearfott R. Baker (1996) *Rigorous global search continuous problems by R. Baker Kearfott.*Dordrecht [etc.] Klumer Academic Publishers cop. 1996.

Omersbee L.E., Lingireddy S. *Calibrating hydraulic network models.* Journal of American Water Association. February 1997. 42–50.

Ormsbee L.E. (1989) *Implicit network calibration.* Journal of Water resources Planning and Management. ASCE March 1989. 115(2), 243–257.

Passy U., Wilde D.J. Generalized polynomial optimizations, SIAM J. Appl. Math., 15 (1967) pp.1344–1356.

Piyavskii S.A. *An algorithm for finding the absolute extremum of a function.* USSR Computational Mathematics and Mathematical Physics, 12: 57–67, 1972.

Puig V. *Aportación a la generación de Umbrales adaptativos.* Tesis doctoral Universida Politecnica de Catalunya. February 1999.

Rahal C.M., Sterling M.J.H., Coulbeck B. (1980) *Parameter Tuning for Simulation Models of Water Distribution Networks.* Proc. Institution of Civil Engrs., London September 1980. 69, 751–762.

Shamir U. *Optimal design and operations of water distribution systems.* Water Resources res. February 1974. 10(1), 27–36.

Walski T.M. (1983) *Technique for calibrating network models*. Journal of Water resources Planning and Management, ASCE, October 1983, 109(4), 360–372.

Optimal scheduling of South Staffordshire water supply system using the FINESSE package

P.L.M. Bounds, K. Ulanicka & B. Ulanicki
Water Software Systems, De Montfort University, UK

B. Dacre & G. Cummings
South Staffordshire Water, UK

ABSTRACT: This paper presents a part of the project concerning optimal scheduling of the South Staffordshire Water Supply System. The objective of this part was to introduce a computer aided water network engineering package called FINESSE into the company to support operational planning and decision-making. The package enables new hydraulic models to be constructed graphically by direct screen manipulation. An optimal network scheduler compliments a hydraulic simulator by calculating operational schedules and all other hydraulic results to the same accuracy. The pumping cost is evaluated using the hydraulic characteristics of the pumps and for the system. Other costs include source and treatment costs. It is possible to solve the scheduling problem over a one-week time horizon minimising the operational cost and satisfying all hydraulic constraints. The scheduler can handle complex tariffs with irregular time steps and can produce both continuous and discrete control schedules for each pump station.

1 INTRODUCTION

This paper presents a part of the project concerning optimal scheduling of the South Staffordshire Water (SSW) Supply System with the overall aim of reducing operating costs. Two alternative approaches were investigated, one based on a spreadsheet model (Tischer et al, 2003) and another on a FINESSE model (2003). This paper describes results of the FINESSE approach. The SSW company required a strategic simulation and optimisation model for weekly and daily operational planning purposes in order to manage energy more efficiently. The Company's existing approach already achieves significant operational cost savings through knowledge and experience of staff and the application of an operational planning process that is based on a weekly and daily planning horizon. The Company also uses some spreadsheet models based on mass-balance and energy cost calculations to support planning decisions. The objective of this part of the project was to introduce a computer aided water network engineering package called FINESSE into the company, in order to further support this planning process. The main aim of FINESSE is to model and optimise the SSW supply network in order to reduce total operating costs. The package enables new hydraulic models to be constructed graphically and alterations to pipes, pumps, reservoirs etc. can be made directly. Tools are included to automatically simplify, merge and simulate models as required. Furthermore, FINESSE includes an optimal network scheduler that compliments the hydraulic simulator by calculating operational schedules to the same accuracy. The optimisation model takes into account the head-flow hydraulic characteristics of the system as well as the mass balance for the reservoirs. The pumping cost is evaluated using the hydraulic characteristics of the pumps and for the system. Other costs include source and treatment costs. The optimal flow schedules and pump schedules for each pump station are calculated. It is possible to solve the scheduling problem over a one-week horizon minimising the operational cost and satisfying all hydraulic constraints. The scheduler can handle complex tariffs with irregular time steps and can produce both continuous and discrete schedules. A general algebraic modelling language and non-linear programming solver were integrated into the FINESSE package to solve these optimisation problems. The scheduler is not just used to analysing the overall cost savings but also to experiment with operating constraints. The system is operated within many volume, pressure and flow constraints. These have been derived over a long period of time according to operational experience and local considerations. Constraints effect operating costs and an overall analysis of constraints and their effects can reduce operating costs.

The following section describes the development of an optimisation model from spreadsheet and simulation models. The optimal scheduling algorithm and its implementation in a wider modelling environment is presented in section 3. Section 4 analyses the results of scheduling the optimisation model and then there are some conclusions.

2 MODEL BUILDING

2.1 *Network description*

SSW supplies drinking water to 1.2 m people over 1500 square km in the West Midlands, South Staffordshire, South Derbyshire, North Warwickshire and North Worcestershire areas of the UK. The system comprises of: 2 major river sources (290 Ml/D), 23 borehole stations, plus some transfers to and from other water companies, and 20 supply zones of varying complexity represented as approximately 18 network models. The total component numbers of these models are 45,694 pipes; 40,611 nodes, 41 sources, 242 valves, 44 pumps and 31 reservoirs. Electricity tariffs typically have 5 periods over a day.

2.2 *Simulation model*

The water company provided models of 18 zones of the network in WATNET format. The typical size of a zonal model was around 3000 to 5000 nodes. They were imported into FINESSE one by one, simulated and validated. The node coordinates were adjusted according to the correct geographical position and all zonal models were merged on one workspace using facilities available in FINESSE. The boundary conditions for each zone were removed and replaced by appropriate interconnections. The model of the whole SSW distribution network was created. The model includes:

- 42195 nodes
- 48109 components, among others:
  - 31 variable head reservoirs
  - 50 pump stations
  - 191 valves

It was used to view the whole system in detail, locate nodes and components etc. The model was then simplified to enable simulation experiments. To obtain the best possible layout of the simplified model the zonal models were reduced separately, merged and then reconnected. An automatic tool is available in FINESSE to accomplish simplification. The simplification algorithm is described fully in Ulanicki et al (1996). The demand was adjusted and the model was calibrated against SCADA data. Apart from two zones the simulation results were very close to SCADA values. It was concluded that the model could be used for simulation trials. The detailed and simplified network models in FINESSE are shown in figures 1 and 2.

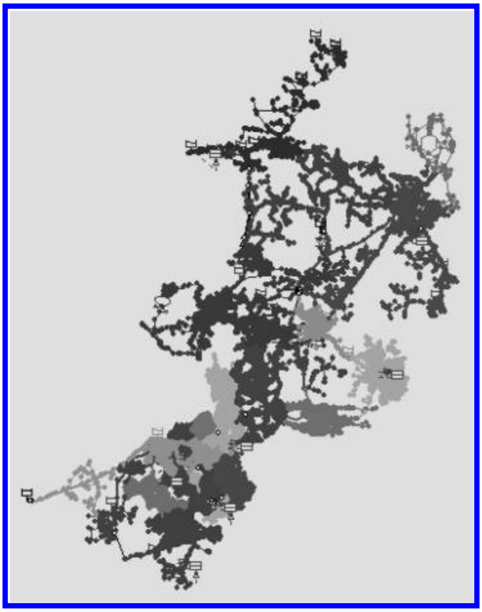

Figure 1. SSW network detailed model.

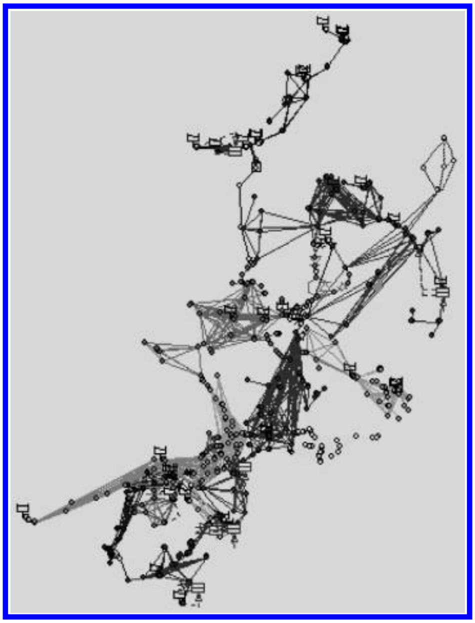

Figure 2. SSW network simplified model.

The process of updating and validating the detailed zonal models took longer than expected due to the underestimated size of the models and the many changes in the field since their last calibration. The simplification process substantially reduced the size of zonal models but not to the extent required for optimisation studies. Most of the simplified models still have too many elements (pipes and valves). This is due to the complicated network structure inside the zones, the large number of PRVs (e.g. 18 in one zone) often interacting, no simple supply line, and/or no clearly defined and separated DMAs. Nevertheless the simplified zonal models were merged and the model of the whole SSW network can be used successfully for trial and error simulations. Another benefit of the work was a better understanding of the supply system structure and its operation. The information and knowledge gained from this exercise enabled a further manual simplification of the model for optimisation purposes.

2.3 *Optimisation model*

The initial optimisation model was built using some of the structural data extracted from simulation models and the "mass balance" data from spreadsheet models within the company. The number of reservoirs was reduced leaving only major ones. Pipe data were estimated based on the simulation model. Each pump station is separated by a reservoir, which prevents the pump stations from interacting. The pipes between pump stations and reservoirs were independently calibrated from system curves taken from SCADA measurements. The model has been extended to include demand flows from SCADA measurements, additional boreholes and recent tariffs. The model is presented in figure 3 and includes:

11 reservoirs (9 variable head, 2 fixed head)
60 junctions
53 pipes
16 pumps (8 variable speed, 8 fixed speed)
15 valves
20 boreholes

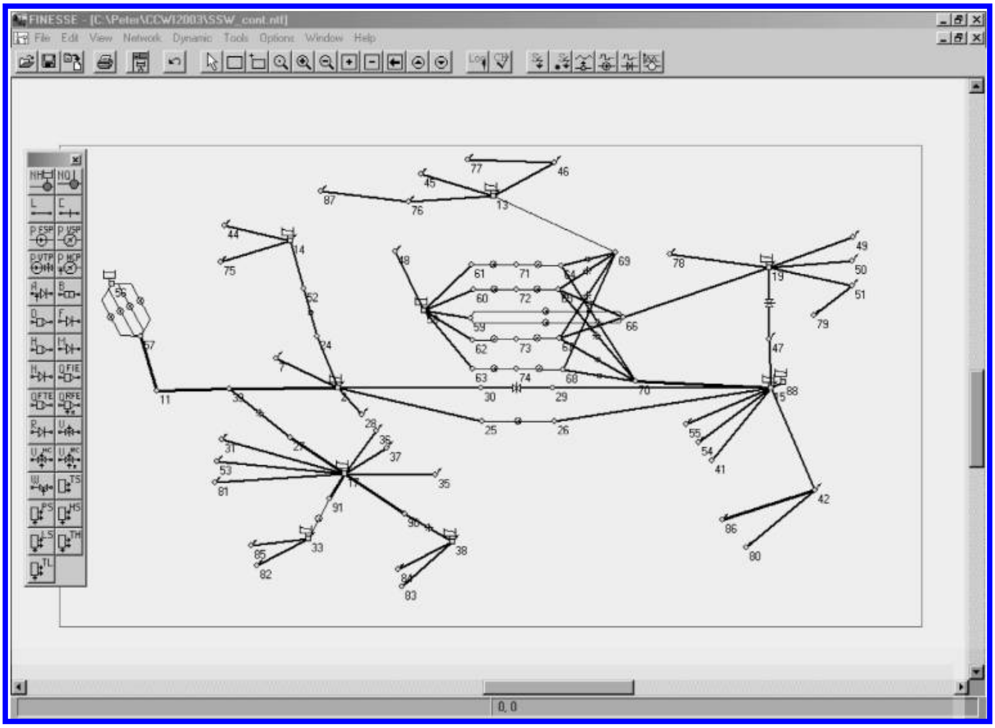

Figure 3. The SSW optimisation model built in FINESSE.

The total water consumption of the model is 330 Ml/d. The network is fed by three major sources: one from the West (node 56), a second from centre (node 58) and a large number of boreholes that are distributed throughout the network. Over a third of the water is pumped from the Western source to major reservoirs (nodes 2 and 17). Water is also pumped from the Central source to three major reservoirs (nodes 13, 15 and 19). A booster pump transfers water across the network from reservoir 2 to reservoir 15.

3 METHOD

The main objective was to model and optimise the SSW supply network to reduce total operating costs. There are complex relationships between different components of cost and operating constraints in the whole network that can only be understood through use of an advanced optimal scheduler.

Optimal network scheduling calculates least-cost operational schedules for pumps, valves and treatment works for a given period of time, typically 1 day or 1 week. The problem is expressed as:

1. Minimise pumping cost + treatment cost + source cost
2. Subject to network equations and operational constraints

The decision variables are the operational schedules for control components, such as pumps, valves, and water works outputs. Optimal scheduling belongs to a class of dynamic optimisation problems where the models are expressed in the form of differential algebraic equations and the decision variables are functions of time. The full mathematical formulation is described in detail in Ulanicki, Bounds et al (1999).

There are very efficient modelling environments and solvers for solving large-scale non-linear programming problems. Therefore, in order to solve the problem numerically, the optimal network scheduling problem is converted into a non-linear programming problem using time discretisation.

A discrete time grid $\{t_0, t_1, t_2, \ldots, t_K\}$ where $\Delta t = t_{k+1} - t_k$ is a time step is assumed. The time grid can be referred to as $\{0, 1, \ldots, k-1, k, k+1, \ldots, K\}$. This scheme allows the application of irregular time steps which was of importance for SSW because of their complex time grid of tariffs.

The objective function is represented as three costs which are all associated with a pump station: a unit electricity cost, a treatment cost and an auxiliary source cost:

$$\phi = \sum_{j \in J_p} \sum_{k=0}^{k=K} \gamma_u^j(k)\ f_j(q_p^j(k), c_p^j(k)) \times \Delta t + \gamma_t^j(k) \times q_p^j(k) \times \Delta t + \gamma_s^j(k) \times q_p^j(k) \times \Delta t \tag{1}$$

where J_p is the set of indices for pump stations. The term $f_j(q_p^j(t), c_p^j(t))$ represents the electrical power consumed by pump station j. The mechanical power of water is obtained by multiplying the flow (q) and the head increase across the pump station. The consumed electrical power can then be calculated by dividing the mechanical power of water by the pump efficiency. The head increase variable $\Delta h^j(t)$ can be expressed in terms of flow in the pump hydraulic equation, so that the cost term becomes dependent on the operational variables: pump station flow $q^j(t)$ and the control variable $c^j(t)$ as illustrated in equation 1. The $c^j(t)$ vector represents the number of pumps on and/or pump speed. The function $\gamma_u^j(t)$ represents the electricity tariff. The treatment cost and the auxiliary source cost are proportional to the flow output of the pump with the unit price of $\gamma_t^j(t)$and $\gamma_s^j(t)$ respectively.

The reservoir storage is discretised using a simple Euler scheme in equation 2:

$$h_i(k+1)S_i = h_i(k)S_i - q_i(k)\Delta t \qquad \forall i \in I_r,\ k = 0,1,\ldots,K-1 \tag{2}$$

where I_r is the set of indices for reservoirs, h_i is the head at node i, q_i is the reservoir flow at node i, and S_i is the cross-sectional area of reservoir i.

Each component has an equation. For a variable speed pump, for example, the equation is:

$$(-h_{outlet}(k) + h_{inlet}(k))u_j(k)^2 + A_j q_j(k)^2 + B_j q_j(k) u_j(k) s_j(k) + C_j u_j(k)^2 s_j(k)^2 = 0 \qquad \forall j \in J_p,\ k=0,..,K-1 \tag{3}$$

where the set of pumps is denoted as $J_p \subseteq J$, the pump hydraulic model is obtained by a quadratic fit (coefficients A_j, B_j, C_j) of the set of points provided by a manufacturer, u_j is the number of pumps switched on and s_j is the normalised speed of the pump.

The Kirchhoff's law I for flow balance at each node must be satisfied. Each node I $\in I$, has two sets of branches M_i^+ where i is the inlet node of the branches and M_i^- where i is the outlet node of the branches:

$$\boxed{\begin{array}{l}\sum_{j \in M_i^+} q_j(k) - \sum_{j \in M_i^-} q_j(k) - q_{R,i}(k) + q_{D,i}(k) - q_{S,i}(k) = 0 \\ \forall i \in \mathrm{I},\ k=0,..,K-1\end{array}} \tag{4}$$

where the first summation gives the total flow for all branches whose inlet is node $i \in I$, while the second summation gives the total flow for all branches whose outlet is node $i \in I$, $q_{R,i}(k)$ is a reservoir flow, $q_{D,i}(k)$ is a demand flow, and $q_{S,i}(k)$ is treatment flow. A reservoir, demand or source flow will not exist in a node equation if the node is not of this type.

The operational constraints have the form of inequalities to maintain the feasible operating range. Typical examples are for reservoir levels, which must be kept inside minimum and maximum bounds, as indicated below:

$$h_i^{\min}(k) \le h_i(k) \le h_i^{\max}(k) \qquad \forall i \in I_r,\ k=1,2,\ldots,K \tag{5}$$

3.1 *The FINESSE environment*

The optimal network scheduler is fully integrated into a wider modelling environment, called FINESSE (2003), to solve the network scheduling problem. FINESSE includes general features and tools to complement optimisation. FINESSE has a comprehensive Microsoft Windows user interface based on a visual schematic of the water network (figure 3). The SSW model was built and edited interactively using the FINESSE model building features. The network schematic was created by "drag and drop" from a palette of network components (figure 3). Then the network data was added directly by clicking on the components of the schematic, or alternatively in tables listing all components and all parameters. This feature proved to be an advantage over the spreadsheet approach. In FINESSE, the data model is shared by the whole environment, including the optimal scheduler. If the data model had already existed as a text file from another modelling package it could have been imported automatically. Currently, FINESSE imports EPANET, GINAS and WATNET files. Other external connections include interfaces to SCADA systems and a general copy/cut/paste feature to transfer data from the Microsoft clipboard to Microsoft Office applications such as Microsoft Excel spreadsheets.

FINESSE includes an hydraulic extended-period simulator which was used to test and calibrate the SSW model. An engineer uses the simulator to produce a feasible set of pressures and flows. These simulation results are transferred to the scheduler in order to initialise the decision variables for solving the optimisation problem. The optimal network scheduler contains the same network equations (eqns. 3 and 4) as the simulator and so they calculate pressures and flows to the same accuracy although the commercial solvers are different. Because the scheduler contains the network equations found in the simulator it requires the same network data. In addition, the scheduler contains the objective function (eqn. 1) and the operational constraints (eqn. 5) and so tariff data and maximum and minimum constraint settings are also required. This data is added to the common and shared database of FINESSE using the graphical features outlined above.

3.2 *Optimal network scheduler*

The scheduler, as with all tools in FINESSE, is general purpose in that it takes any data model of a network in FINESSE and if the model is feasible it calculates the optimal schedules. The scheduler has solved other networks such as Kilham, Yorkshire Grid and Aix Nord/Trevaresse networks (Ulanicki et al, 1997). Because the scheduler not only takes into account the flow mass balance (eqn. 4) but also the head-flow non-linear characteristics of the network components of the system (eqn. 3), its results are more accurate than linear or mass balance spreadsheet models.

There are very efficient modelling environments and solvers for resolving large-scale non-linear programming problems. The optimal scheduling problem is written in a mathematical modelling language called GAMS (Brooke et al, 1998) which automatically calls up a non-linear programming solver called CONOPT (Drud, 1985). GAMS has been integrated into FINESSE by using a Dynamic Link Library as shown in figure 4. The scheduler is general purpose. If a specific network is to be solved then modelling code is generated from the network data model stored in the FINESSE database. The optimisation procedure is initiated by the engineer by operating the user interface. Data is transferred from FINESSE to the GAMS manager via the FINESSE data interface library. The GAMS manager produces the GAMS source file containing a complete and concise formulation of the problem for the specific network (Ulanicki, Bounds et al, 1999). The GAMS executable is then called to compile and execute the source file. The GAMS software environment itself controls the solver via its solver interface. When the solution is completed GAMS outputs the results into an output file according to instructions given in the source file. This output file is read by the GAMS manager and translated back to FINESSE via the FINESSE Data Interface Library.

CONOPT is a non-linear programming solver, which uses a generalised reduced gradient algorithm (GRG) (Drud, 1985). Hence it has the advantage of performing iterations in a reduced state space. Some variables are changed freely by the algorithm (super-basic variables) and some are

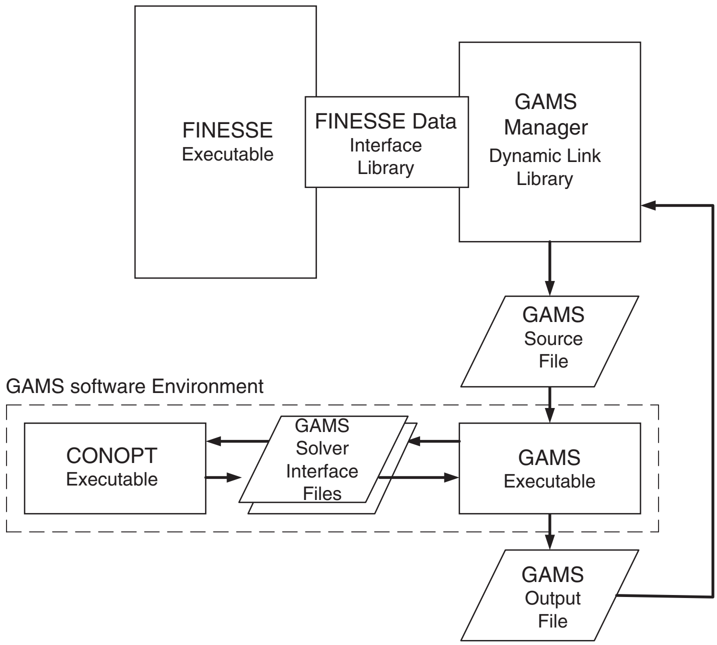

Figure 4. Software mechanism to integrate the GAMS Modelling Language into the FINESSE environment.

calculated from the equality constraints of the problem (basic variables). The gradient is calculated only with respect to the super-basic variables. GRG algorithms are efficient for models with few degrees of freedom. CONOPT can solve the optimal scheduling problem in reasonable operational time for medium-to-large sized networks. The size of the SSW optimisation model is given in section 2.3 and the speed of calculation is given in section 4.

3.3 *Discrete scheduler*

The scheduler can produce both continuous and discrete schedules. Decision variables such as pump speed, valve position and pump flow are naturally continuous while others such as the number of pumps switched on are integer. Current operational planning in SSW calculates optimal pump flows and leaves their translation into pump control switches to local operations personnel. In practice the presence of reservoirs enables the problem to be relaxed to a continuous one. In SSW, the reservoir storage tends to be reduced during weekdays to save pumping costs because of expensive tariff periods and refilled by pumping at the weekends. Reservoirs head trajectories express the optimal storage policy because they average out pump flow. The change in a reservoir volume over some period of time (time-step) is governed by its net inflow/outflow during that period, which in turn affects the average flow pumped during the period. The scheduler in FINESSE initially assumes that the problem is a purely continuous one to calculate the optimal pump flows. The non-linear programming algorithm efficiently solves this relaxed continuous optimisation problem.

The continuous relaxation of the scheduling problem does assume that the number of pumps switched on can vary continuously between constraints. For example, 1.5 pumps switched on would be an acceptable solution. In order to translate this into an integer solution for local operational use a discrete scheduler is employed. As a starting point, the reservoir trajectories are taken from the continuous solution. For each tariff period, a Branch & Bound algorithm is applied to follow the original reservoir trajectories as closely as possible.

The relaxed optimal continuous solution is cheaper than any optimal discrete solution. The discrete solution that is found in the neighbourhood of the continuous solution is assumed to be close to the optimal discrete solution.

4 RESULTS

Optimal scheduling calculated the least-cost operational schedules for pumps, valves and treatment works for a one week time horizon. The SSW network contains 9 reservoirs and 16 pump stations. The problem was solved over a 1 week time horizon and included the irregular tariff time steps. FINESSE and scheduler were executed on a Pentium 4 PC with a clock speed of 2.2 GHz. Two sets of results are presented one for the continuous optimisation solution and the other for the discrete optimisation solution:

4.1 *Results of the continuous optimisation scheduler*

The total calculation time for the whole weekly horizon was approximately 30 minutes.

The reservoir levels were kept between 100% and 45% of the maximum level during the week and the final reservoir level must be 95% or more. These constraints were satisfied and the results

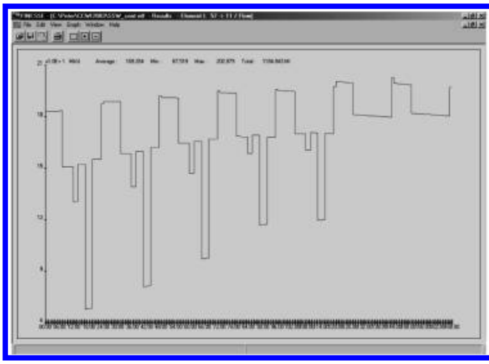

Figure 5. Optimised pump flow in pipe 57-11.

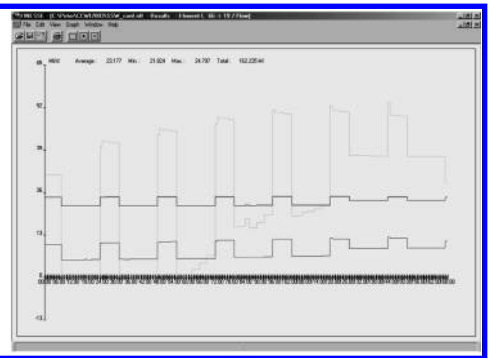

Figure 6. Optimised pump flows in pipes 70-15, 66-19, and 69-13 (in descending order of magnitude).

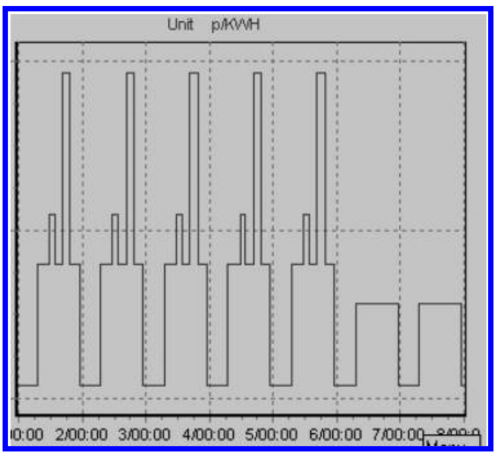

Figure 7. Weekly electricity tariff for Western pump station (56-57).

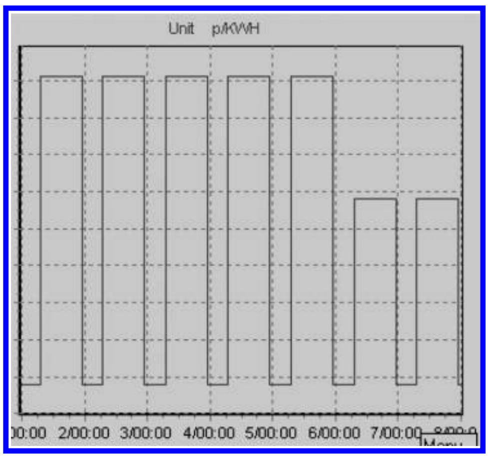

Figure 8. Weekly electricity tariff for Central pump station (58-64, to 58-68).

for the four major reservoirs are displayed in figures 9 to 12. The original reservoir level of node 2 (figure 9) was conservatively operated high throughout the week, but the optimal trajectory is kept low for the efficient operation of the downstream pumps.

The optimal pump flows (figures 5 and 6) have a direct correspondence with the electricity tariffs (figures 7 and 8). There are higher flows during lower tariff periods and vice versa.

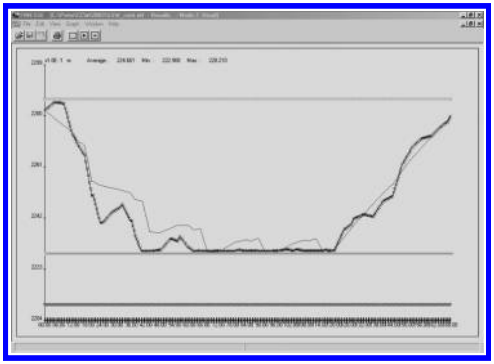

Figure 9. Continuous (thin line) and discrete (thick line) reservoir head trajectory at node 2.

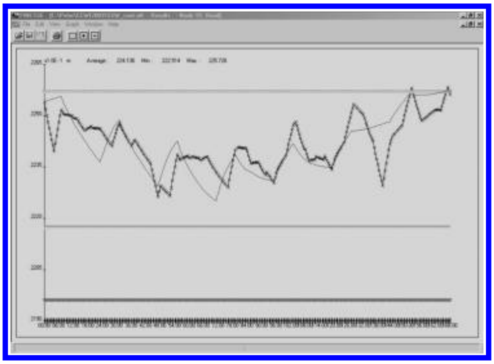

Figure 10. Continuous (thin line) and discrete (thick line) reservoir head trajectory at node 15.

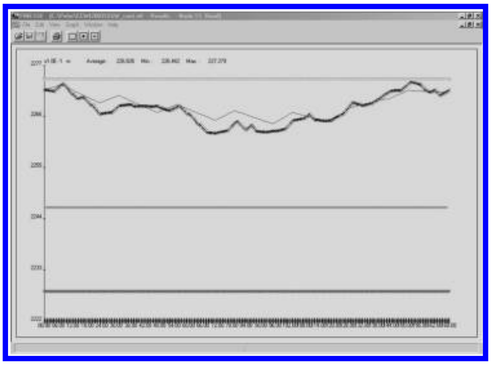

Figure 11. Continuous (thin line) and discrete (thick line) reservoir head trajectory at node 13.

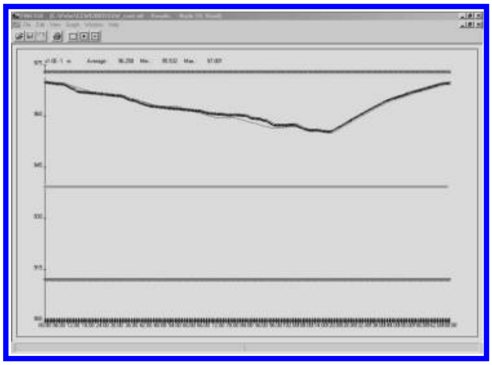

Figure 12. Continuous (thin line) and discrete (thick line) reservoir head trajectory at node 19.

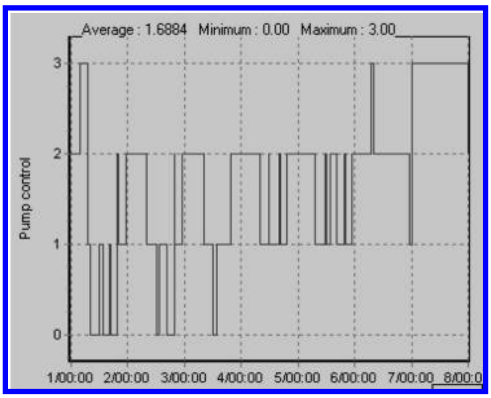

Figure 13. The discrete schedule for the major fixed speed pump 56–57.

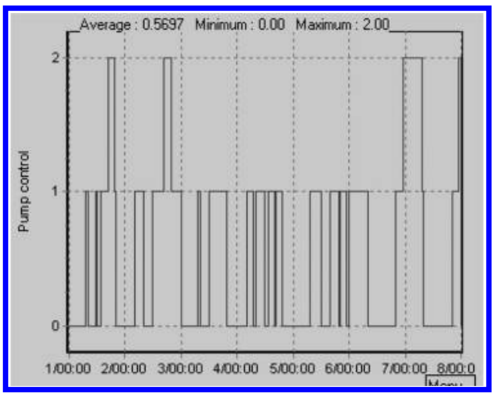

Figure 14. The discrete schedule for the major variable speed pump 56–57.

4.2 *Results of the discrete optimisation scheduler*

The discrete solution closely follows the reservoir trajectories from the continuous solution (figures 9 to 12). The discrete schedules were calculated and the control schedules for two major sets of pumps are illustrated in figures 13 and 14.

At the time of writing the exact energy savings were under evaluation and will be prepared for presentation at the conference.

5 CONCLUSIONS

This study shows that it is possible to model and calculate the optimal network schedules for a company-wide network. The scheduling problem was solved over a one-week horizon minimising the operational cost and satisfying all hydraulic constraints. The scheduler handled complex tariffs with irregular time steps and a network with 9 variable head reservoirs and 16 pump stations. A general algebraic modelling language and non-linear programming solver were integrated into the FINESSE software environment to solve the optimisation problem. Other tools integrated into FINESSE such as a simplifier, merger and simulator were used to build the optimisation model. The simulator also produced a reasonable starting point for the scheduler. Both continuous and discrete schedules were calculated. The optimisation model continues to be improved and tested against current operational cost at the time of writing.

REFERENCES

Brooke A., Kendrick, D., Meeraus, A. and Raman, R. (1998) *GAMS: A user's guide*, GAMS Development Corporation, Washington, USA.

Drud, A.S. (1985) CONOPT: A GRG Code for Large Sparse Dynamic Non-linear Optimisation Problems, *Mathematical Programming*, 31, pp. 153–191.

FINESSE (2003) *WSS Software Description*, [Online], Water Software Systems, De Montfort University, UK. Available from: http://www.eng.dmu.ac.uk/wssys/software.html.

Tischer, M., Henkel, S., Ulanicki, B., Dacre, B. and Cummings, G. (2003) Optimal Scheduling of South Staffordshire Water Supply System using a Spreadsheet Solver. In: CCWI 2003, *Proceedings of International Conference on Advances in Water Supply Management. Imperial College London, UK, 15–17 September 2003*.

Ulanicki, B., Bounds, P.L.M. and Rance, J.P. (1999) Using GAMS modelling environment to solve network scheduling problems, *Measurement and Control*, 32(4), pp. 110–115.

Ulanicki, B., Coulbeck, B. and Ulanicka, K. (1997) Generalised Techniques for Optimisation of Water Networks. In: Powell, R. and Hindi, K.S. ed. Computing and Control for the Water Industry, *International Conference on Computing and Control for the Water Industry, Brunel University, Uxbridge, UK, 4–5 September 1997*, Baldock, Research Studies Press, pp. 163–175.

Ulanicki, B., Zehnpfund, A. and Martinez, F. (1996) Simplification of Water Network Models, In: Muller, A. ed. HYDRINFORMATICS '96, *Proceedings of the 2nd International Conference on Hydroinformatics, ETH Zurich, Switzerland, 9–13 September 1996*, vol. 2, Rotterdam, A.A.Balkema, pp. 493–500.

Optimal scheduling of South-Staffordshire water supply system using a spreadsheet

M. Tischer & S. Henkel
Technische Universität Ilmenau, Fakultät für Informatik und Automatisierung, Ilmenau, Germany

B. Ulanicki
Water Software Systems, De Montfort University, Leicester, UK

B. Dacre & G. Cummings
South Staffordshire Water plc, Walsall, UK

ABSTRACT: This paper presents a part of the project concerning optimal scheduling of the South-Staffordshire Supply System. Two alternative approaches were investigated, one based on a spreadsheet model and another on a FINESSE model. This paper describes results of the spreadsheet approach. The spreadsheet model takes into account mass balance in the major reservoirs, water transfer between the main subsystems, the system demands, chemical and electrical tariffs and operational constraints. The constraints are concerned with the reservoir levels and flows for sources and pumps. The operational cost is evaluated using a nonlinear power/flow characteristic for pump stations and an electricity tariff. The power/flow characteristic was obtained from physical measurements by South-Staffordshire Water. It was possible to solve the scheduling problem over a one week time horizon, minimising the operational cost and satisfying operational constraints. This approached fitted well into the existing working practices of the company.

1 INTRODUCTION

This paper describes an optimisation tool for an existing decision support system, which is a part of an optimisation study covering the South Staffordshire Water (SSW) supply network with the overall aim of reducing operating costs.

The Company's existing approach already achieves significant energy savings. Short-term operational control is based on daily and weekly planning horizons. Overall reservoir storage is reduced through the week and replenished at weekends. The Company uses decision rules based on experience and the knowledge of their particular system. The rules incorporate power tariffs and rely on following the proposed reservoir storage profiles at major reservoirs. Energy consumption is monitored on line to facilitate the control decisions. The Company uses some spreadsheet models based on mass-balance and energy cost calculations to support planning decisions. Further savings can be accomplished by using advanced numerical optimisation algorithms and hydraulic models.

The main objective is to model and optimise the SSW network to reduce total operating costs. There are complex relationships between different components of cost and operating constraints that can be understood only through use of advanced optimisation tools. For example, the following key issues have been discussed as possible areas of investigation:

- Analysis of overall operation costs, including pumping costs (which depend upon the various electricity tariffs at different pump stations and the option to use the Company's own diesel generation at some sites), and water production costs (which depend on treatment and energy cost).

- Analysis of operational constraints: The system is operated within many volume, pressure and water quality constraints. These have been derived over a period of time according to operational experience and local considerations. Constraints affect operation costs and an overall analysis of constraints and their interactions could reduce operating costs.
- More specifically, blending of water from different sources is used to reduce nitrate removal treatment. However, blending introduces relative constraints on source outputs that may have impact on other costs.
- There may be a trade-off between heavier pumping to take advantage of off-peak electricity (mostly during the night), and increased pressures leading to more leakage.

SSW supplies high quality drinking water to 1.2 m people over 1500 km^2 in the West Midlands, South Staffordshire, South Derbyshire, North Warwickshire and North Worcestershire areas. The system comprises of:

- 2 major river sources Hampton Loade (220 ML/D) and Seedy Mill (70 ML/D)
- 27 borehole stations, plus some transfers to and from other water companies.
- 20 supply zones of varying complexity represented as approximately 18 network models. The total numbers of components of these models are 45,694 pipes, 40,611 nodes, 41 sources, 242 valves, 44 pumps and 31 reservoirs.

2 THE SIMPLIFIED OPTIMISATION MODEL

Currently, SSW uses a simulation spreadsheet developed by Brian Dacre for scheduling the sources and the connections between reservoirs. This basic spreadsheet is based on a simplified model of the South Staffordshire system. The aim of this task was to calculate optimised schedules using this spreadsheet model. It was planned to have an optimisation tool, which can calculate the cheapest schedules to provide the planned output. The optimisation tool should be based on MS Excel and be able to use the available information from the basic spreadsheet. The tool should be accurate, easy to use by operators and, if necessary, easy to introduce changes to the model.

To accomplish the task it was useful to create a graphical representation of the model "hidden" in the basic spreadsheet. Through this graphical representation it was a lot easier to understand the structure and operation of the system.

In Figure 1 the simplified model is shown which consists of:

- 6 main reservoirs
- Inputs and outputs from and to other zones
- 15 boreholes
- 2 treatment works
- 6 inter reservoir connections

The optimisation tool manipulates some decision variables of the model. Other decision variables are set fixed by SSW.

The following components are subject to control decisions:

- Pump stations at two treatment works
  - Seedy Mill
  - Hampton Loade
- Two inter reservoir connections
  - Warstone Valve
  - West Brom Booster

The model is quite simple but represents the most important features of the network including: the main reservoirs, connections between reservoirs and all boreholes. The model is accurate enough to be used in the decision support system, the differences between the model flows and the physical system flows are insignificant, the same can be said about the reservoir trajectories.

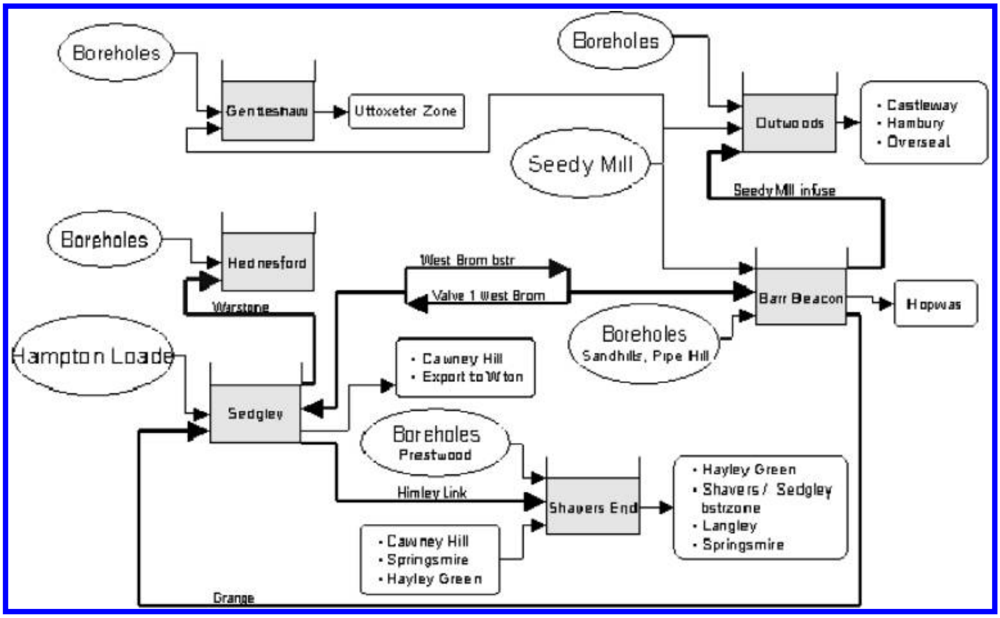

Figure 1. Simplified model.

This model is represented in spreadsheet tables and formulas that relate different cells. The information from the basic spreadsheet is linked to the actual optimisation spreadsheet, where the calculations take place.

3 THE OPTIMISATION SPREADSHEET AND SOLVER

The optimisation tool is a spreadsheet solver that uses the information about the simplified model from the basic spreadsheet. The simplified model is mainly based on mass balance equations. The objective function is represented by a formula that calculates the overall weekly cost of the system. The decision variables are the outflows of the treatment works and the flows of controlled inter reservoir connections. The constraints are imposed on the reservoir levels and major flows.

The overall weekly cost consists mainly of electrical and chemical costs. The electrical cost is calculated directly from the flows and the electrical tariffs. The pump stations are represented by nonlinear power/flow characteristic (regression curves) and are assumed to be unaffected by the changing situation in the remaining network. This enables the used power to be calculated directly from the flow. The power/flow characteristics were obtained from physical measurements carried out by SSW.

Different pump stations, can have different tariffs. The energy savings can be accomplished by shifting the pumping between expensive to cheap tariff periods.

The standard solver which is in the original MS Office package was not able to solve the optimisation problem because of the high number of variables, bounds and constraints. It was necessary to use more advanced solvers. There were two versions of the optimisation problem considered, each solved by a different solver. The first optimisation problem consisted of 138 variables, 103 constraints and 276 bounds. This problem was solved using the "Premium Solver Platform V5.0" by Frontline Systems, Inc which is able to solve optimisation problems with up to 500 variables, 250 constraints, 1000 bounds and 500 integer variables. Another optimisation problem was even larger and required more advanced "Large Scale Solver" provided by the same vendor.

The Premium Solver Platform provides a multi-start option. This option is very helpful especially for problems with suspected local minima. This option might be unnecessary for a good starting point but otherwise is useful in finding a global minimum. It is possible to learn more about Premium Solvers Platform by visiting the website http://www.solver.com/xlscompare.htm.

3.1 *Run the solver*

After installation the solver becomes a part of the add-ins of MS EXCEL. The user can change settings in the spreadsheet dialog boxes. Different cells have different functions, some are designated to keep the objective function, some decision variables and some keep constraints. The solver recognises the type of the problem, which can be linear, quadratic, non-linear or non-smooth, and automatically proposes the appropriate algorithm. Of course, the result of the optimisation depends on many other settings. After the solver has finished calculation it is possible to keep the solution, restore the original values and also generate different reports.

3.2 *Further options/settings*

The solver provides many options which may affect the solution. In this paper only the main options are described. These are: multi-start option, the search methods and the available reports. All options are specified in the user manual for the Premium Solver Platform.

3.2.1 *The multi-start method*

This method may give better results for an optimisation problem but increases the time required to solve the problem. The multi-start method is very helpful if you do not have a good starting point or if you want to increase the possibility of getting a global optimum. If the problem is being solved with this option, the solver runs many times with different starting points and selects the best solution.

3.2.2. *Different search methods*

3.2.2.1 Estimates

It is possible to choose between tangential and quadratic estimates. The tangential approach might be slower in many cases because it uses only linear approximation of the objective function, but if you do not know the behaviour of the objective function it is the more secure alternative. If the objective function is nearly quadratic, the quadratic approach chooses a better initial point. This means that fewer iterations are required to reach the minimum.

3.2.2.2. Derivatives

The options “Forward” and “Central” select two methods for calculating derivatives. With the forward differentiation the point from the previous iteration is used in the conjunction with the current point. Whilst the central differentiation uses only the current point for calculations. The second choice might be better if the derivatives are rapidly changing. Again, the disadvantage of this method is the calculation time, this method is normally slower than the other choice.

3.2.2.3 Search

There are two fundamental search methods available. The choice is between a quasi-Newton method (also known as BFGS) and a conjugate gradient method. The quasi-Newton method uses an approximation of the Hessian matrix. This approximation performs very well but needs a lot of memory storage. The conjugate gradient method performs nearly as well and does not need the storage to keep the Hessian matrix. There is an automatic switching mechanism, which switches between these two methods depending on the current situation, the choice of this option is not critical for the calculations.

3.3 *Outline reports*

For better analysis of the solution the solver provides different kinds of reports, for example, an answer report, a sensitivity report and a limits report. If you want to produce the reports you have specify them with the help of a special option. The reports are especially helpful for diagnosis purposes if any problems occur during the solving process or within the model.

4 CALCULATION RESULTS

In Figure 2 and Figure 3 some important results are displayed. Figure 2 shows the optimised flow schedules for one of the treatment works and the corresponding electrical tariff. It is possible to see the direct relation between the tariff and the resulting flow. More water is pumped during the cheap periods than in the expensive tariff periods. This behaviour is only possible because the reservoir which is connected to this treatment works acts as a buffer for these flow fluctuations.

Figure 3 displays the original and the optimised trajectory of one of the six reservoirs of the system. The optimised trajectory is feasible (satisfies the constraints) and at the end of the week the reservoir is 95% full.

The most obvious benefit of using optimiser was that it generated feasible trajectories satisfying all operational constraints. At the time of writing this paper the comparative field studies were not available but the modelling studies indicated of 1.5% of potential savings.

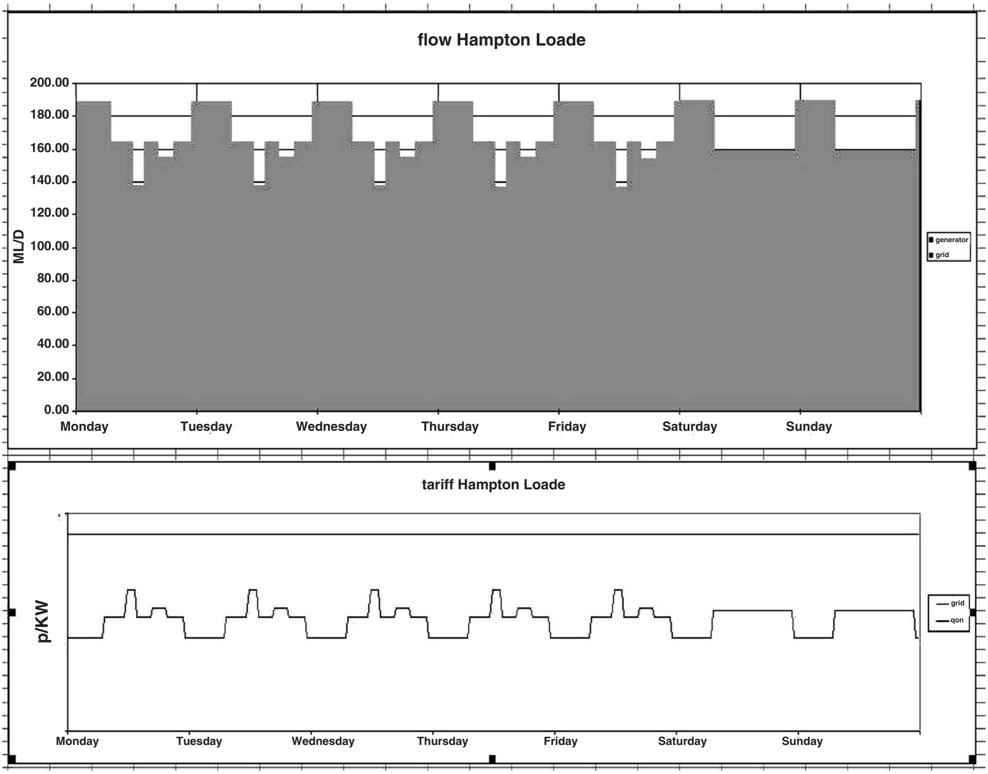

Figure 2. Result 1.

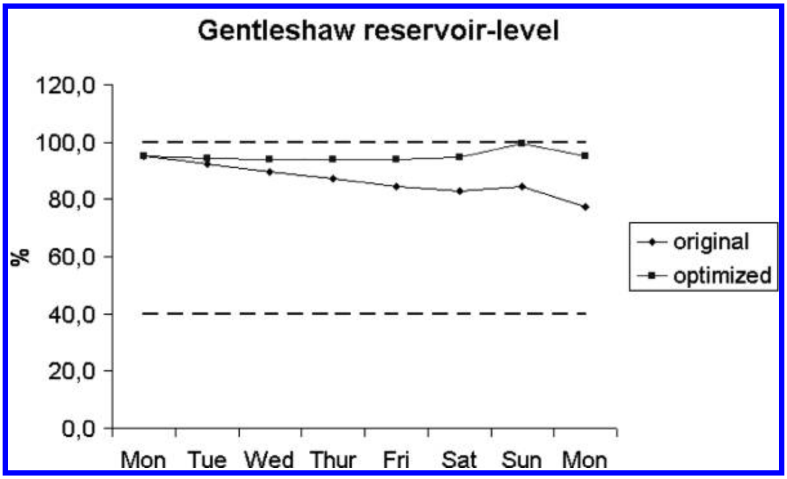

Figure 3. Result 2.

5 CONCLUSIONS

With the developed optimisation tool every engineer of the company can use the decision support system. This approach fitted well into the existing working practices in the company. Because of its speed and accuracy the calculations can be repeated very often. The contingency situations within the water network, like infrastructure failures or cleaning processes, can be easily handled. The optimisation schedules are only slightly better than the results of the existing decision support system but this is not surprising. This shows the correctness of the tool and the experience of SSW engineers.

The advantages of the optimisation tool compared to the existing practice are: energy savings, reduction in time spend on preparing the schedules, the optimised schedules can be prepared by less skilled staff in a repeatable manner. The major drawback of the spreadsheet approach is a lack of a network GUI to build and maintain the optimisation model interactively.

REFERENCES

Frontline: Compare Excel Solvers, http://www.solver.com/xlscompare.htm, 17.12.2002.
Frontline: Optimisation Software Overview, http://www.solver.com/xlsplatform.htm, 17.12.2002.
Frontline: User manual for the Premium Solver Platform V5.0.
Ulanicki, B., Bounds, P.L.M., and Rance, J.P. 1999. Using a GAMS Modelling Environment to Solve Network Scheduling Problems, *Measurements and Control*, Vol. 32, No 4, May.

Optimal rerouting to minimise residence times in water distribution networks

T. Devi Prasad
Research Fellow, School of Engineering, University of Exeter, Exeter, UK

Godfrey A. Walters
Professor, School of Engineering, University of Exeter, Exeter, UK

ABSTRACT: Management of water quality is a major issue for water companies, especially as many systems are old and have excess capacity. This paper presents a methodology that optimizes a water quality parameter using an evolutionary program. The water quality parameter considered is the age of the water at various nodes in a network. A steady-state water quality model is used to find the water age. The evolutionary program re-configures the network by selecting a set of pipes in the network for closure. The optimal network configuration is achieved when the water age at nodes in the network is minimized subject to maintaining hydraulic feasibility and connectivity. The methodology is then applied to an example network to demonstrate the efficacy of the proposed method.

1 INTRODUCTION

Most water distribution systems in the UK were built with excess capacity and many of these still have oversized pipes and unnecessary system redundancy. This can lead to low velocities and very long residence times, and thereby to water quality problems. The deterioration of water quality is caused mainly by turbidity and pathogenic organisms. Of these two, pathogenic bacteria are the main concern. In order to reduce the risk of waterborne diseases, chlorine is typically used as a disinfectant. However, excess amounts of chlorine cause bad taste and smaller additions may not kill the bacteria completely. As water flows through the network chlorine concentration changes in a complex way that is dependent on a number of factors such as temperature, chemical and biological properties of the water. However, it is clear that the length of time the water remains in the system after treatment is very significant in determining the residual chlorine concentration. Effective control of water quality could be achieved through network reconfiguration using pipeline closure, throttling of flow control valves, zoning/rezoning, increasing night flows, re-circulating systems, and booster disinfection. It should be recognized that no one strategy is a complete solution to the water quality deterioration problems. The best solutions are likely to involve combinations of these strategies.

In view of the uncertainties involved in knowledge of the state of a network and of the physical process that occur, it is decided to use water age as a surrogate measure of water quality in this study. One of the strategies for the reduction in age of water at remote nodes of a network is through reconfiguration using pipeline closures. That is, at places where excess network capacity exists, selective closure of pipes in the network causes increase in velocities in other parts of the system, thereby reducing residence times. The pipes to be closed must carefully be selected so that the ability of the system to supply water at adequate pressure is not adversely affected. Closure of pipes can be achieved through isolation valves. As such valves will normally be present in all links of a network, implementation of pipe closures may have little associated cost for considerable potential benefit.

Evolution Programs (EPs) have attracted the attention of many researchers in the optimization of water distribution networks (Savic and Walters 1995; Savic and Walters 1997). These programs mimic natural evolution and are best suited for solving combinatorial optimization problems that cannot be solved using more conventional optimization methods. They can be applied to large complex non-linear problems with relative ease. Since their performance relies heavily on the evaluation of a fitness function, an efficient method of evaluating the objective function and thereby the fitness of each candidate solution (chromosome) is required.

The present work involves determination of the age of water at various nodes of the network under steady-state hydraulic conditions. Steady-state hydraulic analysis is carried out using EPANET Toolkit (Rossman 1999). These hydraulic results are then used to solve a steady-state water quality model. The values of age of water at various nodes, obtained from the solution of the water quality model, are then used to derive a representative water quality measure for the network. Minimization of a water quality measure is taken as the objective with the settings of isolation valves as decision variables and minimum allowable pressures at nodes as constraints. The model is applied to an example network.

2 PROBLEM STATEMENT

The optimal rerouting model to reduce residence times of water in a network can be stated as:

$$\min_{CV \subseteq V} J = I_q \tag{1}$$

subjected to:

$$H_i \geq H_i^{\min}; \qquad i = 1,\ldots,N \tag{2}$$

where V is a set of all valves in the network, CV is a set of closed valves, I_q is a water quality measure defined in the following section, H_i is the head at node i, and $H_i^{\min}$ is the minimum required head at node i. As evolutionary algorithms cannot solve the constrained problems directly, the above model is conveniently transformed into an unconstrained optimization problem using a penalty function. The penalty (P) function is given by,

$$P = (C \times t)^{\alpha} \sum_{i=1}^{N} B_i^{\beta} \tag{3}$$

$$\text{and } B_i = \begin{cases} \left(\dfrac{H_i^{\min} - H_i}{H_i^{\min}} \right) & \mathit{if} \quad H_i < H_i^{\min} \\ 0 & \mathit{Otherwise} \end{cases} \tag{4}$$

where B_i is the pressure violation at node i; t is the generation number; and C, α, and β are constants. This penalty function penalises solutions with constraint violations, and the pressure on infeasible solutions is increased as the number of generations increases. The unconstrained optimization model is given by,

$$\min_{CV \subseteq V} J = I_q + (C \times t)^{\alpha} \sum_{i=1}^{N} B_i^{\beta} \tag{5}$$

A reasonable choice for the parameters is $C = 0.5$, and $\alpha = \beta = 2.0$ (Michalewicz, 1995). In this study the following network water quality parameters are used as objective functions.

(i) Minimize maximum water age at any node $\left(I_q = \max_{i=1}^{N}(A_i) \right)$.

(ii) Minimize average age of water at nodes $\left(I_q = \frac{1}{N}\sum_{i=1}^{N} A_i \right)$.

(iii) Minimize weighted average age of water at nodes $\left(I_q = \frac{\sum_{i=1}^{N} A_i Q_i}{\sum_{i=1}^{N} Q_i} \right)$.

3 EVOLUTIONARY ALGORITHM

The evolutionary program (EP) used in this paper uses artificial methods based on the natural process of evolution. It is based on the natural evolutionary concepts of preferential survival, reproduction of fittest members, and diversity among members of a population. The following is a brief description of the EP used in this study.

Candidate solutions, namely network configurations generated from the base graph (the complete network with no closed valves), are represented by chromosomes using an integer set of coding. A chromosome contains a set of integer numbers defining the pipes that are open. The order in which pipe numbers are given in the set are not important and C++ set type variables are particularly useful for economical storage and easy manipulation. The initial population is generated at random subject to rules that ensure connectivity of candidate solutions. The approach adopted

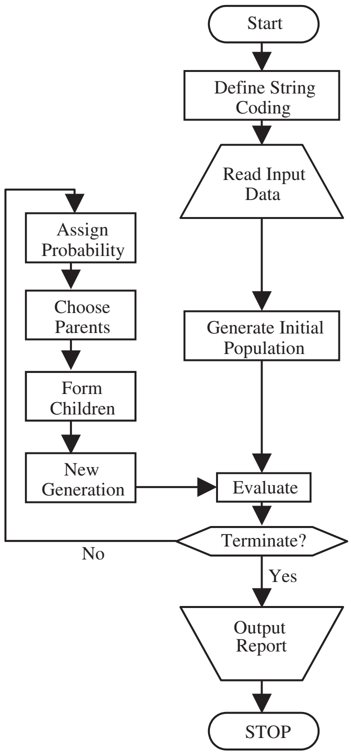

Figure 1. Flow chart for the evolutionary algorithm.

here is hierarchical, i.e., at the first level it creates a spanning tree so that the created network is fully connected and at the second level it allows for loop existence in the network. The fitness function returns a measure of how good a candidate solution is. Since the problem is a minimization problem, the fitness is a function of J. The following relation between the objective function value J_k for the solution k and the corresponding fitness f_k is used.

$$f_k = \frac{J_{\max} - J_k}{J_{\max} - J_{\min}} \tag{6}$$

where J_{max} and J_{min} are the best and worst solution candidates of the population respectively. This linear scaling scheme discriminates between candidates when all members have similar values of objective function. After generating the initial population, the generational replacement scheme is applied next to generate a new population from the old one. The new population then becomes old and the process is repeated. In order to maintain the elitism the best member in the previous generation is inserted back into the new population at a random slot. Creation of members of the new population is performed through the mating of a pair of members of the old population. The process starts by selection of parents for recombination. The second step, creation of children, differs from the conventional GA but follows the basic idea of EP by which genetic information of parents is conveyed to the children. This step is performed by means of new genetic operators analogous to crossover and mutation. The process is terminated when a particular criterion is met. A more detailed methodology can be found in the paper by Walters and Smith (1995). The structure of the main program implementing these basic steps is given in Figure 1.

4 COMPUTATION OF WATER AGE

Males et. al., (1985), introduced the concept of average age (A_i) of water, or average time of travel from all sources, at node i (Fig. 2). This concept was applied to find the age of water at various nodes of a network under steady-state flow conditions.

The age of water at any node i is given by,

$$\boxed{A_i = \frac{q_{ji}\left(A_j + R_{ji}\right) + q_{ki}\left(A_k + R_{ki}\right) + Q_i A_{si}}{q_{ji} + q_{ki} + Q_i}} \tag{7}$$

here q_{ji} = steady-state flow in pipe connecting nodes j and i; A_i = average age of water at node i; Q_i = flow of water from an external source at node i; A_{si} = age of external source of water at node i; and R_{ji} = residence time of water in the pipe connecting nodes j and i. The values of residence time R for any pipe can be calculated as:

$$R_{ji} = \frac{L_{ji}}{\left(4q_{ji} / \pi D_{ji}^2\right)} \tag{8}$$

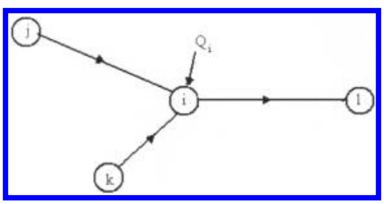

Figure 2. Network junction.

where L_{ji} and D_{ji} are the length and diameter, respectively, of the pipe connecting nodes j and i. From the known steady-state hydraulic solution, the values of R for any pipe can be calculated. Eq. 7 is a linear equation in terms of unknowns, A_i, A_j, and A_k. For each junction node an equation, similar to Eq. 7, can be written, forming a system of linear equations. The coefficient matrix of this system of equations is sparse, and can profitably take advantage of sparse-matrix simultaneous equation solution techniques.

5 APPLICATION OF THE MODEL

A program based on the methodology presented has been written in C++ and implemented on an Intel P4 computer. The model is applied to an example problem to understand the performance of the model with each of the objectives presented.

The network layout of the example problem is shown in Figure 3. The network has 34 nodes and 47 pipes. Table 1 shows the demand and elevation values for all the nodes, which are supplied from one source at a head of 80 m. The minimum required head at all nodes is set to 15 m. Relevant pipe data are given in Table 2. In this example there are 1.4×10^{14} (2^{47}) possible candidate solutions and a complete enumeration is impracticable. To assess the impact and relevance of each of the three objectives defined earlier, the GA model is solved considering each objective separately. Also, to assess the reduction in each objective their values are calculated for the network with all pipes open (original network).

The GA model was applied with different random seeds and GA parameters. The optimal results are presented in Table 3. Minimization of the maximum age of water at nodes in the network has reduced the maximum age of water at any node from 16.31 hrs (original network) to 6.29 hrs. The optimal solution for this case has 9 closed valves. These valves are located in pipes 2, 3, 6, 11, 22, 26, 28, 32, and 39. The standard deviation of age is also reduced considerably. However, the average age of water for this solution is at the higher end compared to the other solutions. The optimal solution with weighed average as objective has 11 closed valves located in pipes 3, 4, 5, 11, 13, 16, 21, 26, 37, 42, 45. Minimization of weighted average produced a solution that has very large values of maximum age (29.63 hrs) and standard deviation (5.05 hrs). One of the reasons why this large maximum age is produced is the small weight given to it since the demand is very small at that node. Hence, its effect on the objective function is small. Finally, the solution obtained by minimizing the average age has smaller quantities for almost all the parameters. This solution also has

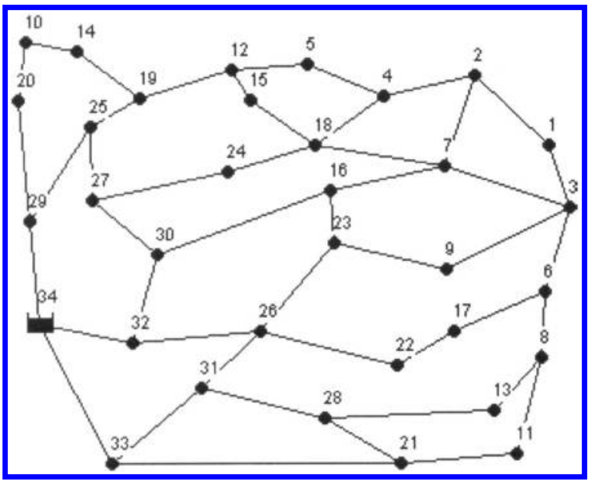

Figure 3. Example network.

Table 1. Node data for example network.

Node	Demand (L/s)	Elevation (m)	Node	Demand (L/s)	Elevation (m)
1	20.0	22.0	18	20.0	21.0
2	10.0	25.0	19	13.0	20.0
3	15.0	30.0	20	10.0	20.0
4	20.0	20.0	21	25.0	21.0
5	30.0	21.0	22	15.0	18.0
6	25.0	30.0	23	18.0	19.0
7	70.0	25.0	24	40.0	10.0
8	40.0	20.0	25	20.0	22.0
9	15.0	23.0	26	30.0	18.0
10	18.0	30.0	27	10.0	20.0
11	10.0	20.0	28	20.0	25.0
12	5.0	20.0	29	10.0	20.0
13	12.0	22.0	30	40.0	28.0
14	10.0	30.0	31	10.0	30.0
15	11.0	18.0	32	20.0	20.0
16	20.0	20.0	33	10.0	20.0
17	30.0	20.0	34 (Reservoir)	− 672.0	80.0

Table 2. Pipe data for the example network.

Pipe	Nodes U/s	D/s	Length (m)	Diameter (mm)	Hazen-Williams coefficient
1	1	2	1000	300	100
2	1	3	1200	300	100
3	2	4	2000	400	100
4	2	7	1000	400	100
5	3	6	1000	250	100
6	3	9	1800	450	100
7	3	7	1000	300	100
8	4	18	1000	450	100
9	4	5	1200	300	100
10	5	12	2200	400	100
11	6	8	2000	400	100
12	6	17	1000	300	100
13	7	16	2000	500	100
14	7	18	1200	400	100
15	8	13	1000	400	100
16	8	11	1000	500	100
17	9	23	1000	350	100
18	10	14	2000	400	100
19	10	20	1200	400	100
20	11	21	1000	400	100
21	12	19	2000	600	100
22	12	15	3000	600	100
23	13	28	1200	400	100
24	14	19	1200	400	100
25	15	18	1500	200	100
26	16	23	2000	600	100
27	16	30	2000	500	100
28	17	22	1000	400	100

(Continued)

Table 2. *Continued.*

	Nodes				
Pipe	U/s	D/s	Length m	Diameter mm	Hazen-Williams coefficient
29	18	24	1200	300	100
30	19	25	1200	600	100
31	20	29	2200	500	100
32	21	28	1800	400	100
33	21	33	1000	500	100
34	22	26	1000	400	100
35	23	26	1200	450	100
36	24	27	2000	600	100
37	25	29	1800	650	100
38	25	27	1800	500	100
39	26	31	1000	250	100
40	26	32	1000	400	100
41	27	30	800	400	100
42	28	31	1200	600	100
43	29	34	1400	600	100
44	30	32	2000	300	100
45	31	33	1000	700	100
46	32	34	800	700	100
47	33	34	1000	600	100

Table 3. Results of Example Network.

Objective function	Max. age*	Average age*	Weighted average age*	Standard deviation*
Original Network	16.31	4.64	4.69	4.06
$\min\left(\max_{i=1}^{N} A_i\right)$	6.29	3.48	3.58	1.91
$\min\left(\frac{1}{N}\sum_{i=1}^{N} A_i\right)$	7.39	2.78	2.96	2.09
$\min\left(\sum_{i=1}^{N} A_i Q_i \Big/ \sum_{i=1}^{N} Q_i\right)$	29.63	3.45	2.82	5.05

*All units are in hrs.

11 closed valves located in pipes 3, 7, 11, 13, 22, 23, 26, 30, 37, 42, and 45. Therefore, average age is taken as the water quality measure that best represents the entire network in this study.

6 CONCLUSIONS

The paper presents an evolutionary algorithm that minimizes a water quality parameter by re-configuring the network. A steady-state water quality model was used to find the age of water at various nodes of the network. The evolutionary algorithm was used to select a set of pipes of the network for closure, such that the water quality parameter is minimized. The water quality parameters considered were maximum age of water, average age of water, and weighted average age of water. The evolutionary algorithm ensures that the re-configured network is hydraulically feasible and is connected. Application of the model to an example network revealed that use of the average age of water as a water quality measure gave the best overall results in terms of standard deviation, average and maximum age values.

REFERENCES

Males, R.M., Clark, R.M., Wehrman, P.J. and Gateset, W.E. (1985). Algorithm for mixing problems in water systems. *ASCE Journal of Hydraulic Engineering*, Vol. 111, No. 2, pp. 206–219.

Michalewicz, Z. (1995). Genetic Algorithms, numerical optimization and constraints. *Proceedings of the 6th International Conference on Genetic Algorithms, Pittsburgh*, July 15–19, pp. 151–158.

Rossman, L.A. (1999). The EPANET Programmer's Toolkit for analysis of water distribution systems. *Proceedings of the Annual Water Resources Planning and Management Conference*, CD-ROM, ASCE, Reston, Va.

Savic, D. and Walters, G.A. (1995). An evolution program for optimal pressure regulation in water distribution networks. *Journal of Engineering Optimization*, Vol. 24, No. 3, pp. 197–219.

Savic, D.A. and Walters, G.A. (1997). Genetic Algorithms for the Least-cost Design of Water Distribution Networks, *ASCE Journal of Water Resources Planning and Management*, Vol. 123, No. 2, pp. 67–77.

Walters, G.A. and Smith, D.K. (1995). Evolutionary design algorithms for optimal layout of tree networks. *Journal of Engineering Optimization*, Vol. 24, No. 4, pp. 261–281.

A progressive mixed integer-programming method for pump scheduling

G. McCormick & R.S. Powell
Systems Engineering Department, Brunel University, Uxbridge, UK

ABSTRACT: One drawback of some pump scheduling methods is that results are continuous, but discrete schedules are needed. Heuristics are sometimes employed to derive discrete schedules from flow or reservoir profiles. Integer programming could produce discrete schedules directly but is impractical for networks with substantial numbers of pumps. Progressive mixed integer programming (P-MIP) solves an LP then progressively increases the discrete portion of the solution until a complete schedule is obtained. After iterative improvement costs are typically within 1% of the LP lower bound. The method is practical for realistic sized networks.

1 INTRODUCTION

Many different methods have been used for pump scheduling in clean water networks. Dynamic programming can give discrete schedules directly but problem complexity increases exponentially with the number of reservoirs in the network, which makes the method impractical with more than about three reservoirs. Gradient methods and linear programming can give discrete schedules directly if the decision variables are pump durations and all pumps are supposed to switch on at the beginning of a time-slot (if switched on at all). Unfortunately this can lead to schedules with excessive pump switching (Andersen & Powell, 1999).

Gradient methods and LP can alternatively be used to find optimal flow profiles or reservoir profiles. A heuristic solution phase must then follow to find discrete schedules which approximate to the continuous optimum. The development of a discrete solution from a continuous solution is not a trivial problem, and the discretisation heuristics have been a critical element in some earlier methods (e.g. Andersen & Powell, 1999; Burnell, Race and Evans, 1993). Integer programming formulations have been proposed which can provide discrete schedules directly (De Oliveira, Sousa et al 2001), but the problem size can be excessive: with 24 timeslices and 30 pumps there would be over 600 integer variables.

In this paper a progressive mixed integer heuristic is proposed which can provide discrete schedules for large networks. A continuous solution is found then the discrete portion of the solution is increased step by step until the whole solution is discrete. Subsequent passes refine the discrete solution. Costs of the final solution compare well with the LP lower bound. Further refinements (e.g. reduced pump switching) are possible using hill climbing.

The formulation of the scheduling problem as a linear program is described in section 2, following which the P-MIP heuristic is outlined in section 3. A greedy heuristic used after P-MIP to take pump switching costs into account and make small improvements is described in section 4, and results are tabled in section 5. After a discussion in section 6 conclusions are drawn in section 7.

2 PROBLEM FORMULATION

The concept of progressively increasing the discrete portion of a problem can be applied with many formulations. That used here divides the scheduling period into a number of timeslices of say one hours duration each, during which demands are almost constant and reservoir variations modest. Pumps are grouped into sets such that if there is a significant hydraulic interaction between two pumps they are placed in the same set. Sets of pump combinations are supposed to have been generated for each interacting group of pumps. The decision variables are the proportions of each combination to be used in each timeslice. This effectively linearises the problem if the effect of reservoir levels on flows is small, which is commonly the case. The coefficients of this formulation may be determined by simulating pump combinations and observing their effects on reservoirs, sources and energy consumption. Water demands are implicit.

Let $x_{gc}(t)$ be the proportion of timeslice t for which combination c of group Gg is switched on (a dimensionless number). A schedule is a valid assignment of a proportion $x_{gc}(t)$ to each group G_g for each timeslice t. The proportions must sum to 1 for each group and timeslice (there is a null combination in each group).

$$\forall t, g: \quad \sum_{c \in Gg} x_{gc}(t) = 1 \tag{1}$$

A mixed integer formulation can be obtained by constraining chosen proportions

$$\text{For selected } t, g, \forall c \in G_g: \quad x_{gc}(t) \in \{0,1\} \tag{2}$$

The objective function can be stated as minimize cost = energy cost + source cost + target shortfall cost.

$$\text{Min } K = \sum_{t=1}^{N_T} \sum_{g=0}^{N_G} \sum_{c \in Gg} \kappa_{gc}(t) x_{gc}(t) v(t) + \sum_{t=1}^{N_T} \sum_{s=1}^{N_S} \eta_s F_s(t) v(t) + \sum_{r=1}^{N_R} \pi_{T_r} O_{T_r} \tag{3}$$

Here r = reservoir, s = source, $v(t)$ = timeslice duration and $\kappa_{gc}(t)$ is the energy cost of a combination. $F_s(t)$ is the source output rate and η the cost of water at source. π_{T_r} is the price imposed if a reservoir end target is not met (to avoid horizon effects). O_{T_r} is the target shortfall. Minor deviations from target are reasonable when discrete pump durations are necessary.

Let L = reservoir volume, $\lambda_{rgc}(t)$ =a reservoir rate of change. Initial volumes $L_r(-1)$ are given. Mass balance requires that for all r, t

$$\boxed{L_r(t) = L_r(t-1) + \sum_{g=0}^{N_G} \sum_{c \in Gg} \lambda_{rgc}(t) x_{gc}(t) v(t)} \tag{4}$$

If O_{T_r} is the degree to which reservoir r falls short of its end of day target

$$L_r(t = last) + O_{T_r} \geq Ltarget_r \tag{5}$$

Source flows are derived from linear coefficients $\phi_{sgc}(t)$ and the proportions x.

$$\boxed{F_s(t) = \sum_{g=0}^{N_G} \sum_{c \in Gg} \phi_{sgc}(t) x_{gc}(t) v(t)} \tag{6}$$

Reservoirs must stay within control limits

$$Lmin_r \leq L_r(t) \leq Lmax_r \tag{7}$$

Sources must also stay within instantaneous and cumulative limits

$$Fmin_s \le F_s(t) \le Fmax_s \tag{8}$$

$$\sum_{t=1}^{NT} F_s(t) \le Fcumax_s \tag{9}$$

In many cases pressures will be satisfactory as long as reservoirs are in bounds. When this is not the case pressure constraints should be satisfied by limiting the combinations available in each group.

3 THE P-MIP HEURISTIC

The heuristic employs three phases:

Phase 1: LP solution,
Phase 2: A progressive increase in the proportion of the problem which is binary,
Phase 3: Iterative improvement of the solution.

The first phase is a conventional LP solution in which the decision variables are the proportions of combinations to be used in each timeslice. This provides a lower bound to the optimal schedule cost, but cannot be implemented directly: it is necessary to find binary values (0 or 1) of the proportions.

After the first (LP) phase, any timeslices in which combination proportions are already 1 or 0 are frozen: they are in effect removed from the problem. Decision variables in one or two timeslices are then declared to be binary and the remaining decision variables are continuous. A mixed integer solution is found and this process is repeated until binary decisions have been made for all timeslices.

Once binary values have been obtained for all decision variables attempts are made to improve the solution. In phase 3 one or two timeslices are optimized at a time, keeping decision variables in all other timeslices unchanged. This improvement process is repeated until either no further improvement is obtained or a time or iteration limit is reached.

It was found empirically that better results were obtained in phases 2 and 3 by starting with the first and last timeslices and then working from the extremes into the middle of the schedule.

P-MIP was implemented using a mathematical programming language (MPL: see MPL Manual, 2003) and the Delphi programming language. Delphi code decided which variables should be frozen. The frozen combinations became data for the MPL interpreter, which was able to efficiently calculate corresponding boundary conditions on the remaining part of the problem.

The P-MIP heuristic is described in figure 1.

4 FURTHER IMPROVEMENTS

The LP solution cost (continuously variable proportions or durations) cannot usually be met using binary proportions and fixed timeslices, as they imply extra constraints. Longer timeslices may lead to poor schedules due to inflexibility, for example Lansey & Awumah (1994) found empirically that with a DP formulation the solution cost increased noticeably after timeslice durations reached 3 hours. Smaller timeslices should allow a closer approximation to the continuous solution. However smaller timeslices means more timeslices, and the solution time of the P-MIP heuristic should be approximately proportional to the square of the number of timeslices (LP size increases and number of P-MIP steps also increases). As will be seen from section 5 results with 24 timeslices were close to the LP lower bound, and further improvements were obtained by using P-MIP with 24 timeslices, then splitting the solution into 96 timeslices, and using a simple greedy heuristic to find further improvements. An additional benefit was that the heuristic also considered pump switching costs. It is shown in figure 2.

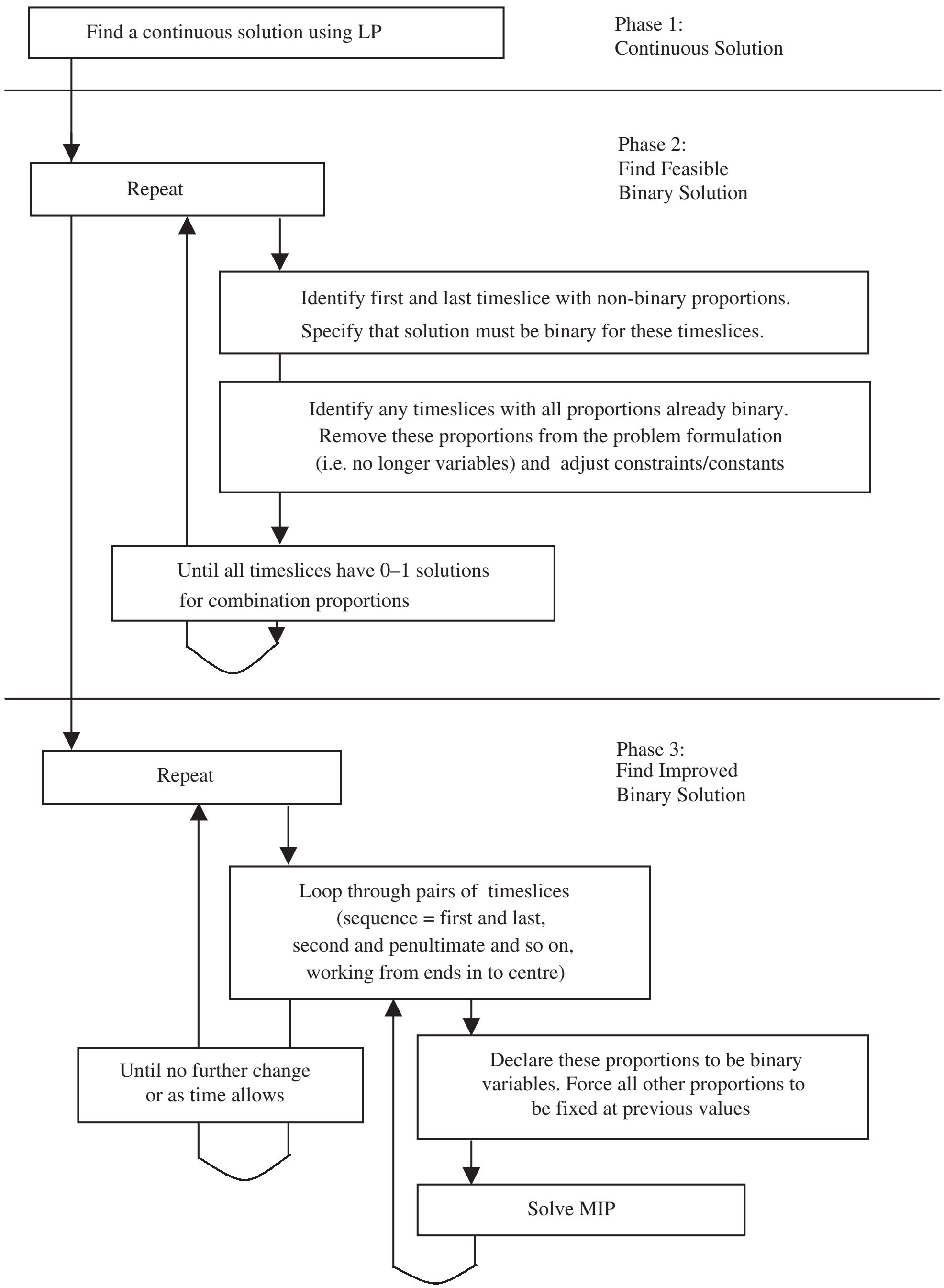

Figure 1. The P-MIP heuristic.

This local search systematically explores the neighbourhood of a schedule and accepts the most improving changes as they are found without deeper exploration. It usually finds a local optimum which it cannot climb out of, but this is acceptable if the starting schedule is near-optimal.

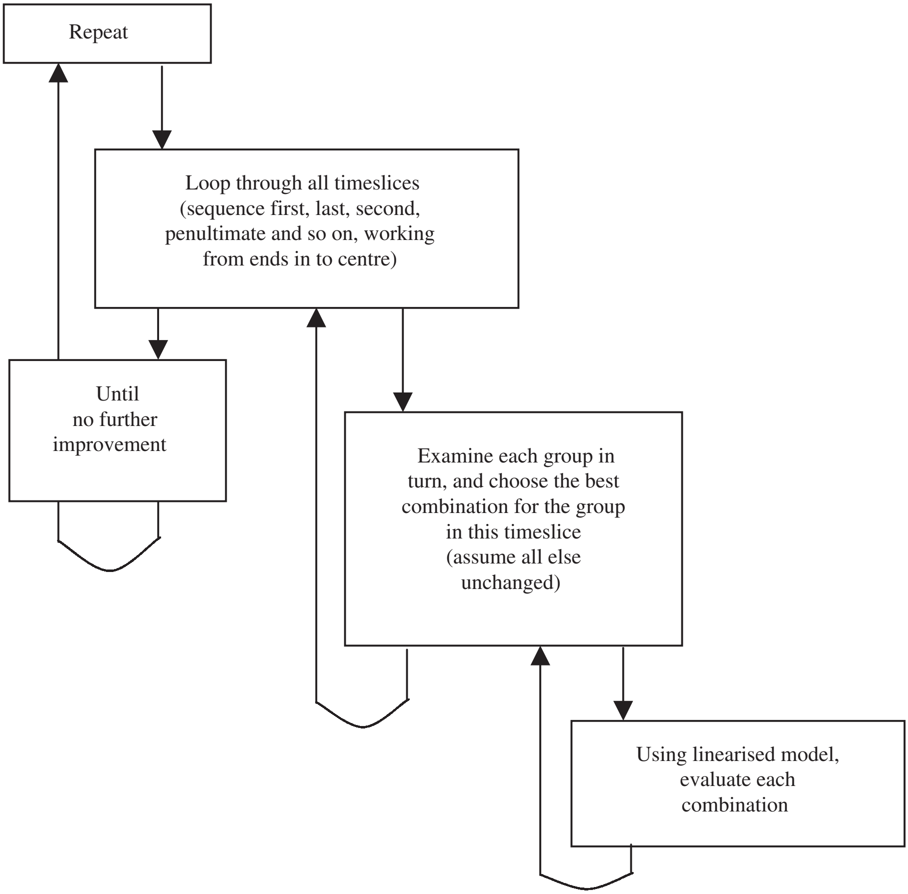

Figure 2. A greedy schedule improvement heuristic.

5 RESULTS

The "Mathematical Programming Language" (MPL) was used with the FortMP solver (see FortMP Description, 2003). For a problem with 24 timeslices, 10 reservoirs, 13 sources and 35 pumps LP was able to produce a continuous solution in less than 5 seconds on a 1 GHz PC using the primal simplex method. P-MIP involved several iterations and took correspondingly longer. Results are given in table 1.

6 DISCUSSION

P-MIP requires no tuning, and was easy to implement using a modelling language. There is a degree of flexibility in P-MIP. For smaller problems or using more powerful solvers or computers it might be possible to work with three or four timeslices at a time. Conversely, only one timeslices could be made binary at a time, reducing the size of each MIP problem, but this has not so far been found necessary.

P-MIP does not consider pump switching costs directly, but they may be taken into account by the greedy heuristic. Switching costs could be included in the integer program, but the formulation

Table 1. Results from P-MIP.

Network size	LP Cost £	P-MIP 24 timeslices Time (s)	Cost £	% of LB	P-MIP + Greedy 96 timeslices % of LB	Time (s)
1 Source, 1 Reservoir, 4 Pumps	294.96	3	295.83	100.3		
2 Sources, 2 Reservoirs, 7 Pumps	1831.61	7.3	1860.30	101.6	101.0	9.1
1 Source 5 Reservoirs, 9 Pumps	1079.51	10	1089.75	100.95	100.01	13
13 Sources, 10 Reservoirs, 35 Pumps	3920.57	233	3947.20	100.7	100.1	289

would be intricate because the timeslices chosen for optimisation in the second and third phases of P-MIP are not normally adjacent.

There may be some advantage to using variable length timeslices, e.g. inserting a few shorter length timeslices to increase scheduling flexibility without increasing the total number of timeslices very much. This has not yet been tried.

7 CONCLUSIONS

A progressive LP based method has been described that discretises only a portion of the problem at a time, which is much more practical than attempting a complete integer-optimal solution. The method requires no tuning.

P-MIP solution times are short enough for practical application, even with a network which has 13 sources, 10 reservoirs and 35 pumps. Scheduling networks with more reservoirs or pumps would be slower, but still practicable. With 24 timeslices P-MIP shows 0.3 to 1.6% loss of optimality relative to the LP lower bound. With a small additional expenditure of computation time, splitting into 15 minute timeslices allowed results within 1% of the lower bound to be obtained using a greedy heuristic using the 24 timeslice schedule as a starting point. In three of the four cases results were within 0.1% of the lower bound.

P-MIP provides high quality solutions and makes it practical to apply integer programming to larger problems than would be possible if attempting to solve the entire problem at once.

REFERENCES

Andersen JH & Powell RS 1999, "The Use of Continuous Decision Variables in an Optimising Fixed Speed Pump Scheduling Algorithm", in Powell RS & Hindi KS (eds) *Computing and Control for the Water Industry*, Research Studies Press. 119–128.

Burnell D, Race J & Evans P 1993, "The Trunk Scheduling System for the LWRM", in Coulbeck B (ed) *Integrated Computer Applications in Water Supply Vol 2: Applications and Implementations for Systems Operation and Management.* Research Studies Press 203–217.

De Oliveira Sousa JJ, da Conceicao Cunha M & de Almeida Sa Marques JA 2001, "Optimal Pumping Scheduling Model for Energy Cost Minimisation", in Ulanicki B, Coulbeck B & Rance J (eds). *Water Software Systems: Theory and Applications*, Vol 2. Research Studies Press. 99–109.

FortMP Description 2003, at http://www.optirisk.co.uk/fortmp.asp.
Lansey KE & Awumah K 1994, "Optimal Pump Operations Considering Pump Switching", *Journal of Water Resources Planning and Management,* ASCE, 120 (1) 17–35.
MPL Manual (2003), http://www.maximal-usa.com/mplman/mplwtoc.html.

NOTATION

c	pump combination index.
g	pump group index.
r	reservoir index.
s	source index.
t	timeslice index (discrete intervals are used).
$v(t)$	duration of timeslice t.
$x_{gc}(t)$	proportion of combination c group g in pump schedule at t (dimensionless).
$F_s(t)$	flow rate from source s in timeslice t.
$Fcumax$	maximum permitted daily source output.
$Fmax$	maximum permitted flow rate.
$Fmin$	minimum permitted flow rate.
G_g	pump group or set of pump combinations index.
K	cost.
$L_r\,(t)$	volume of reservoir r at time indexed by t.
$Lmax$	permitted maximum reservoir level.
$Lmin$	minimum permitted reservoir level.
$Ltarget_r$	target end of day level, reservoir r.
Nc_g	the number of combinations in group g.
N_G	number of groups.
N_R	number of service reservoirs.
N_S	number of sources.
N_T	number of timeslices or stages.
O_{T_r}	amount by which final reservoir level is short of target.
η_s	price of water from source s.
$\phi_{sgc}(t)$	flow from source s due to combination c of group g.
$\kappa_{gc}(t)$	energy cost per unit time of combination c group g.
$\lambda_{rgc}(t)$	rate of change of reservoir r due to combination c of group g.
π_{T_r}	penalty rate for shortfall relative to reservoir end of day target.

Multi-objective optimization to the rehabilitation of a water distribution network

P.B. Cheung, L.F.R. Reis & I.B. Carrijo
São Carlos School of Enginerring, University of São Paulo, São Carlos, São Paulo, Brazil

ABSTRACT: Many engineering problems involve optimization of multiple and competing objectives. Conventionally, analysts have treated these problems as single objective optimization problems instead of multi-objective ones. In multi-objective optimization, there is not a single optimum but a set of trade-off solutions known as Pareto-optimal, non-dominated or non-inferior solutions. Recognizing the multi-objective nature of the rehabilitation problem of water supply distribution networks, this paper presents an application of an elitist multi-objective evolutionary algorithm, namely, Strength Pareto Evolutionary Algorithm (SPEA), to generate a series of non-dominated solutions. The rehabilitation analyses were conducted on a simple hypothetical network for minimum cost, minimum pressure requirement and maximum hydraulic benefit, dealt with a three-objective problem. Analyses were performed to identify suitable crossover and mutation operators as well as parameters to this rehabilitation problem, for which SPEA presented high performance in producing Pareto fronts.

1 INTRODUCTION

Most of the existing water supply distribution systems were developed to operate over a determined planning period. Over time, however, failures caused by the deterioration of pipes and hydraulic components become frequent in such systems. In addition, the increasing demand for water caused by the urbanization has lead to problems such as insufficient discharges to meet demand and low-pressure levels in the network. Thus, the decision process of rehabilitation and replacement of existing components to meet current and future demands constitutes a subject of great interest.

Improvements in a distribution system performance can be achieved through rehabilitation of specific pipes, other components and/or the addition of new components to the existing network. In general, limited funding is available to modify the systems in order to guarantee a satisfactory level of water supply service. Researchers (Kim & Mays 1994, Kleiner et al. 1998) have applied optimization techniques to rehabilitation of water distribution systems, focusing on the economic considerations. Techniques such as linear, integer, non-linear and dynamic programming have been exhaustively used in water distribution system optimization.

Some researchers (Engelhardt et al. 2000, Walski 2001) have pointed out the disadvantages of the conventional optimization methods to treat the rehabilitation as a complex and discontinuous problem with many local optima. Many conventional optimization methods do not guarantee that a global optimum can be found. Further, they are based on a single objective, whereas many real situations require simultaneous optimization of multiple objectives.

Optimization has evolved over the recent years due to the introduction of a number of non-conventional algorithms as the Genetic Algorithms (GAs), which mimic the evolutionary principles of nature to drive the search towards optimal solutions. One of the most striking differences between classical search methods and GAs is the use of a set of solutions instead of only one solution (Deb 2001).

In single objective optimization, the best (the global minimum or maximum) design or operational strategy is obtained depending on the nature of the problem to be solved. Based on a very different concept, a typical multi-objective method seeks a set of solutions that are superior to the remaining ones in the search space. This set is denominated Pareto optimal front. Because GAs work with populations of possible solutions, a number of optimal solutions can be captured during their iterative search process. Thus they are naturally well suited to treat multi-objective problems.

According to Fonseca & Fleming (1995) evolutionary techniques for multi-objective optimization can be classified into several classes: objective reduction approaches, classified population approaches, weight-randomizing approaches, preference relationship approaches and Pareto-based approaches. In Pareto-based approaches the objectives are dealt with simultaneously, and, differently when compared to other cited classes methods which require simplifications.

Deb (2001) classified the Pareto-based approaches on two main classes: non-elitist and elitist algorithms. Three types of non-elitist implementations were first proposed as: Multi-objective Genetic Algorithms (MOGA) (Fonseca & Fleming 1993), Niched Pareto Genetic Algorithms (NPGA) (Horn & Nafpliotis 1994) and Non-Dominated Sorting Genetic Algorithms (NSGA) (Srinivas & Deb 1995). Two elitist implementations stand out: Strength Pareto Evolutionary Algorithm (SPEA) (Zitzler 1999) and Non-Dominated Sorting Genetic Algorithms II (NSGAII) (Deb et al. 2000). In spite of such a variety of methods, there are few comparisons available in the literature (Cheung et al. 2003) for water resources problems.

Several researchers (Simpson et al. 1994, Dandy et al. 1996, Reis et al. 1997, Savic & Walters 1997) have applied GAs to the optimization of water distribution systems. However, some studies (Halhal et al. 1997, Walters et al. 1999, Dandy & Engelhardt 2001, Kapelan et al. 2002, Cheung et al. 2003, Formiga et al. 2003) have suggested the use of multi-objective optimization techniques in water distribution system problems. More recently, MOGA was applied (Dandy & Engelhardt 2001) to the problem of rehabilitation of a hypothetical network, considering cost and reliability criteria.

Cheung et al. (2003) compared the performance of the non-elitist (MOGA) and elitist (SPEA) methods in terms of Pareto front achieved, analyzed processing time and discussed population sizes from both methods in a water distribution network rehabilitation problem. The comparison of methods was based on a performance metric (Set Coverage Metric) proposed by Zitzler (1999), which evaluates the relative spread of solutions between two sets of solution vectors. The results showed that the Pareto fronts obtained through MOGA were entirely dominated by the fronts obtained through SPEA for the various population sizes. Besides, SPEA is faster than MOGA.

This paper presents an effort to identify suitable crossover and mutation operators to be considered in elitist multiobjective evolutionary algorithm (SPEA) applied to the rehabilitation of a hypothetical network (Gessler 1985).

2 THE PROBLEM OF WATER DISTRIBUTION NETWORK REHABILITATION

The performance of water networks can be improved in terms of their hydraulic capacity by cleaning, relining, duplicating or replacing existing pipes; increasing their physical integrity by replacing structurally weak pipes; increasing system flexibility by additional pipe links; and improving water quality by removing or relining old pipes (Halhal et al. 1997).

Generally, high costs are involved in remedial works and available financial resources are limited to implementation of such task. Thus, there is a need for implementation and development of optimal rehabilitation plans as the funding must be optimally invested over the planning period.

3 MULTI-OBJECTIVE OPTIMIZATION MODEL

Five objectives can be pointed out with regard to the operation of water distribution networks, namely, hydraulic capacity, physical integrity, flexibility, water quality and economy, each of them can be expressed by means of several attributes, constituting a complex multi-objective problem.

Classical methods are not efficient for multi-objective problems as they often lead to a single solution instead of a set of Pareto optimal solutions. Multiple runs cannot guarantee generation of different points on the Pareto front each time and some methods cannot even handle problems with multiple optimal solutions (Deb 2001)

Evolutionary methods, on the other hand, maintain a set of solutions as a population during the course of search and thus result in a set of Pareto optimal solutions in a single run (Fonseca & Fleming 1993, Veldhuizen 1999, Zitzler 1999, Deb 2001).

3.1 *Problem formulation*

Water distribution systems frequently require rehabilitation (cleaning, lining, reinforcement, among others) to maintain the satisfactory services for the population. However, the rehabilitation of an existing system is a complex task if it is to be implemented in the most effective and economic manner. It necessitates a systematic and thorough approach, backed up by skillful engineering judgment, and considerable capital resources. The examination and evaluation of design alternatives is a field in which optimization models can play an important role, particularly when economic resources are limited and the problems are of large size (Walters et al. 1999).

Many objectives can be incorporated in rehabilitation decision models. We prefer to formulate a three-objective network rehabilitation problem in order to compare our results with those reported in literature (Simpson et al. 1994, Cheung 2003) that consider single-objective and two-objective, respectively. Thus, the present paper formulates the rehabilitation problem as that of minimization of cost (Equation 1) and pressure deficit (Equation 2) as well as maximization of hydraulic benefit (Equation 3), considering various combinations of rehabilitation choices. The individual objectives are expressed as follows.

$$\boxed{\text{Minimize Cost, } F_1 = \sum_{\ell \in \Im} c_\ell L_\ell + \sum_{k \in \pi} c_k L_k} \tag{1}$$

where ℓ is the index of the pipes to be rehabilitated (cleaned or left unaltered); k is the index of the new pipes (replaced or duplicated); $\Im$ is the set of alternatives related to the pipes requiring rehabilitation; π is the set of alternatives for new pipes; L is length of the pipe; c_ℓ are rehabilitation unit costs and c_k are unit costs of new pipes. The decision problem corresponds to the identification of pipes to be added in parallel or as a new pipe.

$$\text{Minimize Pressure Deficit, } F_2 = \sum_{i=1}^{LC} \max\left(H_j - H_{j\min}\right)_i \quad j = 1,2,\ldots,nn \tag{2}$$

where pressure deficit is the sum of maximum nodal deficits on the network for each demand pattern; j is the index of nodes; nn is the total number of nodes in the system; H_j is the energy and H_{jmin} is the required minimum energy at node j. LC denotes the number of demand patterns considered. In this study three demand patterns shall be investigated: peak, average and minimum demands.

Finally, the maximization of hydraulic benefit is based on formulation proposed by Halhal et al. (1997). However, in this paper it has been modified to give a physic characterization to the hydraulic benefit formulation. This objective function (Equation 3) is quantified as the difference between the pressure deficiencies in the network before improvement (DEFO) and after improvement (DEFP) represented by each solution found which is calculated by Equation 4.

$$\text{Maximize Hydraulic Benefit, } F_3 = \text{DEFO} - \text{DEFP} \tag{3}$$

$$\boxed{\text{DEFO} / \text{DEFP} = \gamma \sum_{j \in \Im} \left|H_{min} - H\right|_j Q_j} \tag{4}$$

where γ represents the specific weight of water; $\Im$ is the set of nodes related to the energy below minimum required energy at node j; H is the energy and H_{min} is the required minimum energy at node j and Q is the demand at node j. Observe that this formulation (Equation 3) produces a benefit in terms of power and which differs from formulation proposed by Halhal et al. (1997).

3.2 *Multi-objective evolutionary algorithm*

Ziztler (1999) introduced elitism by explicitly maintaining an external population of possible solutions in the resolution of such multi-objective problem. This population stores a fixed number of the non-dominated solutions that are found until the beginning of iteration. In each iterate, newly found non-dominated solutions are compared with existing external population and the resulting non-dominated solutions are preserved. This algorithm is called Strength Pareto Evolutionary Algorithm (SPEA). It does more than just preserve the best solutions but also uses these elite solutions to participate in the genetic operations along with the current population in the hope of influencing the population to steer towards good regions in the search space.

This study employs the SPEA method based on elitism, implementing the code developed by Andrzej Jaszkiewicz in the Multi-objective Methods Metaheuristic Library for C++ (MOMHLib++). This library allows the user to build the genetic operators (recombination and mutation) most proper to the specific problem in hand. Three recombination operators (Linear, BLX-α and Uniform) and three mutation operators (Boundary, Random and Non-Uniform) were implemented whose details are presented here.

Recombination (crossover) and mutation aim at generating new solutions within the search space by the variation of existing ones. The recombination operator takes a certain number of parents and creates a certain number of offspring by recombining the parents features. To mimic the stochastic nature of evolution, a crossover probability is associated with this operator. By contrast, the mutation operator modifies individuals by changing small parts in the associated vectors according to a given mutation rate (Zitzler 1999).

3.2.1 *Recombination operator*

Linear recombination: Wright (1991) proposed one of the earliest implementations where a linear operator creates three new solutions (Eqs. 5–7) from two parents solutions $x_i^{(1,t)}$ and $x_i^{(2,t)}$ at generation t, with the two best solutions being chosen as offspring. In this paper, the best solution of them was chosen utilizing the non-dominance concept.

$$x_i^{(1,t+1)} = 0.5x_i^{(1,t)} + 0.5x_i^{(2,t)} \tag{5}$$

$$x_i^{(1,t+1)} = 1.5x_i^{(1,t)} - 0.5x_i^{(2,t)} \tag{6}$$

$$x_i^{(1,t+1)} = -0.5x_i^{(1,t)} + 1.5x_i^{(2,t)} \tag{7}$$

where x is the decision variable; i is the index of the decision variable; t is the index of the generation; 1 and 2 are index of parents vectors

Blend recombination: Eshelman and Schaffer (1993) suggested this operator (BLX-α) for real-parameter GAs. Starting from parent solutions $x_i^{(1,t)}$ and $x_i^{(2,t)}$ (assuming $x_i^{(1,t)} < x_i^{(2,t)}$), the BLX-$\alpha$ randomly picks a solution in the range $[x_i^{(1,t)} - \alpha(x_i^{(2,t)} - x_i^{(1,t)}), x_i^{(2,t)} + \alpha(x_i^{(2,t)} - x_i^{(1,t)})]$. Thus, considering u_i a random number between 0 and 1, the BLX-α operator can be described in Equation 8.

$$x_i^{(1,t+1)} = (1-\gamma_i)x_i^{(1,t)} + \gamma_i x_i^{(2,t)} \tag{8}$$

where $\gamma_i = (1 + 2\alpha)u_i - \alpha$. Deb (2001) reported that α equals 0.5 performs better than BLX-α with another α values.

Uniform recombination: Goldberg (1989) and Michalewicz (1992) described the uniform recombination method which operates on individual genes of the selected chromosomes. They suggested this operator for the binary representation. However, in this study the uniform recombination operator was modified to the rehabilitation problem that was represented by real code (decision variables). Initially, a template vector (m_i) that assumes 0 or 1 value is randomly generated (the same size of solution vector) in each recombination operation. Equation 8 demonstrates it.

$$\boxed{x_i^{(1,t+1)} = \begin{cases} x_i^{(1,t)} & \text{if} \quad m_i = 0 \\ x_i^{(2,t)} & \text{if} \quad m_i = 1 \end{cases}} \tag{9}$$

3.2.2 *Mutation operator*

Mutation is an important process that permits new genetic material to be introduced to a population during iterative process. The task of mutation operator is to disturb every solution in the parent population to create a new population.

Boundary mutation: Michalewicz (1992) reported that this operator (Equation 10) substitutes a selected gene by random gene which it is picked in $[x_i^{(L)}, x_i^{(U)}]$ interval where $x_i^{(L)}$and $x_i^{(U)}$ are the lower limit and upper limit of decision variables, respectively.

$$\boxed{y_i^{(t+1)} = \begin{cases} x_i^L & \text{if} \quad r < 0.5 \\ x_i^u & \text{if} \quad r > 0.5 \end{cases}} \tag{10}$$

where y_i is the new solution and r is a random number in [0,1].

Random mutation: Deb (2001) reports this operator (Equation 11) as the simplest mutation scheme to create a solution randomly from entire search space.

$$y_i^{(t+1)} = r_i(x_i^{(U)} - x_i^{(L)}) \tag{11}$$

Non-uniform mutation: This operator depends on the current generation and the maximum number of allowed generations to create a new solution (Deb 2001). Equation 12 describes it.

$$y_i^{(t+1)} = x_i^{(1,t+1)} + \tau(x_i^{(U)} - x_i^{(L)})(1 - r_i^{(1-\frac{t}{t\max})^b}) \tag{12}$$

where τ takes a Boolean value, -1 or 1, each with a probability of 0.5; t max is the maximum number of allowed generations and b is a user-defined parameter. In this study, the b parameter was chosen 6 (Michalewicz 1992).

3.3 *Hydraulic simulator model*

A steady-state hydraulic analysis was used to evaluate the consequences of a rehabilitation plan in terms of the objectives F_1, F_2 and F_3 using the EPANET 2 code (Rossman 2000) that was linked to our C++ code. It should be noted that EPANET 2 represents an efficient code for hydraulic calculations related to water distribution networks.

3.4 *Performance metric*

According to Deb (2001), performance measures should be used in the multiobjective analysis. In this study, the set coverage metric (Zitzler 1999) was adopted as performance index to compare the efficiency of genetic operators. The metric is used to get an idea of the relative spread of solutions between two sets of solution vectors A and B. The set coverage metric C (A, B) calculates the proportion of solutions in B, which are weakly dominated by solutions of A (Equation 13):

$$\boxed{C(A,B) = \frac{\left|\{b \in B \mid \exists a \in A : a \preceq b\}\right|}{|B|}} \tag{13}$$

The expression $a \preceq b$ denotes that a dominates b. The metric value C (A, B) $=$ 1 means that all the members of B are weakly dominated by A. On the other hand, C (A, B) $=$ 0 expresses that no member of B is weakly dominated by A. Since the domination operator is not a symmetric operator, C (A, B) is not necessarily equal to 1 $-$ C (B, A). Thus, one must calculate both C (A, B) and C (B, A) to understand how many solutions of A are covered by B and vice versa.

4 APPLICATION EXAMPLE

The rehabilitation study of the hypothetical network in Figure 1 was initially proposed in Gessler (1985). Later various authors (Simpson et al. 1994, Wu & Simpson 2001, Cheung et al. 2003) used this problem as the basis for comparisons of their formulations. The network in Figure 1 has 14 pipes, 2 constant level reservoirs (nodes 1 and 5) and 9 demand nodes (2, 3, 6, 7, 8, 9, 10, 11 and 12), where the solid lines represent the existing system and dashed lines depict new pipes. The hydraulic data can be found in the original paper (Gessler 1985) and several other papers (Simpson et al. 1994, Cheung et al. 2003).

The problem as posed in (Gessler 1985) has some interesting features that include: selection of diameters for five new pipes; three existing pipes may be cleaned, duplicated, or may remain unaltered; three demand patterns are considered; and two supply sources are available, whose options are described in Table 1. The respective costs also can be found in (Gessler 1985, Simpson et al. 1994, Cheung et al. 2003).

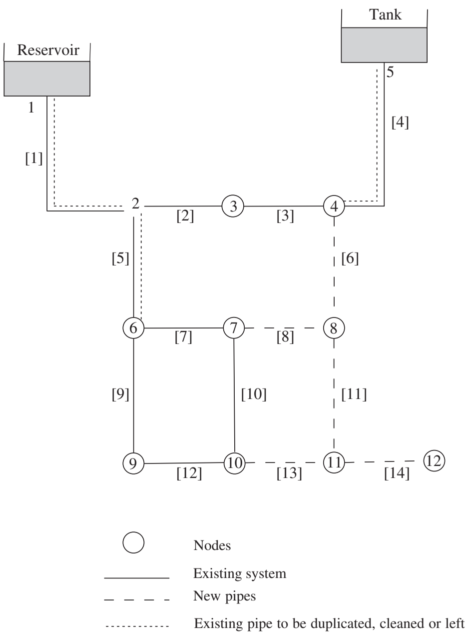

Figure 1. Hypothetical network.

Table 1. Decision options (rehabilitation and design).

Rehabilitation option	New pipe option (mm)	Real code
Leave as existing	152	0
Clean existing pipe	203	1
Duplicate with 152 mm	254	2
Duplicate with 203 mm	305	3
Duplicate with 254 mm	356	4
Duplicate with 305 mm	407	5
Duplicate with 356 mm	458	6
Duplicate with 407 mm	509	7

4.1 *Genetic algorithm implementation*

GA applications are important to find the appropriate representation of decision variables through strings of fixed length. While many coding schemes are possible, it is convenient to avoid the decoding phase in order to reduce processing time. Several authors (Michalewicz 1992, Goldberg 1991) have suggested the use of real code instead of the binary one in order to keep one gene-one variable correspondence. Hence, real code has been used in this study for decision variables representing the rehabilitation options to be implemented in the network to improve its hydraulic performance. The first three variables in the string refers to the decision in pipes 1, 4 and 5, for which values in the range from 0 to 7 have to be determined, according to the options defined in Table 1. The next five variables in the string refer to the decision for pipes 6, 8, 11, 13 and 14, for which values in the range from 0 to 7 have to be determined according to the options defined in Table 1.

4.1.1 *Genetic algorithm parameters*

For this application example, a population size (n) of 300 strings was considered following suggestions of Cheung et al. (2003). A probability of recombination (p_r) of 0.9 and a mutation probability (p_m) of 0.1 ($1/n < p_m \leq 1/\ell$, where ℓ represents string length) were chosen following Simpson et al. (1994). The SPEA was permitted to run for 1000 generations, starting from three different initial populations of solutions (random seeds).

5 RESULTS AND DISCUSSIONS

The results obtained from application of SPEA method to the example problem (Fig. 1) are presented in this section in order to identify the influence of different recombination and mutation operators on the Pareto fronts produced. Three recombination and mutation operators have been considered. Table 2 presents the computational simulation design which was performed in this work. All simulations were run in an AMD Athlon (TM) XP 1800+.

Table 2 presents the average processing time for each type of considered configuration (genetic operators) in this study. Observe that the configurations #2, #5 and #8 presented the highest processing time. In common, these configurations used the linear recombination operator. The difference in the behavior of linear recombination operator is due to search algorithm of nondominated solutions. This recombination operator takes more processing time than other operators because it always creates three new individuals in each evaluation and the best individual is chosen according to the Pareto dominance concept (search algorithm of nondominated solutions). It can be observed that the mutation operator does not significantly change the processing time.

As mentioned before (item 1), the multiobjective optimization methods always look for a set of optimal solutions. The visualization of final solutions is very difficult, mainly when the problem has more than two objectives. In some studies, researches still get to plot (Halhal et al. 1997,

Table 2. Computational simulation design and average processing time.

Configuration	Recombination operator	Mutation operator	Random seeds	Average run time (minutes)
1	BLX-α	Boundary	0, 1000 and 2000	11.12
2	Linear	Boundary	0, 1000 and 2000	20.13
3	Uniform	Boundary	0, 1000 and 2000	11.30
4	BLX-α	Random	0, 1000 and 2000	10.18
5	Linear	Random	0, 1000 and 2000	17.61
6	Uniform	Random	0, 1000 and 2000	10.3
7	BLX-α	Non-uniform	0, 1000 and 2000	11.8
8	Linear	Non-uniform	0, 1000 and 2000	20.8
9	Uniform	Non-uniform	0, 1000 and 2000	10.9

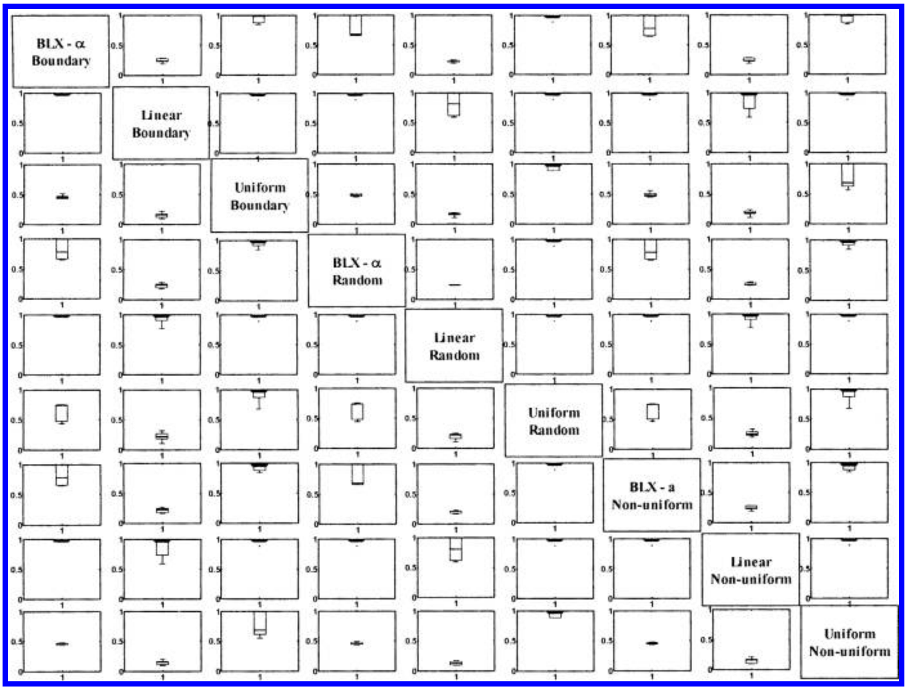

Figure 2. Box plot based on measure C defined (3.5).

Walters et al. 1999) and to perform sensitivity analysis (Cheung et al. 2003) to infer about genetic parameters (generations, population size). However, for three-objective problems comparison studies the performance metric (Equation 13) should be used and the visual analyses should be used as complement. In as much as, the main objective of this paper was to investigate the relative merit of different genetic operators (recombination and mutation) in a three-objective problem, the performance metric (3.5) was used to evaluate the different simulation configurations (Table 3).

Three runs were made for each configuration starting from distinct initial populations (random seeds) and the comparisons were performed considering the combinations between all possible pairs of configurations. The metric in (Equation 13) was used and the results presented in matrix box plot form in Figure 2, where each rectangle contain a box plot representing the distribution of

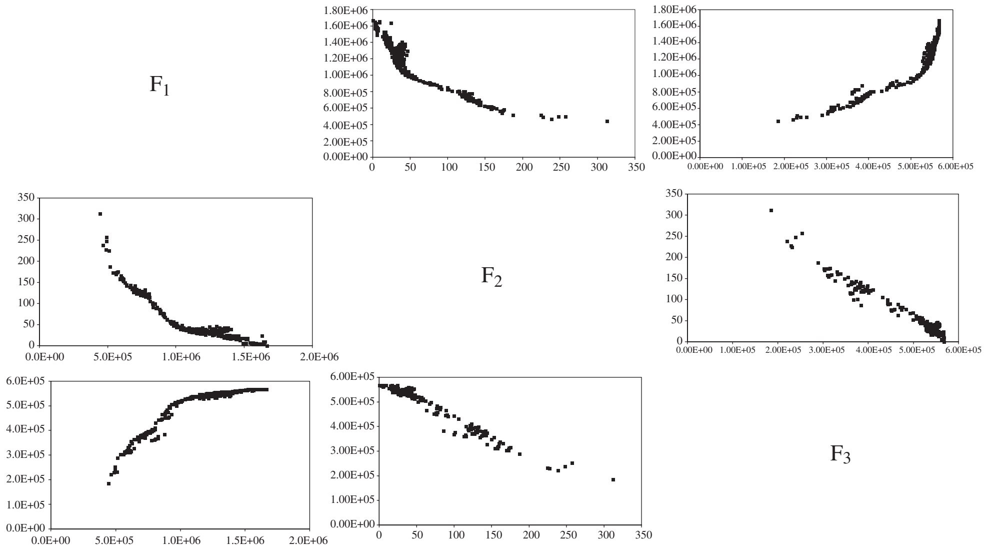

Figure 3. Scatter-matrix of a set of nondominated solutions at #9 configuration (random seed 1000).

the C values for all combinations (nine) of pairs of two configurations. Each graph presents distribution of C calculated from results (A) obtained from the configuration indicated in the row in combination with those (B) from the configuration indicated in the column through definition in (Equation 13) for C (A, B).

Box plots are used to visualize the distribution of these samples. The upper and lower ends of the box are the upper and lower quartiles, while the line within the box encodes the median. According to this figure (Figure 2) the box summarize the spread and shape of the distribution.

Figure 2 shows the direct comparison based on the measure C (3.5) for both configurations. The Pareto fronts achieved by configuration that utilizes linear recombination operator configuration are entirely dominated. The configurations that contain BLX-α operator are dominated by the configurations that contain uniform operator. It is seen that the configurations that contain uniform recombination operator dominate all other configurations that not contain it. There is not clear evidence of superior performance when the results obtained for several configurations are compared among themselves for a given case. In general, it seems that the recombination operators have more influence on the Pareto front than mutation operator.

Figure 3 presents a set of nondominated solutions for configuration #9, among which a solution (F_1 = \$1.67E+06, F_2 = 0 m and F_3 = 567359 kW) can be identified as similar to the one that has already been pointed out by Simpson et al. (1994), considering a single objective problem, and Cheung et al. (2003) considering a two-objective problem.

The solutions of SPEA permits to the decisor-maker to visualize trade-offs (costs, pressure deficit and hydraulic benefit) and can choose a satisfactory solution that meets the needs of his system. For example, if the decision-maker accepts the solution of Figure 3, he is aware of that the cost is not minimum value and that the benefit is not obtained maximum value. It demonstrates that the multi-objective optimization technique has become a great interest area to the decision making of water distribution systems problems.

6 CONCLUSION

This paper employs an elitist multi-objective evolutionary algorithm (SPEA) to the rehabilitation problem of a water distribution network, considering the objectives of minimization of costs and pressure deficit and maximization of hydraulic benefit of network to meet water demand. It represents an effort to compare the performance of the recombination and mutation operators applied to the problem that suggests the uniform recombination operator as suitable operator to the rehabilitation problem using SPEA. Another contribution presented in this paper refers the hydraulic benefit formulation that transforms the deficiency of nodal pressures to power (kW).

Direct comparison based on set coverage metric (Equation 13) shows the poor performance of the linear recombination operator in terms of the proximity of the calculated solution to the Pareto region as in terms of processing time. However, the uniform recombination operator presents the highest performance in relation to the others considered recombination operators. These analyses also demonstrates that recombination operator has more influence on the Pareto solution than the mutation operator.

In order to obtain satisfactory results using multi-objective evolutionary algorithms for water distribution networks, it is important to choose appropriated recombination and mutation operators for reading a stable Pareto front. Finally, the potentialities of the elitist multi-objective evolutionary algorithm SPEA are demonstrated, and several future possibilities are open for resolution of water resources problems including the treatment of more realistic and complex objectives than those dealt with here.

ACKNOWLEDGEMENTS

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REFERENCES

Cheung, P.B., Reis, L.F.R., Formiga, K.T.M., Chaudhry, F.H. & Ticona, W.G.C. 2003. Multiobjective evolutionary algorithms applied to the rehabilitation of a water distribution system: a comparative study. *Evolutionary Multi-Criterion Optimization.* Springer, Heidelberg. LNCS 2632: 662–676.

Dandy, G.C., Simpson, A.R. & Murphy, L.J. 1996. An improved genetic algorithm for pipe network optimisation. *Water Resources Research,* 32(2): 449–458.

Dandy, G.C. & Engelhardt, M.O. 2001. Optimum rehabilitation of water distribution system considering cost and reliability. *Proceedings of the World Water and Environmental Resources Congress, Orlando, Florida, 2001.*

Deb, K. 2001. *Multi-Objective Using Evolutionary Algorithms*. John Wiley & Sons, Ltd.

Deb, K., Pratap, A., Agarwal, S. & Meyarivan, T. 2000. A fast and elitist multi-objective genetic algorithm – NSGAII. *KanGAL Report Number 2000001.*

Engelhardt, M.O., Skipworth, P.J., Savic, D.A., Saul, A.J. & Walters, G.A. 2000. Rehabilitation strategies for water distribution networks: a literature review with a UK perspective. *Urban Water* (2): 153–170.

Eshelman, L.J. & Schaffer, J.D. 1993. Real-coded genetic algorithms and interval-schemata. In *Foundations of Genetic Algorithms 2 (FOGA-2)*: 187–202.

Fonseca, C.M. & Fleming, P.J. 1993. Genetic algorithms for multiobjective optimization: formulation, discussion and generalization. *Proceedings of the Fifth International Conference, San Mateo 1993.*

Fonseca, C.M. & Fleming, P.J. 1995. An overview of evolutionary in multiobjective optimization. *Evolutionary Computation* 3(1): 1–16.

Formiga, K.T.M., Chaudhry, F.H., Cheung, P.B. & Reis, L.F.R. 2003. Optimal design of water distribution system by multiobjective evolutionary methods. *Evolutionary Multi-Criterion Optimization.* Springer, Heidelberg. LNCS 2632: 677–691.

Gessler, J. 1985. Pipe network optimization by enumeration. *Proceedings Computer Applications for Water Resources, ASCE, New York, N.Y.*, 572–581.

Goldberg, D.E. 1989. *Genetic Algorithms in Search, Optimization, and Machine Learning.* Addison-Wesley, Reading, Massachusetts.

Goldberg, D.E. 1991. Real-coded genetic algorithm, virtual alphabets, and blocking. *Complex Systems*, 5 (2): 139–168.

Halhal, D., Walters, G.A., Ouazar, D. & Savic, D.A. 1999. Water network rehabilitation with structured messy genetic algorithm. *Journal of Water Resources Planning and Management* 123(3): 137–146.

Horn, J. & Nafpliotis, N. 1994. Multiobjective optimization using the niched pareto genetic algorithm. *Proceedings of the First IEEE Conference on Evolutionary Computation, IEEE World Congress on Computational Intelligence,* 1: 1–32.

Jaskiewicz, A. MOMHLIB++: Multiple Objective Metaheuristics Library in C++. *http://www-idss.cs.put.poznan.pl/~jaszkiewicz/MOMHLIB.*

Kapelan, Z.S., Savic, D.A. & Walters, G.A. 2002. Multi-objective sampling design for water distribution model calibration. *Proceedings 3rd International Conference on Environmental Management*, Johannesburg, RSA.

Kim, J.H. & Mays, L.W. 1994. Optimal rehabilitation model for water distribution systems. *Journal of Water Resources Planning and Management*, ASCE, 120(5): 674–692.

Kleiner, Y., Adams, B.J. & Rogers, J.S. 1998. Long-term planning metodology for water distribution system rehabilitation. *Water Resources Research*, 34(8): 2039–2051.

Michalewicz, Z. 1992. *Genetic Algorithms + Data Structures = Evolution Programs*. Springer-Verlag, New York, N.Y.

Reis, L.F.R., Porto, R.M. & Chaudhry, F.H. 1997. Optimal location of control valves in pipe networks by genetic algorithm. *Journal of Water Resources Planning and Management.* 123(6): 317–326.

Rossman, L.A. 2000. *EPANET 2, USERS MANUAL*. U. S. Environmental Protection Agency, Cincinnati, Ohio.

Savic, D.A. & Walters, G.A. 1997. Genetic algorithms for least-cost of water distribution networks. *Journal of Water Resources Planning and Management*, ASCE, 123(2): 67–77.

Simpson, A.R., Dandy, G.C. & Murphy, L.J. 1994. Genetic algorithms compared to other techniques for pipe optimization. *Journal of Water Resources Planning and Management*, ASCE, 120(4): 423–443.

Srinivas, N. & Deb, K. 1995. Multiobjective optimization using nondominated sorting in genetic algorithms. *Evolutionary Computation*. Massachusetts Institute of Technology, 2(3): 221–248.

Veldhuizen, D.V. 1999. *Multiobjective Evolutionary Algorithms: Classifications, Analyses, and New Innovations*. Ph.D. Thesis, Dayton, OH: Air Force Institute of Technology.

Walski, T.M. 2001. The wrong paradigm, why water distribution optimization doesn't work. *Journal of Water Resources Planning and Management* (Editorial), ASCE, 123(3): 203–205.

Walters, G.A., Halhal, D., Savic, D.A. & Ouazar, D. 1999. Improved design of anytown distribution network using structured messy genetic algorithms. *Urban Water*, 1: 23–38.

Wright, A. 1991. Genetic algorithms for real parameter optimization. In *Foundations of Genetic Algorithms 2 (FOGA-1)*: 205–218.

Wu, Z.Y. & Simpson A.R. 2001. Competent genetic-evolutionary optimization of water distribution systems. *Journal of Water Resources Planning and Management*, ASCE, 15(2): 89–101.

Zitzler, E. 1999. Evolutionary algorithms for multiobjective optimization: methods and applications. Ph.D. Thesis, Switzerland, Swiss Federal Institute of Technology Zurich.

Chapter 5 – Network operation

Security of supply for service reservoirs

G. McCormick & R.S Powell
Systems Engineering Department, Brunel University, Uxbridge UK

ABSTRACT: A rapid method of assessing security of supply from service-reservoirs is described. A method of determining the minimum storage profile needed to ensure compliance with a prescribed service standard is also indicated. The probability distribution of bulk supply interruptions is calculated from component reliability data. Consequences of interruptions are inferred using a mass-balance heuristic. The probability and distribution of resulting customer supply problem durations is then calculated from the resulting reservoir profiles. Control curves are obtained from an inverse procedure. Calculations are fast enough for sensitivity analysis and for interactive evaluation of alternative operational or design options. Monte Carlo simulation has verified the method.

1 INTRODUCTION

Many water distribution networks are designed in such a way that customers will have adequate supply pressures as long as there is water in the reservoir. There are "rule of thumb" criteria for reservoir sizing (e.g. Twort, Ratnayaka & Brandt, 2000, pp 500–501), but they do not allow cost vs. benefit to be explicitly examined. Most published work on formal assessment of reservoir reliability depends on Monte Carlo studies or case studies which are slow and hard to generalise. For example Nel & Haarhof (1996) described the simulation of plant failures during thousands of years of simulated operation, with response to each failure decided by optimisation. The process was repeated for each alternative to be investigated. Case studies typically investigate a small number of carefully chosen incidents and then apply probabilities to estimate the general implications of these incidents. They have the benefits and problems of Monte Carlo studies in an accentuated form (e.g. Germanopoulos, Jowitt & Lumbers, 1986).

When supply to a service reservoir and distribution from it are hydraulically separate, by making a few reasonable assumptions about operating policy and failure statistics a rapid enumeration becomes possible which is equivalent to Monte Carlo simulation but fast enough to evaluate a range of alternative designs interactively. The enumeration can also be used to calculate constant security reservoir profiles, and as a basis for improved schedule optimisation techniques which take reservoir security into consideration.

The enumeration method and the physical assumptions it is based on are outlined in section 2. This method is based on statistical assumptions, described in section 3. The detailed calculations are given in section 4. In section 5 a hypothetical example is used to illustrate how the method can be applied to a system with two sources, and one main reservoir which feeds three smaller reservoirs. The method can be used to support pump schedule optimisation subject to security constraints, and this is discussed in section 6. The method was verified using Monte Carlo simulation, described in section 7. Results are discussed in section 8 and conclusions are drawn in section 9.

2 THE PROBLEM: DEFINITIONS AND METHODOLOGY

“Reliability” is used here to describe plant failure. “Security” will indicate the satisfactory meeting of consumer demands, which might or might not be affected by plant failures. “Availability” will be measured by the time free from customer problems.

- Outline network topology and arc capacities are specified, including source capacities, optional infusions, and onward pumping to secondary reservoirs.
- Plant failure modes, probabilities, duration distributions and rates are specified. Fire flow requirements are represented as a failure type similar to bursts, with a specified severity, probability and duration distribution.
- Demand distributions and a typical reservoir profile are also specified.
- The consequences of each failure mode on the ability of the network to feed the reservoir are calculated.
- For each combination of failure mode, demand level and time of day, operational responses are efficiently calculated using a mass balance heuristic. Reservoir profiles are thus obtained.
- Making suitable statistical assumptions, the probability and mean duration of customer problems is deduced from the set of reservoir profiles.
- Distributions of customer problem duration may then be calculated by a series of convolutions.
- Constant security control curves may be obtained by using a bracketing algorithm to invert the security calculations.

The calculations consider only one main reservoir, which sits between a relatively simple supply network and a distribution network. It is assumed that poor distribution pressures or other problems will result when the main reservoir volume is below a given safe value. Demand for water will be met if and only if there is water in the main reservoir. A cascade or tree of other downstream reservoirs may be supplied from the main reservoir. Each cascade or tree reservoir may supply several others but only takes supply from one reservoir.

3 STATISTICAL ASSUMPTIONS

- Plant failures (index b) occur randomly, with equal probability at any time of the day or night. The overall failure rate (F_b per year) for any given type of failure is constant.
- Different types of interruptions occur independently.
- Plant failures are short enough so that they never overlap or occur simultaneously.
- Each type of failure causes a constant loss of capacity but is of variable duration.
- Stochastic diurnal demand profiles are adequately modelled by iteration over a number of simpler demand profiles (index d), each with a specified probability.
- The calculations do not depend on assumptions about the form of the probability distribution of durations, except that durations should be positive, and delays may need to be modelled.

4 CALCULATIONS

Let there be a breakdown type b at time t while demand follows profile D_{0d}. Plant failure on a specified arc reduces the capacity of that arc and network supply capacity S_b during the breakdown is calculated from the local capacity loss using the Ford Fulkerson algorithm.

Initial volume at the time of the plant failure is based on a standard profile $W(t)$

$$V_{bd}(t,0) = W(t) \tag{1}$$

Note the distinction between the routine profile by time of day and the post failure profile $V_{bd}(t,n)$ which is relative to t, the time of failure. The time of day at the start of interval n is, $t + n\Delta$, so mass balance calculations give the subsequent reservoir profile (ignoring bounds) as

$$\boxed{U_{bd}(t,n) = V_{bd}(t,n-1) + \Delta t \left\{ \begin{array}{l} S_b + \sum_{i=1}^{N_I} I_i(n) - \sum_{c=1}^{N_R} R_{0c}(n) \\ -L_b(n) - D_{0d}(t+n\Delta t) \end{array} \right\}} \tag{2}$$

This heuristic mass balance calculation assumes that in order to maintain supply, inflow to the reservoir will be increased to S_b as soon as the failure occurs, and available infusions I_i will be opened. $L_b(n)$ represents water loss due to a burst or fire and D_{0d} is a demand profile. Values for onward pumping $R_{0c}(n)$ are obtained from another heuristic, described later. If the reservoir reaches its physical bounds (Vmax and 0) it is assumed that either demand is not met or supply is adjusted.

$$V_{bd}(t,n) = \mathrm{Max}\left(\mathrm{Min}\left(U_{bd}(t,n), Vmax\right), 0\right) \tag{3}$$

4.1 *Reservoir profile and problem duration for a particular demand and breakdown*

Basic statistics of customer problems can be calculated from the post failure profiles $V_{bd}(t,n)$. We first find K, the cumulative duration of unsatisfactory main reservoir levels. We assume that customers will have problems unless $V_{bd}(t, n) > 0$.

$$K_{bd}(t,0) = 0 \tag{4}$$

$$\boxed{\begin{array}{ll} \text{if} & V_{bd}(t,n) > 0 \\ \text{then} & K_{bd}(t,n) = K_{bd}(t,n-1) \\ \text{else} & K_{bd}(t,n) = K_{bd}(t,n-1) + \Delta t \end{array}} \tag{5}$$

This calculation can readily be extended to assume that there will be customer problems if reservoir volume is less than a given safe level.

4.2 *Mean duration of customer problems*

Calculations in the previous section have been for a specific t, n, b and d. We must now consider calculations which are conditional only on a particular b, and which therefore average over t, n and d.

Let $\Pr(n/b)$ = the probability that the duration of failure b will be $n\Delta t$ seconds, $\Pr(d)$ = the probability specified for demand profile d, μ_b = mean duration of customer problems and F_b = frequency (per year) for breakdowns of type b. Conditional expected problem durations μ_b are calculated from $K_{bd}(t, n)$ multiplied by the demand and duration probabilities. If breakdowns are equally probable at any time of day

$$\boxed{\mu_b = \frac{1}{N_T} \sum_{t \in H} \left(\sum_{d=1}^{N_D} \Pr(d) \sum_{n=0}^{\infty} \Pr(n \mid b) K_{bd}(t,n) \right)} \tag{6}$$

The mean consumer problem duration per year is

$$\mu = \sum_{b=1}^{N_B} F_b \mu_b \tag{7}$$

μ depends on the routine profile $W(t)$ via equation (1). It is desirable to be able to assess the implications of variations in reservoir size on the assumption that the routine profile *relative to top level* will be the same. To this end one may specify a relative profile $\tilde{W}$ and calculate an absolute profile W as a function of Vmax. If H is a set of evenly spaced times

$$\forall \; Vmax \geq \underset{t \in H}{Max}\left[\tilde{W}(t)\right] : \; W(t) = Vmax + \tilde{W}(t) - \underset{t \in H}{Max}\left[\tilde{W}(t)\right] \tag{8}$$

It is then possible to calculate mean problem duration as a function of Vmax.

4.3 *Security: Probability that there will be m customer problems*

Each type of failure occurs at random. For each t, b and d the consumer problem duration K is zero until a certain failure duration is reached. If the probability of demand profile d is Pr (d) the mean number of customer problems per year λ_b due to breakdown type b is thus

$$\begin{aligned} \lambda_b &= F_b \Pr\left[K_b \neq 0\right] \\ &= F_b \left(1 - \frac{1}{N_T} \sum_{t \in H} \left(\sum_{d=1}^{N_D} \Pr(d) \sum_{n \mid K_{bd}(t,n)=0} \Pr(n \mid b) \right)\right) \end{aligned} \tag{9}$$

Since failure durations are short and we assume no overlap then numbers of problems due to each type of failure will have a Poisson distribution:

$$\Pr(m \text{ customer problems in a year}) = \frac{\left(\sum_{b=1}^{N_B} \lambda_b\right)^m \exp\left(-\sum_{b=1}^{N_B} \lambda_b\right)}{m!} \tag{10}$$

4.4 *Probability distribution of customer problem duration*

For breakdown b and routine reservoir profile W we can calculate the problem duration distribution for a single failure. Let Tc be the duration of customer problems. The conditional probability of any given Tc given a single failure of type b is

$$\Pr(Tc \mid b,1) = \frac{1}{N_T} \sum_{t \in H} \left(\sum_{d=1}^{N_D} \Pr(d) \sum_{n \mid K_{bd}(t,n)=Tc} \Pr(n \mid b) \right) \tag{11}$$

The overall probability distribution of problem duration for any other number of failures is obtained by a series of convolutions

$$\Pr(Tc \mid b,m) = \sum_{x=0}^{Tc} \Pr(Tc - x \mid b, m-1) \Pr(x \mid b,1) \tag{12}$$

The distribution of customer problem duration for all possible numbers of failures type b is then

$$\Pr(Tc \mid b) = \sum_{m=0}^{\infty} \Pr(Tc \mid b,m) \frac{(F_b)^m}{m!} e^{-F_b} \tag{13}$$

The overall duration distribution can be calculated from a series of convolutions of customer problem distributions due to these different types of failures.

$$\Pr\left(Tc \in \{1..n\} \mid b\right) = \sum_{x=0}^{T_C} \Pr\left(Tc - x \in \{1..n-1\} \mid b\right) \Pr\left(\left[x = n\right] \mid b\right) \tag{14}$$

4.5 *Constant availability or security control curves*

A conditional mean problem duration can be calculated assuming that the breakdown happens at time step t and the reservoir volume at the start of the breakdown is w.

$$\mu\left(w,t\right) = \left\{ \sum_{b=1}^{N_B} F_b \sum_{d=1}^{N_D} \Pr\left(d\right) \sum_{n=0}^{\infty} \Pr\left(n \mid b\right) K_{bd}\left(t,n\right) \right\} \mid \left[\mathrm{W}\left(t\right) = w \right] \tag{15}$$

This relationship makes it possible to derive a profile $W(t)$ such that expected problem duration is constant throughout the day, by taking each value of $t \in H$ and searching the interval [Vmax, 0] for a suitable value of w, using a bracketing algorithm based on the "regula falsi" method. A constant probability profile can be produced similarly.

4.6 *Reservoir tree heuristics*

The simplest way of representing a multiple or cascading reservoir situation is to lump the operating capacities together, and similarly to amalgamate the local demands. If the reservoirs are arranged in a tree structure so that each reservoir can supply several further reservoirs, but may only take a variable supply from one the mass balance heuristic can be extended. Calculation begins at the end of the tree's branches (from which there is no onward pumping: $R_{cr}(n) = 0$). It is assumed that pumping to any tree reservoir is reduced to a set minimum as soon as there is a failure, but only until that reservoir falls to its safe level. After calculating the profile and supply profile for one reservoir all its supplying reservoirs are considered and so on in turn. By the end of this chain of calculations $R_{oc}(n)$ is known for all tree reservoirs c. Mean demands are used, so these calculations need only be done once for an entire tree.

5 ILLUSTRATION OF THE METHOD

A hypothetical example was constructed to demonstrate the capability of the method. This had three tree reservoirs, two sources, 5 levels of stochastic demands and 7 different types of failures. A diagram of the example network is shown in Figure 1. A graph of μ vs Vmax, two constant security profiles and many other graphs were obtained in only 17 seconds using a 1 GHz PC.

Failure rates, associated network capacities for the various breakdowns, and the demand profiles are in table 1.

The resulting annual security and availability as a function of reservoir volume is shown in figure 2. Service loss distributions are shown in figure 3. The jagged shape of one histogram is not statistical noise; it reflects the effect of diurnal demand and volume profiles when a reservoir regularly varies between nearly full and nearly empty. Figure 4 shows an evaluation of some design options. Reducing delays in opening infusions to nearly nil has a similar effect to arranging for a larger infusion (55 l/s instead of 35 l/s) with the same delay as the base case (4 hours). Installing a twin supply pipeline with the same total capacity has a much greater effect on security even through the number of bursts is increased. This is because with the given figures bursts on pipe 1 are the dominant source of problems.

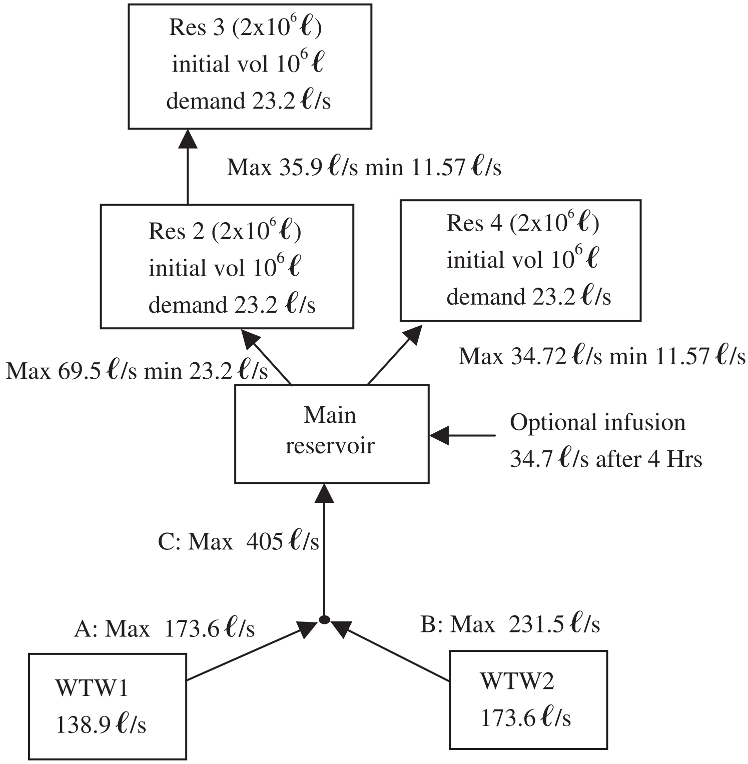

Figure 1. Example network.

Table 1. Failure characteristics.

Failure	Nb	Distribution (mean, SD etc)	ARC	Capacity l/s Loss	Net
Fire	0.1	Uniform (4, 20) 139 l/s	–	0	313
Pump 1	0.1	Uniform (5,15)	A	174	174
Pump 21	0.1	Gamma (9,5)	A	116	232
Source 1	0.1	Cens. exp (min 1 SD 10)	A	139	174
Pump station	0.1	Truncated normal (20,15)	B	232	139
Transformer	0.1	Lognormal (20,9)	C	197	208
Pipe 1 burst	0.1	Triangle (4,20,30), 35 l/s for 4 hrs	C	405	0

Other investigations could be carried out rapidly, for instance to assess the possibility of supplying remote demands directly from the main reservoir without using the "tree" of smaller local reservoirs, or the value of extra reliability due to increased pump maintenance.

6 PUMP SCHEDULE OPTIMISATION SUBJECT TO SECURITY CONSTRAINTS

The mean service loss function $\mu(w, t)$ is shown in figure 5. Horizontal contours provide reservoir profiles with constant security at different times of day. Such profiles could replace the usual minimum constraint on reservoir levels, which is a proxy for reservoir security constraints. Mean service

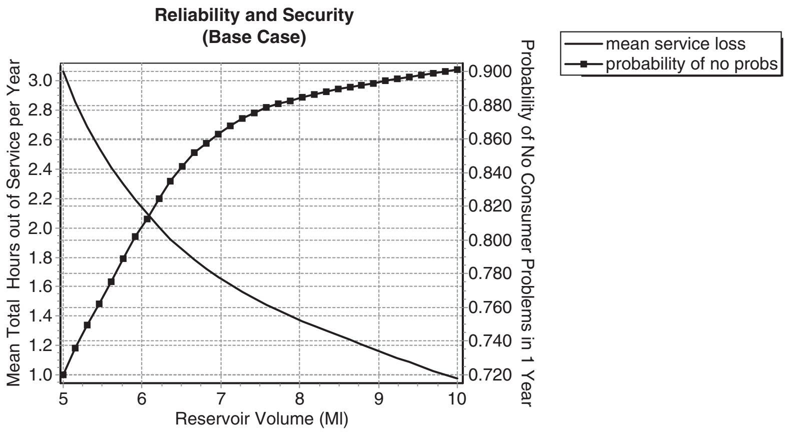

Figure 2. Estimated annual security (base case).

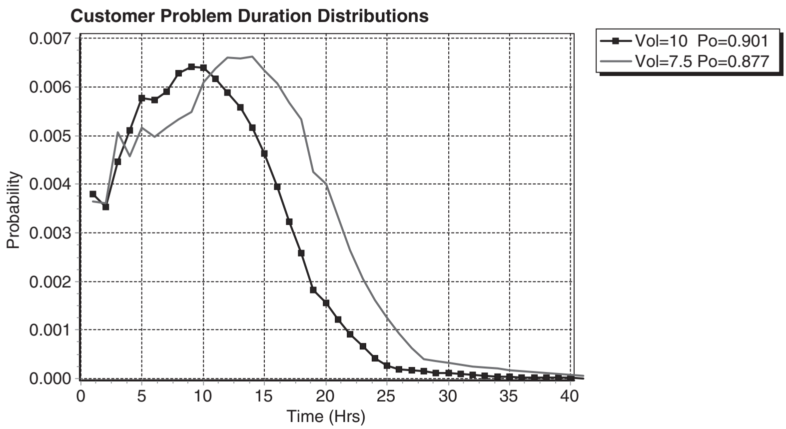

Figure 3. Distribution of service loss durations.

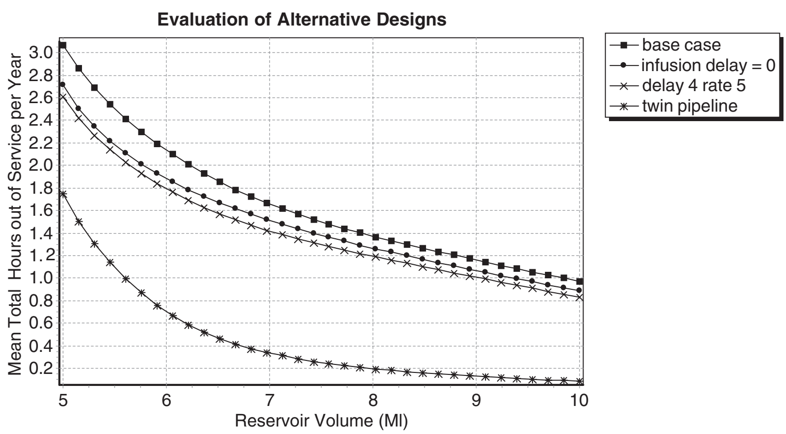

Figure 4. Comparison of operational response options.

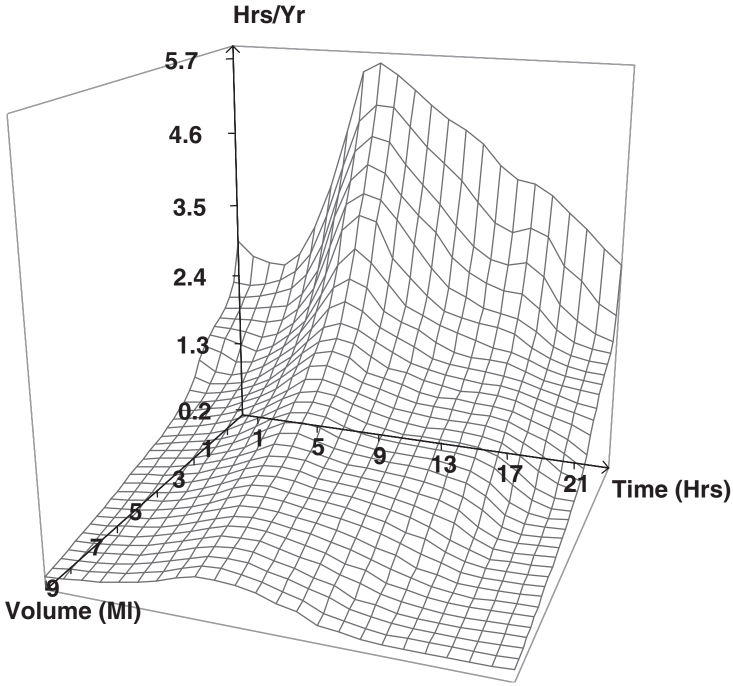

Figure 5. Mean service loss function $\mu(w, t)$.

loss functions can also be considered directly when pump scheduling methods are based on some form of non-linear optimisation.

7 MONTE CARLO SIMULATION

In order to check the method described in this paper a set of Monte Carlo studies was performed based on the same hypothetical example. The Monte Carlo study simulated 2000 years of operation of the supply system. Each of the failure periods was scheduled optimally. The process was repeated a number of times using the same demand sequence but different starting conditions and reservoir volumes. A comparison of results from Monte Carlo simulation and from the fast method described in this paper is shown in Figure 6. The security estimates were usually within 5% of one another and never more than 8% different.

8 DISCUSSION

The assumption that failures never overlap is a good approximation when failure rates are low and durations are relatively short, which should be the case in normal operation. The assumption that demands will be fully met if and only if there is water in the reservoir is fair if the distribution network is designed in a typical, conservative fashion.

Some pump stations have large numbers of pumps, or stand-by pumps. In such cases failures of groups of pumps should be considered together.

To be able to maintain a standard reservoir profile, pump capacity must be rather greater than peak day demand. The method would need to be modified if that is not the case.

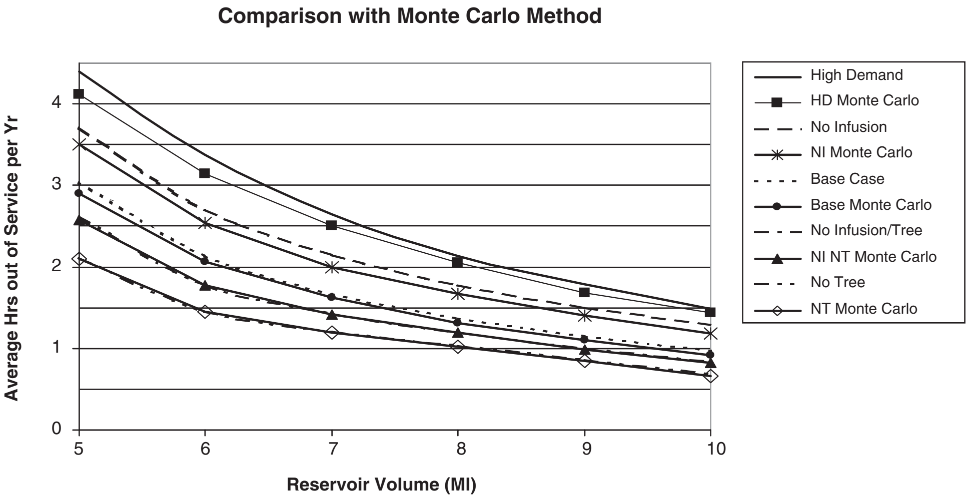

Figure 6. Comparison with Monte Carlo study.

9 CONCLUSIONS

A methodology has been developed to assess the security and availability of a bulk water supply system with storage, and procedures to determine time-varying minimum reservoir levels have been indicated. The failure patterns of supply components are estimated by the system operator, who also describes demand patterns and an outline of the network. Given the reservoir size and a typical diurnal profile customer security is then calculated in terms of mean total customer problem hours per year or the probability of no problems per year. A heuristic can be used to deal with multiple reservoir cases.

The method is very fast, and simple to use. A wide range of studies relating risk and engineering or management options can be based on it. Time-varying minimum storage required in the service reservoir can also be derived. A Monte Carlo study has verified the method.

REFERENCES

Germanopoulos G, Jowitt P.W. & Lumbers J.P. 1986, "Assessing the Reliability of Supply and Level of Service for Water Distribution Systems", *Proc. ICE*, 80, 413–428.

Nel D. & Haarhof J. 1996, "Sizing Municipal Water Tanks with Monte Carlo Simulation", *Aqua* (45) 203–212.

Twort A.C., Ratnayaka D.D. & Brandt M.J. 2000, "*Water Supply*", 5th ed, Edward Arnold, London.

NOTATION

b	index for types of breakdowns.
c, r	index for reservoirs. $r = 0$ indicates the main reservoir.
d	index for demand profiles.
m	no. of customer problems.
n	number of intervals (of length Δt seconds) since a breakdown began.
i	index for infusions.
t	time of day at which a breakdown began (s).
$D_{0d}(t)$	profile of demand on the main reservoir (l/s).

$\bar{D}_r$	mean demand on reservoir r. ($\bar{D}_0$ refers to main reservoir).
F_b	frequency (per year) for breakdowns of type b.
H	a set of discrete times evenly spaced throughout the whole day.
$I_i\,(n)$	profile of infusion i following a breakdown (l/s). (takes account of delays in opening).
$K_{bd}\ (t,n)$	duration of customer problems if the breakdown at time t ends at time $t + n\Delta t$.
$L_b\,(n)$	profile of loss of water (l/s) during a breakdown of type b. Non-zero for bursts.
N_B	number of types of breakdowns.
N_D	number of demand profiles.
N_I	number of available infusions.
N_R	number of cascade or tree reservoirs.
N_T	number of times of day at which failures may take place (=size of H).
$R_{0c}\,(n)$	rate of pumping from the main reservoir to reservoir c following a breakdown.
S_b	network capacity (l/s) throughout a breakdown of type b.
Tc	duration of customer problems.
$U_{bd}(t, n)$	volume implied by mass balance without taking reservoir bounds into consideration.
$U_{bd}(t, n)$	main reservoir profile, following breakdown type b at time t with demand profile d.
$V_{\max}$	maximum possible volume in main reservoir.
$W(t)$	a standard reservoir volume profile in normal conditions.
$S_{\max}$	maximum network capacity.
Δt	duration of an interval used for calculation (s).
λb	mean number of customer problems per year due to breakdown type b.
μ	expected problem duration.

Simulation of mixing in axisymmetric service reservoirs with denser inflow

O. Nordblom
Dept. of Water Environment Transport, Chalmers University of Technology, Gothenburg, Sweden

ABSTRACT: The influence of denser inflow on the transient mixing process in cylindrical service reservoirs was investigated using computational fluid dynamics (CFD). The inlet pipe was directed upwards along the symmetry line in the centre of the reservoir, which allowed the flow to be simulated using a two-dimensional axisymmetric model. The standard k–ε model of turbulence was used to calculate turbulent mixing and a deforming mesh technique was used to handle the rising water surface. First, the penetration height of negatively buoyant jets was simulated for different jet densimetric Froude numbers and inlet pipe diameters. The results were found to agree reasonably with experimental data. In the next step, mixing processes were simulated in reservoirs with different height-to-width ratios. From these tests, the critical conditions for mixing were quantified in terms of the inflow conditions, the water depth and the height-to-width ratio of the reservoir.

1 INTRODUCTION

The potential risk of water quality degradation in service reservoirs owing to long residence times is an important issue in drinking water distribution. For safe operation, it is essential that the water reserve is exchanged continuously with new water. Previous investigations of the hydraulics of service reservoirs have found that this requirement is best accomplished using the momentum flux of the turbulent jet formed at the inlet to mix the water (Grayman et al. 2000, Hammer & Marotz 1986). The turbulent jet is suitable for mixing since it causes efficient entrainment of ambient water and creates a circulation in the reservoir.

Mixing processes in service reservoirs are, however, always affected to some extent by buoyancy forces originating from temperature differences between the stored water and the entering water. These differences may arise as a result of heat transfer through tank walls or abrupt shifts in the temperature of the inflowing water. In situations with strong buoyancy flux and weak momentum flux lighter inflows could collect at the water surface whereas denser inflows could sink to the bottom. The phenomenon has been observed both in small-scale physical models (Pantzlaff & Lueptow 1999, Rossman & Grayman 1999) and in large-scale field measurements (Chuo et al. 2001, Grayman et al. 2000). The stable stratification, which develops in these situations, generally leads to some kind of plug flow. Since inlets and outlets are usually located near the bottom, denser inflows could lead to plug flows of the last in-first out (LIFO) type, which reduces the water exchange considerably. Denser inflows are, therefore, of particular interest when investigating mixing processes in service reservoirs.

When the jet fluid is denser than the ambient fluid and the discharge is directed upwards, the jet is called a negatively buoyant jet. In unbounded homogeneous water bodies, negatively buoyant jets create a characteristic fountain-like flow with a maximum height of rise. For fully turbulent conditions and negligible mass at the discharge point, it has been found that the penetration height of the jet is determined entirely by the relative strength of the momentum and buoyancy fluxes of the discharge (Baines et al. 1990, Turner 1966, Zhang & Baddour 1998).

In reservoirs and other confined geometries, the downward flow of a fountain will spread along the bottom and create a layer of dense water, which mixes very slowly with the lighter water above the fountain top (Baines et al. 1990, Pantzlaff & Lueptow 1999). To avoid this situation, which inevitably leads to bad mixing conditions, the initial momentum flux of the jet has to be large enough to generate a circulation region extending from the water surface to the bottom (Pantzlaff & Lueptow 1999). This situation requires that the jet can penetrate to the surface, spread horizontally and reach the sidewalls where the buoyancy force can drive the flow downwards and complete the circulation. No other flow patterns should result in better mixing since in the case of a circulation along the boundaries the water is diluted as much as possible before being recycled back into the jet.

It follows from the discussion above that some momentum must be left in the jet when it hits the water surface in order to prevent the jet from falling back on itself. Further, it is likely that the required excess of momentum depends in some way on the height-to-width ratio of the tank. A mixing criterion of this kind was proposed in the scale-model study by Rossman & Grayman (1999). Their analysis of the experimental data was generally satisfying but did not involve the height-to-width ratio of the tank. Some details about flow patterns and transient characteristics of confined negatively buoyant jets have been reported in other studies (Baines et al. 1990, Pantzlaff & Lueptow 1999), but for the geometric and operational conditions typical of service reservoirs information is still very limited.

The purpose of the present study is to use computational fluid dynamics (CFD) to investigate flow patterns and mixing processes in cylindrical service reservoirs driven by upward negatively buoyant jets and to establish mixing criteria for different height-to-width ratios. The mixing processes under consideration take place during the fill period in reservoirs with a fill-and-draw mode of operation. Since this flow, being three-dimensional and transient, is computationally expensive only a two-dimensional axisymmetric case will be studied. The numerical results are compared with reference data of penetration heights and mixing criteria reported in the literature.

2 DESCRIPTION OF THE FLOW PROBLEM

2.1 *Geometry*

The reservoir is a standing cylinder with a vertical inlet directed along the symmetry axis in the centre of the tank. The inlet pipe terminates flush with the bottom. The parameters defining the geometry are the water depth, H, the tank diameter, D, and the inlet pipe diameter, d. When the fill period starts, the reservoir is partially filled with water with a constant temperature, T_1. The inflowing water has a constant velocity, U_0, and a constant temperature, T_0, different from T_1. A definition sketch for the flow domain is shown in Figure 1.

2.2 *Dimensional considerations*

As mentioned in the introduction, the penetration height of negatively buoyant jets is the determining factor in the mixing process under consideration. Thus, it is essential that the model can predict the penetration height with sufficient accuracy.

Based on dimensional consistency and experimental evidence it has been found that the penetration height, x_m, of turbulent negatively buoyant jets issuing into unbounded environments scales as $x_m B_0^{1/2}/M_0^{3/4}$, where $M_0 = AU_0^2$ is the specific momentum flux, $B_0 = AU_0 g|\rho_a - \rho_0|/\rho_0$ is the specific buoyancy flux, A = cross sectional area of the discharge, g = gravitational acceleration, ρ_0 = discharge fluid density and ρ_a = ambient fluid density (Turner 1966). The relation is based on the assumption that the mass flux of the discharge is negligible. It is often expressed in terms of the jet densimetric Froude number, $\mathsf{F} = U_0/(gd|\rho_a - \rho_0|/\rho_0)^{1/2}$, and the pipe diameter, d, as

$$\frac{x_m}{\mathsf{F}d} = c_m \tag{1}$$

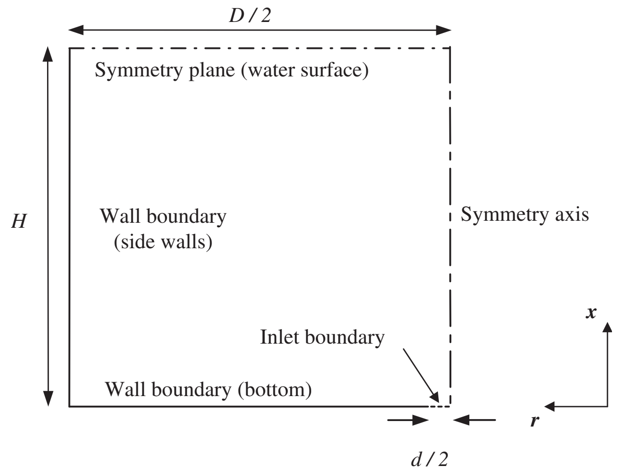

Figure 1. Definition sketch for the flow domain.

where c_m is an empirical constant.

The penetration height in reservoirs should be controlled by F and d in the same way as in (1) since the inflow conditions are fully turbulent and the mass flux is negligible. The latter condition holds for F > 5 (Zhang & Baddour 1998), which is the Froude number range of interest here. The floor of the reservoir could have some influence on the flow near the discharge point and, thereby, on the penetration height. The water depth and the tank diameter should, however, be unimportant as long as the jet is far from the water surface and the sidewalls. Investigation of the penetration height and its dependence on the governing parameters F and d is the first part of this study.

In order to avoid a fountain flow, the momentum flux must be large enough to generate a circulation along the boundaries from the water surface to the bottom. Since this flow pattern requires that the jet can penetrate at least to the surface, a simple mixing criterion could be formulated as $x_m > cH$, where c is some unknown constant. However, it is likely that the height-to-width ratio, H/D, also influences the flow and should be included in the relation. Assuming that the penetration height is proportional to the product of F and d as in (1), the following mixing criterion can be formulated:

$$\frac{\mathsf{F}d}{\mathsf{H}} > f\left(\frac{H}{D}\right) \tag{2}$$

where the constant, c, has been replaced by a function, f, of the height-to-width ratio. The length scale ratio on the left side in (2) expresses the relative strength of the input momentum and buoyancy fluxes in relation to the water depth and was originally proposed in the experimental study by Rossman & Grayman (1999). Analysis of the mixing criterion (2) for different height-to-width ratios forms the second part of the study.

3 MATHEMATICAL MODEL

3.1 *Governing equations*

The governing equations are the Reynolds averaged Navier-Stokes equations with the Boussinesq eddy-viscosity approximation (Wilcox 1998), the continuity equation, the standard equations for turbulent kinetic energy and dissipation rate of the $k-\varepsilon$ model and a species mass fraction equation. The mass fraction, which represents the fraction of dense water originating from the inflow, is used to calculate the temperature and the density. The symmetry of the flow makes it possible to formulate the equations in two dimensions in a cylindrical coordinate system.

If r and x denote the radial and axial directions, respectively, and v and u the corresponding mean velocity components, the continuity equation and the momentum equations read (Panton 1996):

Continuity

$$\frac{\partial \rho}{\partial t} + \frac{1}{r}\frac{\partial(\rho r v)}{\partial r} + \frac{\partial(\rho u)}{\partial x} = 0 \tag{3}$$

Axial momentum

$$\frac{\partial(\rho u)}{\partial t} + \frac{1}{r}\frac{\partial(r\rho v u)}{\partial r} + \frac{\partial(\rho u^2)}{\partial x} = -\frac{\partial p}{\partial x} + \frac{1}{r}\frac{\partial(r\tau_{rx})}{\partial r} + \frac{\partial \tau_{xx}}{\partial x} + \rho g_x \tag{4}$$

Radial momentum

$$\frac{\partial(\rho v)}{\partial t} + \frac{1}{r}\frac{\partial(r\rho v^2)}{\partial r} + \frac{\partial(\rho u v)}{\partial x} = -\frac{\partial p}{\partial r} + \frac{1}{r}\frac{\partial(r\tau_{rr})}{\partial r} + \frac{\partial(\tau_{rx})}{\partial x} \tag{5}$$

where p = pressure; ρ = density; and g_x = gravitational acceleration ($= -9.81\ \mathrm{m/s^2}$).

The viscous stresses in (4) and (5) are defined as (Panton 1996):

$$\tau_{rx} = (\mu + \mu_t)\left(\frac{\partial u}{\partial r} + \frac{\partial v}{\partial x}\right) \tag{6}$$

$$\boxed{\tau_{xx} = (\mu + \mu_t)\left(2\frac{\partial u}{\partial x} - \frac{2}{3}(\nabla \cdot \vec{v})\right)} \tag{7}$$

$$\tau_{rr} = (\mu + \mu_t)\left(2\frac{\partial v}{\partial r} - \frac{2}{3}(\nabla \cdot \vec{v})\right) \tag{8}$$

where $\nabla \cdot \vec{v} = (1/r)\partial(rv)/\partial r + \partial u/\partial x$; μ = molecular viscosity ($= 0.001\ \mathrm{kg/(m,\ s)}$); and μ_t = eddy viscosity.

The set of equations above is closed using the standard equations for turbulent kinetic energy, k, and dissipation rate, ε, of the $k - \varepsilon$ model (Rodi 1993). In the present study the effect of buoyancy on the turbulence, which is sometimes accounted for in the $k - \varepsilon$ model, has been omitted. The turbulence equations read:

Eddy viscosity

$$\mu_t = \rho C_\mu \frac{k^2}{\varepsilon} \tag{9}$$

Turbulent kinetic energy

$$\boxed{\begin{aligned}&\frac{\partial(\rho k)}{\partial t} + \frac{1}{r}\frac{\partial(r\rho v k)}{\partial r} + \frac{\partial(\rho u k)}{\partial x} = \frac{1}{r}\frac{\partial}{\partial r}\left(r(\mu + \frac{\mu_t}{\sigma_k})\frac{\partial k}{\partial r}\right) + \\ &\frac{\partial}{\partial x}\left((\mu + \frac{\mu_t}{\sigma_k})\frac{\partial k}{\partial x}\right) + G_k - \rho\varepsilon\end{aligned}} \tag{10}$$

Dissipation rate

$$\boxed{\begin{aligned}\frac{\partial(\rho\varepsilon)}{\partial t}+\frac{1}{r}\frac{\partial(r\rho v\varepsilon)}{\partial r}+\frac{\partial(\rho u\varepsilon)}{\partial x}=\frac{1}{r}\frac{\partial}{\partial r}\left(r(\mu+\frac{\mu_t}{\sigma_\varepsilon})\frac{\partial\varepsilon}{\partial r}\right)+\\ \frac{\partial}{\partial x}\left((\mu+\frac{\mu_t}{\sigma_\varepsilon})\frac{\partial\varepsilon}{\partial x}\right)+C_{1\varepsilon}\frac{\varepsilon}{k}G_k-C_{2\varepsilon}\rho\frac{\varepsilon^2}{k}\end{aligned}} \tag{11}$$

where the shear production, G_k, takes the form:

$$\boxed{G_k=2\mu_t\left(\left(\frac{\partial u}{\partial x}\right)^2+\left(\frac{\partial v}{\partial r}\right)^2+\frac{1}{2}\left(\frac{\partial v}{\partial x}+\frac{\partial u}{\partial r}\right)^2+\left(\frac{v}{r}\right)^2\right)} \tag{12}$$

The constants in the turbulence model take the following standard values (Rodi 1993): $C_\mu = 0.09$, $\sigma_k = 1.00$, $\sigma_\varepsilon = 1.30$, $C_{1\varepsilon} = 1.44$ and $C_{2\varepsilon} = 1.92$.

The species mass fraction, ϕ, is obtained from the following transport equation:

$$\frac{\partial(\rho\phi)}{\partial t}+\frac{1}{r}\frac{\partial(r\rho v\phi)}{\partial r}+\frac{\partial(\rho u\phi)}{\partial x}=\frac{1}{r}\frac{\partial}{\partial r}\left(r(\Gamma_\phi+\Gamma_{\phi,t})\frac{\partial\phi}{\partial r}\right)+\frac{\partial}{\partial x}\left((\Gamma_\phi+\Gamma_{\phi,t})\frac{\partial\phi}{\partial x}\right) \tag{13}$$

where Γ_ϕ and $\Gamma_{\phi,t}$ are the molecular and turbulent mass diffusion coefficients, respectively. Γ_ϕ is set to 10^{-6}kg/(m, s), which is a typical value for various chemicals dissolved in water (Thibodeaux 1996) and should be appropriate also when calculating the spreading of water in a different water mass. $\Gamma_{\phi,t}$ is related to the turbulent viscosity according to Reynolds analogy between mass and momentum transport (Rodi 1993): $\Gamma_{\phi,t} = \mu_t/Sc_t$, where Sc_t is the turbulent Schmidt number. The value of Sc_t is set to 0.7, which is supported by measurements of spreading rates of mean velocity and tracers in round jets (Panchapakesan & Lumley 1993) and plane jets (Pope 2000).

Based on the mass fraction, which represents the fraction of water originating from the inflow, the temperature T is calculated from the relation:

$$T = T_1 + \phi(T_0 - T_1) \tag{14}$$

where T_0 = temperature of the inflowing water; and T_1 = temperature of the water that is stored in the reservoir initially.

The density, which appears in all transport equations above, is calculated from the temperature (in degrees Celsius) using the standard equation of state (UNESCO, 1981):

$$\rho(T) = a_0 + a_1T + a_2T^2 + a_3T^3 + a_4T^4 + a_5T^5 \tag{15}$$

where $a_0 = 999.842594$; $a_1 = 6.793952\cdot10^{-2}$; $a_2 = -9.095290\cdot10^{-3}$; $a_3 = 1.001685\cdot10^{-4}$; $a_4 = -1.120083\cdot10^{-6}$; and $a_5 = 6.536332\cdot10^{-9}$.

3.2 *Boundary and initial conditions*

The wall boundary condition is based on the standard wall function approach by Launder & Spalding (1974), which includes boundary conditions in the momentum equation and the turbulence equations. In the mass fraction equation a zero flux condition is prescribed at all walls. The water surface is considered as a symmetry plane with a prescribed constant vertical velocity corresponding to the rate of mass inflow. At the inlet, the species mass fraction is set to one and the axial velocity is given a constant value, U_0. The inlet values of the turbulent kinetic energy and the dissipation rate are set as follows: k is calculated from the turbulence intensity, $I = (2k/3)^{1/2}/U_0$, assuming I is 1%, and ε is calculated from the definition: $\varepsilon = C_\mu^{3/4} k^{3/2}/l$, where the mixing length, l, is set to one-tenth of the pipe diameter according to Rodi (1993). With these estimates of k and ε the turbulent to laminar viscosity ratio, μ_t/μ, at the inlet will be of order 10^2.

The simulations start with zero velocities and zero species mass fraction, i.e. the initial temperature is constant and equal to T_1. The initial value of ε is estimated from the average energy dissipation rate: power input, $P = \rho\pi d^2 U_0^3/8$, divided by the initial water mass. The corresponding initial value of k is determined from (9) assuming a turbulent to laminar viscosity ratio, μ_t/μ of order 10^1.

The sensitivity of the results to the inlet values of k and ε can be expected to be weak since most kinetic energy and dissipation rate is generated in the shear layer of the turbulent jet. For the same reason, the specific choice of initial values of k and ε is not very important – the turbulence produced by the jet will soon rule out the initial turbulence.

3.3 *Details of calculations*

The equations were solved using the CFD code *Fluent* (Fluent 2003), which is based on the finite volume method (Versteeg & Malalasekera 1995). A second-order upwind differencing scheme and a first-order implicit time integration scheme was applied to all flow variables. The PISO algorithm (Versteeg & Malalasekera 1995) was used in the pressure-velocity coupling. The computational mesh consisted of orthogonal hexahedral cells. The typical number of cells was about 8000 with higher resolution near the symmetry axis and the wall boundaries. The time step was set to a constant value in the range of 0.05 to 0.15 s and was always smaller than one third of the smallest time needed to pass through any grid cell in the mesh. Normally, 3 to 5 iterations per time step was needed to reach convergence on each step. The results were tested for grid independence by repeating a few of the simulations on finer meshes.

The rising water surface was handled using a deforming mesh technique combined with dynamic layering in the uppermost cell layer. During the transient simulations the cells in the uppermost layer were allowed to expand at a constant rate corresponding to the rate of mass inflow. When the initial cell height, h, had increased to $1.4\,h$, the uppermost layer was divided into one layer of height h and one layer of height $0.4\,h$ on which the cell expansion proceeded in the next step. Further details about the deforming mesh module are given in the program manual (Fluent 2003).

4 RESULTS

4.1 *Penetration height*

To investigate the dependence of the penetration height on the governing parameters described in Section 2 numerical simulations were done for inflow conditions typical of service reservoirs. The penetration height, x_m, is defined as the distance along the symmetry axis from the discharge point at the bottom of the reservoir to the point of zero axial velocity.

Figure 2 shows the variation of x_m with time in a typical case. The development of the flow field is characterized by the formation of a front that moves upwards until the opposing buoyancy force has reduced the initial momentum flux to zero. After that, the heavy fluid falls back around the jet towards the bottom. When the downward flow has reached the bottom it spreads radially, hits the sidewalls and forms a layer of dense water at the bottom. As seen in Figure 2, the penetration height falls rapidly from the initial maximal value of the first front (15 m) to a lower, almost steady, value (9.5 m). The decrease is a result of the downward flow of the fountain, which slows down the upward jet. The simulated steady-state penetration height was generally about 60% of the maximum penetration, which is comparable with the experimental value: 70% (Pantzlaff & Lueptow 1999, Turner 1966).

Table 1 shows the variation of the steady-state penetration height with F and d for some values of the inlet velocity, U_0, inlet temperature, T_0, and initial temperature, T_1. It is seen in the last column

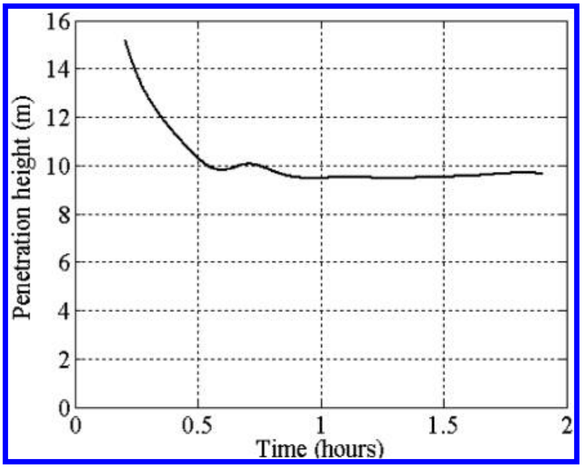

Figure 2. Variation of penetration height with time.

Table 1. Simulated steady-state penetration heights.

d (m)	U_0 (m/s)	T_1 (°C)	T_0 (°C)	F –	x_m (m)	$x_m/(\mathrm{F}d)$ –
0.40	0.25	15.0	14.0	10.5	6.1	1.46
0.40	0.20	15.0	14.5	11.7	6.8	1.44
0.30	0.25	15.0	14.0	12.1	5.1	1.40
0.50	0.20	10.0	9.5	13.9	9.5	1.37
0.30	0.40	15.0	13.0	14.0	5.8	1.38
0.40	0.35	15.0	14.5	20.5	11.0	1.34
0.50	0.40	15.0	14.5	21.0	14.0	1.33
0.30	0.80	15.0	14.0	38.7	14.1	1.21

that the penetration height scales with F and d as in (1) if the relatively small decrease of c_m from 1.46 to 1.21 with increasing Froude numbers is ignored. According to the original measurements by Turner, c_m takes the value of 1.74. However, the constant is uncertain and values ranging from 1.5 (Pantzlaff & Lueptow 1999) to 2.16 (Zhang & Baddour 1998) have also been reported.

4.2 *Mixing criterion*

The critical value of the function, f, in (2) was investigated in three reservoirs with the same initial water depth, $H_1 = 10\,\mathrm{m}$, but different diameters ranging from 10 m to 40 m. The inflow conditions represented by F and d were chosen so that the desired circulation along the boundaries developed soon after the start of the fill period. The simulations continued until the circulating flow pattern started to transform into a fountain, which marks the critical condition. By keeping track of the velocity field just below the rising water surface during the transient simulations, changes in the flow pattern were easy to detect.

Figures 3a–3b show the change in the flow pattern, marked by short path lines, in a typical case. First, the circulation generated by the jet extends over the whole width of the tank (Fig. 3a). After some time, the horizontal flow just below the water surface turns down before reaching the sidewalls due to the force of gravity (Fig. 3b). At this stage, the mixing capacity of the jet has been reduced since some flow is recycled directly back into the jet. With increasing simulation time, the point at the water surface where the flow turns down gradually approaches the jet axis until the jet can no longer penetrate to the surface.

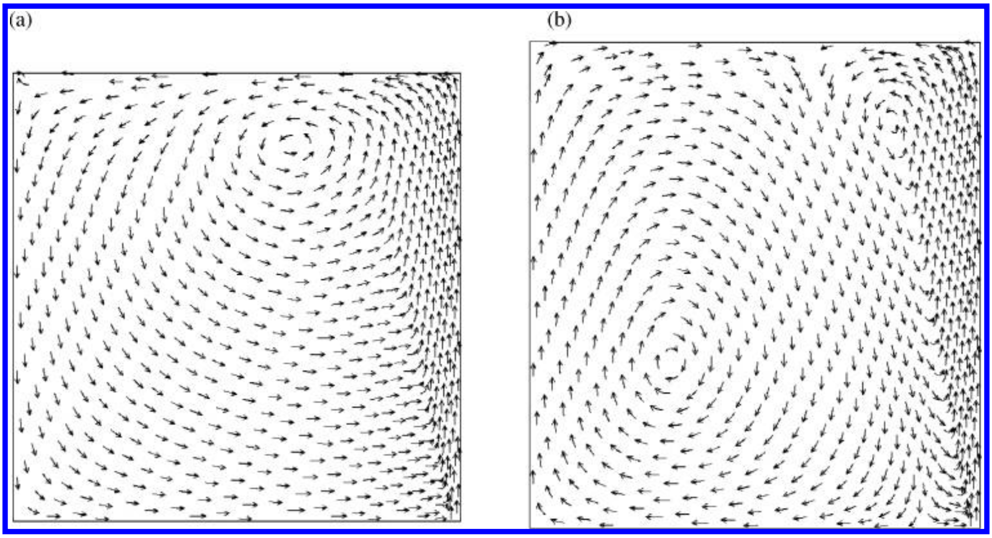

Figure 3. Flow pattern before (a) and after (b) the change from a circulating flow to a fountain flow.

Table 2. Critical values of the ratio Fd/H for three different height-to-width ratios.

D (m)	H_1 (m)	H_1/D –	d (m)	U_0 (m/s)	T_1 (°C)	T_0 (°C)	F_1 –	F_1d/H_1 –	H_2 (m)	F_2d/H_2 –
10.0	10.0	1.00	0.40	0.27	15.0	14.5	15.8	0.631	10.2	0.628
10.0	10.0	1.00	0.40	0.28	15.0	14.5	16.4	0.658	20.0	0.467
20.0	10.0	0.50	0.40	0.40	15.0	14.5	23.5	0.939	10.1	0.934
20.0	10.0	0.50	0.40	0.42	15.0	14.5	24.7	0.986	20.0	0.701
40.0	10.0	0.25	0.40	0.75	15.0	14.5	44.0	1.761	10.1	1.753
40.0	10.0	0.25	0.40	0.85	15.0	14.5	49.9	1.996	20.0	1.419

Table 2 lists a subset of all simulations. The table shows the inflow conditions, the initial water depth, H_1, and the initial length scale ratio, F_1d/H_1. The parameters d, T_0 and T_1 were fixed whereas U_0 was varied. The outcomes of the simulations are presented as the water depth, H_2, and ratio, F_2d/H_2, at which the flow pattern starts to change from a circulation to a fountain. If the flow pattern did not change before the water depth was doubled, the simulation was stopped and the mixing criterion was considered fulfilled. In this case, the table shows the final water depth, $H_2 = 20$ m, and the corresponding value of F_2d/H_2. In the calculation of F_2d/H_2, the Froude number was based on the difference between the density of the inflow and the average density in the reservoir. This practice is appropriate since the density of the water surrounding the jet is approximately equal to the average density before the flow pattern has started to change. The value of Fd/H, computed in this way, always decreases during the fill period because the water depth increases faster than the Froude number.

As seen in Table 2, if F_1d/H_1 is less than a certain critical value the flow starts to change almost directly after the beginning of the fill period. However, if F_1d/H_1 only just exceeds the critical value the circulation proceeds at least until the water depth has doubled. Thus, within a narrow interval the outcome of the mixing process changes completely and once the circulation has been established it proceeds although Fd/H falls below the initial value. With reference to the mixing criterion (2), Table 2 shows that f can be bound to the interval [0.63, 0.66] for $H_1/D = 1.0$, [0.94, 0.99] for $H_1/D = 0.5$ and [1.76, 2.00] for $H_1/D = 0.25$.

5 CONCLUSIONS

The important role of the penetration height in the present mixing problem motivated a preliminary study of this parameter. The linear relation between the penetration height and the product of F and d, which follows from a dimensional analysis, was tested for velocities and temperatures typical of service reservoirs. The constant of proportionality was found to decrease somewhat with increasing Froude numbers and was generally less than the experimental values reported in the literature. The discrepancy may partly be attributed to different methods of defining the terminal height of rise. Measurements of penetration heights usually rely on visual methods whereas the numerical model uses the more accurate definition as the point of zero axial velocity. Taking into account the significant variations in the reported values of the constant, the numerical results are considered reasonable.

Based on the penetration height a mixing criterion, which includes the length scale ratio, Fd/H, and the height-to-width ratio, H/D, was formulated. From a number of simulations the critical conditions for mixing, expressed in terms of Fd/H, were established for height-to-width ratios in the range of 0.25 to 1.0. It was found that the critical length scale ratio increases with decreasing height-to-width ratios, which was expected since a higher momentum flux is required to overcome the force of gravity acting on the radial flow in a wide tank. Another interesting result was that length scale ratios just below the critical value resulted in a break down of the large circulation region almost immediately after the start of the fill period. On the other hand, if the critical value was exceeded at the beginning, the circulation continued at least until the water depth was doubled, although this implied that Fd/H fell below its initial value. This result was unexpected but is most likely caused by the favourable change of the height-to-width ratio that takes place during the fill period. For example, in the widest tank where $H/D = 0.25$, Fd/H decreases from 2.0 to 1.4 as the water depth is doubled. However, 1.4 is still well above the critical value corresponding to $H/D = 0.5$. Although not tested, the circulation would probably remain if the depth was doubled again. Then, Fd/H would decrease to 1.0, which is still above the critical value corresponding to $H/D = 1.0$. Thus, the effect of the decreasing length scale ratio during the fill period seems to be compensated by the increasing height-to-width ratio.

The results can be compared with the scale-model experiments by Rossman and Grayman (1999). Using tracer methods, they determined the critical value of Fd/H to 0.8 in the axisymmetric case for height-to-width ratios in the range of 0.3 to 0.5. The corresponding value predicted by the numerical model should be larger than 1. The discrepancy may be a consequence of the underestimation of the penetration height described above or a result of the different methods used to evaluate the mixing conditions. Additional experimental data are required for a more detailed validation of the model.

The results obtained in this study can serve as reference values for evaluation of mixing conditions in reservoirs with a similar geometry and a vertical inlet at the bottom. To this end the temperature difference between the stored water and the entering water has to be estimated or measured and the inlet velocity must be known. If it turns out that the mixing criterion is not fulfilled, the critical length scale ratio corresponding to the actual height-to-width ratio provides information about the required increase of the inlet velocity.

The most important result is the evidence that there exists a critical initial value of Fd/H below which the circulation along the boundaries cannot be maintained during the fill period. Hence, Fd/H can be interpreted as a measure of the capability to drive a circulation along the boundaries and prevent the jet from falling back on itself. Future work should be directed towards analyses of other kind of geometries and inlet locations to find out which parameters are involved in the mixing process and to establish critical conditions for mixing.

REFERENCES

Baines, W. D., Turner, J.S. & Campbell, I. H. (1990). Turbulent fountains in an open chamber. *Journal of Fluid Mechanics* 212: 557–592.

Chuo, P.Y., Ball, J.E. & Fisher, I.H. Thermal Stratification in Drinking Water Service Reservoirs. *6th Conference on Hydraulics in Civil Engineering* 159–167, Hobart, Australia.

Fluent. (2003). *Fluent 6.1 User's Guide.* Fluent Inc., Lebanon.

Grayman, W.M., Rossman, L.A., Clifford, A., Deininger, R.A., Smith, C., Smith, J.F. & Schnipke, R. (2000). *Water Quality Modeling of Distribution System Storage Facilities*. AWWA Research Foundation and American Water Works Association, Denver, CO.

Hammer, D. & Marotz, G. (1986). Improvement of Water Exchange in Drinking-Water Reservoirs by Hydraulic Measures. *Wasserwirtschaft* 76(4): 145–150.

Launder, B.E. & Spalding, D.B. (1974). The Numerical Computation of Turbulent Flows. *Computer Methods in Applied Mechanics and Engineering* 3: 269–289.

Panchapakesan, N.R. & Lumley, J.L. (1993). Turbulence measurements in axisymmetric jets of air and helium. Part 2. Helium jet. *Journal of Fluid Mechanics* 246: 225–247.

Panton, R.L. (1996). *Incompressible flow.* New York: Wiley.

Pantzlaff, L. & Lueptow, R.M. (1999). Transient positively and negatively buoyant turbulent round jets. *Experiments in Fluids* 27: 117–125.

Pope, S.B. (2000). *Turbulent flows.* Cambridge: Cambridge University Press.

Rodi, W. (1993). *Turbulence Models and Their Application in Hydraulics: A state-of-the-art review*. Rotterdam: Balkema.

Rossman, L.A. & Grayman, W.M. (1999). Scale-Model Studies of Mixing in Drinking Water Storage Tanks. *J. Envir. Engrg.* 125(8): 755–761.

Thibodeaux, L.J. (1996). *Environmental Chemodynamics – Movement of Chemicals in Air, Water, and Soil.* New York: Wiley.

Turner, J.S. (1966). Jets and plumes with negative or reversing buoyancy. *Journal of Fluid Mechanics* 26: 779–792.

Versteeg, H.K. & Malalasekera, W. (1995). *An Introduction to Computational Fluid Dynamics: The Finite Volume Method.* Harlow: Longman Scientific & Technical.

Wilcox, D.C. (1998). *Turbulence modeling for CFD.* La Cãnada, Calif.: DCW Industries.

Zhang, H. & Baddour, R.E. (1998). Maximum Penetration of Vertical Round Dense Jets at Small and Large Froude Numbers. *J. Hydr. Engrg.* 124(5): 550–553.

Coordinated control for improving water supply

P.I. Wade & R.A. Bartindale
Parsons Brinckerhoff Ltd

ABSTRACT: Water is a critical resource, and will increasingly become a major political issue, as communities seek to secure their access to reliable and sustainable water resources.

This paper explores the means by which monitoring and control technology can be used to improve the management and performance of water supply networks. By applying relatively simple control techniques in an imaginative way, leakage rates can be reduced dramatically and supply continuity better assured. Both these have a direct impact on water quality, enabling significant reduction of contaminants. Economically, such developments can be justified on the basis of reduced water losses with an associated reduction in production costs.

Co-ordinated control of the water network can enable much better response to supply and demand changes, and better utilisation of key assets, to the benefit of supply and distribution costs. These advantages can be gained through intelligent application of well-understood technology that is available today. The availability of a range suitable communications media can also be used to advantage.

Real time control systems have played an increasing role in successful water projects world-wide. The paper uses two diverse case studies to show how computer and communications technology can be applied to improved water service delivery.

1 INTRODUCTION

1.1 *A scarce resource*

For a large proportion of the world's population, water is a scarce resource. Increasing population and industrialisation place ever more stringent demands on the available supplies. The challenge for governments and water utilities is to explore more cost effective ways of improving the quality and reliability of water supplies to keep pace with growing demand in a sustainable way.

This paper explores the means by which telemetry and control technology can be used to improve the management and performance of water supply networks. Applying simple control techniques in an imaginative way, leakage rates can be reduced dramatically and supply continuity ensured. This can have a direct impact on water quality, enabling a significant reduction of contaminants. Economically, such developments can be justified on the basis of reduced water losses and/or effective reduction in production costs.

1.2 *Water stress*

This paper uses two case studies to explore the ways in which computer and communications technology can be applied to improve water service delivery. The examples have been selected to illustrate applications of technology to alleviate problems of water supply under different degrees of water stress.

Water stress can be considered to be the ratio of water available at a location relative to the local needs.

Table 1. Water stress comparison.

Case	Qatar	UK	Australia
Annual fresh water per capita available in m^3.*	90	1,222	19,198
Annual water per capita abstracted in m^3.	235	1,660**	1,117

* Ref 1, 1995.
** UK Environment Agency (1995 excluding electrical use).

In each application described, the aim has been to obtain more useful water with the use of fewer resources. Although conservation has been the driver for these projects the technology has also delivered the operational benefits of improved leakage management, enhanced water quality, better security of supply, improved incident response and increased asset knowledge.

2 CASE STUDY 1 – QATAR

2.1 *Simulation of leakage control options*

The availability of abundant energy resources has made possible the production of water (by means of desalination) in a water-stressed environment. This availability of water has facilitated rapid economic development such that the water supply and distribution system installed only 25 years ago now needs to be managed in a more sophisticated and sustainable way in order to balance available resource and demand.

As part of the system development strategy, water towers are being progressively replaced by variable speed pumps that achieve the same engineering objectives but in a more flexible way. This flexibility can be enhanced by using a SCADA system to reduce the water losses that are feature of all water networks. Use of SCADA technology permits a much more rapid detection of bursts and other leaks and consequentially better response. Also, it becomes possible to actively control supply pressure to align with actual demand patterns. Both of these measures serve to reduce leakage.

Chart 1 illustrates how the potential for leakage reduction is created by the growth in the water network. The increasing gap between production and consumption represents water that is currently lost through leakage. A proportion of this not inconsiderable volume can be conserved through intelligent use of control technology.

The water network is shown conceptually in Figure 1. Each of the 70 zones within the network (which comprises several leakage control districts) is supplied by a variable speed pump. A spreadsheet-based computer model was used to represent each zone in the network and the parameters of pressure, flow and leakage were analyzed as they changed across the zone and over each day.

The schematization of the model is indicated in Figure 2. The model uses simplifications of the network performance such as that pressure is proportional to the distance moved into the network and that leakage is proportional to pressure at a location.

The results of using the model (Chart 2) show how the time modulated system was an improvement on the designed system that had no modulation. With the un-modulated water tower design, water pressure are high at night throughout the network giving high night-time leakage. Closing the system at night (as per current practice) stops night leakage, but higher pressure must be sustained during the day to make good the demand so losses remain high. The low night pressure has a high cost in water quality as contamination of the water through main depressurization often occurs. The model also estimates diurnal pressure profiles within networks.

SCADA technology provides the flexibility to allow pressure modulation. Pressure feedback from the network permits optimal pressure for the expected demand to be achieved. This reduces losses further without the quality penalty. It has been known for some time that reducing pressure

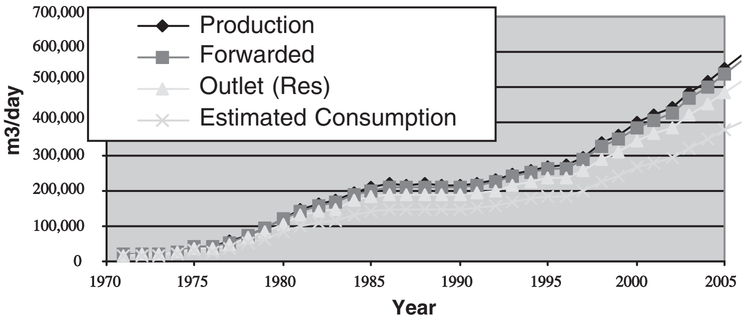

Chart 1. Water production and consumption.

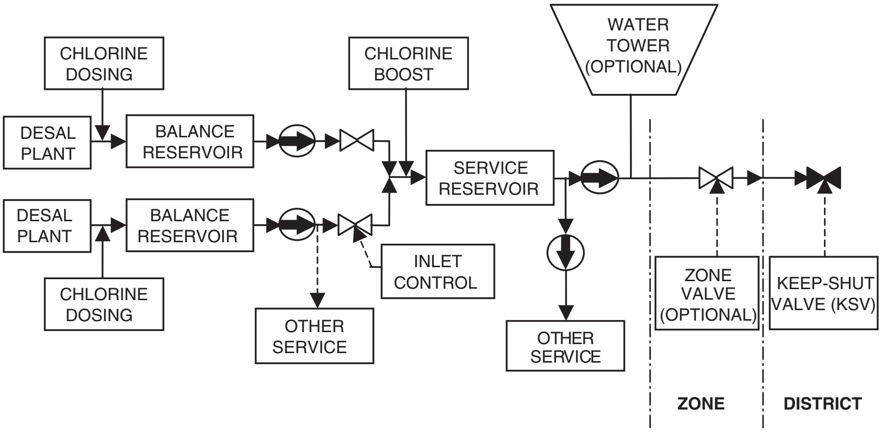

Figure 1. Generic representation of water network.

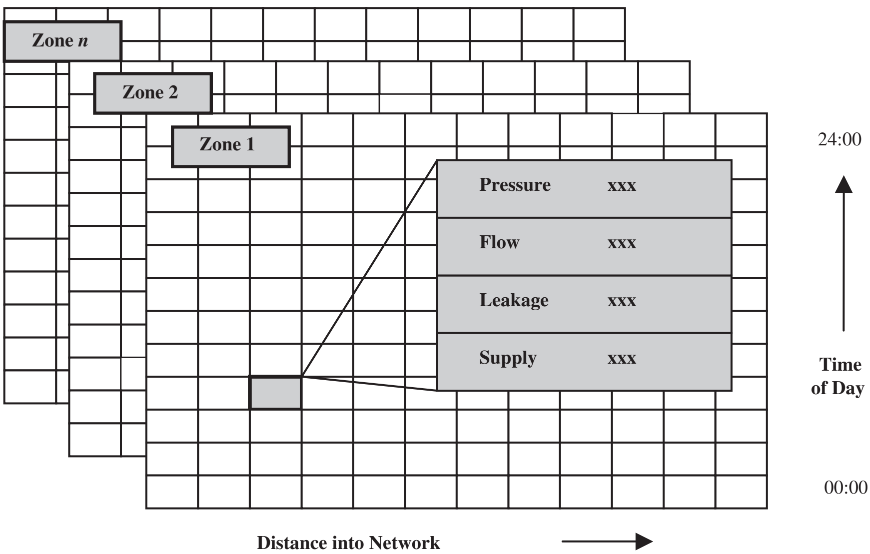

Figure 2. Schematic of computer model.

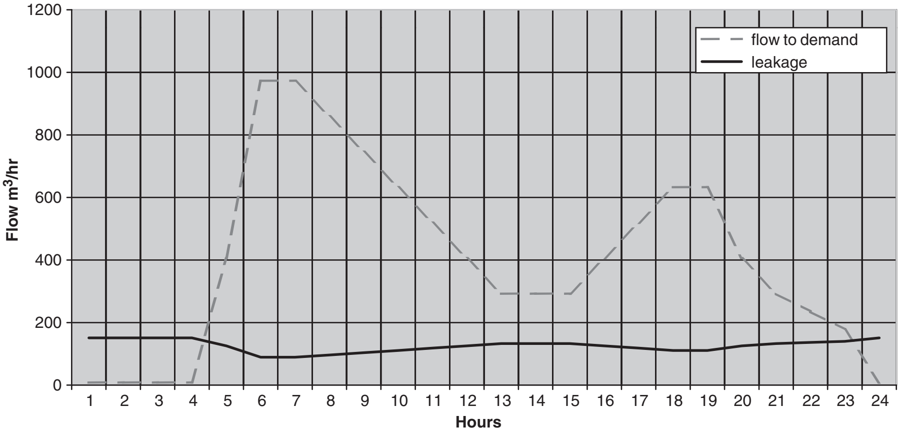

Chart 2. Typical results for a zone with no modulation (full night pressure).

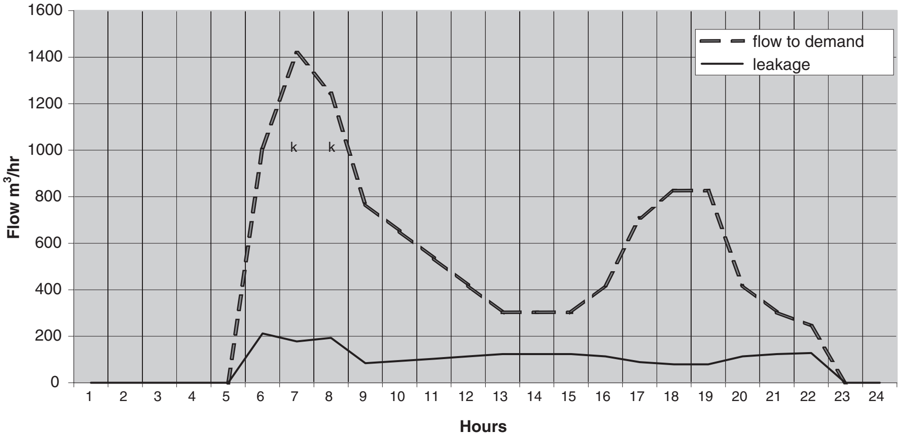

Chart 3. Typical results for a zone with time modulation.

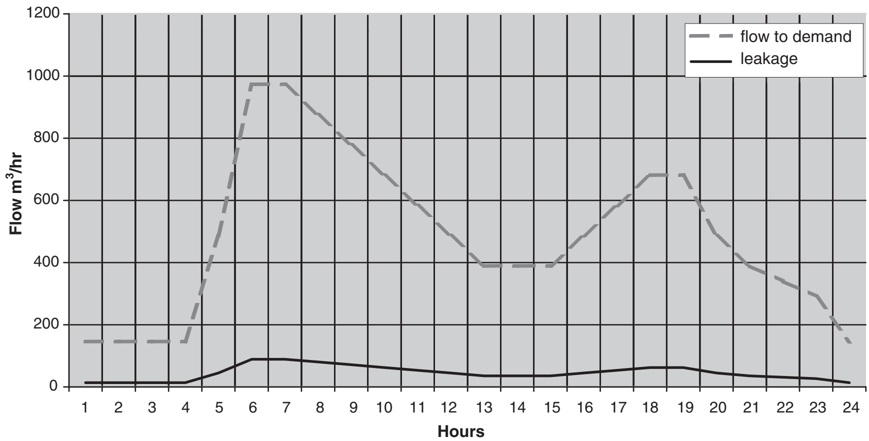

Chart 4. Typical results for a zone with pressure modulation.

reduces leakage but even higher savings can be realized if pressure can be modulated by demand flow. The use of distributed valves to do this has previously proved unreliable and until recently the use of SCADA to do this has been too costly. This case study has shown that advanced SCADA control can now achieve excellent cost to benefit ratios for pressure or flow modulation.

2.2 *Control concept*

A generic control loop for a typical water supply zone is shown in Figure 3. Primary telemetry outstations are located at the water tower sites. These utilize existing UHF radio communications facilities to provide continuous feedback monitoring.

Secondary outstations are located at nodes in each zone that have been determined to be the most critical from a water pressure perspective. Secondary outstations make use of GSM mobile digital communications to provide dial-up links to the telemetry system. Reporting is by exception, on schedule or on demand. The secondary outstations contain calculated diurnal profiles of the expected pressure and/or flow at the node.

When the recorded pressure or flow value deviates from this value by a predetermined amount, the outstation dials out with the exception data. This exception data is used by the telemetry system to adjust the set point of the pressure reducing valve (PRV) at the zonal supply points, and under extreme conditions, the set point of the pump control system. The control system assumes that the system is operating to the expected diurnal pressure profile unless it is notified otherwise. The expected diurnal profile can be updated as required to improve system performance by making use of empirical supply/demand data collected by the telemetry system.

2.3 *Costs and benefits*

The scheme as described demonstrated a highly robust return on investment. The projected capital and revenue costs of the system over a nominal twenty-year cycle, taking into account expected system expansion, upgrade and replacement costs were compared to the financial benefits of leakage reduction from pressure control. These include the reduced water leakage resulting from pressure

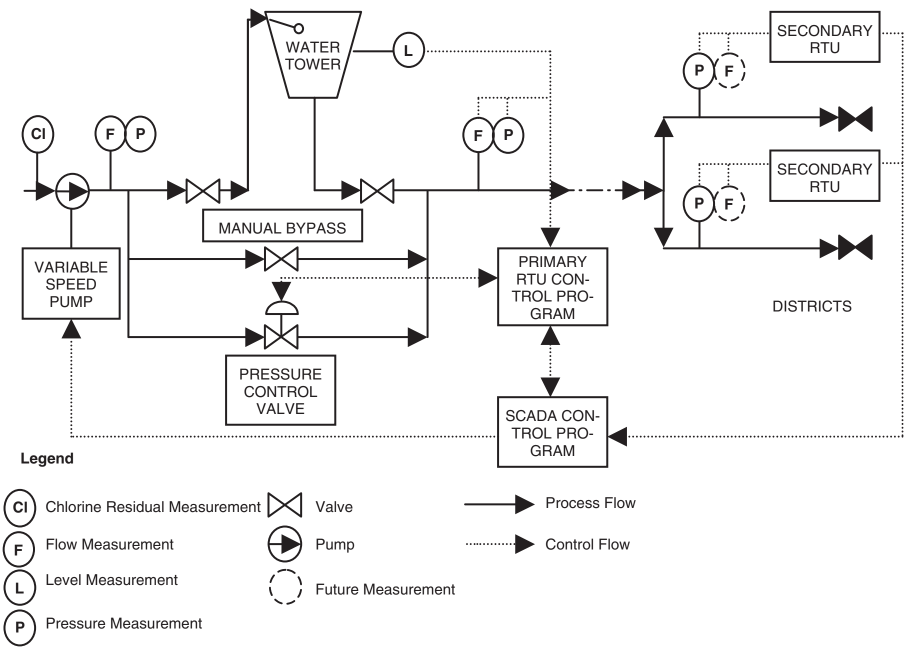

Figure 3. Block diagram of generic pressure control loop.

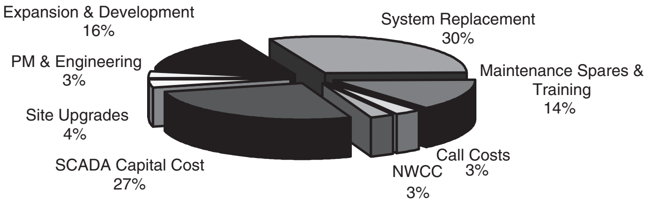

Chart 5. Cost makeup for water SCADA system (typical).

control, labour efficiencies due to automated data collection and energy savings from reduced pump operation. Typical benefit to cost ratios of up to 3 were obtained even with pessimistic estimates of achievable leakage reduction.

Intangible benefits include significantly improved consistency of water quality made possible through full-time system pressurization. Pressurization of the network enables much more effective leak monitoring and bursts can be responded to and repaired rapidly. The water saved by such a rapid response to bursts is also not included in the above analysis.

3 CASE STUDY 2 – SYDNEY, AUSTRALIA

3.1 *Integrated water supply control system*

The water supply and wastewater systems serving the city of Sydney and surrounding areas (approximately 13,000 km^2) comprises some 12 Impounding Reservoirs, 11 Water Filtration Plants, 165 Water Pumping Stations, 262 Service Reservoirs, 620 Sewage Pumping Stations and Sewage Treatment Plants. A pilot control scheme in Illawarra went live in 1994 with the system for Sydney's water being introduced in stages from 1996. The system has continued to expand as wastewater assets are progressively upgraded and integrated into IICATS (Integrated Instrumentation, Control, Automation and Telemetry System).

As with most large information technology based projects many things changed during the time from concept to operation, not only due to rapid advances in the technology itself, but also business and process changes such as a drastic reduction in the water company's workforce and the introduction of 4 large Build Own Operate (BOO) Water Filtration Plants at the heart of the water supply system. The cost of the system, some A$150 M, has been more than justified by the enormous benefits, some direct but many indirect, resulting from the investment in the IICATS technology.

IICATS was designed to provide integrated operation of assets across the water system, and the widespread application of systemic control is one of the distinguishing features of the scheme. Control scenarios can be adjusted across multiple sites to reflect climatic or water demand changes. The water inventory in each sub-system can be controlled dynamically based on projected demand patterns – a feature which has proved most useful during recent Sydney bush-fire events which represent peak demand for Sydney's water supply.

3.2 *Application of IICATS*

IICATS enabled 4 new BOO Water Filtration Plants to be more tightly specified (saving an estimated A$35 M in capital outlay) and for the plants to be smoothly and successfully integrated into the existing water networks. The provision of full control facilities at 400 key nodes on the water supply network allowed optimal specification of the operation of the new plants which were brought into operation over the period 1995–1997. IICATS has enabled Sydney Water to optimise the demand pattern for each BOO plant, in order to reduce operating costs, whilst also closely monitoring the operation of each plant within its level of service and commercial operating parameters. IICATS is one of the essential tools used in the day-to-day negotiation with the plant operators, ensuring demand and quality targets are met.

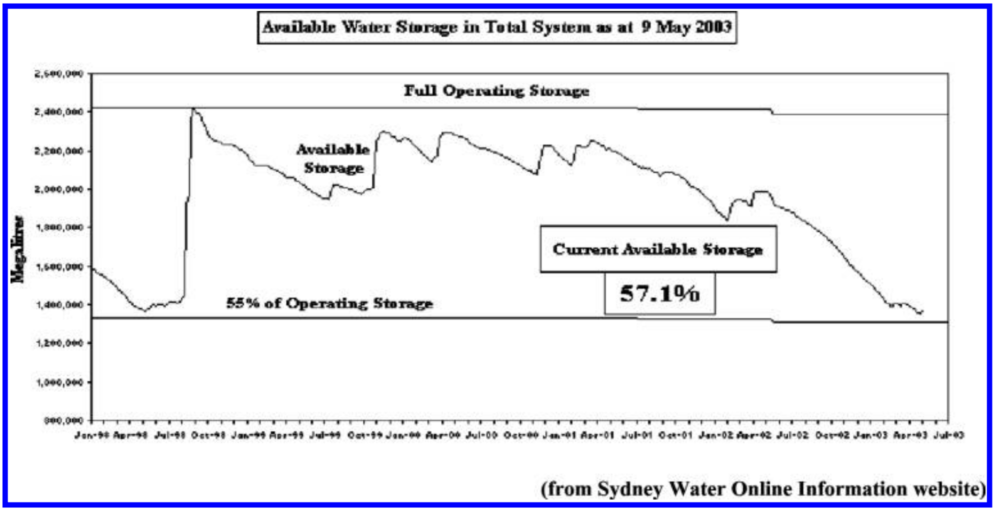

Figure 4. Supply and demand profile for Sydney's Water Network.

Another benefit of the investment in IICATS technology has been savings in operational costs. For example, the scheduling of water pumping stations and reservoirs has been altered so that pumping only occurs during non-peak electricity hours. Essentially the reservoir operates at different parameters depending on the time of day. During nights, many reservoirs will be filled, and then drawn down throughout the day. This works in conjunction with the water filtration plants so that the treatment rates remain relatively constant and do not vary greatly depending on the time of day, also increasing the quality of the water and reducing production cost variations. Sydney Water has used this capability to modify the pumping schedules which determine the demand flow out of one of its major storage and balancing reservoirs at Potts Hill, reducing the need to bring a second, 800 ML reservoir on the site into operation as often as before – whilst also saving several thousand dollars per annum in operating costs at the 600 ML/day Ryde Water Pumping Station and other associated stations.

Another example of the operational advantage obtained as a direct result of use of IICATS was the optimisation of reservoir duty cycles. Conventional operating practice was to try to maintain each reservoir constantly full hence minimising the risk of supply interruption to water users. The set points (switching thresholds) for pump operation are chosen to meet the maximum demand requirement, which can result in frequent pump cycling with consequent detriment to pump life and energy usage. Under this scenario regular flushing of the reservoir must be carried out to ensure consistent water quality. Analysis of demand patterns using the real-time data collected by IICATS allowed optimisation of the pumping schedules to match the demand pattern more closely. This resulted in less pump cycling and better use of the actual storage capacity.

3.3 *Security of water supply*

Integrated telemetry systems enable operations staff to be alerted rapidly and provided with appropriate contextual information. Trends of water usage and pressures have been analysed and set up so that any sudden changes in the daily quantities and pressures in certain areas are highlighted in reports and alarms. Operators can now respond to emergencies, such as trunk bursts, quicker and more effectively. The technology has proved its worth also in allowing the security of the water supply to be improved and has proved invaluable in isolating and flushing out water contaminated by the cryptosporidium pathogen. In these days when malicious attacks on water supplies have to be considered to be a real threat, telemetry increases confidence that incidents can be detected rapidly, isolated and successfully dealt with.

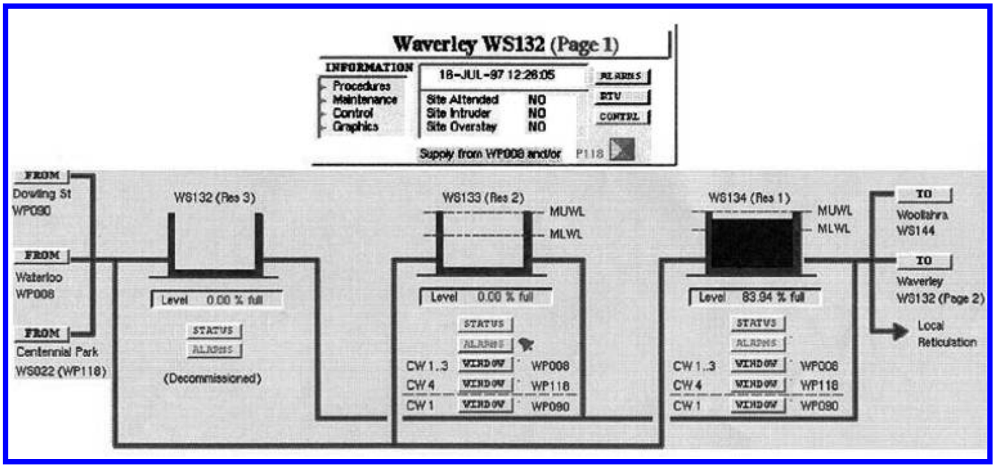

Figure 5. Example IICATS mimic for systemic control of water storage assets.

4 CO-ORDINATED ASSET OPERATION

This paper has described applications in two diverse locations, neither of which would appear to share many characteristics with the UK water industry. The principal driver for the application of technology in both Qatar and Australia was water conservation and the efficient use of the available water resource. In the case of Qatar, the scarcity of water and its production cost drive the requirement for conservation. In Sydney, where raw water is (surprisingly) abundant, continuity of supply and associated storage costs are the drivers for conservation.

The UK water industry is recognized as a world leader in network management, leakage control and delivery of sustainable water quality. This is an enviable position, yet real challenges still lie ahead, including:

- Ever increasing regulatory and reporting demands
- The relentless drive towards Asset Management efficiencies driven by the Regulator including the challenges of AMP4
- The need to address true resource sustainability, with its necessary emphasis on River Basin Management
- Competition and commercial efficiency, including the development of new public/private commercial models.

While parts of the industry agonise over the most efficient division between Asset and Operational Management, and the way in which Opex and Capex demands interact, is it time to reconsider what the real information needs of the business are? Telemetry and SCADA systems, long recognized as key operational tools are ripe for integration into business systems to inform Asset Management decisions and provide a much more accurate understanding of plant and network assets. The statistical techniques provided by Data Mining can enable investment decisions to be based on much more robust data than previously possible, to the economic benefit of consumers, utilities and the community.

Has telemetry and SCADA technology finally come of age?

REFERENCES

Sustaining Water, Easing Society (a second update), T Gardner-Outlaw & R Engelman, Population Action International, 1997.

Integrated Information Systems for Improved Water Asset Management, J Hutton & R Bartindale, National Hydrological Seminar (Ireland), 2002.

Automatic derivation of linearised hydraulic models from Epanet

G. McCormick & R.S Powell
Systems Engineering Department, Brunel University, Uxbridge, UK

ABSTRACT: Linearised hydraulic models are useful when scheduling using linear programming and other methods. Accurate linearised hydraulic models can be built by dividing the day into intervals with homogenous demands, and evaluating the effect of pump combinations on reservoirs and sources. The number of combinations to consider can be impractically large unless distinctions between hydraulically interacting and non interacting pumps are taken into account. An automatic procedure is described which interrogates an Epanet based hydraulic model and automatically derives groups of interacting pumps and lists of pump combinations. A linear model based on these groups and combinations is then automatically calibrated by running Epanet under program control. Calibration in this manner allows model adaptation to daily network and demand distribution changes.

1 INTRODUCTION

Pump scheduling of water networks with storage requires a model of the hydraulic network, a compatible formulation of the problem and a solution method. Linear programming is a fast and flexible optimization method which is well suited to solution of large scheduling problems. It is particularly suited to stochastic optimization problems, such as scheduling which takes water demand uncertainties into account. Linearised models are essential for linear programming.

Models based directly on detailed hydraulic simulation have been used for scheduling (e.g. Powell et al., 1999), but they are inappropriate for linear programming and too slow for methods which require large numbers of schedule evaluations such as dynamic programming and metaheuristics. Macroscopic and linearised models based on regression have been used, for instance Lansey and Awumah (1994), but they are not "spatially responsive", i.e. if there is a change in network connectivity or in the geographic distribution of demand they can be misleading (Ormsbee, 1991). A linearised model which can be built automatically and rapidly updated from a detailed hydraulic model should be both able to respond to frequent network changes and compatible with many different solution methods, including linear programming. This paper describes such a model, and explains how to build and update it.

The linear model is described in section 2 and the automatic model building procedure is shown in section 3. Accuracy is considered in section 4, a discussion follows in section 5 and conclusions are drawn in section 6.

2 A LINEAR MODEL

Hydraulic demand/head/flow relationships are non linear and pump curves are non linear. The flow resulting from switching on two pumps will not usually be double the flow from just one pump, and the outcome may be quite different depending on water demands and reservoir levels. Solution methods typically divide the scheduling period (usually a day) into shorter periods or timeslices during which water demand is almost constant and electricity tariffs are constant. Coefficients then vary according to timeslice. If the timeslice is short enough (say an hour) reservoir levels will often vary only a little during the timeslice. When there is a significant head difference between pumps and reservoirs the effect of reservoir levels on flows is small. If the decision variable is then the use of a combination of the available pumps for a certain proportion of a timeslice the effect of such a combination on flows will be near-linear within a timeslice.

If there are more than a few pumps in a network the number of combinations to consider may be prohibitively large. With 20 fixed speed pumps there are over a million combinations. Pumps interact strongly when they are hydraulically close (e.g. if they pump into a common main and the hydraulic resistance of the link between them is low). Conversely if there are few and limited hydraulic connections, or if the connections are through reservoirs which break the direct communication of pressure, then pumps may be split into groups such that pumps in a group interact non-linearly, but pumps in different groups have hydraulic effects which are independent and which can be estimated as the linear sum of the individual effects. 20 pumps in four groups of five would require only 32 combinations to be considered in each group.

The proposed linear model therefore breaks the day into timeslices of an hour or less in which tariffs are constant and demands are nearly constant, divides pumps into groups such that pumps within a group interact strongly but there is little interaction between different groups, and estimates flows from combinations of pumps switched on in each group.

A best practice network design would arrange all demands to be fed by gravity from reservoirs. When demands are zoned onto distinct reservoirs in this way demands can be included in the model as an explicit variable with purely linear mass-balance effects (e.g. MISER, as discussed by Jowitt & Germanopoulos, 1992). The model proposed here is more general, allowing demands to be located between pumps and reservoirs, and therefore relies on re-estimating the timeslice-dependent coefficients each day. The model can be expressed as follows.

Let $x_{gc}(t)$ be the proportion of timeslice t of duration $v(t)$for which combination c of group Gg is switched on. Reservoir changes are derived from linear coefficients $\lambda_{grc}(t)$ and the combination proportions x. Here $L_r(t)$ is the volume of reservoir r.

$$\boxed{L_r(t) - L_r(t-1) = \sum_{g=0}^{N_G} \sum_{c \in Gg} \lambda_{rgc}(t) x_{gc}(t) v(t)} \tag{1}$$

Source flow rates F_s are derived from linear coefficients ϕ_{sgc}(t) , where s is a source.

$$\boxed{F_s(t) = \sum_{g=0}^{N_G} \sum_{c \in Gg} \phi_{sgc}(t) x_{gc}(t) v(t)} \tag{2}$$

Energy cost depends on linear coefficients κ_{gc}(t) and is given by

$$K = \sum_{t=1}^{N_T} \sum_{g=0}^{N_G} \sum_{c \in Gg} \kappa_{gc}(t) x_{gc}(t) v(t) \tag{3}$$

The coefficients $\lambda_{rgc}(t)$, $\phi_{sgc}(t)$ and $\kappa_{gc}(t)$ can be obtained quite simply from these relationships by examining each timeslice and in each timeslice simulating each combination for each group. Note that demands tariffs and pressures do not appear explicitly in this model.

3 AUTOMATIC MODEL BUILDING AND CALIBRATION

3.1 *Outline of the automatic procedures*

Epanet is a hydraulic network simulator which has been made publicly available by the USA Environmental Protection Agency (Rossman, 1994). An automatic system for building linearised models of the above form from Epanet models has been built using the Delphi programming language and the Epanet Programmers Toolkit. The procedure is:

- Load Hydraulic Simulation Model (Epanet)
- Switch on all pumps (1 at a time), note effects in all timeslices
- Switch on all pairs of pumps (1 pair at a time), and compare effects
- Note pairs of pumps which interact and pairs which are identical
- Form pumps which may interact non linearly into sets
- Generate all possible pump combinations for each set to create "groups"
- Use identical pump information to remove un-needed combinations
- Remove combinations which break key constraints (e.g. efficiency)
- Calibrate the model by simulating all combinations, for all timeslices and groups.

3.2 *Measurement of interaction*

While it is possible to determine that certain pumps will interact by examining a network diagram, it is sometimes harder to be sure that pumps will not interact, if there is a distant or moderately restricted hydraulic connection between them. The model building procedure is therefore based on objective measurements of interaction between network elements.

Pump interaction is judged by examining coefficients for reservoir level changes and instantaneous source flows (λ and ϕ), measured relative to the null situation: all pumps off. Let p and q represent different pumps and

$$\lambda_{\max}(p,q) = \underset{r=1\cdots N_R,\ t=1\cdots N_T}{\mathrm{Max}} \left(\frac{\lambda_{r,p}(t) + \lambda_{r,q}(t) - \lambda_{r,p+q}[t]}{\lambda_{r,p+q}(t)} \right) \tag{4}$$

$$\phi_{\max}(p,q) = \underset{s=1\cdots N_R,\ t=1\cdots N_T}{\mathrm{Max}} \left(\frac{\phi_{s,p}(t) + \phi_{s,q}(t) - \phi_{s,p+q}(t)}{\phi_{s,p+q}(t)} \right) \tag{5}$$

The percentage of interaction or non-linearity $I(p,q)$ may be defined as

$$I(p,q) = 100 Max\left[\lambda_{\max}(p,q), \phi_{\max}(p,q)\right] \tag{6}$$

For several of the hydraulic models examined a threshold of 0.9% led to the construction of pump groups identical to those expected from examination of the network. Pumps in the same pump station and pumping into the same main typically had interactions of 5% or more. Interactions of rather less than 1% were attributed either to remote hydraulic interactions, or to the limited convergence accuracy of the hydraulic simulation together with low flows. The interaction threshold may be chosen either according to the degree of linearity and accuracy required or empirically. Lower thresholds produce greater accuracy but larger groups and therefore greater numbers of combinations to consider.

3.3 *Grouping and calibration algorithms*

Having determined which pumps interact non linearly it is necessary to form the interacting pumps into groups. The determination of these groups is described in figure 1. The next step is to identify all possible distinct combinations of pumps in each group. The calculation of combinations is described in figure 2. This includes procedures which eliminate combinations which are indistinguishable in their effects. This step is also a convenient point at which to eliminate some infeasible combinations. For instance it may be decided to eliminate any combination which results in low pump efficiencies, not because of the energy cost but because this may be dangerous for the pumps. Instantaneous pressures are a function of the combinations, so it may be possible to ensure pressure feasibility by

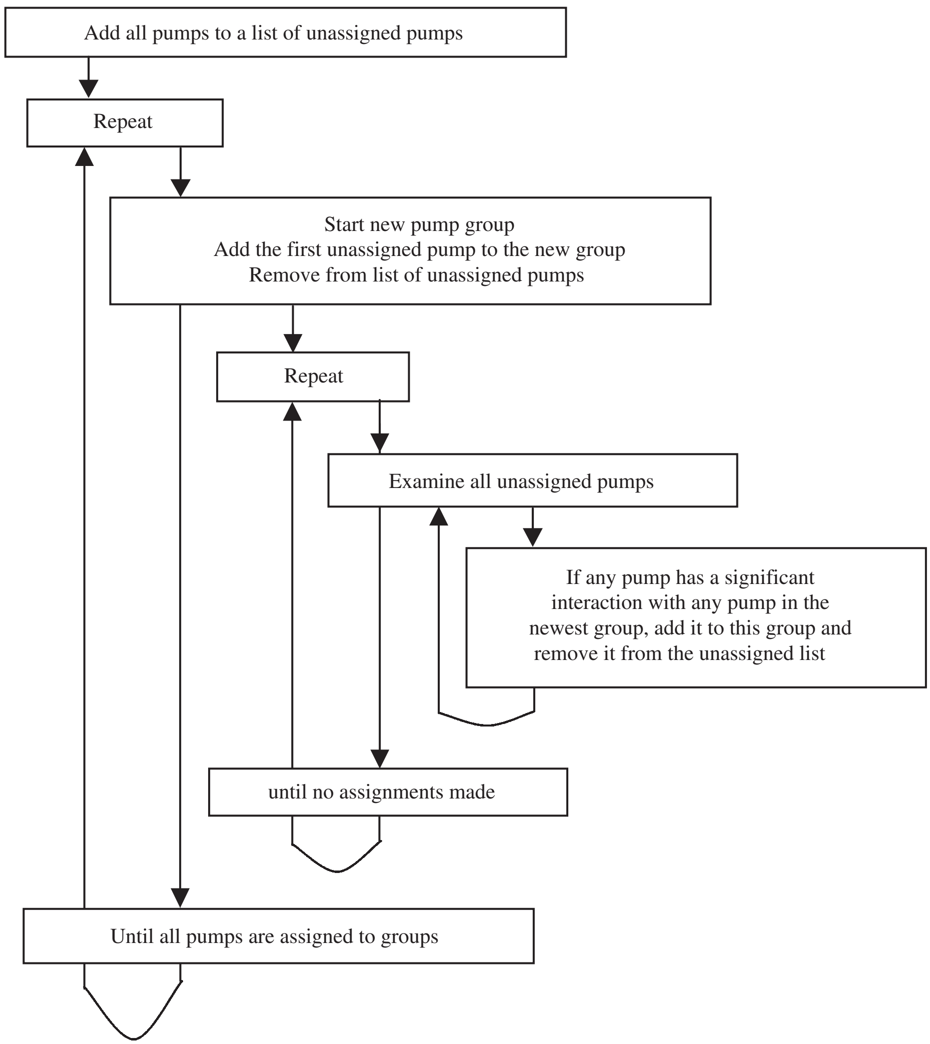

Figure 1. Grouping algorithm.

allowing or disallowing combinations. If a source is served by only one group then it will also be possible to ensure that instantaneous flow rates stay in bounds by allowing or disallowing combinations.

Once groups of pumps and pump combinations have been determined it is necessary to calibrate the linearised model. This calibration may be repeated as necessary (typically daily) and is described in figure 3. Note that it is possible to specify a time step duration to Epanet. However this is the maximum timestep used: in some circumstances the simulation will use a shorter time-step. Since calibration is based in part on reservoir level changes it is therefore necessary to find the time-step actually used and normalize changes to standard durations in order to obtain consistent model coefficients.

4 ACCURACY

There are two obvious sources of inaccuracy in the proposed model. The first consists of residual non-linearities in the interactions between pumps from different groups. This is controlled by selecting

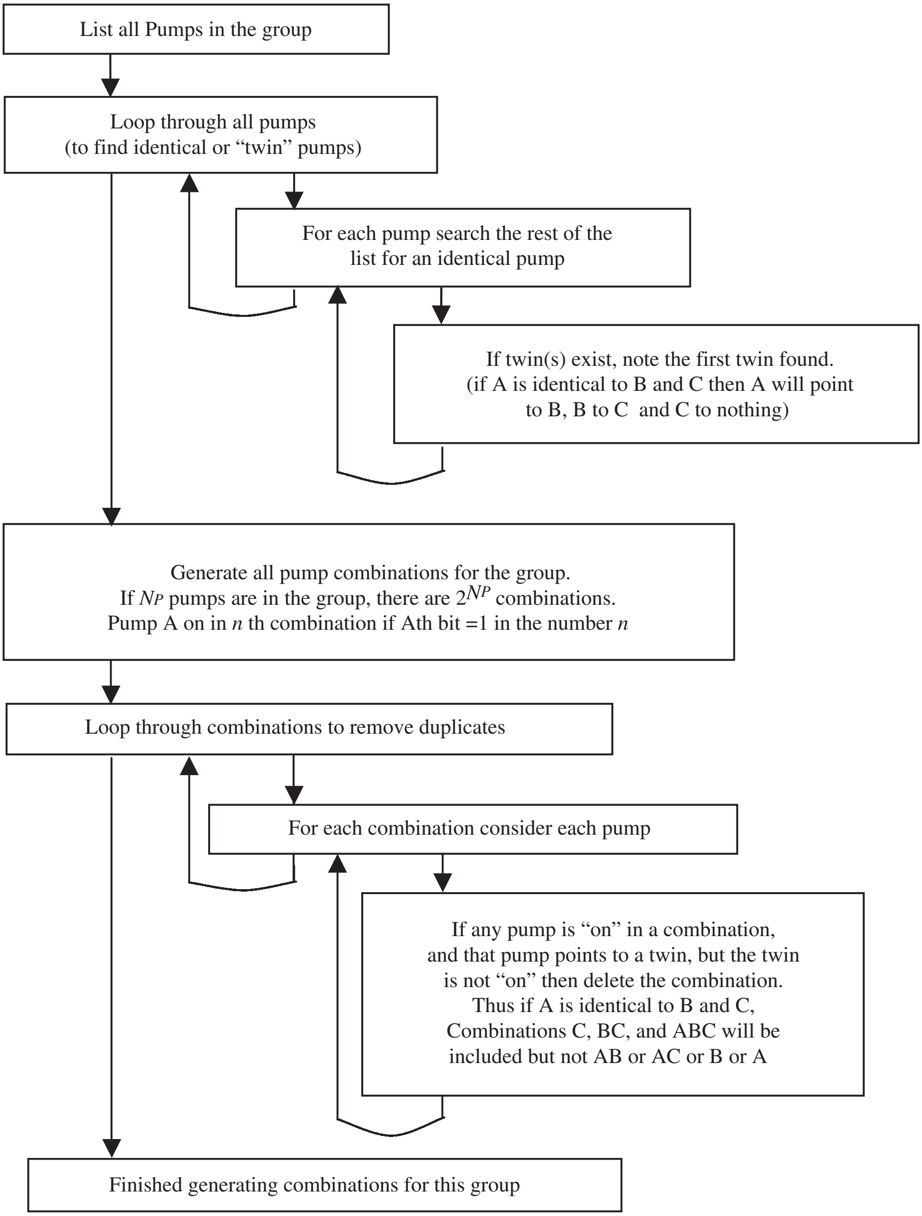

Figure 2. Algorithm for determination of combinations for a group.

the interaction threshold when building pump groups. The second is the effect of reservoir levels on network flows. If reservoirs are top fed then reservoir level will have no effect on inflow, otherwise changes in level will increase the head which pumps must provide and may therefore reduce flows a little.

Where there are significant headlosses in a network flows will not be sensitive to small changes in service reservoir level. Consider a simple network consisting of a source (at zero head) feeding a pump, connected by a delivery main to a service reservoir. Assume that the normal or duty flow and head are F_{DUTY} and H_{DUTY} respectively. Assume further that the pump curve is quadratic, and that the

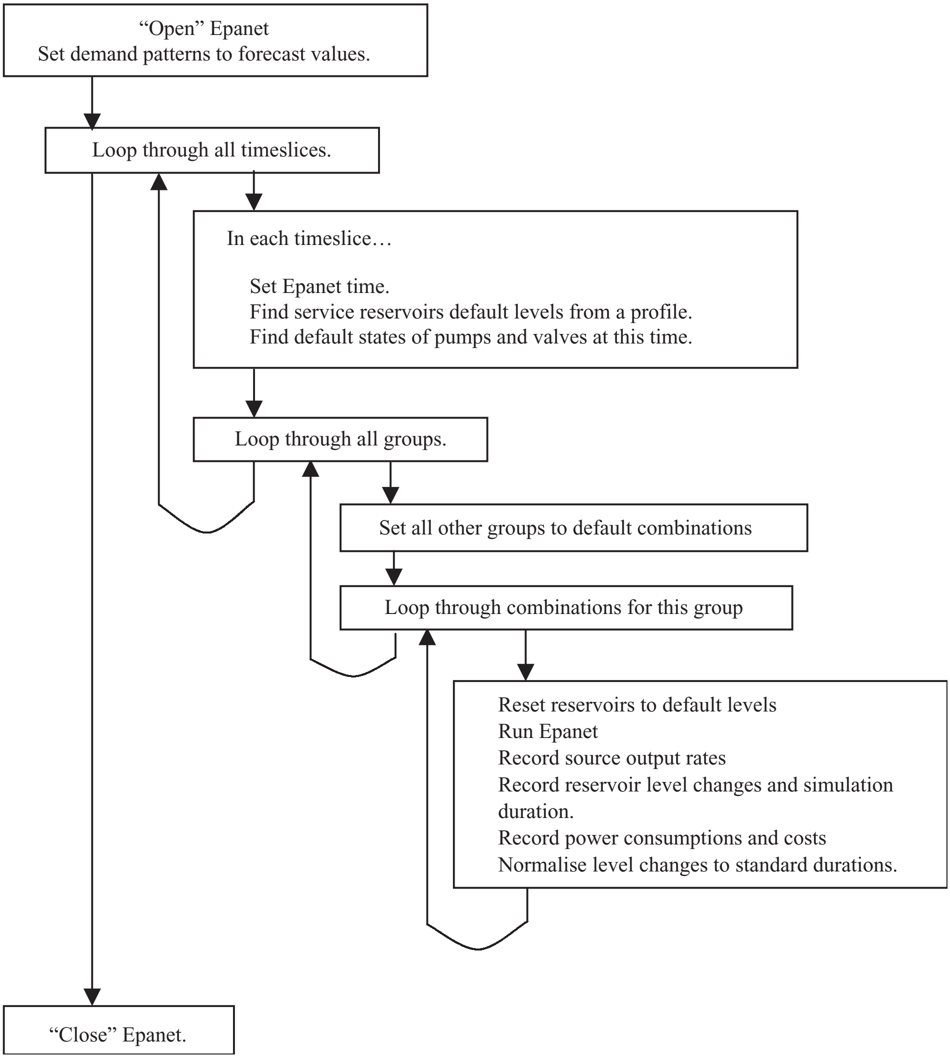

Figure 3. Calibration of model using Epanet.

stall head of the pump is 1.33333 H_{DUTY}. (These are the Epanet defaults). Let F be the flow and H the pump delivery head, while the water in the reservoir is on average Δ_R metres above the source. If there is a perturbation dX of the service reservoir level it can be shown that as a percentage of flow the sensitivity Ψ is

$$\boxed{\Psi = 100\frac{dF}{dX} = \frac{-100F_{DUTY}\%}{2.519H_{DUTY} - 1.852\Delta_R}} \tag{7}$$

With say 40 m headloss plus some lift a 1 m change in reservoir level would reduce flow by less than 1%. Some sample values are shown in Table 1.

We can also estimate flow changes within a timeslice. Assume that the pump is sized so as to be able to maintain levels in the service reservoir with a 50% duty cycle. Assume also that the service reservoir contains 0.5 day's supply and is 10 m deep. With no demand such a pump could fill

Table 1. Non-linearity analysis.

Typical percentage flow error for a 1 m change in reservoir level (Linear Model)					
Res-pump elevation difference (m)	Duty head (m)				
	10	25	50	75	100
0	4.0	1.6	0.8	0.5	0.4
10	15.0	2.2	0.9	0.6	0.4
25	6.0	1.3	0.7	0.5	
50	3.0	1.0	0.6		
75	2.0	0.9			
100	1.5				

the reservoir in 6 hours. A 1% change in flow rate would then produce a 1.5 cm error in 1 hour, 0.15% of depth. If differences between the intended and default profile are 1 m or less, and are not systematic, then we can conclude that when headlosses are 25 m or more a linear model will predict flows within a small percentage of the actual flows, and a reservoir profile which is within a small percentage of the actual profile. This source of error is minimized by centering calibration on a typical reservoir profile.

5 DISCUSSION

The methods outlined apply to fixed speed pumps. Variable speed pumps and continuously variable valves have not yet been implemented. Valve parameters could be treated as continuous variables when the valves do not interact with all other network elements – i.e. the valve would have linear effects and should form its own "group", with settings represented by varying the proportions $x_{gc}(t)$. Variable speed pump settings are likely to be associated with non linear changes in flows. Pump speed choices should be represented as sets of pseudo combinations. The same is true of valve parameters if effects are non-linear.

Demands are implicit. They could be made explicit if demands are zoned onto reservoirs. Daily recalibration would then be less necessary but the model would be less general. Tariffs are also implicit. They could be made explicit, which would be an advantage if daily recalibration is unnecessary but irrelevant otherwise.

Daily calibration takes only a minute or two with a network model which has 10 reservoirs 13 sources and 35 pumps, using a 1 GHz PC. However determining pump groups and combinations takes a few minutes longer. Separate files are therefore used to retain this information and other parameters which are not stored in Epanet files.

In many cases hydraulic interactions between pump stations are small, and the numbers of pumps at any one station are not usually enough to give rise to very large numbers of distinct combinations. However there are some networks with many interactions – e.g. ring main based systems. The result could then be a very large number of distinct combinations to consider. None the less, the linearisation described in this paper is suitable for many networks, It is possible to envisage networks with significant hysteresis or where discontinuous behaviour can be induced by small changes in circumstances and the linearised model would probably not be suitable for such difficult cases, but this potential problem is shared by many optimisers and does not seem to be a problem in practice.

It should be borne in mind that hydraulic models are never perfect. Because much of the water distribution infrastructure is typically old, buried and hard to inspect there is structural uncertainty. It is usually impossible to measure the resistance of individual pipes, and pump characteristics deteriorate in time so there is parametric uncertainty. Finally, demands cannot be perfectly predicted. Barker (1993) pointed out that continued value from hydraulic models depends on updating the basic data. Models are also subject to errors from rogue logger results, underestimates of peak flows, inadequate metering, and permeable boundaries. Powell et al (1999) described a number of typical modelling errors. These errors and uncertainties may give rise to errors greater than those due to imperfect linearisation.

Some arrangements of active or non linear elements in a network could potentially result in many significant non-linear interactions or in hysteresis. The linear model might not be suitable in such cases. However there are many networks where the linearised model is well suited.

6 CONCLUSIONS

In this paper a choice of variables for hydraulic optimisation has been described which makes many schedule optimisation problems nearly linear. The day is divided into timeslices with approximately constant demands and constant tariffs, and pumps are placed into groups such that there is no interaction between groups. An automatic method for deciding the groups was described. A calibration procedure was also described which, being automatic, improves the responsiveness of these linear models to network changes. This procedure is in fact essential, because the model does not assume that the effects of demand variations can be assigned linearly to different reservoirs.

With the formulation presented here the main error is due to the non-linear effects of changing reservoir levels, which are small in many circumstances. In some cases modelling and demand prediction uncertainties will be greater. This source of error is minimized by centering calibration on a typical reservoir profile. When there is no typical reservoir profile calibration about a single, average reservoir level is safe in the sense that true inflows will be higher than predicted when reservoir levels are low and vice versa.

Linear models based on this formulation could be used for stochastic optimisation using LP. The models are fairly general: they do not depend on demands being assigned to specific reservoirs and they can allow for the interaction of pumps in different stations. They are easy to keep up to date using an automatic calibration procedure.

REFERENCES

Barker RA 1993, "Network Analysis at Ground Level", in Coulbeck B (ed) *Integrated Computer Applications in Water Supply Vol 2: Applications and Implementations for Systems Operation and Management* Research Studies Press, 3–15.

Jowitt PW & Germanopoulos G 1992, "Optimal Pump Scheduling in Water Supply Networks", *Journal of Water Resources Planning and Management,* ASCE, 118(4) 406–422.

Lansey KE & Awumah K 1994, "Optimal Pump Operations Considering Pump Switching", *Journal of Water Resources Planning and Management,* ASCE, 120 (1) 17–35.

Ormsbee LE (ed) 1991, "Energy Efficient Operation Of Water Distribution Systems", *University Of Kentucky Dept Of Civil Engineering* Research Report No. UKCE9104.

Powell RS, Sterling MJH, Andersen JA & Burgess A 1999, "A Pump Optimisation Suite for Large Scale Water Systems" in Powell RS & Hindi KS (eds) *Computing and Control for the Water Industry*, Research Studies Press. 177–190.

NOTATION

Indices

c	pump combination index.
g	pump group index.
p, q	pumps.
r	reservoir index.
s	source index.
t	timeslice index (discrete intervals are used).

Other variables

$v(t)$	duration of timeslice t.
$x_{gc}(t)$	proportion of combination c group g in pump schedule at t (dimensionless).
$F_s(t)$	flow rate from source s in timeslice t.
F_{DUTY}	pump duty flow.
G_g	pump group or set of pump combinations index g.
H	pump delivery head.
H_{DUTY}	duty delivery head for a pump.
$I(p, q)$	percentage of non-linearity for interaction of pumps p, q.
K	cost.
$L_r(t)$	volume of reservoir r at time indexed by t.
dX	perturbation of reservoir level (m).
Nc_g	the number of combinations in group g.
N_G	number of groups.
N_R	number of service reservoirs.
N_S	number of sources.
N_T	number of time slices or stages.
Δ_R	average difference in level (metres) between source and reservoir.
$\phi_{\max}(p, q)$	maximum interaction between pumps p and q (measured via flow rates).
$\phi_{sgc}(t)$	flow from source s due to combination c of group g.
$\phi_{s,p+q}(t)$	water demand on source s due to pumps p and q.
$\kappa_{gc}(t)$	energy cost per unit time of combination c group g.
$\lambda_{\max}(p, q)$	maximum interaction between pumps p and q (measured via reservoir levels).
$\lambda_{rgc}(t)$	rate of change of reservoir r due to combination c of group g.
$\lambda_{r,p+q}(t)$	rate of change of reservoir r due to pumps p and q.
Ψ	sensitivity of flow to reservoir level.

Pressure control policy in water distribution networks: sector 35, Bogotá, Colombia

F.S. Contreras-Jimenez
University of Newcastle, Newcastle upon Tyne, UK

J. Saldarriaga
Universidad de los Andes, Bogotá, Colombia

ABSTRACT: This article describes the implementation of the Optimal Pressure Methodology in 35th Hydraulic Sector of the water distribution system of Bogotá City, developed as a pressure control policy to reduce leakage losses and update the GIS of the network. The pressure was reduced from 47 m to 33 m at the entry point and as a result the flow to the hydraulic sector was also reduced from 90.28 l/s to 66.74 l/s. The water consumption in the sector was not affected and the water losses in the sector were reduced to 29% from 49% as a result of this pressure reduction.

KEY WORDS: Pressure control, water demand, hydraulic model, GIS, water distribution, leakage losses.

1 INTRODUCTION

This project was developed by the Water and Sewer Systems Research Centre (WSSRC) of the University of Los Andes for the Empresa de Acueducto y Alcantarillado de Bogotá (EAAB). The main objective of the project was to study the effect of a pressure control policy in the water consumption planning. The technical unit of the EAAB was interested in the pressure control policy in order to reduce the leakage losses, maintenance and operational costs while the commercial unit thought that a reduction in the network pressure would significantly affect the revenues to the water company. The EAAB selected the 35th Hydraulic Sector (HS-35) of the network to undertake an experimental study to determine the effect on water consumption produced by a pressure control policy under real conditions. The HS-35 is located in the north-east part of Bogotá City, with a population of 45.000 people and 9.000 households, over an area of 44 Ha, completely isolated from other hydraulic sectors with only one water entry where a Pressure Reduction Valve (PRV) is installed.

2 BACKGROUND

The EAAB was building a storage tank with a capacity of 88.000 m^3 in the west of Bogotá whose operation would increase the pressure in the network in the west part of the city thus raising the amount of burst pipes and the leakage losses, especially through the fittings. Consequently, the EAAB wanted to implement a pressure control policy to mitigate the effects of the new tank without a reduction in the water consumption.

It is a well known fact that the pressure control policy is cost-effective for reducing leakage losses, but the EAAB wanted to study the problem under real conditions, also including:

– The construction and calibration of a hydraulic model of the HS-35
– Updating the information of the network

– Analysing the HS-35 users' data base
– Measuring flow and pressure (over a period of six months)
– Reading the household meters (two times per month)
– The development of a Methodology to reduce the water pressure that could be followed in each of the hydraulic sectors of the water distribution network.

The policy implemented for the project, uses all the information provided by the calibrated hydraulic model to find the optimal pressure at the entrance of the HS. This optimal pressure basically guaranties three things: leakage reduction, minimal service condition in the network and higher water savings costs (due to reduced leakage losses) than the losses in water consumption revenues.

3 METHODOLOGY

The methodology for the optimal pressure determination (Saldarriaga & Contreras Jimenez, 2002) for the project consisting of three stages is shown in Figure 1. The first stage covers the gathering of the information of network and users including the construction of a preliminary hydraulic model and the design of appropriate field works. The second stage of the methodology includes all the field works during six months under three different pressure conditions (high, medium and low) for which continuous flow and pressure measurements were required. It also includes the household meter reading every 15 days. Finally, the methodology includes the calibration of the hydraulic model and the determination of the optimal pressure at the entrance of the HS and if it is necessary the determination of optimal pressures in smaller areas inside the HS.

All the information required for the project was provided by different units of the EAAB.

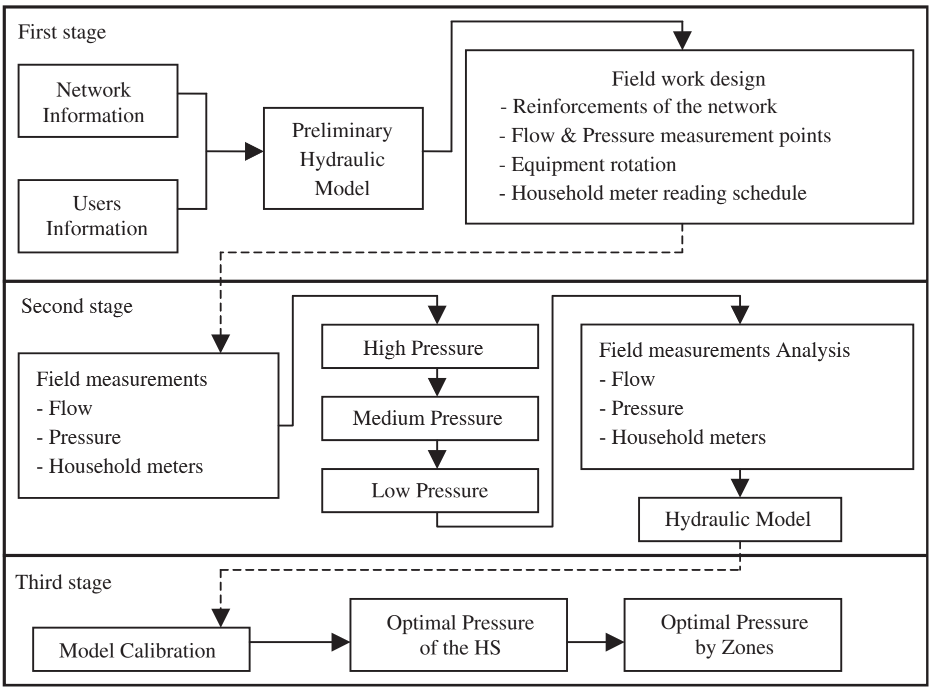

Figure 1. Methodology for the optimal pressure determination.

3.1 *First stage*

This stage is divided in four main activities: gathering information of the network and of the users, construction of a preliminary hydraulic model and the design of appropriate field works (WSSRC, 2001a).

3.1.1 *Network information*

All the information regarding the water distribution network was divided into two large groups, one related to the network pipe sizes (75 mm and above) and devices searching for their real and nominal diameters, material, installation dates and minor loss coefficients for each pipe. Some 90% of the pipe diameters in the HS-35 are between 75 mm and 150 mm, and 70% of them were installed during the 60's through 80's and 70% of the pipe material is asbestos–cement. The second group is related to the connectivity of the components and the operational scheme.

3.1.2 *Users' information*

Users' data base contains the information of all the 9.000 householders in HS-35, including the addresses, account numbers, status, etc. and their consumption records (historical average consumptions, the last six consumptions and the total volume registered by the water meter). In addition, it includes the x and y coordinates of the users crossed with a Bogotá cadastral map to verify the correct location of all the users in the city blocks. The identification process emphasises especially the location of big water consumers or users (such as industrial buildings, commercial centres, hospitals, etc.) likely to be more sensible to the possible reductions in water pressures.

3.1.3 *Construction of the preliminary hydraulic model*

A preliminary hydraulic model was built to understand the initial hydraulic operation of the HS-35 Figure 2 and was proved by a new hydraulic operation scheme. This new hydraulic operation was totally different to the initial operation in the sector using the historical consumptions, to calculate the water demand in each node. These consumptions were calculated with the historical volumes,

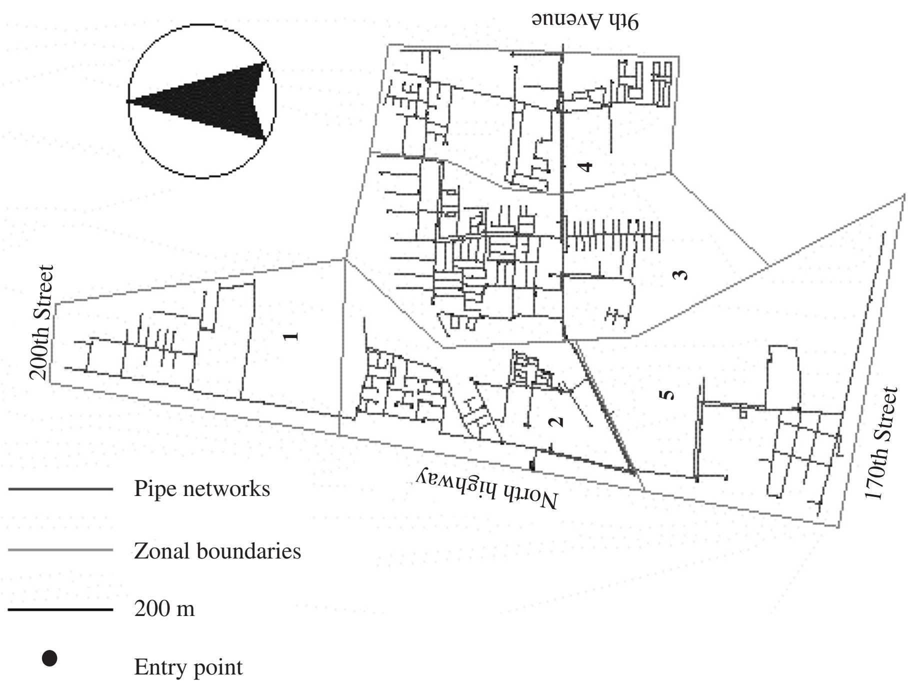

Figure 2. Preliminary hydraulic model HS-35.

either theoretical or real (if these were available). The preliminary model was used to identify various zones with high and low pressures and the ones in which it was necessary to make topological changes so as to find the best theoretical operation scheme of the network. All the information of the network and the users was also used to update the GIS of the EAAB.

3.1.4 *Design of the field works*

The design of the appropriate field works included the location of six flow meters and eleven pressure meters in the sector. The flow and pressure measurement equipment was programmed to be read continuously every minute during the six months and downloaded every week. In addition, the reading of the household meters of this sector (HS-35) was also programmed amounting to twelve household readings in total in this period of six months.

3.2 *Second stage*

The second stage is divided in three activities: measurements of flow, pressure and water meters under three different pressures, analysis of the field information and the adaptation of the hydraulic model (WSSRC, 2001a).

3.2.1 *Field measurements*

The field works were commenced with the first household reading and with a high pressure scheme in the network. The first pressure tested in the HS-35 was 47 m. Two months later the pressure was changed to 42 m and for the final two more months the pressure was further reduced to 33 m corresponding to a total 30% reduction in the pressure. There were no significant problems with the information of flow and pressure readings in the network. During this six months eleven more household readings were also made during which some important problems appeared, the quality of the users' data base was not as good as it should be; 690 householders (7.6% of the total) were not consuming any water at all. There were other problems concerning the users' addresses and reading routes. A re-reading process was implemented to solve these problems.

3.2.2 *The analysis of field measurements*

The next step was the analysis of the information of flow and pressure readings involving the build up of the flow and pressure curves for each pressure stage. These curves were the input for the calibration of the hydraulic model of the network. Further, an analysis of the water consumption readings was also made which included a comparison during the project period and the consumptions a year before and also with the historical ones.

3.2.3 *Hydraulic model*

Finally, the hydraulic model was developed which was verified by Epanet (Figure 3), Watecad and Redes models. The hydraulic model contained 1172 nodes, 987 consumption nodes, 1268 pipes and 1 source.

3.3 *Third stage*

This stage is divided in three activities: model calibration, optimal pressure determination at the entrance of the HS-35 and optimal pressure determination by zones.

3.3.1 *Model calibration*

The calibration process (Saldarriaga & Salas, 2002) under different leakage hypotheses in HS-35 was the most important product of the research work carried out by the WSSRC. The variable range of the topological data was defined and a first leakage hypothesis was outlined.

Then the roughness and the minor losses were adjusted in each pipe to reach the preliminary calibration. A further calibration was achieved by classifying each pipe in different groups by material and original diameters. After testing different combinations of diameters, their absolute roughnesses and minor loss coefficients a final calibration was reached.

Figure 3. Pressure levels under average water consumption (Epanet).

3.3.2 *Optimal pressure at the entrance of the HS-35*

Other leakage hypotheses were tested (spatial distribution in the sector) until the best hypothesis that calibrates the network was reached. This final calibration is the one used to test other pressures scenarios and to establish the optimum pressure at the entrance of the HS-35. With the flow measurements and the household meter readings, the unaccounted water losses could be established under the pressures tested. Water losses could be calculated as a function of the pressure, and can be discriminated between the commercial (commercial losses) and technical (leakage losses) ones. These losses functions could be implemented into the calibrated model in order to establish a pressure to the entrance which minimizes the unaccounted water and this was found to be 33 m at the entrance of HS-35.

3.3.3 *Optimal pressure by zones*

Using the calibrated model, the water losses were calculated in each of the zones of the sector was divided. No additional changes in the pressure at the entrance of each zone were needed in HS-35.

4 RESULTS

The flow and pressure measurements were used to calibrate the hydraulic model. The household meter readings were used to calculate the amount of water consumed during the different operational pressures (WSSRC, 2002). The accounted water was constant during the three pressure periods. The results of the water balance in HS-35 are shown in Figure 4.

The reduction of the water pressure at the entrance of HS-35 was 30.25%. The initial pressure was 48.98 m and the optimal pressure after calibrating the model was 32.95 m corresponding to the water loss factors of 48.96% and 29.05% respectively. Figure 5 shows the evolution of the water losses factor and the reduction of the pressure during the project.

The most important result after the implementation of the optimal pressure methodology in HS-35 is that the accounted water does not depend on the supplying pressure. The validation of the previous statement in other hydraulic sectors is necessary. In the event in which there is no dependence between the accounted water and the supply pressure, the optimal pressure for a given network is found in the minimum service pressure for the critical pressure point. If the above hypothesis is not valid the financial analysis reducing the associated costs to the water losses subject to maintain a proper sales level must be done. The most relevant result which can be extracted of the pressure, flow and household measurements carried out during the second stage is that the pressure does not have apparent effect on the users' water consumption.

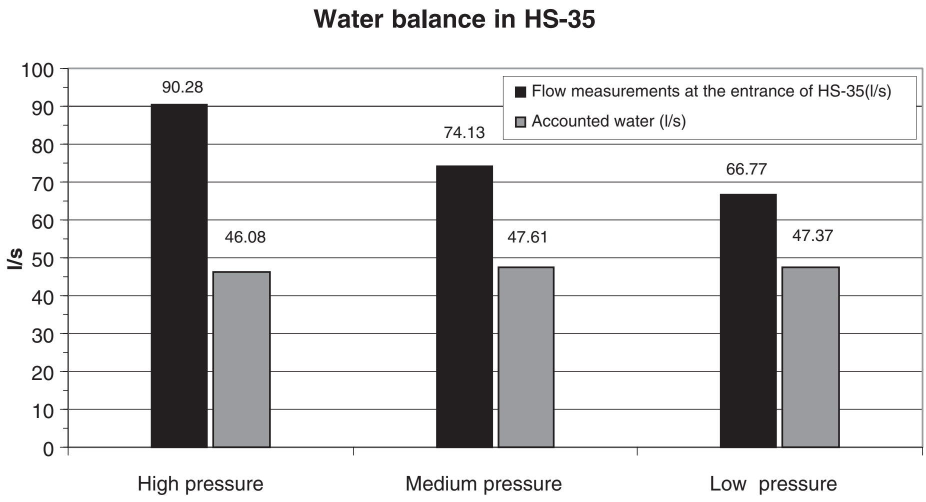

Figure 4. Water balance in HS-35.

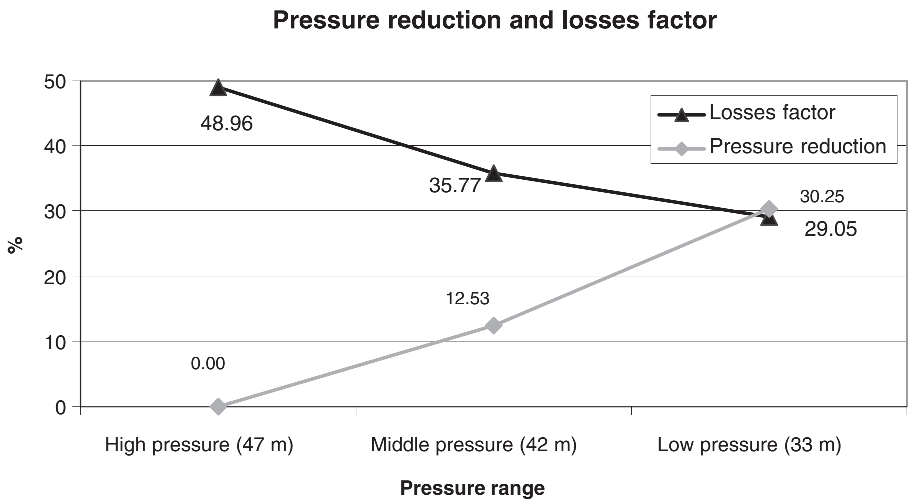

Figure 5. Pressure reduction and losses factor in HS-35.

Finally Figure 6 shows the leakage curve for HS-35. The curve (see Equation 1) was the result of six months of field woks, and 4 months of office work. The curve describes the behaviour of the water losses (technical and commercial) with respect to the pressure variations.

$$Q = 0.0087h^{2.1906} \tag{1}$$

where Q = leakage in l/s; and h = water pressure in m at the entrance of the HS-35.

5 CONCLUSIONS

- The accounted water does not depend on the supply pressure. The pressure was reduced from 47 m to 33 m at the entry point and as a result the flow in the hydraulic sector was also reduced

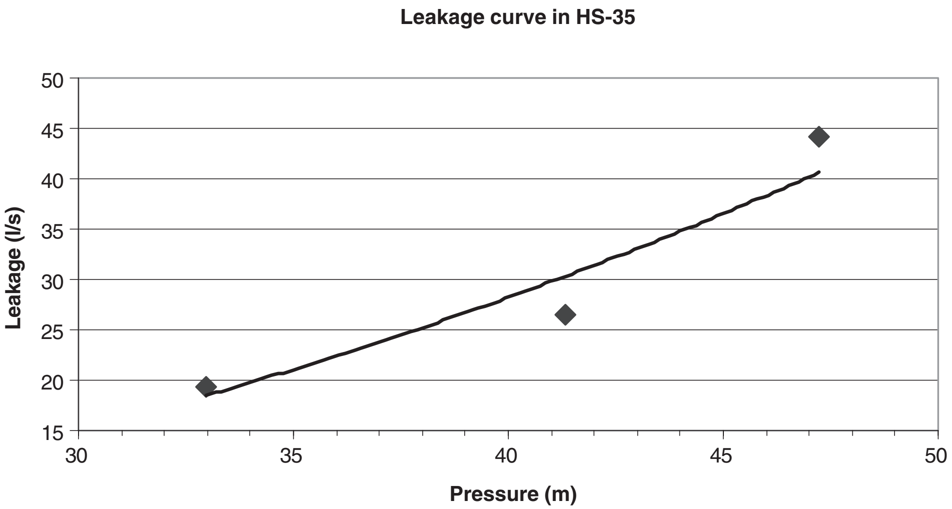

Figure 6. Leakage curve HS-35.

from 90.28 l/s to 66.74 l/s, but the users' water consumption was almost the same during the three stages of pressures studied.

- The total water loss factor in HS-35 was reduced to 29% from 49% due to the reduction in pressures to 30 m.
- The hydraulic model of the network includes pipes of 75 mm and above and the user information in a GIS platform. This tool could be used as the first step of a networks management programme. This programme could include: network & updated information of the users, spatial location of the consumers who reported zero consumption (0), renewal and/or rehabilitation of distribution networks, etc.
- The average water demand per household during the second stage of the project in HS-35 was 5 m^3 lower than in the historical water consumption; this is the result of water saving programmes implemented by the local authorities and the water pricing policies implemented by the EAAB during the last couple of years.
- This optimum pressure implementation is a cost-effective solution with savings of around £18.000 per month.
- With this detailed hydraulic model the approximation to the water demand in each network nodes is more accurate, this is essential for the mathematical modelling and its calibration.
- The calibrated model allows finding the location of the leaks and illegal connections in small areas.
- Where losses were too high, a renewal and/or pipeline rehabilitation programme should be implemented.

REFERENCES

Saldarriaga, J. & Contreras-Jimenez, F.S. 2002. Determination of the optimal pressure plane methodology. XX Latin-American Hydraulics Congress, La Havana, Cuba.

Saldarriaga, J. & Salas, D. 2002. Calibration in water distribution networks under a leak environment. XX Latin-American Hydraulics Congress, La Havana, Cuba.

Water and Sewer Systems Research Centre. 2002. Optimum pressure plane project. Final Report. University of Los Andes. Bogotá, Colombia.

WSSRC (Water and Sewer Systems Research Centre). 2001a. Optimum pressure plane project. First Stage. University of Los Andes. Bogotá, Colombia.

WSSRC (Water and Sewer Systems Research Centre). 2001b. Optimum pressure plane project. Second Stage. University of Los Andes. Bogotá, Colombia.

Making defensible decisions in the face of uncertainty

E.J. Langman & S. De Rosa
FaberMaunsell

ABSTRACT: A new approach which offers considerable potential to the management of complex water supply systems is described. The methodology was developed at the University of Bristol in collaboration with a group of UK infrastructure owners, regulators and engineering consultants. FaberMaunsell have successfully applied the ideas and accompanying software tool, Perimeta, in sectors where the main asset-base consists of complex infrastructure systems. The approach involves representing the infrastructure system hierarchically, incorporating (as appropriate) processes at all levels within the organisation, from high-level business process, right through to day-to-day operational decisions. The performance of each sub-system is captured by a set of performance indicators drawn from a number of sources including inspection records, design calculations, model studies and expert judgements. Some evidence can also be drawn from analogous situations or from the historical performance of the system in question. Uncertainty in the available evidence is made explicit in order to support robust, auditable, defensible decision-making.

1 INTRODUCTION

The management of water supply systems is very complex, not least because many of the assets are hidden. In addition, the loading on the system varies over time and the assets can be spread over a large geographical area through differing types of terrain. Finally, one of the key challenges facing engineers in this field is that the assets are not easily accessible and the degradation patterns are not generally well understood.

It is therefore necessary for those managing water supply assets to make decisions in the face of uncertainty. Because the assets are network-based, decision "rules", particularly those relating to degradation cycles, can have a large economic impact. For example, a change in inspection frequency for a particular asset might make a difference of a few pence per year (per asset or per metre length), but across the entire asset-base, this change can result in a substantial investment or, if the frequency is reduced, cost-saving.

Given this background of uncertainty and costs, it is not surprising that there has been an increasing focus on "risk assessment" in recent years. However, many of the methods used to carry out these risk studies have necessitated an estimation of statistical factors, such as, for example, the probability of failure of a particular asset. Unfortunately, while in other sectors, such as the nuclear or automotive industries, such probabilities of failure can be determined through testing and statistical analysis, this is not so with water supply systems where even basic "facts", required to support assessments of the probability of failure are not generally available. For example, the expected life and degradation patterns for particular pipes may not be known with any degree of certainty. Furthermore, even if these modes of behaviour were rigorously developed, this would be of little benefit to those organisations where the asset records are not complete and the asset age and/or material of which it is made, are not always known.

The interesting issue to consider, however, is whether this state of lack of asset knowledge is detrimental to the safe and efficient performance of the asset-base. Whilst there may be some who would argue that complete asset registers are essential to the management of these assets, it could be argued that, while new assets should be documented, old assets (particularly buried or otherwise inaccessible assets) may not need to be comprehensively recorded unless there is a specific area of concern. This is because, in some cases, the failure of a particular asset may be acceptable, particularly if the consequences will be limited, the cost of prevention is outside the ALARP region, or mitigation measures are sufficient to safeguard life and property.

The aim of this paper is to present a methodology which would allow organisations working within the water supply sector to make medium- to long-term investment decisions that balance their commercial aims with risk limitation in such a way that all stakeholders: customers, regulators and shareholders, can be left satisfied.

2 ASSET MANAGEMENT CHALLENGES IN THE WATER SUPPLY SECTOR

As has already been outlined in the introduction to this paper, water supply systems, in common with other complex infrastructure assets, have physical failure mechanisms that are not fully understood. This is partly due to the fact that the asset stock is aging (and has not been at this age historically, so behaviour is difficult to predict), and partly due to the fact that the exact mode of failure for a particular part of a network, or a particular point asset, will be site-specific. Existing failure models may be relatively complete, but even when the failure mode is understood, the moment of failure is far less predictable (due to the lack of degradation predictive modelling). Even as knowledge improves and the probability of failure begins to be estimated quite accurately, the exact location of any given failure at any point in time, will still be very difficult to predict with any great certainty. One cause of this is the fact that many of the assets are hidden (buried), while another issue is that the exact loading regime can be difficult to predict, since the actual demand and forces on the asset may vary both temporally and spatially.

Partly due to the difficulty in accessing many of the assets, and partly due to financial considerations, monitoring information is very scarce in this sector. Therefore, it has become increasingly necessary to rely on expert judgement and condition characterisation techniques, which typically provide rather uncertain information. Carrying out monitoring work to inform asset management decisions can be time consuming and may not result in a great increase in certainty regarding the performance of the system. It is therefore essential that monitoring is not carried out where there would be only limited benefit (and great cost), but does take place where it can be of benefit. Thus there is a clear need for a decision support mechanism that can assist engineers in deciding (through a risk-based assessment) *where* to collect monitoring information, *what* information to collect and *when* to collect it.

Of course, condition and performance monitoring is not the only challenge. The next requirement is to decide what all the data that have been collected mean. For example, is a measure of 3.2 mm (in a particular context) significantly worse than a value of 3.1 mm? What is the largest value that might be expected? At what point should action be taken? The methodology described in this paper does not answer these questions, but it does ensure that the knowledge and expertise of those who *can* answer these questions, is captured in a comprehensible and consistent format, so that decisions that are made on the basis of this knowledge are truly defensible.

This means that many forms of evidence, both qualitative and quantitative, of varying degrees of quality and completeness must be captured in such a way that the associated uncertainty is fully acknowledged, while still allowing informed and reliable decisions to be made so that monitoring and remediation resources can be targeted more efficiently.

The remainder of this paper describes the application of a new approach to asset management, namely Perimeta, which can address these issues in a structured way.

3 THE PERIMETA METHODOLOGY

3.1 *Development*

The Perimeta methodology was developed by members of the Civil Engineering Systems Group at the University of Bristol, between 1999 and 2002. FaberMaunsell have since applied the approach in a number of innovative projects in the water, rail and highway sectors. The aim of the research programme was to develop new decision-support techniques to enable performance-based management of complex civil engineering infrastructure systems. Through consultation with experts from engineering organisations (consultants and contractors) as well as government organisations, it became clear that many sectors have similar problems with regard to managing their physical assets. Although the case study described in this paper relates to a network-based system, typical of the water supply sector, the methodology has also been applied to other complex systems including dams, flood and coastal defences and engineered and natural slopes.

3.2 *Overview*

The diagram in Figure 1, is an overview of the Perimeta methodology. The pipe in the bottom left-hand corner represents the system being modelled. The system is represented as a hierarchical process model, which provides an overview of system performance. All available performance indicators (PIs) relating to the system are entered into a database (to avoid double-entry of data, the software uses XML, so data can be extracted from existing databases). Each PI is then projected through a value function, to produce a Figure of Merit (Italian Flag), which can be associated with one or more processes. All the PIs associated with each process are then weighted according to their relative importance to give a combined figure of merit (FM) for each process being considered. All the FMs are propagated up through the hierarchy to give an overall FM for the system.

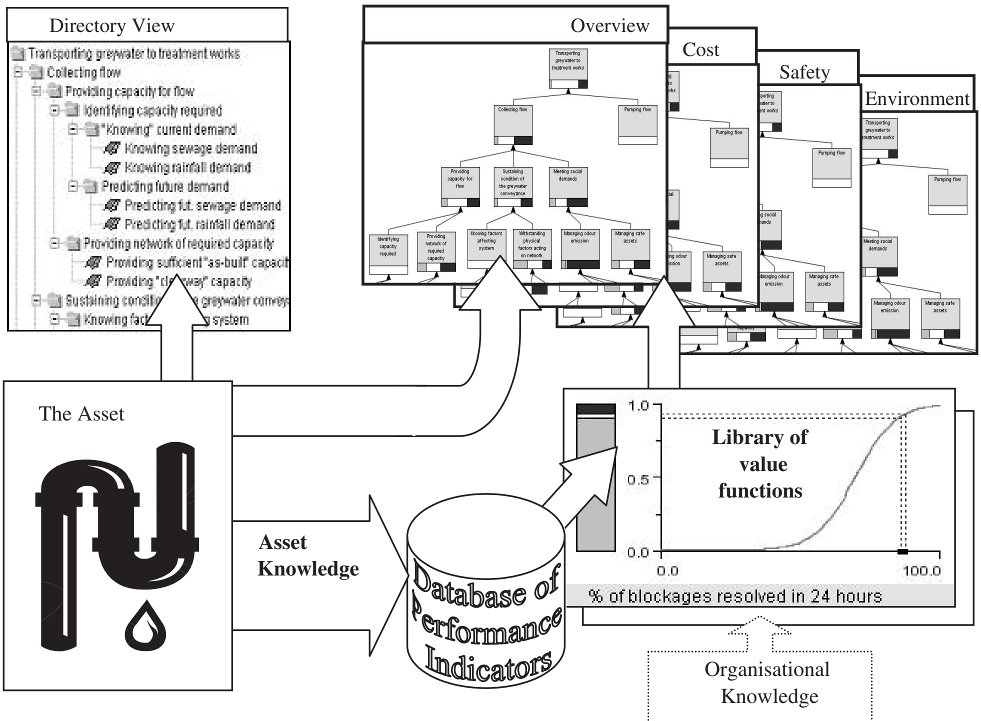

Figure 1. Overview of the Perimeta approach.

4 KEY PRINCIPLES AND IMPLEMENTATION

4.1 *Modelling systems through the processes they enact*

The first stage in improving the performance of water supply systems using this methodology is to identify the processes that are taking place. This is done through consulting with experts in the field through workshops (Davis & Hall, 1998) and small group sessions, and capturing their view of what processes are taking place. In addition, organisational literature, such as Business Plans, Inspection Guidance and so on, is used to provide further richness to the process model. The conventions of Value Engineering and patterns developed at the University of Bristol and at FaberMaunsell are used to help ensure that the model produced is robust and meaningful. The boxes that represent processes are commonly referred to as "holons".

In accordance with modern process thinking (e.g. Checkland, 1981, Rummler & Brache, 1995) the model should not be built according to the existing organisational structure; rather it should focus on the processes that are being undertaken within the organisation. This allows cross-departmental processes to be identified and included within the hierarchy. The model will be complete when it contains sufficient detail to inform the decisions that require support from it. The process-model can then be used, not only for improving performance through the Perimeta process, but also to inform other organisational processes, such as Knowledge Management studies.

In the context of the water supply function, relevant processes include:

- Water source management
  - Understanding source characteristics and quality and their variability
  - Predicting and meeting supply/demand requirements
  - Managing the Environmental effects
  - Meeting legislative demands

- Water treatment
  - Controlling costs
  - Carrying out treatment stages
  - Analysing input water quality
  - Meeting supply/demand requirements
  - Obeying legislation

- Water distribution
  - Managing trunk main system
  - Providing sufficient storage for requirements
  - Distributing water through mains
  - Meeting supply/demand requirements
  - Predicting demand growth
  - Following legislation
  - Managing network
  - Assigning capital and operational maintenance

- General
  - Identifying business values
  - Satisfying investment drivers
  - Delighting stakeholders
  - Meeting Industry Regulations

The Perimeta methodology is applied through a combination of Performance Indicators (PIs), Value Functions and Figures of Merit as described below, and generates a view of performance in a graphical manner that highlights areas of uncertainty.

4.2 *Performance indicators (PIs) and evidence*

The assets involved in water supply are complex and varied. Therefore, the performance indicators used in their management can be difficult to compare. Unlike some engineering assets, failure of many elements cannot be tolerated. This is because the cost of failure, in lost customer confidence, compensation, or even, in the case of safety-critical assets, such as dams, loss of life, is an unacceptable cost. When looking at the assets in terms of performance, it becomes clear that an element cannot be considered to be performing well simply because it is not in a state of failure. Performance must be measured against an asset meeting some stated performance objective. The degree to which the asset is meeting these objectives can be assessed through the evaluation of various Performance Indicators (PIs), which are in effect variables that describe the state of the system (system state variables).

Performance Indicators can be assigned to any particular process and, where a number of PIs are associated with one process, they can be weighted according to their relative importance. In addition, there may be some higher level PIs (sometimes called KPIs – Key Performance Indicators) that are only measured at the top level of a system. Lower level indicators for a water supply system might include assessments of the condition of part of a network, or a gauge measurement of the actual thickness of some critical element. Higher-level indicators might include, for example, customer satisfactions scores, water quality ratings and so on. A useful element of work that can be carried out is to prepare a Cascade of Performance Indicators. These are useful for checking that lower level performance indicators are in line with the organisation's objectives. Also, lower level performance indicators can be propagated up through the hierarchy to give an overall indicator of performance. This can be done by applying weightings between processes and propagating them through these, or, where appropriate, simply summing the raw values (for example the number of near-misses of injuries for each process may be totalled to give an overall PI for the near-misses company-wide). Higher-level indicators tend to relate to the organisation's financial performance, health and safety record, or compliance with statutory demands, while lower-level PIs are generally more closely linked to the physical performance of an asset.

It must not be forgotten that expert judgement is another form of evidence that must be considered alongside numerical PIs. This is a key function within the Perimeta methodology.

4.3 *Value functions*

In order to compare PIs it is necessary to map the measured (or linguistic) evidence through a value function to produce a dimensionless value. These are then represented as Figures of Merit, as described in section 4.4. Six generic value functions are used to represent performance. These can be thought of as "performance curves". In five cases expert judgement is used to translate the numerical performance indicator to a value of performance. These five functions are defined by their shape: "linear", "stepped", "convex", "concave" and "s-shaped". The sixth function is used to express uncertain expert judgements. These are expressed in the format "judgement, certainty" (e.g. "good performance, low certainty"), and, once "translated" through the linguistic value function, can be expressed as a Figure of Merit in just the same way as the numerical PIs are shown. This makes it possible for several PIs to be connected to one process holon (weighted as appropriate). The Figure of Merit is explained in the following section.

The value function is chosen using expert judgement as being the best representation of how the system is performing with reference to a particular piece of evidence. This evidence could be, for example, the way that performance declines with age. However, using the six functions available, any type of evidence (including judgement) can be mapped in this way.

4.4 *Figure of merit*

The Perimeta methodology can be used to demonstrate visually how a system is performing. This is done by "translating" performance indicators into figures of merit through a value function. A number of performance indicators can then be weighted against one another and attached to any

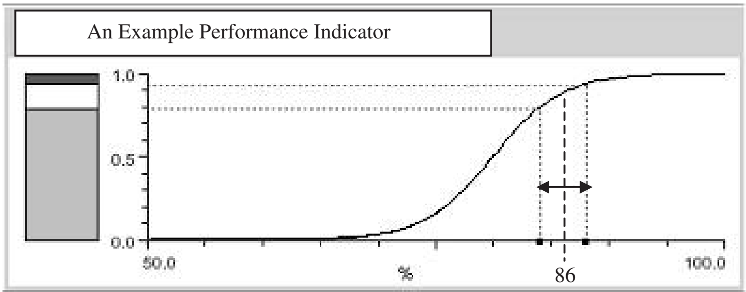

Figure 2. Using a value function to evaluate a figure of merit of one Performance Indicator.

given process (or number of processes) to give a figure of merit for that process. The Figure of Merit (FM) is frequently referred to informally as "The Italian Flag" (Blockley et al. 2000) and is used to represent the performance of a process. Each PI has a value function and several VFs can be combined together to illustrate the overall performance of any holon. Many processes within the system will have more than one PI related to them. The FM is made up of three colours, green (left hand lighter shade), red (the darkest block) and white (centre). The green represents all the evidence that is available that a process is performing well, while the red represents evidence of unsatisfactory performance, and the white represents the uncertainty in the performance of the holon.

In the example above, consultation with experts has revealed that, for a particular performance indicator, a value of 90% would be very good, while a value of 75% or less would be represent near failure. The actual value was 86% plus or minus 2%. Evidence of good performance is represented by the green (lower grey) part of the bar on the left, while red (dark grey) indicates evidence of poor performance. The area of white corresponds with the level of uncertainty. Hence, in the example above the level of performance may be up to 0.92 "green" or as much as 0.21 (1–0.79) "red", with an uncertainty of 0.13 (0.92–0.79).

When more than one PI is associated with any particular holon, it can be weighted according to its relative importance. For example, a PI related to "safety" might be weighted much more highly than one giving an indication of cost. The case study below demonstrates how these individual figures of merit are brought together into an overall view of the complete process with an indication of the criticality of each sub-process.

4.5 *Multi-attribute views of system performance*

The top right-hand corner of Figure 1 contains four views of the system. The three over-lapped windows represent a view of performance with regard to an area of particular interest (e.g. cost, the Environment, safety). The value functions for a particular PI can be varied for each of these views to allow for the fact that while a particular measure, such as "overtime" may have a value that indicates good performance in one area (output, for example) it may suggest poorer performance with regard to another area of interest (e.g. cost).

4.6 *Handling uncertainty and propagating evidence*

The PIs and figures of merit have uncertainty associated with them which arises both from the way in which performance is measured and from the way in which it is valued (as represented by the value function). These can both be accounted for by setting bounds of uncertainty for the measured value of the PI and the shape of the value function used. This uncertainty is handled through the value function and illustrated by the white part of the "Italian flag".

Evidence of performance can be recorded and interpreted locally at any level within the hierarchy, but it can also be propagated up through the system using an inference mechanism for probability intervals (Hall et al, 1998). Figures of merit are calculated for the locally inputted evidence and for evidence propagated from lower levels of the system to give a combined figure of merit for any particular process.

5 THE CASE STUDY

5.1 *Constructing the model and locating the data*

The methodology outlined above is considered suitable for application to the asset management of water and waste water systems. It needs to be said at this point that the application of Perimeta to these sectors is in its infancy. However, the authors feel that the approach offers considerable potential to both the water supply and the wastewater functions. This case study focuses on wastewater systems, simply because development in the model building is more advanced. Nonetheless, the fundamental principles relating to process model building and data appraisal are considered directly transferable to water supply systems.

In order to test out the methodology a simplified model of a hypothetical wastewater catchment, including pumping stations, was developed for eventual application to real systems. The criterion used was that the model should demonstrate the processes that the catchment is designed to fulfil, i.e. transportation of sewage to treatment works, with these linked to particular assets where appropriate. At higher levels, processes such as company overall performance on the wastewater service can be identified. At lower levels, processes such as "providing sufficient clearway capacity" or "providing sufficient pumping power" are associated with physical assets such as "sewers", "rising mains" and "pumps". It has been recognised that physical assets are not purely inanimate objects but that the processes which they enact require the contribution of associated control systems and, most importantly the knowledge and skill of the people who are responsible for operating, maintaining, inspecting and refurbishing these assets. Thus the process-orientated model is essential for capturing the important elements of the system, both "hard" and "soft". (Related discussion can be found in Hall et al, 2002b with reference to a model built for hydroelectric power generation.)

The top levels of the model can be defined by considering the vision of the organisation in question. For example, this might be "to be the most efficient, effective and environmentally-sensitive water company in the UK". Then, by using the principles of Value Engineering and repeatedly asking the question "How?", the model can be constructed in a top-down manner through interviews and workshops with those people who manage the system.

For the wastewater scenario, the lower levels in the hierarchy were developed through the use of narrative and storyboarding; a method of describing the system through exploring the processes which must take place for the system to be successful. Questions such as "For process X to perform well, what other processes need to be performing well?" were used to identify the processes that were required to satisfactorily deliver the higher level process of "transferring sewage to the treatment works and eventually to meet the top objectives of the company. The work also took advantage of the findings of a project being undertaken by FaberMaunsell staff at the same time. This project involved using failure modes and effects analysis to carry out a risk assessment for a sewerage system. Thus, the risk analysis (if this fails, the following process will fail) also informed the model-building process for the Perimeta model.

5.2 *Determining value functions and adding performance indicators*

The wastewater processes were in general represented by an s-shaped function, this being considered most applicable to the particular situations being modelled. This function was useful because in many cases there was some "soft" step point at which performance began to drop off slowly. For example, a value of 80% might be considered to be 50% good (Score of 0.5), while a value of 85% might be very much better, and a score of 75%, very much worse. At the same time, once a performance indicator value of, say 90% was achieved, very little extra performance would be gained through increasing that value towards 100% (indeed it might not be cost-effective or best value for

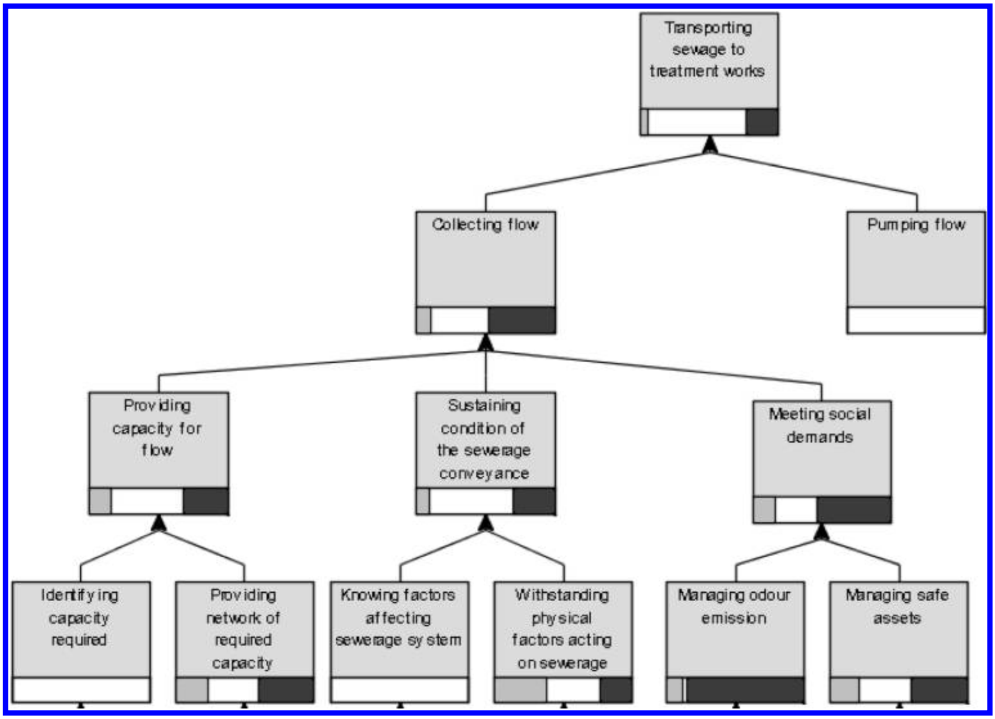

Figure 3. Part of the case study Perimeta model.

money). Similarly, once the value had fallen to, say 60%, the performance would be so bad that further deterioration would have almost no perceivable impact on performance.

In other cases, such a "soft" step was not required. This would be the case where a particular performance indicator value results in a total "switch" in performance. For example, a record of zero deaths in one year would result in a score of 1.0 (best possible performance), but, since one death on site is unacceptable, a performance indicator value of 1 or more would result in a performance score of 0.0 for that PI.

Other performance indicator shapes can also be useful. For example, an organisation may feel that it is very important that safety inspections are carried out on time. Of course, an indicator such as "percentage of inspections carried out on time" is common, although it could be improved if the number of days was also considered (so that one or two inspections are not allowed to become overdue by many months). An indicator of "total days by which inspections were overdue" might be at least as useful. In this case, a concave VF with a negative gradient might be appropriate, so that performance falls away very quickly as the number of days by which inspections were overdue moves away from the optimum, zero.

5.3 *Benefits of the model*

There are several key benefits that could result from the application of a model of this type. The most obvious of these is that the completed model provides a record of what the organisation does, how it is doing it, and how the various processes link in to the top-level "vision". Secondly, by associating the existing performance indicators with the model it is possible to determine, in a highly visual manner, which areas of the system are currently being monitored (e.g. there is some red and green) and which areas are not fully understood. Since part of the work is to consider the resulting risk of poor performance (and uncertainty), it is important to weight the links between the various processes according to the criticality of that process to the success (or failure) of the holon above. In this way, if a particular asset is found to be critical to the success of the system any "red" associated with that holon will be propagated "strongly". In a similar way, it is also possible to demonstrate where the good performance of a particular process is not particularly necessary for success overall. This finding can be used as a justification for not putting additional monitoring in place in a particular area, but instead investing the resources saved into the remediation or monitoring of another aspect of the system.

The flexibility of the approach, with regard to weighting particular aspects of performance (e.g cost, Environment and so on) is particularly useful in a sector where the demands of stakeholders (e.g. customers, shareholders, the Regulator) are so complex and potentially conflicting. Ultimately, a model of this type could prove useful for communicating with the various stakeholder groups. This could be done for an organisation as a whole, or with regard to a particular decision (e.g. a planning application).

Finally, the model provides a platform for testing potential interventions to the system. Because the Figures of Merit propagate up through the model, it is possible to investigate how a particular change at a lower level (such as discovering that the "white" of one process should be the "red" of failure) affects the performance of the system as a whole. In this way, choices can be made, and defended, with regard to determining monitoring, remediation and capital investment works.

6 CONCLUSIONS

The case study outlined in this paper is for a wastewater system. However, the principles can clearly be transferred to the area of water supply. The Perimeta methodology has been applied alongside a risk analysis framework to identify areas of good, poor and uncertain performance for a hypothetical wastewater system (including pumping stations).

It is clear that the process model required is best constructed through the use of experts working together in a group workshop (supplemented by one-to-one and desk studies). In this way, a robust and defensible model can be built that includes weightings between one process and another. The use of these weightings enables scenario testing to be carried out. In practice, this would mean that a particular maintenance regime, or a change in monitoring practice could be visualised through the effect that it would have on related performance indicators (and, through propagation upwards, the effect that this would have on the system as a whole).

The main benefit of all this is that decisions can be made in a robust and defensible manner, through the consideration of both quantitative and qualitative evidence. Even data that contain a high degree of uncertainty (through collection gaps, poor collection practices, etc.) can be used to support the decision, since the uncertainty can be captured in the model. In addition, the simple "Italian Flag" view means that performance can be communicated with all stakeholder groups, regardless of their technical ability or particular viewpoint. Thus, communication with account managers, the Regulator and the general public can be facilitated. Particular aspects of performance (e.g. Environment, cost, safety) can be viewed in isolation where appropriate, while still maintaining an overview of the performance of the system as a whole.

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REFERENCES

Baker-Langman, E.J., Hall, J.W., Taylor, C.A. and Davis, J.P. (2002). Evidence-based Decision Support Tool for Management of Complex Infrastructure Systems. *DMInUCE Conference Proceedings (Electronic)*, London.

Blockley, D.I. and Godfrey, P. (2000). *Doing it Differently Systems for Rethinking Construction.* London: Thomas Telford.

Cui, W.C. and Blockley, D.I. (1990). Interval probability theory for evidential support. *Int. J. Intelligent Systems, 5*, 183–192.

Checkland, P. (1981). *Systems Thinking, Systems Practice.* New York: John Wiley & Sons Ltd.

Davis, J.P. and Hall, J.W. (1998). Assembling uncertain evidence for decision-making. *Proc. 3rd Int. Conf. on Hydroinformatics, Rotterdam*. 1089–1094.

Hall, J.W., Blockley, D.I. and Davis, J.P. (1998). Uncertain inference using interval probability theory. *Int. J. Approximate Reasoning, 19(3–4)*. 247–264.

Hall, J.W., Le Masurier, J.W., Baker, E.J., Davis, J.P. and Taylor, C.A., (2002a). A decision-support methodology for performance-based infrastructure management. *ASCE Journal of Infrastructure Systems*. 37–48

Hall, J.W., Le Masuier, J.W., Baker, E.J., Davis, J.P. and Taylor, C.A. (2002b). Multi-attribute performance monitoring for reservoir systems. *Reservoirs in a changing world – Proceedings of the 12th conference of the BDS held at Trinity College, Dublin (ed. Tedd, P.)*: 534–550.

Rummler, G.A. and Brache, A.P, (1995). *Improving Performance: How to Manage the White Space on the Organization Chart*. Second Edition. San Francisco: Jossey-Bass Inc.

Wymore, A.W. (1993). *Model-Based System Engineering,* Raton: CRC Press.

Strategy for improving the performance of trunk main systems in water supply networks

A. Wallen
Thames Water Research and Technology, Spencer House, Reading, UK
Department of Systems Engineering, Brunel University, Uxbridge, UK

C. Ashton & C.A. Woodward
Thames Water Research and Technology, Spencer House, Reading, UK

R. Powell
Department of Systems Engineering, Brunel University, Uxbridge, UK

ABSTRACT: The primary objective of a water supply network is to provide a constant, reliable supply of potable water, and to enhance reliability levels many contain multiple transmission routes, resulting in heavily meshed topologies.

Whilst it is generally believed that multiple supply routes increase performance reliability, or security of supply, certain disadvantages remain. Additional pipes add to the amount of infrastructure which may leak or cause a burst event, especially where the increase in the number of supply routes has been achieved by retaining older pipes that may be in poor condition. Increased residence time for water within the network could also give rise to undesirable water quality characteristics.

A methodology has been developed which can be used to estimate the impact to overall network performance of abandoning mains in poor condition. Deterministic simulation techniques are carried out on a trunk main system within a water supply network to assess network supply security, flood risk, leakage and water residence times, which can then be combined to provide a value for overall network performance.

1 INTRODUCTION

This paper presents a methodology for estimating the performance of the trunk main systems that transmit water across water supply networks. The prevailing industry premise is that each trunk main in current operation increases network stability and performance. A detailed examination could demonstrate that there are some mains which could be taken out of service without serious consequences to the security of supply. Such a move could raise the standard of overall network performance through reduction of network flood risk, leakage risk and water age.

The primary function of a water supply network is to provide water of adequate pressure, flow rate and quality to customers. Network reliability, or supply security, can be seen as a measure of how well this service is performed, and a system is said to be reliable if it functions properly for the specified evaluation time (Ostfeld 2001).

However, it is no longer adequate to judge system performance by supply security alone. In UK, the demand for clean water is increasing due to population growth and the rise in per capita water consumption (Engelhardt et al. 2000), while the Environment Agency has identified areas of England and Wales where they believe that water abstraction regimes may already be damaging the environment (Environment Agency 2001). In order to help satisfy the increasing demand for clean water without unduly raising abstraction levels, the UK water industry regulator, Ofwat, has imposed targets for each water supply company to reduce system losses.

Losses from trunk mains may be divided into two categories. Corrosion holes, joint failures and lesser fractures give rise to subsurface leakage, which can be difficult to detect, whereas bursts can be said to occur where a pipe fails to the degree that the pressure in the pipe forces water to the ground surface. The high pressures in trunk mains mean that a burst may cause significant surface flooding, which can lead to traffic disruption and property damage. Any main which could cause damaging floods in the case of a burst could be a prime candidate for abandonment, if the option were feasible while maintaining reliable supply.

The purpose of this research is to build a network optimisation tool to aid decision makers working in asset management in the water supply industry. The network optimisation tool would investigate the impacts, in terms of network performance, of reconfiguring a trunk main system by removing selected high risk mains from service.

2 LITERATURE REVIEW

2.1 *Reliability analysis*

There is a lack of agreement in the field of water distribution system research as to how to define or calculate network reliability (Ostfeld & Shamir 1996, Xu & Goulter 1998, Shin & Park 2000). The definitions and methodologies which are available can broadly be divided into three categories: (1) Topological analysis (Ormsbee & Kessler 1990) covers methods where reliability is assumed if a designated surrogate "topological" measure is satisfied; (2) simulation analysis seeks to reduce the computation time required for deterministic analysis through hydraulic analysis of a network reduction or schematic, or quasi-hydraulic analysis of the full network; (3) deterministic analysis involves modelling the flows and pressures within the network using the non-linear hydraulic equations, in order that network reliability can be calculated explicitly.

2.1.1 *Topological analysis*

Lee (1980) introduced the concept of *network connectivity* as a measure for analysing water distribution system reliability. Connectivity analysis enumerates the pathways connecting a system source to a customer or system demand node, for each demand node in the network; the more pathways exist, the more reliable a system is seen to be. Network redundancy is an alternative definition for the existence of more than one adequate path between source and demand nodes in a hydraulic network (Ormsbee & Kessler 1990) while reachability is defined as the probability that all nodes are connected to at least one source (Wagner et al. 1986).

Hydraulic connectivity analysis was developed to enumerate the number of hydraulically capable network paths between the source and demand nodes (Goulter & Coals 1986, Shin & Park 2000). Similarly, Awumah et al. (1991) developed entropic redundancy, where the head loss in each pipe is used as a measure for the hydraulic capability of different supply routes.

All topographical analysis methods are based on enumerating the number of independent supply routes available between source and demand node as a surrogate measure for system reliability. The assumption is that increasing the number of pipe routes increases the supply security. The methodology presented in this paper seeks to investigate this assumption to see whether it is possible to remove one of a set of multiple pipe routes without significantly decreasing network supply security, while increasing overall network performance. Topographical methods are therefore not appropriate for this purpose as a more detailed breakdown of the effect of pipe removal is required.

2.1.2 *Simulation analysis*

Hybrid methods have been developed over the last decade, where system reliability is directly calculated from nodal pressures, but the model used to predict the pressures is not fully deterministic. Xu & Goulter (1998) developed a linear probabilistic model where the expected value of pipe demands and coefficients are used in a linearised model of the system to predict the distribution of nodal heads. Ostfeld (2001) uses stochastic simulation of failures in a schematic of the distribution network to provide results which are evaluated by comparison with iso-reliability lines developed from lumped-supply-lumped demand analysis.

The drawback to this type of methodology lies in the lack of transparency. Where asset management decisions are to be based on the results from a system, it is necessary that the errors arising from the various assumptions can be calculated and understood by the decision maker. Increasing the number of assumptions on which the system analysis is based can increase the complexity of error, making this type of technique less appropriate for decision support models.

2.1.3 *Deterministic analysis*

A typical procedure involves repeated modelling of the flows and pressures around a network after simulation of random mechanical component failures, using the non-linear hydraulic equations. The results are then evaluated to provide a measure for system reliability, using whichever performance indices are appropriate for the methodology (Bao & Mays 1990, Fujiwara & Ganesharajah 1993). The main drawback to deterministic analysis is that it is computationally intensive; however, the increase in computer power over the last two decades, the relative simplicity of the networks to be considered with this method, and the transparency required in order that fully informed decisions can be based on the results make this type of method the most appropriate for this analysis.

2.1.4 *Types of failure*

Reliability analyses require a strict definition of the types of failure that shall be included in the evaluation. There are two basic types: mechanical or hydraulic failure (Bao & Mays 1990). Mechanical failure occurs where a piece of infrastructure does not operate correctly, due either to component failure, or failure of the power supply, e.g. for pumps. Hydraulic failure occurs where demand exceeds network capacity, which may be due to flow requirements for firefighting (Cullinane 1986), droughts, quality assurance at peak and night flows (Coelho 1997), or the decrease in mains capacity due to tuberculation (Xu & Goulter 1998). This paper is limited to assessing the impacts of the mechanical failure of trunk mains.

2.2 *Network losses and damage from burst flooding*

Network leakage analyses deal mainly with methods for estimating system losses (Joy & Oakes 1999) or methods and equipment for locating subsurface leakage while on the street (Golby & Woodward 1999, Fuchs & Rielhe 1991). Lambert (1994) developed a modelling technique in which losses are categorized by flow rate, with a division at 0.5 m^3/h between background losses and bursts. He also derived formulae from technical reports to estimate trunk main background losses, and approximated trunk main burst frequencies from company records and leakage control reports.

Habibian (1991) describes possible reasons why trunk mains fail although he also states that further research is required to fully understand failure mechanisms. Mechanisms are seen to depend principally on the pipe material, year laid, pressure within the pipe, surrounding soil type and traffic loading (Laske 1989, Aikman 1993, Blakey et al. 1999, Marshall 2001). It may be possible to develop a leakage rate probability for a pipe from the dependent variables listed above, adjusted with company data records, similarly to Lambert (1994).

The effect of flooding from trunk mains has been investigated by Blakey (1999), where a runoff model is used to allocate a burst consequence score to each pipe main. The methodology could be adjusted to estimate curves for the damage and disruption levels caused by each burst main, dependant on the flow rate of the water escaping.

2.3 *Water quality and water age*

The literature for the study of water quality modelling for water supply systems is extensive, and a summary of the principal methods can be found in Maier (1999). A full water quality model would require too much computation time to be feasible for inclusion in this model, but water age has been suggested as a surrogate measure for water quality analysis (Coelho 1997, Engelhardt 2000). Cardew et al. (2000) provide validation of water age modelling where they report that good correlation can be obtained between the water age predicted by modelling and water age found using tracers in an actual system.

2.4 *Network optimisation*

Reported methods for selecting the optimal design for a water supply network commonly balance infrastructure costs and an appropriate level of reliability (Awumah & Goulter 1992, Ostfeld & Shamir 1996). The author has found no published research that seeks to improve the reliability of an existing network by reducing the amount of aging infrastructure within the network. There are no optimisation studies which take into account the financial, social and environmental costs of increased water residence time, burst flood damage or water losses.

3 METHODOLOGY

A trunk main system, or transmission main system, serves to deliver water across a supply network from treatment works to municipal distribution areas. A system consists of sources (treatment works, service reservoirs), the network of transmission mains, valves, some storage tanks, and demand nodes from which distribution mains connect to the municipal distribution networks. Although different municipal areas may be interconnected through the distribution system, as well as through the transmission network, the prevalence of zonal and district metering has ensured that these areas can be isolated at distribution level. The ability to model a trunk main network as an isolated system is the key to the methodology presented in this section.

The methodology consists of two iterative stages: (1) simulation of the network in TransNet, an adaptation of the public domain software EpaNet, developed by the Author for analysis of system performance under mechanical pipe failure conditions, and (2) use of a genetic algorithm to create generations of sample "reduced" trunk main systems through manipulation of the original network. After stage (2) the sample networks are returned to (1) for analysis.

Stage (1) may be subdivided into five steps, the first four each relating to one of the system performance variables: network supply security, flood risk, leakage risk and water age risk. In step five, cost functions are allocated to the four variables so that they may be combined to give a single network performance score. Stage (2) is subdivided into: (i) selection of the fittest networks to parent a new generation, using the performance score, and (ii) manipulation of the selected networks to create a new sample generation.

This paper describes the methodology on which TransNet is based, focusing on the methods by which the four performance factors are evaluated and combined.

3.1 *Network supply security*

Germanopoulos et al. (1986) first introduced security of supply as a distinct term to define the probability that a network can supply all customers with an adequate water supply throughout an evaluation period. The term was used to separate mechanical system reliability: the probability of components not failing, from hydraulic system reliability: the probability that a network can continue to provide satisfactory service despite mechanical failure.

Network supply security, $0 \leqslant W_s \leqslant 1$, can be defined as the probability that a network will provide water of adequate pressure and flow rate throughout the evaluation time period. Water quality is dealt with separately within the water age risk variable. Pipe failure probability, B_k, is defined for supply security evaluation as the probability of a burst occurring on a main within a fixed period (e.g. a year) and can be estimated from pipe failure records for an existing system (Lambert 1994). The consequence to supply of each pipe failure, S_k, can then be assessed in terms of the severity and duration of customer supply shortfall while the pipe is first isolated and then repaired. Network supply security is evaluated from pipe burst probability and consequence using equation 1.

$$W_S = 1 - \frac{1}{M}\sum_{k=1}^{M} B_k \cdot S_k \tag{1}$$

where M = total number of network pipes k.

In order to model and evaluate the consequence of trunk main burst to the customers, a demand allocation is required at each distribution main connection equal to the lumped demand across that distribution zone. A 24-hour demand pattern is consistent across the network. The number of households supplied within each zone, C_n, is also recorded as a parameter of the demand node on the trunk main. Connections for large commercial customers (e.g. breweries) are weighted according to volume required, and essential connections such as those supplying hospitals or dialysis patients are also allocated a criticality weighting.

$$S_K = \frac{1}{NT}\sum_{n=1}^{N}\sum_{t=1}^{T} C_n \cdot V_{n,t} \tag{2}$$

where N = total number of network nodes n; T = total number of evaluation time steps t.

Consequence is calculated for a burst occurring at the period of peak demand using equation 2, where the flows and pressures across the network are simulated for the period before the burst is isolated, and the subsequent period while the pipe is closed and the burst is being repaired.

After modelling the nodal pressures in the trunk main system a performance violation, $V_{n,t}$, can be calculated for each demand node, for each time step within the total evaluation period, using the equation for the curve in figure 1.

Evaluation of the probability and consequence of failure is carried out for each pipe and aggregated across the network to give the network supply security. Although this method is computationally intensive, the small size of trunk main networks makes it feasible. However, as trunk main bursts can also have serious consequences at ground surface level due to the large volumes of water escaping, network supply security evaluation is not sufficient to give a comprehensive assessment of the performance of a system.

3.2 *Network flood risk*

The methodology used by Blakey et al. (1999) can be adapted to allocate two burst flood consequence coefficients: a damage coefficient, λ_k, and a disruption coefficient, τ_k, to each pipe in the network.

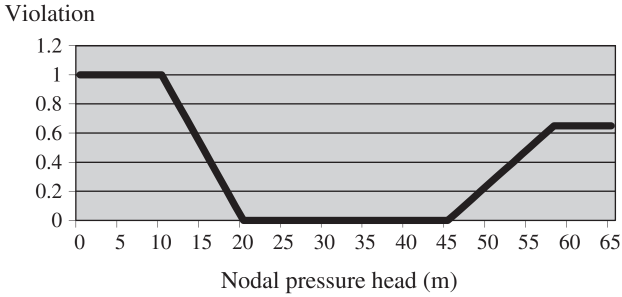

Figure 1. Performance curve for nodal pressure head.

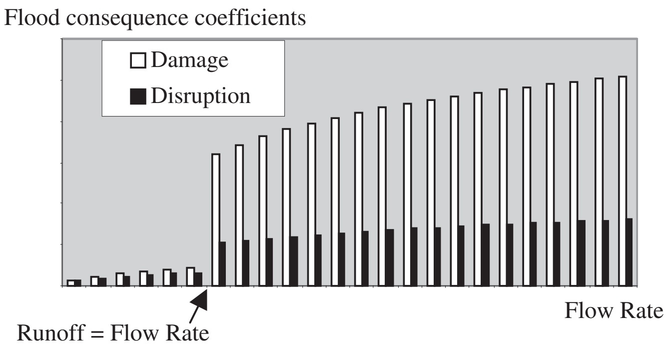

Figure 2. Damage and disruption as a function of burst flow rate for a pipe k.

The flood consequence curves for each pipe are calculated in the runoff model by estimating the flow rate from the burst, Q_k, and simulating surface dispersion for the volume that escapes over the time to isolate the main. A topological surface map superimposed over the network is used to evaluate runoff vectors and velocities, potential sinks, and damage that could be caused en route from burst to sink. Flood consequence curves for each pipe (Fig. 2) would only need to be calculated once, for repeated use in the performance model, and therefore this method minimizes use of the complex topological runoff model.

Burst records indicate that the majority of damage caused by flooding from a burst pipe occurs within the initial period after the burst first occurs, and therefore the damage coefficient is to a great extent not dependent on the duration of the burst flow, D_k. Disruption continues while the burst is flowing, and the total disruption level is therefore time dependent. The flood consequence coefficients for a particular burst flow rate are found from figure 2 and the network flood risk, W_F, is calculated using equation 3.

$$\boxed{W_F = \sum_{k=1}^{M} B_k \cdot \left(\lambda_k(Q_k) + \tau_k(Q_k) \cdot D_k\right)} \tag{3}$$

Reducing the number of mains in a network, and therefore reducing the number of pipes which may burst, will improve overall network performance by reducing the network flood risk.

3.3 *Water age*

The concentration of disinfection residuals in a water supply network decreases the longer the treated water remains in the network, and the rate of decrease is dependent upon the type of disinfectant used. A water quality performance index for each node is taken from a water age-probable deterioration in water quality curve such as figure 3, which can be selected or input by the operator.

A value for the network water age risk, $0 \leqslant W_A \leqslant 1$, can then be calculated using equation 4.

$$W_A = \frac{1}{NT} \sum_{n=1}^{N} \sum_{t=1}^{T} Q_{n,t} \tag{4}$$

where $Q_{n,t}$ = probable deterioration in water quality at node n at time t.

Threshold limits set by the operator also restrict the number of hours for which Q_n at any node can exceed 0.5, 0.75 or 1. If these limits are exceeded, then the layout under consideration is not considered adequate for the existing dosing plan, and the network water age risk is automatically set to 1.

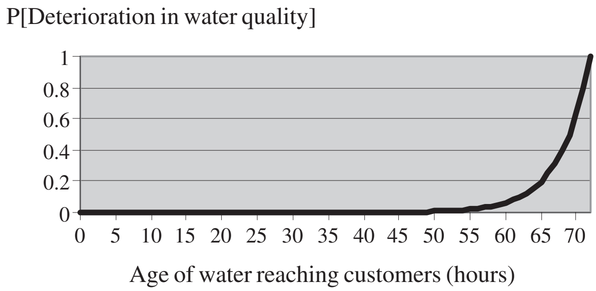

Figure 3. Sample curve relating nodal water age to the probability of water quality deterioration.

3.4 *Leakage risk*

Leakage levels within a supply network can be inferred from analysis of the difference between the metered input and the water delivered to customers, after allowing for operational use losses (e.g. for mains flushing) (Lambert 1994, Golby & Woodward 1999).

Assessment of whether a pipe is likely to be leaking can be carried out with reference to condition data and records of previous failures. Elements from Marshall (2000), Blakey et al. (1999) and Aikman (1993) are used to develop a coefficient of pipe deterioration, A_k for pipes of various material, buried in different soil types.

$$R_K(t) = R_K(t_L)e^{A_K(t-t_L)} \tag{5}$$

where R_K = rate of leakage per km of pipe k; t_L = year the pipe was laid; Mains leakage rates are taken to generally increase exponentially with time (Eqn. 5).

The network leakage risk, W_L, can then be calculated using equation 6.

$$W_L = \sum_{k=1}^{M} R_K \cdot L_K \cdot T \tag{6}$$

where L_K = length of pipe k.

Network leakage risk will decrease with a reduction in the number of pipes in the network.

3.5 *Network performance*

An appropriate combination of the performance variables should require that each is converted into cost units, as many operational decisions are largely driven by cost considerations and the decision maker will be familiar with this mechanism.

However, before the costs associated with any network reconfiguration are considered, the adequacy of the network in terms of fulfilling statutory requirements is first evaluated from the figures given for network supply security and network water age risk. If either falls outside the required standard, the proposed configuration is abandoned and further cost calculations are not required.

Certain externalities are included in the network performance cost appraisal such as the social effects of being cut off from a water supply (often related to goodwill), and the environmental benefits derived from reducing network losses, and thereby decreasing the abstraction levels required to deliver a set volume of supply. A currency unit measure for these externalities can be assigned after careful appraisal, using a method such as that described by Hanley & Spash (1993), although the subjective nature of these measures should be highlighted to the asset manager using this decision tool.

The costs associated with each of the four performance variables are combined to give a network performance score for any configuration of the network. As abandoning a main in poor condition is likely to have significant impact through reduction in costs due to flooding and water losses, mains with a history of poor performance are most likely to be targeted by this methodology. Where a main has a history of repeated bursts, removal from service may also have beneficial impacts on the overall network supply security, as that main can no longer burst and cause supply disruption. The performance scores obtained through this methodology are available for stand-alone interpretation, or may be passed to a genetic algorithm for comparison with alternative potential network reduction schemes.

4 CONCLUSIONS

Supply system reliability is commonly measured in terms of the ability of the system to provide adequate supply to meet demand at the system delivery points. It is generally accepted within the system reliability research field that reliability, or supply security, may be improved by increasing the number of paths which can be used for transmission. However, supply security is affected by the mechanical reliability of the system, which represents the probability of the components continuing to operate throughout the evaluation time period. Clearly, while increasing the number of network paths should increase the security of supply, the mechanical system reliability will decrease as the number of components that may fail rises. This paper describes a decision model that has been designed to investigate the key impacts arising from reconfiguring a trunk main system to abandon pipes with a poor service record. The model must incorporate supply security and water quality assurances while investigating possible economic, environmental and social improvements arising from the reduction of network losses and number of burst events.

Numerous models and methods have been reported in the literature, although no universal agreement exists as to how network performance should be evaluated. This methodology includes several performance variables and assessment methodologies developed from different areas of the field of water supply network research. The combination of the key performance variables in one methodology should make this tool more applicable for analysis of existing networks, and the development of a cost analysis of the overall performance allows ease of interpretation of the results and comparison between different solutions. Data from one of the trunk main systems within London will be provided by Thames Water Utilities Ltd in order to test the tool.

REFERENCES

Aikman, D. 1993. Asset appraisal of trunk mains, *J IWEM* 7: 37–43.

Awumah, K., Goulter, I.C. & Bhatt, S.K. 1991. *Entropy based redundancy measures in water distribution networks*. *J Water Resources Planning and Management* Division ASCE, 117 (5), 595–614.

Bao Y. & Mays L.W. 1990. Model for Water Distribution System Reliability. *J Hydraulic Engineering* 116 (9): 1119–1137.

Blakey, G., Ta, T. & Sherwin, C. 1999. Delivery of the GIS trunk mains burst risk model. *Proc. '99 AGI conference,* London, September.

Cardew, P., Grayman, W., Marshall, J. & Marshall, P. 2000. Aspects of modelling chlorine decay in water distribution systems. *UK Water Industry Research Ltd.* Report Ref. No. 00/DW/03/15. Drinking Water Quality and Health: Distribution system.

Coelho S.T.1997. *Performance in water distribution – a systems approach*. Taunton, UK: Research Studies Press.

Cullinane, M.J. 1986. Hydraulic Reliability of Urban Water Distribution Systems, *Proc. ASCE Water Forum*, California.

Engelhardt, M.O., Skipworth, P.J., Savic, D.A., Saul, A.J. & Walters, G.A. 2000. Rehabilitation strategies for water distribution networks: a literature review with a UK perspective. *Urban Water* 2: 153–170.

Environment Agency 2001. Water resources for the future. *Environment Agency.* Bristol, UK.

Fuchs, H.V. & Riehle, R. 1991. Acoustical Analysis Detects Leaks. *Water/Engineering & Management*, January.

Fujiwara, O. & Ganesharajah, T. 1993. Reliability assessment of water supply systems with storage and distribution networks. *Water Resources Research*, 29 (8): 2917–2924.

Germanopoulos, G., Jowitt, P.W. & Lumbers, J.P. 1986. Assessing the reliability of supply and level of service for water distribution systems, *Proc. Institution Civil Engineers*. 1(80): 413–428.
Golby, J. & Woodward, T. 1999. Find that leak. *IEE review*. September.
Goulter, I.C. & Coals, V. 1986. Quantitative approaches to reliability assessment in pipe networks, *J Transport Engineering* Division ASCE, 112 (3): 287–301.
Habibian, A. 1991. Leak and Corrosion Monitoring. *Pipes & Pipelines International*. January-February.
Hanley, N. & Spash, C. 1993. Cost benefit analysis and the environment. *Edward Elgar Publishing Ltd*. England.
Joy, J. & Oakes, A. 1999. Leakage estimation from night flow analysis. *UK Water Industry Research Ltd.* Report Ref. No. 99/WM/08/21. Leakage.
Lambert, A. 1994. Accounting for Losses: The Bursts and Background Concept. *J Institution of Water and Environmental Management* 8: 205–214.
Laske, C. 1989. Recent techniques in leak detection. *Water Supply* 7(2–3).
Lee S.H. 1980. Reliability evaluation of a flow network. *IEEE Trans Reliability*, R-19 (1) 24–26.
Maier, S. 1999. Modelling Water Quality for Water Distribution Systems. *Thesis, Brunel University*, Uxbridge UK. February.
Marshall, P. 2001. Understanding Burst Rate Patterns of Water Pipes. *UK Water Industry Research Ltd*. Report Ref. No. 01/WM/02/16. Water Mains, Services and Sewerage – Pipeline Innovation.
Ormsbee, L. & Kessler, A. (1990). Optimal upgrading of hydraulic-network reliability. *J Water Resources Planning and Management* Division ASCE, 116 (6): 784–801.
Ostfeld, A. & Shamir, U. (1996). Design of optimal reliable multiquality water-supply systems. *J Water Resources Planning and Management* Division ASCE, 122 (5): 322–333.
Ostfeld, A. 2001. Reliability analysis of regional water distribution systems. *Urban Water* 3: 253–260.
Shin, H.G. & Park, H. (2000). An optimal design of water distribution networks with hydraulic-connectivity, *J Water Supply: Research and Technology-AQUA*, 49. 4, 2000.
Wagner, J.M., Shamir, U. & Marks, D.H. (1986). Reliability of Water Distribution Systems. Report No. 312, *Ralph M. Parsons Laboratory, MIT*, Cambridge, Massachusetts, pp 150.
Xu, C. & Goulter, I.C. (1998). Probabilistic model for water distribution reliability. *J Water Resources Planning and Management* Division ASCE, 124 (4): 218–228.

Chapter 6 – Water quality management in distribution networks

Water quality in distribution systems: rehabilitation and maintenance strategies

J.B. Boxall, D.M. Unwin, P.S. Husband & A.J. Saul
University of Sheffield, Sheffield, UK

N. Dewis
Yorkshire Water Services, Bradford, UK

J.D. Gunstead
Anglian Water Services, Peterborough, Cambridgeshire, UK

ABSTRACT: This paper details work undertaken in a distribution system network area previously subject to rigorous rehabilitation works and pre and post assessment to confirm that improvements in water quality were made. Approximately 12 months after completion of these works, discolouration complaints were again received from a localized area of the network. Sections of the network were aggressively flushed in response to these complaints and the system response to these activities monitored. The isolated dead end pipe serving the area of customer complaints exhibited significant discolouration in response to the flushing. However, other pipes within the study area also exhibited discolouration response in excess of normal regulatory supply limits. The discolouration measurements obtained from this study were interpreted with respect to rates of material accumulation and discolouration risk based on the novel predictive discolouration modelling approach after Boxall *et al* (2001). The accumulation of material leading to discolouration, particularly in areas previously subject to comprehensive rehabilitation, has been shown to be a significant issue and the quantification of rates of accumulation and associated discolouration risk are essential for the development of distribution operation and maintenance strategies.

KEYWORDS: Potable water, discolouration, rehabilitation, flushing.

1 INTRODUCTION

In the UK, WaterVoice and the Drinking Water Inspectorate (DWI) receive many complaints from the public about the aesthetic quality of water supplies (DWI 13/02). Such public complaints are disparate and primarily of discoloured water. These complaints are often considered to be indicative of more serious quality problems, such as bacterial and chemical contamination (coliforms, faecal coliforms, Escherichia coli, lead, iron, aluminum etc) McCoy and Olsen (1986). Understanding and prediction of discolouration risk is of fundamental importance to the water industry. Unlike most water quality parameters, discolouration is readily apparent to customer, and may be quantifiable through turbidity measurements.

Following privatisation of the water industry in the UK significant rehabilitation work was undertaken, driven by water quality (section 19 undertakings). The quality of these works was assured through pre and post rehabilitation assessment, as specified in DWI 15/97, to demonstrate that the renovation had been effective. Such undertakings commonly encompassed replacement or scrap lining of all non-preferred material (i.e. cast iron) and rigorous cleaning, swabbing or air scouring, of all other pipes (i.e. plastics and asbestos cement). Most section 19 undertakings are nearing completion, providing systems that might be considered "clean" and expected to remain free from water quality concerns for the foreseeable future. However such areas are once again receiving complaints of aesthetic water quality.

DWI recognise that the practices employed to operate and maintain distribution systems have a significant impact on the quality performance of those systems. Water companies in the UK have therefore been charged with the need to develop *pro-active* distribution operation and maintenance strategies. In DWI 15/02 the lack of understanding of the complex interactions and processes occurring in distribution systems is recognised: "strategies will continue to evolve as methods and knowledge improve". The relationship between operational (OPEX) and capital (CAPEX) costs must be full quantified to justify the application of strategies. Return periods for maintenance operations are a key unknown in such calculations.

This paper provides insight into material regeneration rates and associated discolouration risk within a previously rehabilitated network area.

2 BACKGROUND

Non-structural "cleaning" methods may be adopted to prevent the occurrence of discolouration events, these are commonly swabbing, air scouring and flushing (WRc 1989). Of these approaches, flushing is generally recognised as the simplest and most robust method. Hydrants are opened to create the hydraulic forces required to remove material that contributes to discolouration events from the localised distribution network. Currently in the UK, flushing is usually carried out as a re-active measure in response to customer complaints, or on arbitrary return period for known problem areas. This is not an acceptable management approach. Understanding and knowledge of causes and mechanisms leading to discolouration must be improved, and proactive management approaches developed.

Boxall *et al* (2001) proposed a new modelling approach based on the fundamental principle that discolouration material was held in stable cohesive layers attached to the pipe walls of the systems and that these layers are conditioned by the usual daily hydraulic regime within the system. The layers have a defined profile of discolouration potential versus layer strength, with an increase in potential corresponding to a decrease in strength. The strength of the layers is dictated by the shear stress imposed within each pipe at the time of peak daily flow and hence peak daily flow controls the discolouration potential. The occurrence of disequilibria hydraulic conditions (burst, re-zoning, increased daily flow etc.) is considered to expose the layers to forces in excess of their conditioned cohesive strength and this leads to a mobilisation of the cohesive layers and results in a discolouration event. Subsequently the cohesive layers *regenerate* and are again conditioned by the peak daily flow. The source of the material is not considered explicitly within the modelling approach, however the layer regeneration rate can be adapted to simulate any material source. Supporting evidence for the selection of model type, consideration of the material source and full mathematical formulation of the model are in presented Boxall *et al* (2001). This model has been programmed into EPANET. Fieldwork results and model validation studies are presented in Boxall *et al* (2002 & 2003).

Quantification of material regeneration rates in distribution systems, in relation to readily attainable parameters, pipe material, source water quality etc. coupled with the Boxall *et al* (2001) modelling approach may be used to directly identify the return periods at which operational maintenance activities would be necessary to minimise discolouration risk. This would facilitate accurate assessment and evaluation of OPEX and CAPEX options for network management strategies.

3 FIELDWORK

A site was located that had been subject to through section 19 rehabilitation and had demonstrated an improvement in water quality through pre and post rehabilitation assessment. Approximately 12 months after completion of rehabilitation works the area started to receive significant numbers of discolouration complaints, particularly within one region. The study network was fed from a surface water source with levels of discolouration, iron, manganese etc. well within regulatory limits.

A programme of monitored flushing was implemented within the network area, specifically targeting the region of the complaints, but also covering a number of other pipe lengths within the network area to investigate the discolouration response of the system. The network is shown in figure 1, identifying the 3 flushing points. The region around flush 3 is the area associated with discolouration complaints.

The turbidity response of the system during flushing operations was measured using Casella, SpectraCense Colour and Turbidity meters. Data quality was confirmed through on-site measurements using a HACH hand held turbidity meter and through laboratory analysis of discrete samples. Flushing operations were undertaken by opening the selected hydrants in two stages up to fully open, to investigate the progressive mobilisation of material by different imposed shear stresses. Flush events were continued for at least double turn over of water within the affected pipe lengths or until the on-site measurements of discolouration returned to acceptable levels (regulatory limit of 4 NTU). Pressure and flow data was collected during the flushing operations, for hydraulic model validation. Table 1 shows the flushing rates that were achieved at each of the flushing points. Discrete samples were collected at the start of each flush, and at each flow increase. These samples were then analysed in the laboratory for a suite of metals and physical parameters.

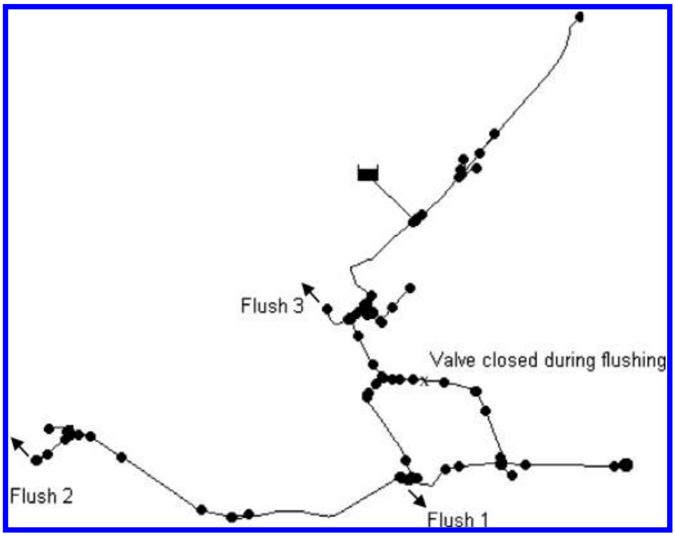

Figure 1. Site layout and monitoring points.

Table 1. Details of flushing flow rates and discrete sample collection.

Flush no.	Flushing flow rate (l/s)	Discrete sample collected	Pipe lengths affected
1a	5.0	Yes	860 m of 150 mm AC
1b	5.6*	No*	and DI pipes
2a	3.9	Yes	1100 m of 100 mm
2b	5.0	Yes	uPVC pipe
3a	2.6	Yes	90 m of 100 mm CI pipe. Area
3b	3.9	Yes	of discolouration complaints

* only slight increase above initial rate due to limitation of system, no discrete sample collected.

4 RESULTS

Figures 2, 3 & 4 shows the turbidity responses measured at each of the flushing points. From these it can be seen that flush three, affecting the shortest pipe length which directly serves the region with the majority of customer complaints, provides the dominant turbidity response; peak turbidity of 500 NTU compared with only 10 to 12 NTU for the other two flushes. Previous flushing studies (Boxall *et al* 2002 & 2003) suggest that turbidities of this magnitude are synonymous with very old poor condition pipes, that have not been subject any cleaning or rehabilitation. It is possible that this pipe length was not included in the rehabilitation scheme or was missed during the works. Alternatively some feature of this pipe length or the conditions it is exposed to mean that it has an exceptionally high corrosion rate. However such corrosion rates are not consistent with the conditions found in distribution systems, AWWARF (1996).

The discolouration responses measured during flushes one and two indicate that accumulation or regeneration of material has occurred within this rehabilitated system, this suggests that there will be long term future discolouration problems. The degree of discolouration induced suggests that

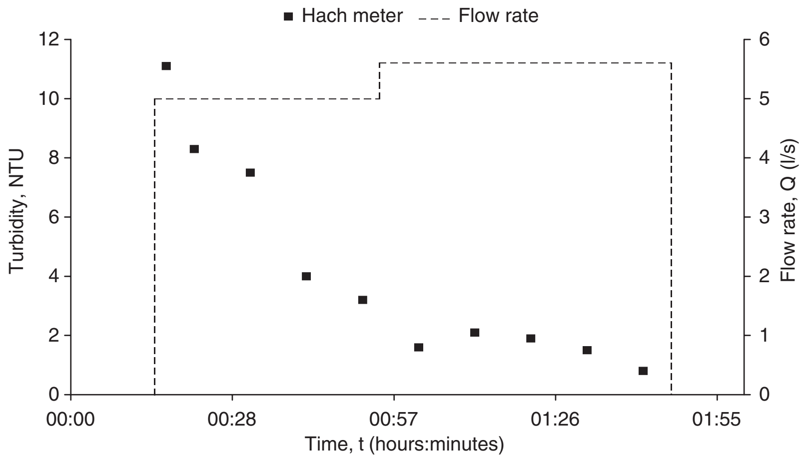

Figure 2. Measured turbidity response from flushing point 1. (No detailed turbidity data was obtained during flush 1 due to an instrument failure).

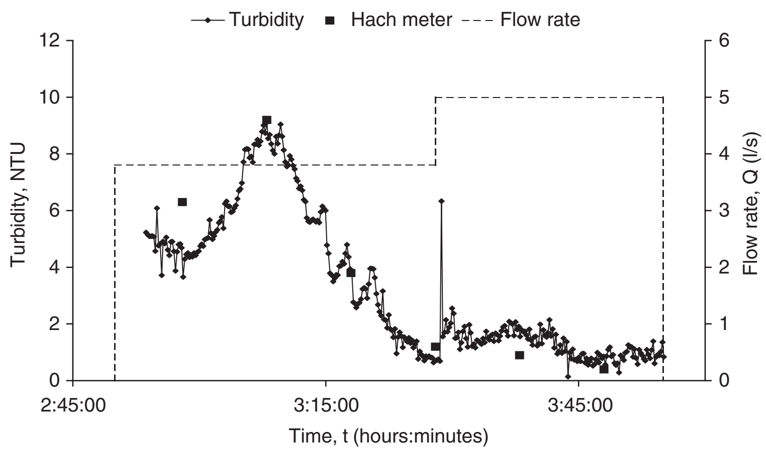

Figure 3. Measured turbidity response from flushing point 2.

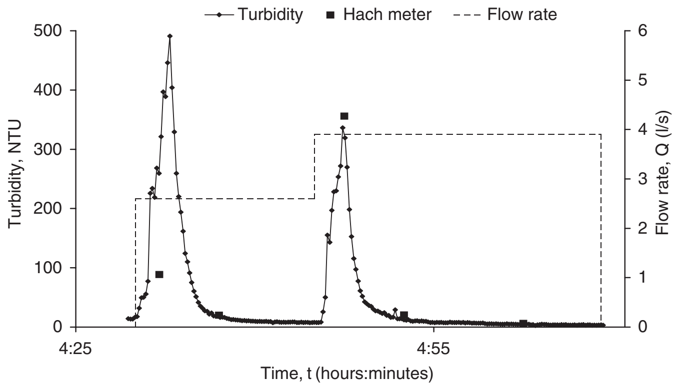

Figure 4. Measured turbidity response from flushing point 1.

Table 2. Results for total concentrations from discrete sample analysis, concentrations from filtered samples was negligible.

Flush number	Turbidity (NTU)	Iron (µg/l)	Manganese (µg/l)	Aluminium (µg/l)	Zinc (µg/l)
1a	0.53	154	39.4	23	29
2a	9.25	1806	421	72	145
2b	1.03	647	76.3	40	40
3a	11.7	2321	135	180	45
3b	150	40310	2200	802	501

regeneration rates in the majority of the pipe lengths of this network area are not likely to pose significant discolouration risk, under normal operating conditions, one year after rehabilitation works.

Table 2 provides results from analysis of the discrete samples collected during the flushing operations, only parameters showing variation during the flushing operations are presented. It should be noted that these samples were collected arbitrarily after each change in flow rate and do not necessarily coincide with the peak turbidities shown in figure 2. From this table it can be seen that the dominant material mobilised from all of the pipe lengths is iron, with manganese, aluminium and zinc also present. All of these parameters show direct relation to the degree of turbidity induced. These results are consistent with those presented by Boxall *et al* 2002 from non-rehabilitated network areas. This is a significant result, suggesting that the materials and hence the accumulation or regeneration processes are consistent between rehabilitated and existing distribution systems.

The source of material leading to distribution have been discussed previously WRc (1989) and Boxall *et al* (2001) from these it could be suggested that the material source is corrosion of cast iron pipes AWWARF (1996). The dominant material mobilised during these operations was cast iron, apparently supporting this conclusion. However, this is a rehabilitated network area where all cast iron pipes were scrapped clean and epoxy lined. It possible that either there are unlined lengths of CI in the trunk main network supplying this area, however limited statutory sampling data does not suggest any significant increase in iron concentrations between source and zone inlet. It could therefore be theorised that the accumulation of materials is a result of prolonged exposure to low background concentrations.

5 PREDICTIVE MODELLING

5.1 *Methodology*

The predictive discolouration modelling approach summarised earlier has been applied to the data obtained during this study. Initially a calibrated hydraulic model is obtained, utilising data collected during flushing operations and under daily conditions. Flushing operations are simulated using emitters and control statements to simulate the stepped flushing undertaken. Empirical discolouration model parameters are then calibrated to recreate the measured turbidity response. Automated search techniques, genetic algorithms, Goldberg (1989) and Simpson *et al* (1994), were employed to assist with both hydraulic and discolouration model calibration.

The resulting predictions obtained from the predictive modelling are shown in figure 3. From this it can be seen that good fits were obtained for all three of the monitored flushing operations. The model parameter values found through the calibration are similar to values, grouped by pipe material and diameter, obtained during previous studies.

The discolouration model requires an estimate of initial layer strength to define the stored discolouration potential, analogous to layer thickness. For old, undisturbed pipes this may be taken from the maximum shear stress imposed by the normal daily hydraulics. In the modelling approach it is assumed that it is this force that limits the accumulation of material within a given pipe. However, for this network area it is unlikely that material accumulation has occurred to such an extent that an equilibrium state has been reached in the year since the rehabilitation works. Hence, to obtain the predictions shown for flushes one and two it was necessary to calibrate for

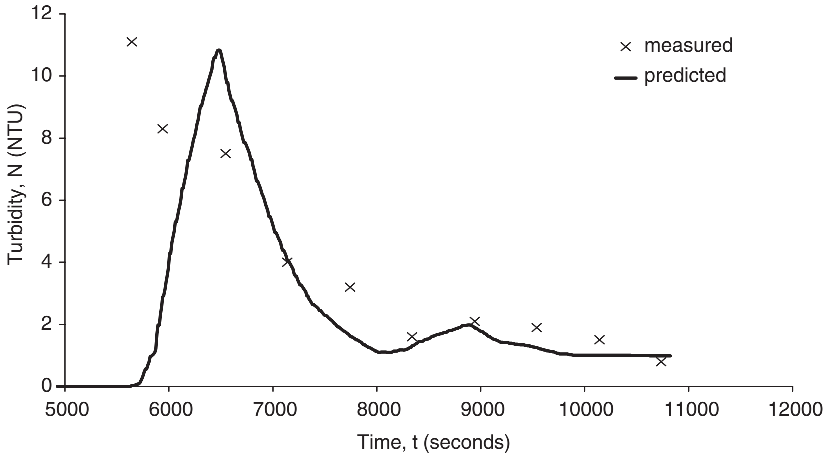

Figure 5. Modelled and measured turbidity response for flushing point 1.

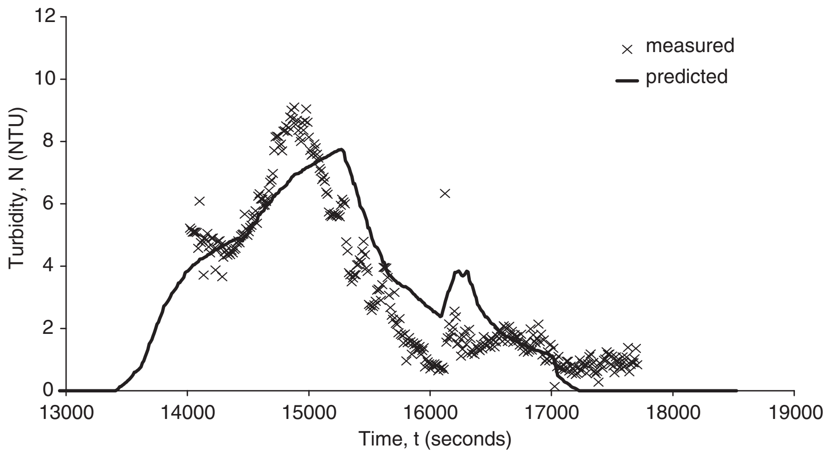

Figure 6. Modelled and measured turbidity response for flushing point 2.

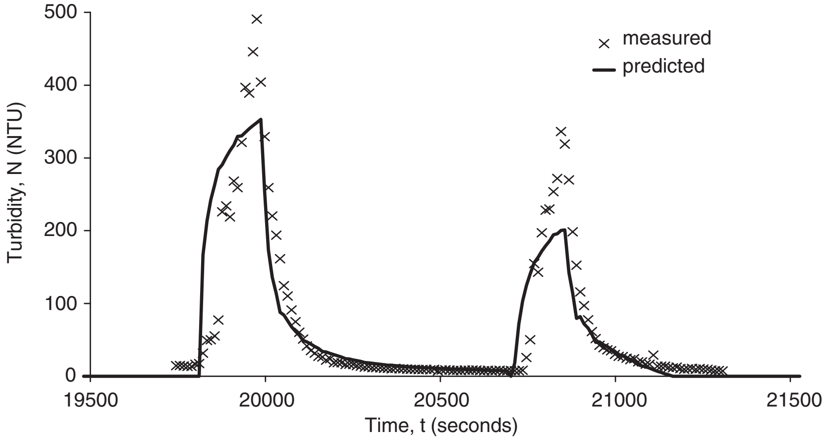

Figure 7. Modelled and measured turbidity response for flushing point 3.

this initial strength. For flush three the predictions were found to be optimised with an initial layer strength equal to that imposed by the peak daily hydraulics. This provides further evidence for the suggestion that the pipe length affected by flush 3 was not included in the rehabilitation works.

5.2 *Regeneration rates*

To produce the predictions shown in figure 3 it was necessary to define model parameters to describe how the turbidity potential of the modelled layers within the pipes varies with imposed shear stress. To model flushes one and two it was also necessary to define the current strength of the layers at the time of flushing. From this initial yield strength and assuming starting conditions of "clean" pipes (maximum strength defined by the discolouration model) it is possible to infer the rate of strength decrease and turbidity accumulation, a regeneration rate. By comparing this rate of decrease in layer strength (material regeneration) with the likely hydraulic forces within a pipe it is possible, for a given acceptable level of risk (i.e. layer strength twice the maximum force imposed by the daily hydraulics) to derive the period from rehabilitation to when operational maintenance would be required. From the data obtained from this network area, for the pipes affected by flushes one and two a period of approximately 3 years is obtained. It should be noted that this is not the return period at which repeat flushing would be necessary as this would be a function of the forces and hence layer strength achieved by the periodic flushing.

The above derivation assumes that the rate of material accumulation, and deterioration of layer strength, follows a linear relationship. Such a linear response is used to accurately describe the mobilisation of material in the predictive modelling. However, there is no information or knowledge of the description of the change in layer strength and discolouration risk during regeneration.

6 CONCLUSIONS

- Material potentially posing discolouration risk has been shown to regenerate in an area previously subject to thorough section 19 undertaking and pre and post rehabilitation assessment.
- Field measurements of system turbidity response to flushing suggest that the amount of material regeneration in 12 months following rehabilitation was not sufficient to pose a significant risk of discolouration in the majority of the study area.
- Iron was the dominant material mobilised during the flushing operations, irrespective of the mains material. This is significant results as all upstream cast iron pipes have been comprehensively scrapped and lined and are not expected to be a direct source of dissolved or suspended iron material.

- Discrete sample analysis shows that the materials responsible for discolouration in this rehabilitated network area are consistent with those found in previous studies. This suggests similar accumulation or regeneration processes, despite the rehabilitation works.
- Discolouration model techniques have been shown capable of recreating the measured turbidity response to change in hydraulic conditions.
- Discolouration regeneration rates have been inferred from the parameters obtained from predictive modelling. These have been utilised to infer, for given risk criteria, the duration from rehabilitation at which discolouration risk would have become unacceptable.

ACKNOWLEDGEMENTS

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REFERENCES

AWWARF (AWWA Research Foundation) (1996) "Internal corrosion of water distribution systems".

Boxall, J.B., Skipworth, P.J. and Saul, A.J. (2001) "A novel approach to modelling sediment movement in distribution mains based on particle characteristics" *CCWI conference*, Leicester, UK, 7–12 September.

Boxall, J.B., Skipworth, P.J. and Saul, A.J. (2002) "Aggressive flushing for discolouration events mitigation in water distribution networks" *IWQ 3rd World Water Congress*, Melbourne, Australia, 7–12 April.

Boxall, J.B., Saul, A.J. and Skipworth, P.J. (2003) "Predictive modelling and understanding of discolouration in water distribution systems" *Journal of Environmental Engineering ASCE.* Under review.

Drinking Water Inspectorate *DWI information letters 15/97, 13,15/02.*

Goldberg, D.E. (1989) "Genetic algorithms in search, optimisation, and machine learning" – Reading, Mass.; Wokingham: Addison-Wesley, 1989.

McCoy, W.F. and Olsen, B.H. (1986) "Relationship among turbidity, particle counts and bacteriological quality within water distribution lines" *Water Resources. Vol. 20, No. 8, pp 1023–1029.*

Simpson, A.R., Dandy, C.D. and Murphy, L.J. (1994) "Genetic Algorithms compared to other techniques for pipe optimisation" *Journal of Water Resources Planning and Management, ASCE, Vol. 120, No.4, pp 423–443.*

WRc (1989) "Removing loose deposits from water mains: Operational Guidelines" *WRc Wiltshire, UK.*

Practical conditions for the use of a first order chlorine decay model in water supply

P. Vieira & S.T. Coelho
National Civil Engineering Laboratory, Lisbon, Portugal

ABSTRACT: The maintenance of a residual quantity of a disinfectant such as chlorine throughout the system, in order to ensure the safety of distributed water is current practice in many countries. Chlorine added at the water treatment plant decays as it travels to the consumer tap, due to reactions in the bulk of the flow and at pipe walls. The classic kinetic model used to describe chlorine bulk reactions in most water quality modelling software packages is a first order decay model. In practice, other models provide considerably better fits in laboratory decay tests. Therefore, when the first order model is preferred to other models for simulation purposes, it is important to evaluate the magnitude of the associated errors. This paper discusses practical conditions for the application of first order/parallel first order models, based on the study of the influence of several water quality parameters (temperature, initial chlorine dosage, organic matter and iron content) on chlorine decay. Laboratory decay tests were carried out using groundwater and the performance of five kinetic models for describing bulk decay has also been assessed.

1 INTRODUCTION

Distribution of water for human consumption free of pathogens is a chief concern of water suppliers. Chlorination is a current treatment practice to achieve this goal by destroying those organisms and preventing waterborne contamination. The most widely used disinfectant is chlorine, either in its gaseous form or as a hypochlorite compound. The maintenance of a residual quantity of chlorine throughout the system, in order to ensure the microbiological safety of distributed water, is current practice in many countries worldwide. The purpose of this chlorine residual is the prevention of regrowth of micro-organisms that have eluded treatment or entered the distribution system due to external contamination caused by pipe failure, maintenance works, intrusions driven by negative pressures, entry of animals or contaminants in tanks, etc.

However, the residual chlorine concentration added to the treated water at the entrance of a distribution system gradually lowers as the chlorine reacts in the bulk phase of the water flowing in the system, and at the wall interface of pipes and tanks. Decay may lead to the total disappearance of the disinfectant, thus increasing the probability of microbiological contamination. This issue becomes more relevant in network zones with high travel times, such as network ends with low consumption.

The study of all these changes in water quality can clearly benefit from the use of computer-based mathematical models. The use of such water quality models requires rigorous calibration of the underlying hydraulic model and an accurate determination of the kinetics of the modelled constituent (if non-conservative, as is the case with chlorine) by means of field and laboratory studies, using the same water that flows in the system.

This paper discusses practical conditions for the application of first order/parallel first order models to the kinetics of chlorine when modelling water supply systems. The analysis is based on the study of the influence of several water quality characteristics such as temperature, chlorine dosage, organic content and iron content on chlorine bulk decay. The performance of five alternative kinetic models for describing bulk decay is equally assessed.

2 CHLORINE DECAY AND MODELLING OF CHLORINE DECAY KINETICS

The chemical species that result from chlorine dissolution in water – hypoclorous acid and hypochlorite ion – participate in several reactions with compounds of organic and of inorganic nature. These reactions are usually grouped in two components: the bulk decay and the wall decay.

In the bulk of the flow, chlorine reacts with inorganic species present in the treated water and easily oxidable, like ammonia, iron, manganese, sulfides, nitrites, cyanides and with less reactive compounds like organic ones (aminoacides, proteins, phenolic forms, etc.) resulting from natural organic mater (NOM), from treatment by-products or from system contamination.

Besides bulk consumption, chlorine concentration also decays due to interaction with the walls of system elements, mainly pipes. The wall decay which includes reactions with the wall material itself (for example, due to corrosion phenomena), with adhering biofilms and with accumulated sediments, is a function of pipe characteristics – material, inner coating, age, diameter and presence of attached biomass. The present paper addresses only the bulk component of the decay.

Studies carried out during the last decade in several countries, concerning chlorine disappearance in supplied water, have put forward several models to describe the kinetics of the decay of this disinfectant. Since the knowledge is limited on the chemical kinetics of the chlorine reactions in distribution systems, modelling has been based on the consideration of the following global reaction: $Cl + Reac \rightarrow Products$, where Cl represents the chlorine species and *Reac* refers to all species that can potentially react with it. This reaction comprises all the reactions that chlorine can participate in drinking water, each characterized by an individual mechanism and kinetics. Due to this complexity and to the fact that the exact composition of *Reac* still remains unknown, the models developed have adopted a "black box" methodology: single reactions that lead to chlorine decay are not taken in consideration separately but as a whole with a global velocity and global kinetics law representing the fate of disinfectant.

The classic kinetic model to describe the chlorine reaction included in most of water quality models is a first order model with respect to chlorine (Biswas and Clark 1993, Chambers *et al.* 1995, Rossman *et al.* 1994, Vasconcelos *et al.* 1997). A rate law of this kind means that the velocity of the reaction is proportional to the concentration of the reagent, the constant of proportionality being the decay constant k (see Table 1). So, according to this model, chlorine concentration decays exponentially over time.

Upgrading this simplistic first order description of chlorine decay, other studies proposed the kinetic laws presented in Table 1. A nth order kinetic law means that velocity of the reaction is proportional to the nth power of chlorine concentration. The limited first order model assumes that a fraction of the initial chlorine concentration, C^*, remains unchanged and only the remainder, $C_0 - C^*$, decays exponentially according to a first order law. A parallel first order rate law assumes that chlorine concentration can be divided in two components, each decaying according to first order models. A fraction x[1] of the initial concentration, $x.\ C_0$, decays exponentially with a rate constant k_1 and the remainder, $(1 - x).\ C_0$, decays also exponentially but with a different rate constant k_2. A second order model in respect to chlorine and all other reactants that contribute to this consumption, *Reac*, assumes that the velocity of the reaction is proportional to both concentrations of these two species.

Although other models that also account for the concentration of organic compounds have been suggested (e.g. Kastl *et al.* 1999), only simple models that require solely the monitoring of chlorine concentration were selected for the present work.

[1] x assumes a value between 0 and 1

Table 1. Kinetic laws proposed by several authors for describing chlorine decay.

Model	Differential equation (dC/dt)	Integrated equation (C)	Adjustable parameters	Reference
First order	$-kC$	$C_0 \exp(-kt)$	k	(Haas and Karra 1984) (Rossman 1993) (Vasconcelos *et al.* 1997)
Second order	$-kCC_R$	$C_0(1 - R)/(1 - R.\exp(-ut)$	R,u	(Clark 1998)
nth order	$-kC^n$	$(kt(n-1) + (1/C_0)^{(n-1)})^{-1/(n-1)}$	k,n	(Haas and Karra 1984)
Limited first order	$-k(C - C^*)$	$C^* + (C_0 - C^*) \exp(-kt)$	k,C^*	(Haas and Karra 1984)
Parallel first order	$-k_1C_1, - k_2C_2$ where $C_{1,0} = C_0x$ $C_{2,0} = C_0(1 - x)$	$C_0\, x \exp(-k_1t) + C_0 (1 - x) \exp(-k_2t)$	k_1,k_2,x	(Haas and Karra 1984)

Table 2. Initial experimental conditions of bottle tests.

Test series	Initial residual chlorine (C0) (mg/l)	Organic content added to the water (mg/l TOC)	Iron (mg/l)	Temperature (°C)
1	0.1–4	—	—	15, 20
2	0.2, 2	1–10 mg/l	—	15
3	1	—	0.02–1	15

3 MATERIALS AND METHODS

3.1 *Determination of chlorine bulk decay constants* (k_{bulk})

As bulk decay is by definition a function of water quality characteristics only, and not influenced by pipe characteristics, the kinetic constants relative to this component of chlorine decay were determined in laboratory using "bottle tests". These experiments were performed on water samples taken from a ground source that supplies a population of 50 000, in Almada (Portugal). Chlorination is the only treatment applied to this water. The residual chlorine concentration entering the distribution system is approximately 0.5–0.6 mg/l. Water samples for bottle tests were collected at the pumping station, just upstream of chlorination, and rapidly transported, in cooled conditions, to the laboratory facilities.

In a total of 42 experiments, three series of bottle tests were carried out with the aim of evaluating the influence of several variables on the bulk chlorine decay. Table 2 summarizes the initial conditions of each series. To study the effect of initial chlorine dosage and of temperature, in series 1, tests were made on water samples spiked only with chlorine in a concentration range of 0.1–4 mg/l and temperature was varied from 15 to 25°C. In series 2, in order to study the effect of the organic content (measured as TOC), tests were run by adding an organic compound that simulates organic mater. For this series, ran at an intermediate temperature of 15°C, two chlorine concentrations were selected: a high value, 2 mg/l, and a low value, 0.2 mg/l. Finally, the influence of iron content was evaluated in series 3, by adding an iron compound to the sample water (which had previously been spiked with chlorine) in the range 0.02–1.5 mg/l. Once more, an intermediate temperature was chosen (15°C) and also an intermediate value of chlorine concentration: 1 mg/l. The ranges tested in the present work are representative of conditions found in real supply systems.

Chlorine was added as a dilute solution, prepared from a 5% sodium hypochlorite commercial solution (Panreac). Organic material was simulated with indulin (Westvaco), which has a molecular structure similar to humic substances present in source waters. The desired iron concentration was obtained from the dilution of a 1010 mg/l commercial standard (Aldrich). All residual chlorine determinations were carried out through a titrimetric method using DPD (N, N-diethyl-p-phenylene-diamine) as an indicator, and ferrous ammonium sulfate (FAS) as titrant (AWWA 1995). A calibrated glass thermometer was used to measure all temperatures. TOC analyses were performed using a Dohrman equipment (combustion/infrared method). The methodology used to perform each bottle test includes the following steps:

1. Preparation of a large volume of spiked water (5 litters) by mixing the water collected in the distribution system with sodium hypochlorite. In test series 2 and 3, indulin or the iron standard solution were also added.
2. Splitting of this mixture by several 300 ml dark glass bottles (Winkler type), completely filled and tightly closed. In order to eliminate any chlorine demand by the material, all glassware was previously treated, by soaking overnight with a concentrated sodium hypochlorite solution.
3. Storage of the bottles in an incubator set at the desired temperature.
4. After predefined time steps, opening of the bottles in order to determine the concentration of residual chlorine by DPD titration.
5. Plot of chlorine residuals against the time elapsed from the beginning of the test. Estimation of decay coefficient by fitting of experimental values.

4 RESULTS AND DISCUSSION

4.1 *Kinetic model for chlorine bulk decay*

Typical results from a bottle test are presented in Figure 1. As expected, chlorine disappeared with time as a consequence of the reactions with other species present in water and already described in the beginning of this paper.

In all tests a fast initial decay was observed, corresponding to the reaction of the more reactive species like iron (II), ammonia, some amines and other highly reactive organic compounds. This phase took about 15 min, for the lower initial chlorine concentrations studied, and 3 h for the higher ones. When TOC is added this value can be as low as 3 min. Chlorine consumption in this phase shows a wide variation from 3%, in the case of high initial disinfectant concentrations (~4 mg/l), to 80%, for low chlorine (~0.2 mg/l).

A second slow and more prolonged decay followed. Now, chlorine is consumed by less reactive compounds like some organic species (humic substances and proteins) and these reactions took, in some cases, more than one month to reach completion. Several authors have already reported this two-phase decay behaviour (Qualls and Jonhson 1983, Jadas-Hécart *et al*. 1992, Kiéne *et al*. 1998, Wen Lu and Kiéne 1999), although the duration of each phase varies significantly from study to study, due to the dependency on water characteristics.

The main objective of performing bottle tests is the determination of bulk decay constants (k_{bulk}) that give a measure of the fastness of reactions between a disinfectant and all chemicals, present in the water, that can react with it. In the present work, those coefficients were determined by fitting experimental bottle test results with the five kinetic models (integrated form) listed in Table 1, using non-linear least-squares regression. Typical profiles obtained this way can be seen in Figure 1.

In general, parallel first order model provided the best fit, with correlation coefficients (r^2) between 0.810 and 0.999. In 74% of the cases this was the most satisfactory model, followed by the nth order model that yielded the best fit in 13% of the cases. Parallel first order decay seemed to be the second more adequate in 44% of the experiments. Only in just a few tests (20%) the classic first order model best fitted the experimental profiles (correlation coefficients between 0.550 and 0.950), although in the remaining cases, the marginal difference from the best models was not significant, justifying the reasonable results obtained in water quality modelling using this simple rate law. It can also be seen from Figure 1 that the shape of a parallel first order adjusts more easily to the characteristic curvature of experimental data than others. Another important advantage of

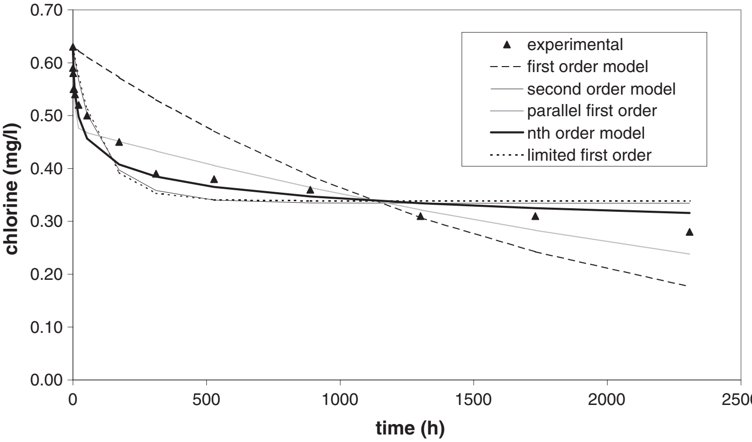

Figure 1. Fitting of experimental results from a bottle test with alternative kinetic models (typical curves).

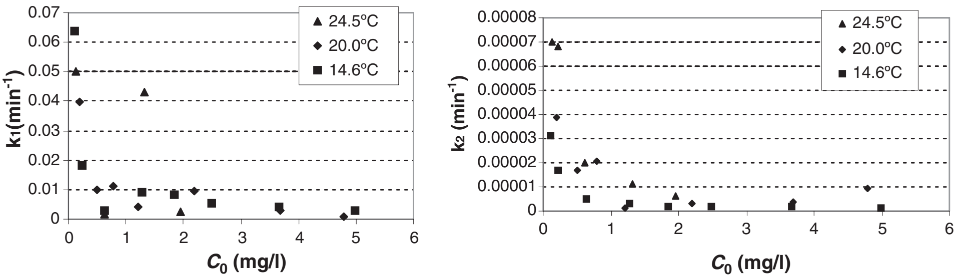

Figure 2. Influence of initial chlorine concentration, C_0, on bulk decay constants.

parallel model relates to the fact that most water quality modelling softwares only allow the use of a simple first order model and it is more correct to input a value of a kinetic constant obtained with a parallel fitting (for example, k_2 if travel times in the system to be modelled are, clearly, on the second decay phase) than one obtained from a first order fitting.

4.2 *Influence of water quality on decay constant*

Determination of decay constants by bottle testing is unquestionably the reference method, but also the most expensive and time consuming. Availability of mathematical relationships between kinetic constants k_{bulk} and the factors affecting it, would be of valuable help to obtain those coefficients in a more expedite way and based only on water quality data already collected in routine monitoring programs carried out by water companies. In order to apply this procedure, it is essential to know the exact dependence of the constants on each of those variables, this study being done for each distribution system. Figures 2 to 5 show how, in the present work, k_{bulk} varied with the four parameters mentioned earlier: initial chlorine concentration, temperature, organic and iron content. The existence of possible correlation between k_{bulk} and those variables was evaluated using the kinetic constants obtained with the parallel first order model, as this was the equation that best described chlorine behaviour in the present system.

As already reported by some authors (Fang Hua *et al.* 1999, Powell *et al.* 2000) for a simple first order model, it can be seen that the initial chlorine concentration influences the rate of disappearance of the disinfectant: higher C_0 values corresponded to slow decays, i.e., lower k_1 and k_2 values (Fig. 2). This variation suggested an inverse relation of the type k_{bulk} vs. $1/C_0$. Indeed it was found that, within experimental limits, k_{bulk} correlates well with $1/C_0$ (correlation coefficients >0.79) (Fig. 3).

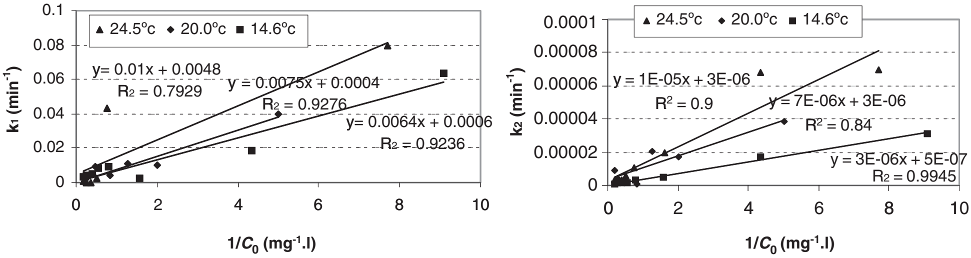

Figure 3. Bulk decay constants as a function of the inverse of initial chlorine concentration, $1/C_0$.

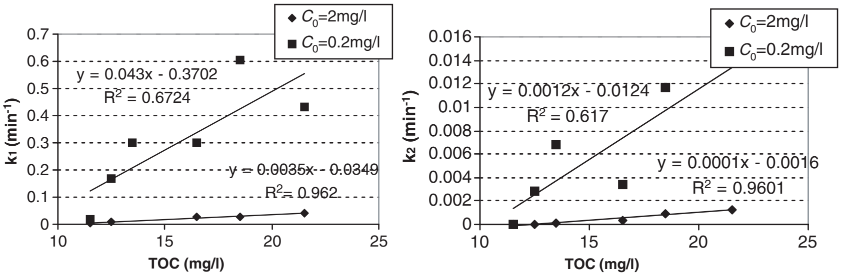

Figure 4. Influence of TOC on bulk decay constants.

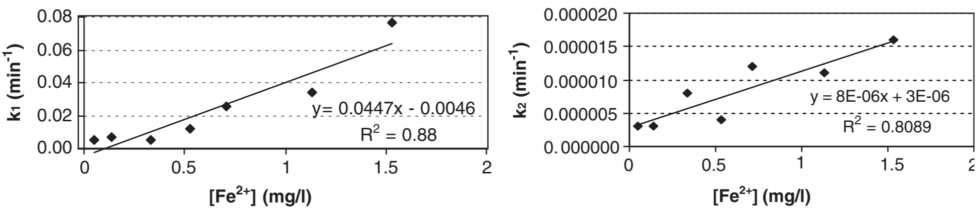

Figure 5. Influence of iron concentration on bulk decay constants.

The Arrhenius law expresses the change in kinetic constants with temperature and, for most chemical reactions, that coefficient increases with increased temperature. In the present work, results from test series number 1 also show that trend in the case of chlorine decay.

Organic and iron content was also found to influence chlorine decay. Figures 4 and 5 present the linear relationships between TOC or iron (II) concentration and kinetic constants. As can be seen, good correlation coefficients were obtained in these cases (>0.81). Correlation was poorer in the case of an initial chlorine concentration of 0.2 mg/l, due to the difficulty in quickly analyze chlorine in such fast experiments. It is interesting to note that k_2 dependence on TOC is stronger (high slopes) than of k_1 (Fig. 4), a fact that might denote the importance of reactions of organic compounds in the second phase of the decay. The situation reverses with iron (Fig. 5) as his reactions are faster and, therefore, more important in the initial phase of chlorine decay. In the second stage, organically bound iron can still be reacting slowly with chlorine, which means that not all of this metal has reacted in the first stage and explains the fact that k_2 still has a little variation.

Finally, taking in to account the expression proposed by Kiéné *et al.* (1998) for the relationship between the kinetic constant, temperature and TOC

$$k = a \times TOC \times \exp\left(\frac{-b}{T}\right)$$

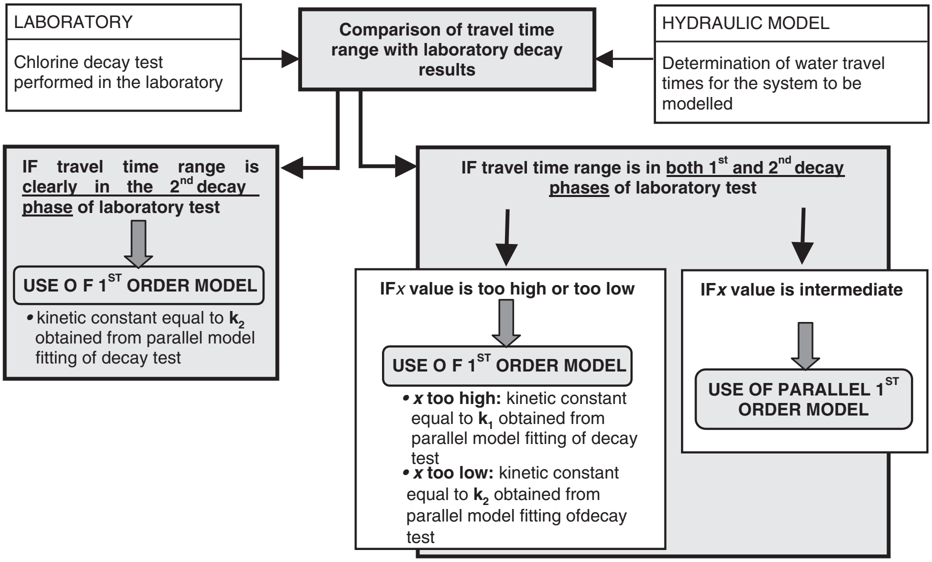

Figure 6. Decision diagram for first order kinetic model use in water quality modelling.

(where k: kinetic constant, $\min^{-1}$; TOC: total organic carbon, mg/l; T: temperature, K), and considering the inverse dependency on C_0, a correlation of the type

$$k = a \times TOC \times \exp\left(\frac{-b}{T}\right) \times \frac{c}{C_0}$$

has been tested in the present work. The following empirical relationships have been obtained with reasonably good correlation coefficients ($R = 0.887$ in the case of k_1 and $R = 0.751$ in the case of k_2):

$$k_1 = 0.01344 \times TOC \times \exp\left(\frac{848}{T}\right) \times \frac{0.01375}{C_0} \quad \text{and} \quad k_2 = 0.000209 \times TOC \times \exp\left(\frac{804}{T}\right) \times \frac{4.839}{C_0}$$

4.3 *Conditions for first order model use*

As already mentioned, most water quality modelling software only allows the use of a simple first order kinetic model to describe the bulk decay of chlorine. In this situation, the user has to input the value of one kinetic constant that, in practice, should be obtained from a laboratory decay test fitted with a parallel first order model. The main question that arises from this procedure is when can the first order model be used with minor errors (i.e., negligible in view of the water quality model use and of the uncertainty associated with analytical methods for chlorine determination).

Figure 6 proposes a methodology to determine whether a simple first order model can be used in water quality simulation. The first step is to compare the range of water travel times in the system to be modelled (obtained from an hydraulic model) with the chlorine decay curve (obtained from a bottle test performed in the laboratory with the water that supplies the system to be modelled). If system travel times correspond, clearly, to the second phase of the decay test, then the behaviour of chlorine can be described solely by k_2 as this is the kinetic constant that characterizes this phase. In these conditions, the use of a simple first order model has minor errors associated. On the other hand, if the range of water travel times comprises both the first and the second laboratory decay phases, then a thorough evaluation has to be made. In a parallel first order kinetic model, if either the first ($C_0.x.\exp(-k_1.t)$) or the second member ($C_0.(1 - x).\exp(-k_2.t)$) of the equation $C = C_0.x.\exp(-k_1.t) + C_0.(1 - x).\exp(-k_2.t)$ is negligible, the expression becomes a simple first

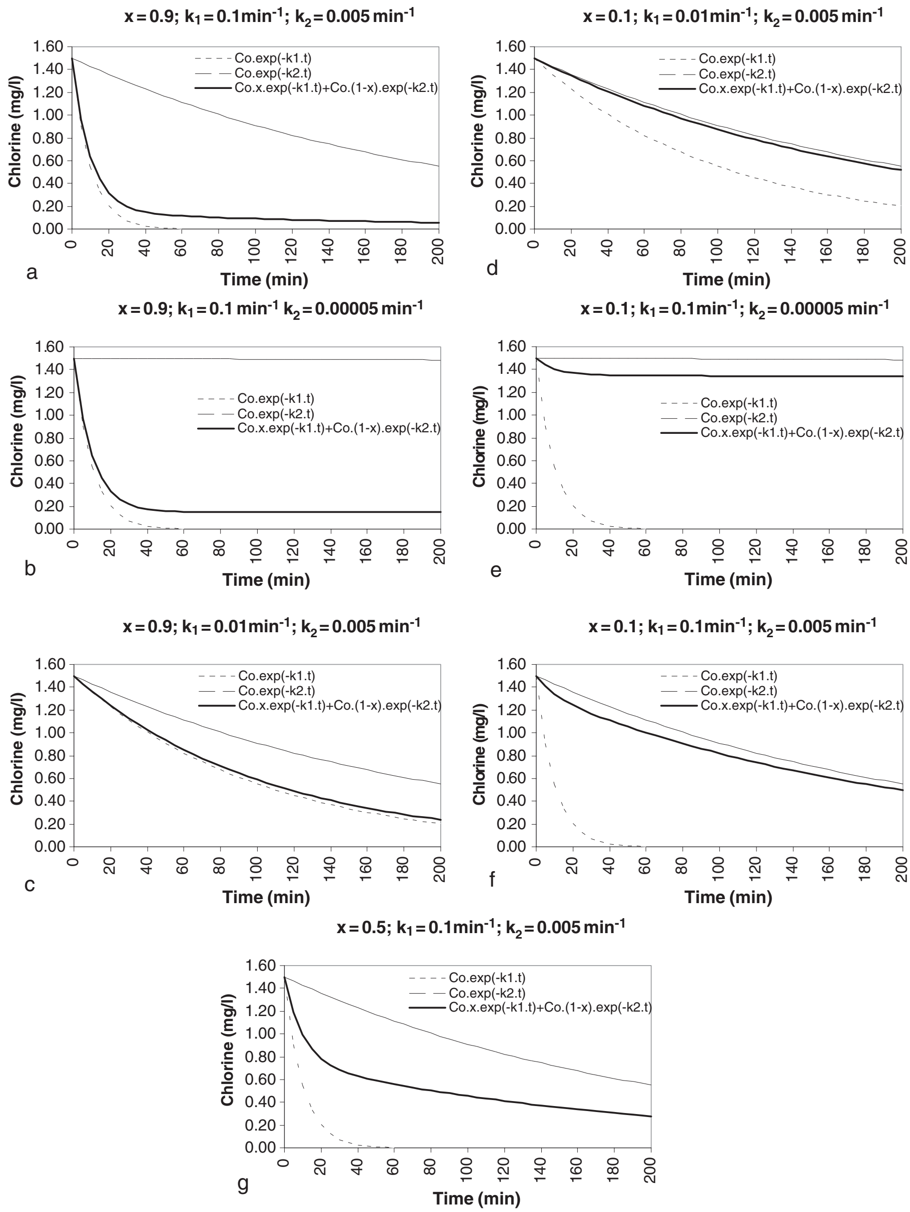

Figure 7. Chlorine decay curves for several conditions (varying x, k_1 and k_2).

order model, with a kinetic constant equal to k_2, in the first case, or equal to k_1, in the second case. x (the fraction of initial chlorine that reacts by a fast mechanism) influences this predominance of the first or the second phase of the decay: for very low or very high values of x the first order model can be used for practical purposes with minor error.

Figure 7, which depicts chlorine decay curves for several conditions, helps to understand this statement. As can be seen, for high x values (Figs 7a, 7b, 7c), the total decay curve approaches the decay curve of the first order model with coefficient k_1. For low x values (Figs 7d, 7e, 7f), the total decay now comes closer to the k_2 model. Finally, if x assumes an intermediate value (Fig. 7f), then, for quality modelling purposes, a software that enables the use of parallel first order kinetic model

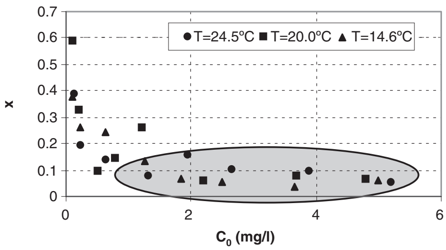

Figure 8. Influence of initial chlorine concentration on fraction x.

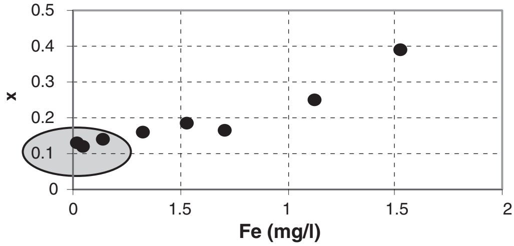

Figure 9. Influence of iron concentration on fraction x.

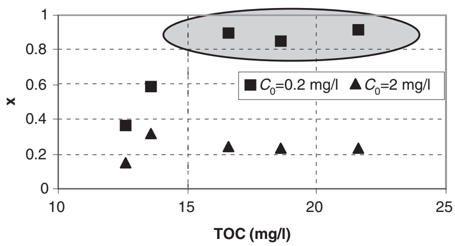

Figure 10. Influence of organic content on fraction x.

must be chosen as the error associated with describing the total decay with a simple first order model can be not negligible.

Furthermore, conditions for first order model application can be set, based upon the variation of fraction x with water quality parameters or, in other words, there can be defined which ranges of water characteristics determine the use of a simple first order model or a parallel. Figures 8 to 10 present the influence of chlorine, organic (as TOC) and iron concentrations on the variable x. x values were obtained from the parallel first model fitting of the bottle tests previously described in section *Materials and methods*.

In a total of 42 experiments, x varies from about 0.05 to 0.9. Figure 8 shows a decrease of x with initial disinfectant concentration, being this behaviour more pronounced at very low C_0. At high concentrations (above 1.5 mg/l) there is only a slight dependency on chlorine concentration and x becomes extremely low (below 0.1). There is a clear increase of the fraction of chlorine rapidly reacting with increased iron content of the water (Fig. 9). Concerning TOC, x can assume very high values (above 0.9) only if chlorine concentration is low and organic content is high (Fig. 10).

In summary, water characteristics that lead to *high x* values are (notice the grey area on Figure 10):

- low chlorine concentrations ($<$0.2 mg/l), specially together with high TOC ($>$15 mg/l) and
- high iron content (although, probably not commonly found in drinking water).

Water characteristics corresponding to *low x* values are (notice the grey areas on Figures 8 and 9):

- high chlorine concentrations ($>$1.5 mg/l) and
- low iron content ($<$0.01 mg/l).

Therefore, the first order kinetic model should be used with care if chlorine concentrations are very low or very high and if iron content is very low.

5 CONCLUSIONS

Improved understanding of the kinetics associated with changes in water quality taking place in the distribution systems, and particularly those regarding residual chlorine, will afford water supply managers better fulfilment of regulatory requirements, as well as customer satisfaction.

The research described in this paper led to the following conclusions:

- Chlorine decay in the bulk of supply water is characterized by an initial fast phase followed by a slower one – the reactions that take place in each phase are not yet entirely known. A parallel first order kinetic model best reproduced this behaviour, as opposed to the first order model frequently employed in water quality modelling. As the latter is often the only choice available in the software, it is important to evaluate the errors associated with its use over the one proposed here. If the travel times in the system clearly correspond to the second phase of the decay curve, then those errors are minor. If, on the other hand, the travel times comprise both phases, then the errors incurred into by the use of a simple first order model increase with the following conditions: very low or very high initial chlorine dosage and very low iron content of the water.
- The rate of bulk decay is a function of initial chlorine dose, temperature, iron concentration and organic content of the water. Therefore, bulk kinetic constants are site specific and should be determined specifically for the water that supplies each particular network. Correlations obtained between bulk constants and water characteristics suggest that those values can be estimated from routine quality control carried out on supply systems. It should be stressed that such relationships are a function of the type of water that flows in the system.
- There is a need for continued research on improved kinetics models for chlorine reactions to be incorporated in water quality network models.

REFERENCES

AWWA, APHA & WEF 1995. *Standard methods for the analysis of water and wastewater*. 19th edition. Washington, DC.

Biswas, P., Lu. C. & Clark, R.M. 1993. A model for chlorine concentration decay in pipes. *Water Research.* **27**(12), 1715–1724.

Chambers, V.K., Creasey, J.D. & Joy, J.S. 1995. Modelling free and total chlorine decay in potable water distribution systems. *J Water SRT-Aqua*. **44**(2), 60–69.

Clark, R.M. 1998. Chlorine demand and TTHM formation kinetics – a second order model. *Journal of Environmental Engineering.* **124**(1), 16–24.

Fang Hua, West, J.R., Barker, R.A. & Forster, C.F. 1999. Modelling of chlorine decay in municipal water supplies. *Water Research*, **33**(12), 2735–2746.

Haas, C.N. & Karra, S.B. 1984. Kinetics of wastewater chlorine demand. *J. WPCF.* **56**(2), 170–173.

Jadas-Hécart, A., El Morer, A., Stitou, M., Bouillot, P. & Legube, B. 1992. The chlorine demand of treated water. *Water Research*. **26**(8), 1073–1084.

Kastl, G.J., Fisher, I.H. & Jegatheeson, V. 1999. Evaluation of chlorine kinetics expressions for drinking water distribution modelling. *J. Water SRT-Aqua*. **48**(6), 219–226.

Kiéné, L., Lu, W. & Lévi, Y. 1998. Relative importance of the phenomena responsible for chlorine decay in drinking water distribution systems. *Water Science and Technology*. **38**(6), 219–227.

Powell, J.C., Hallam, N.B., West, J.R., Forster, C.F. & Simms, J. 2000. Factors which control bulk chlorine decay rates. *Water Research*, **34**(1), 117–126.

Qualls, R.G. & Johnson, J.D. 1983. Kinetics of short-term consumption of chlorine by fulvic acid. *Environmental Science & Technology*. **17**, 692–698.

Rossman, L.A. 1993. The EPANET water quality model. In *Integrated computer applications in water supply. Vol 2 – Applications and implementations for systems operation and management*. Research Studies Press. England. John Wiley & Sons. New York.

Rossman, L.A., Clark, R.M. & Grayman, W.M. 1994. Modelling chlorine residuals in drinking-water distribution systems. *J. Envir. Eng.* **120**(4), 803–819.

Vasconcelos, J.J., Rossman, L.A., Grayman, W.M., Boulos, P.F. & Clark, R.M. 1997. Kinetics of chlorine decay. *Journal AWWA*. **89**(7), 54–65.

Wen Lu & Kiéné, Y. 1999. Chlorine demand of biofilms in water distribution systems. *Water Research*. **33**(3), 827–835.

Water quality assessment through the monitoring of trihalomethane concentration in Curitiba water distribution system

M.C.B. Braga
Chemical Engineering Department, Water Resources and Environmental Engineering Graduate Program, University of Paraná, Brazil

S.M. Braga & C.V.S. Fernandes
Water Resources and Environmental Engineering Graduate Program, University of Paraná, Brazil

R.G. Marins
Undergraduate Student, Chemical Engineering Department, University of Paraná, Brazil

ABSTRACT: Water quality *per se* directly affects public and environmental health. If the water supplied is not adequate for human consumption it can cause serious water born epidemic. High levels of microorganisms in raw water in developing countries, especially faecal bacteria, leads water companies to increase gaseous chlorine concentration in the treatment system in order to guarantee the quality of the final product, that is, potable water. When both residual chlorine and humic substance (DOC) concentrations are available in water, a secondary reaction will occur, resulting in the formation of trihalomethane (THM). Thus, it is important to assess and evaluate the presence of certain substances and organisms in water in order to verify its quality for consumption, especially related to THM. This research was based on the fact that, in January 2001, the water supplied in Curitiba, Brazil, was allegedly not presenting the quality established by a federal regulation (Potable Water Regulation Act – FUNASA n° 1469/00) with respect to residual chlorine and THM. In such a context, a monitoring programme was established to assess the quality of the water supplied as a consequence of the presence of excessive chlorine concentration. As part of the monitoring plan some physico-chemical and microbiological parameters in both raw and treated water were determined, such as: total organic carbon, pH, colour, turbidity, total and faecal bacteria, total THM, and residual chlorine. The results of this research reflect the first serious methodological attempt to evaluate water quality response of Curitiba system for THM formation. Moreover, the basis for the development of a mathematical model to evaluate and predict THM concentration in the above mentioned water supply system will be developed.

1 INTRODUCTION

Water quality deterioration in distribution systems and its causes have been studied since the pioneering work of Larson (1966), and as extended by Rossie (1975) and O'Connor et al. (1975). These papers not only brought attention to this issue but also established a conceptual framework, one that argued for a solid and consistent sequence of research activities.

The quality of water within the distribution systems is known to change after it leaves the treatment plant. These changes are linked to complex physical, chemical and biological activities that take place concomitantly with the transport process. Indeed, water quality problems in water supply and distribution systems are strongly influenced by many different factors including the decay of chlorine-based residuals, bacterial regrowth due to temperature, disinfectant residual and the presence of assimilable organic carbon (AOC). Other factors include biofilm formation on pipe walls inducing depletion of chlorine residuals, chemically and microbiologically induced internal corrosion of the pipe wall, long detention times in pipes or storage tanks and the growth and decay of disinfection-by-products (DBPs).

The goal of this paper is to highlight the first serious methodological approach to understand the trihalomethane formation in Curitiba water distribution system. The results indicate that a significant level of by-product formation has been officially neglected. Remarks towards a more consistent methodological approach are herein included.

2 GROWTH AND DECAY OF DISINFECTANT BY-PRODUCTS

The generation and complex evolution of disinfection by-products (DBPs) in distribution systems are a function of distinct water quality conditions and the chemistry of the associated disinfectant. The increasing disinfection by-products (DBPs) concentrations observed in water distribution systems due to the longer contact time between organic precursors and oxidants is another water quality concern. DBPs are formed as a subproduct of a reaction between the disinfectant and natural organic matter (NOM) in the source water. Chlorine or other disinfectants (e.g., ozone, chlorine dioxide) interact with organic matter (e.g., humic and fulvic acids) in treated water to form disinfection by-products, some of them suspected of being carcinogenic.

The first chlorine by-product compounds were identified 25 years ago, especially in the form of trihalomethanes (THMs), which include halogenated hydrocarbons such as trichloromethane (chloroform), bromodichloromethane, dibromochloromethane and tribromomethane (bromoform). Other forms of by-products include haloacetic acids (HAAs), cyanogen halides, aldehydes and haloacetonitriles (HANs) (Krasner et al., 1989).

In order to reduce the impact of THMs formation in distribution systems, utilities may alternatively use ozonation as disinfectant. Ozone does not produce halogenated DBPs, but can produce by-products such as aldehydes (e.g., formaldehyde), acids (e.g., acetic acid), ketoacids (e.g., glyoxylic) and brominated by-products when bromide ion is present (Hofmann, 2000). In some cases it is common to use ozone with a secondary disinfectant in order to form a stable residual in the system (e.g., ozone + chlorine or ozone + monochloramine). Even in these cases, a new set of by-products have been observed just because ozone alters the nature of the organic matter present in the water, resulting in different precursors than those available in natural water conditions (Speitel et al., 1993). Although chlorinated, brominated and mixed chloro-bromo-derivatives are the most frequent DBPs reported, iodinated trihalomethanes are also formed when iodide is present in water (Cancho et al., 2000). Some by-products are known to decrease due to biological interactions (e.g., aldehydes) (Levi et al., 1993) or chemical instability (e.g., trichloropropanone) (Baribeau et al., 1994). The results in these cases come from water treated based upon pre-ozonation and post-chlorination (Romain, 1996).

The generation and evolution of by-products in water supply systems including both the water treatment and distribution are processes not well understood and have thus been a focus of intense research. One group of researchers has focused their attention on the formation and kinetics of disinfectant by-products, aiming to define analytical and mechanistic models to describe DBP formation. In general, most of the results emphasize the formation of halogenated DBPs.

Despite a better understanding of the main mechanisms inducing DBP formation, strict governmental regulations to address public health considerations have been proposed. The adoption of maximum contaminant levels (e.g., 1989 Surface Water Treatment Rule – SWTR) stresses the necessity of controlling by-product formation in distribution systems. At the same time, these developments have brought into perspective the integrity of the finished water with respect to microorganisms and waterborne diseases, and have taken into account disinfectant residual limitations within the system.

Not surprisingly, research has attempted to evaluate the impact of distinct strategies for controlling formation of by-products, specifically the halogenated DBPs. Such strategies include precursor removal techniques, and the use of alternative oxidants and disinfectants (Singer, 1994 and

Clark et al., 1994b). The first approach attempts to prevent organic precursors by lowering the concentration of natural organic material (NOM) or bromide prior to disinfection within the treatment plant. The major technologies based upon precursor removal are enhanced coagulation, granular activated carbon adsorption and membrane filtration. The second approach emphasizes alternate oxidants supplementing or replacing the use of chlorine. As mentioned previously, this does not exclude the formation of all by-products, although it may change their characteristics.

Thus, DBP formation in water distribution systems is still an area that requires additional research, despite its clear dependence on various water quality parameters and chlorination conditions. The existing numerical models for predicting DBP formation/decay were developed based upon batch-scale experiments or conditions of treatment plants (or pilot plants) (Singer, 1994). However, a comprehensive analytical formulation that accounts for distinct hydraulic conditions, elements inducing biofilm formation, and pipe wall reactions have not been yet developed. Indeed, the existing water quality models generally ignore these factors.

3 WATER QUALITY ISSUES IN BRAZIL

Nowadays, one crucial problem water resources sector faces is the reduction of water availability due to water quality degradation. Present conditions of availability and demand show that, on average, in most Brazilian territory there is no *deficit* of water resources. Furthermore, it is relevant to emphasize that, in accordance to the UN, 8% of world's surface water stock are in Brazil (Solda, 2001). However, urban concentrations in Brazil present critical conditions of sustainability with regard to the supply of potable water (Coelho, 2001).

Water resources situation in Parana is strategic, due to a fast demographic growth, which has affected quantity and quality of water, especially in Curitiba's Metropolitan Region – CMR, (Salamuni et al., 2001; Braga et al., 2001). Most water consumed in Parana State is originated from surface waters, and the State is divided in 16 river basins, being the Alto Iguaçu the longest and the one which is the most intensively occupied. Together with Tibagi and Irai river basins, the Alto Iguaçu river basin shelters 72.5% of the total population of the state (Andreoli, 1999b).

The present condition of water contamination in the CMR is a consequence of irregular land occupation, mainly where preservation areas for water supply have been designated (Salamuni et al., 2001). As a consequence, in order to prevent any disruption in water supply, the water company has to increase the concentration of chlorine in the disinfection process. This fact, associated with a possible presence of humic substances, represented by dissolved organic matter, can lead to the formation of THMs in the distribution system (Macedo, 2001; Braga et al., 2001; OPAS, 1987; OPAS, 1999; International Resources Government, 2002; State of New Mexico, 2002).

In spite of approximately 60% of the total area of the CMR are designated as areas for public water supply, according to the Coordination of Curitiba's Metropolitan Region (COMEC, 2000), in 1997, these areas had been put under increased population and industrial occupation. This fact is worsen by the fact that 12% of the metropolitan population are located in protected areas. In general, these are irregular and clandestine occupations, which have occupied flooding areas of the main rivers, which resulted in the degradation of water quality for public supply of a representative part of the CMR (Salamuni et al., 2001; Braga et al., 2001).

Due to a high rate of demographic growth observed over the last decade, a great number of metropolitan municipalities have absorbed a crescent population demand, which results in conflicts between land use and the maintenance of water quality. The main river basins that are part of the CMR integrated distribution system that is composed of 12 municipalities plus the metropolis, Curitiba, support 93% of the total population of the region (Andreoli, 1999a).

Presently, the water demand of the CMR integrated distribution system is of an order of 7.5 m^3/s. However, if the present dynamics of degradation regarding the water supply areas is considered, the water demand could surpass 42 m^3/s, and would correspond to a population of 8 million inhabitants against the present 2 million (Dalarmi, 1995; COMEC, 2000). According to Dalarmi (1995), although the water supply areas of the CMR could supply the demand for water for the next twenty years, the production is reaching its limit. Besides the provision of water in case of an eventual increase in the production of water, it is also important to guarantee the quality of water. Therefore, a sustained management of water resources demands the adoption of some criteria, which could guarantee the administration of various land use so that multiple uses of water could be preserved.

4 METHODOLOGY

The methodology adopted for the development of this study was based on both the assessment of the present situation of land use in the CMR water supply areas, Alto Iguaçu river basin (Figure 1) and in the assessment of raw and treated water quality, with the aim to evaluate water quality and indicate contributing factors to its degradation.

For the evaluation of the Alto Iguaçu river basin water quality, it was established a four-campaign monitoring programme, which was carried out in August, October, November and December 2001. This programme was implemented through the definition of five sampling points of raw water, which were established as follows: one point in each of the rivers Palmital, Itaqui and Piraquara,

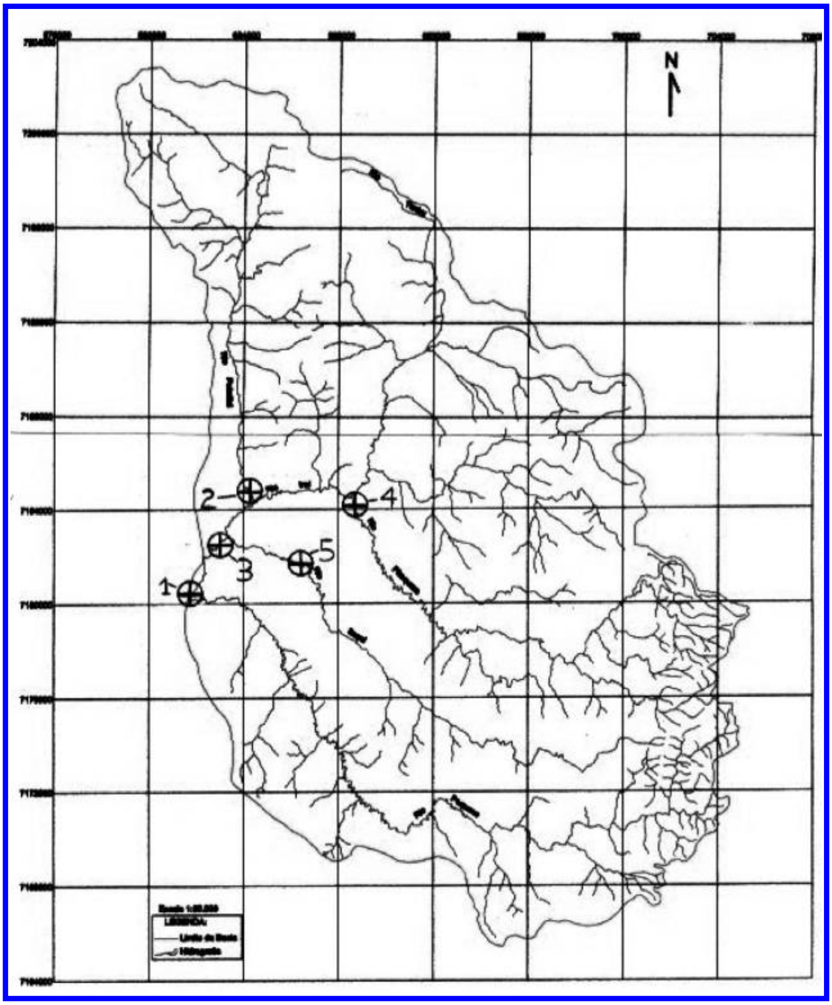

Figure 1. Sampling points in the Alto Iguaçu River Basin.

Table 1. Parameters for the assessment and evaluation of water quality.

Parameter	Raw water	Treated water
Total coliforms	X	X
Faecal coliform	X	X
pH	X	X
Color	X	X
Turbidity	X	X
Total Organic Carbon	X	X
Residual Chrlorine	X	
Total THM		X
Bromoform		X
Chloroform	X	
Dibromochloromethane	X	
Dichlorobromomethane		X

and two points in the Irai River (one downstream of the water company collection point, and one upstream the confluence with Palmital River), according to Figure 1.

The definition of sampling points of treated water was based on a map with geographic division of the areas comprised by the distribution system (SANEPAR, 2001), and carried out from October to December 2001. As there was a lack of information with regard to THMs concentration in the distribution system, and to establish a basis for this and complimentary studies, ten points were picked randomly in order to consider locations near and far from the water treatment plant in the distribution system. This fact was consolidated in order to detect the presence of residual chlorine and eventual THMs formation.

Table 1, presents the parameters analysed for raw and treated water. These parameters were analysed according to the Standard Methods (APHA, 1998).

5 RESULTS AND DISCUSSION

This study presents results of a research carried out in 2001 in the Alto Iguaçu River Basin to evaluate the quality of water supplied in Curitiba, Brazil. The results are summarized in Table 2.

Results of the first campaign (October 2001), with regard to the distribution system, showed that values for chloroform in points T5 and T7 were $57.7\,\mu g\,L^{-1}$ and $79.0\,\mu g\,L^{-1}$, respectively. Considering a guide value established by the *Organização Pan-American de Saúde*/World Health Organization (OPAS/WHO) of $30\,\mu g\,L^{-1}$, in 1987 (WHO, 1987) theses values are twice and two and a half times higher. However, since 1999, the WHO changed the guide value for chloroform to $200\,\mu g\,L^{-1}$ (OPAS, 1999), which would put the quality of water for these two points in accordance with the new value. Total THMs concentrations varied from $69.7\,\mu g\,L^{-1}$ and $94.6\,\mu g\,L^{-1}$, respectively.

It is worth noting that the Brazilian Potable Water Regulation Act – FUNASA n° 1469/2000 establishes a maximum value for THM of $100\,\mu g\,L^{-1}$ but does not establish a value for chloroform. Thus, especially the concentration of total THMs detected in Point T7 ($94.6\,\mu g\,L^{-1}$) almost reached the limit allowed by the 1469/2000 Act, in the first campaign.

As presented in Table 2 and Figure 2, chloroform concentrations in the campaign of November and the two of December, presented values, which varied from $16.4\,\mu g\,L^{-1}$ to $58.0\,\mu g\,L^{-1}$. Point T1 presented the lowest value for the three campaigns and this point represents the nearest point to the water treatment plant. The highest chloroform concentration ($58.0\,\mu g\,L^{-1}$) was in December and it was in Point T7, which is the farthest point in the distribution system with regard to the treatment plant. It can be also observed that in a span of fifteen days, Point T5 presented an increase of approximately 20% in the chloroform concentration.

Table 2. Main results.

		Raw water		Treated water	
Parameter	Date	CONAMA n° 20/86	Sampling points	FUNASA 1469/00	Sampling points
faecal coliform (MPN/100 mL)	30/08/01	3000 (VMP)	15000 (P1)	absence	not detected
			38000 (P2)		
	18/10/01		15000 (P1)		
			45000 (P2)		
	23/11/01		28000 (P1)		
			64000 (P2)		
	28/12/01		12000 (P1)		
			52000 (P2)		
residual chlorine (mg/L)	25/10/01			0,2 < Cl < 2,0	2.20 (T1)
					3.80 (T3)
					1.90 (T6)
					2.70 (T9)
					2.60 (T10)
	21/11/01				2.50 (T1)
					1.90 (T9)
	12/12/01				1.80 (T1)
					1.70 (T5)
					2..40 (T7)
					2.20 (T8)
					2.00 (T9)
					2.00 (T10)
	27/12/01				2.28 (T1)
					2.50 (T10)
total trihalomethane (mg/L)	25/10/01			<=100	69.7 (T5)
					94.6 (T7)
	21/11/01				24.4 (T1)
					41.2 (T5)
	12/12/01				63.5 (T5)
	27/12/01				23.6 (T1)
					47.4 (T2)
					29.4 (T3)
					60.5 (T4)
					65.6 (T5)
					70.2 (T6)
					70.3 (T7)
					68.8 (T8)
					55.0 (T9)
					47.6 (T10)
chloroform (mg/L)	25/10/01			(*)	57.7 (T5)
					79.0 (T7)
	21/11/01				17.9 (T1)
					32.0 (T5)
	12/12/01				48.5 (T5)
	27/12/01				16.4 (T1)
					32.2 (T2)
					20.0 (T3)
					47.7 (T4)
					52.2 (T5)
					58.0 (T7)
					56.7 (T6)
					57.2 (T8)
					46.4 (T9)
					40.3 (T10)

Note: VMP (maximum allowed value). * Not specified.

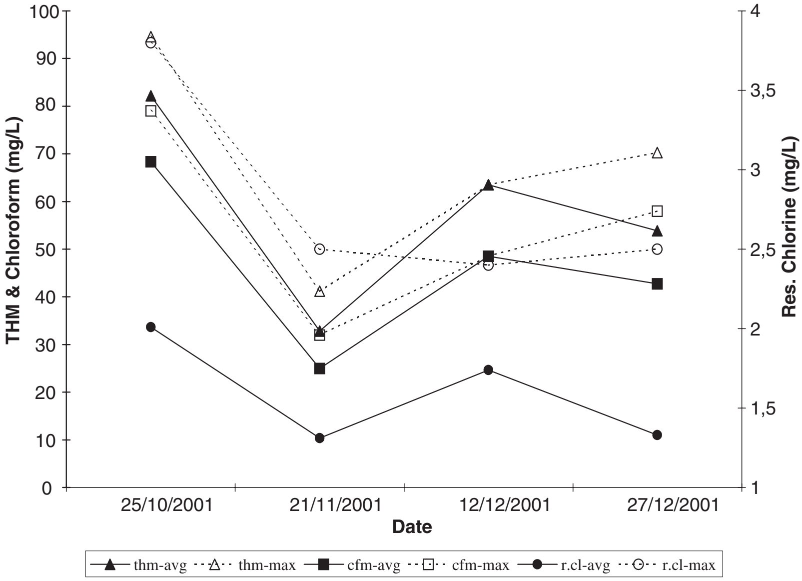

Figure 2. Trihalomethane, chloroform and residual chlorine concentrations.

Another important parameter analysed was residual chlorine, which is a precursor of THM. With regard to residual chlorine, the 1469/2000 Act establishes a value of $2.0\,mg\,L^{-1}$ as maximum allowed in any point of the distribution system. As it can be observed in Table 2, among the ten points assessed in the first campaign five presented chlorine concentrations higher than or approximately equal to $2.0\,mg\,L^{-1}$. Comparing chlorine concentrations between the first and the third campaign, three points (T1, T9, T10) presented values that were close to the limit of $2.0\,mg\,L^{-1}$. It is worth noting that T1 presented high level of THMs concentration, which varied from $1.80\,mg\,L^{-1}$ to $2.50\,mg\,L^{-1}$ in all the five campaigns, and that this point is the nearest to one of the water treatment plants that comprise the integrated distribution system. Regarding point T10, it presented chlorine concentrations varying from $2.0\,mg\,L^{-1}$ to $2.60\,mg\,L^{-1}$ in three out of four campaigns, and point T9 presented a value of $2.70\,mg\,L^{-1}$. The first and the latter point are relatively close to another treatment plant of the distribution system. However, only one point, T3, presented an exceedingly high concentration for all campaigns ($3.80\,mg\,L^{-1}$). This point is relatively close to a third treatment plant of the system.

In the time this research was being carried out, the water company was requested to provide information on the parameters analysed to assess water quality of the distribution system. It should be emphasized that from a database provided with information totalising 4606 analyses, from 01/02/00 a 28/02/01, it was observed that only one point of the distribution system presented concentration higher than $2.0\,mg\,L^{-1}$. This point did not coincide with any of the points assessed by this study and, in August 2000, the chloroform concentration was $3.0\,mg\,L^{-1}$. No mention was done to the associated hydraulic conditions of the system through the company's monitoring investigation.

Another point that is worth mentioning is faecal coliform concentration for raw water. Brazilian Surface Water Regulation Act – CONAMA n° 20/1986 establishes that the faecal coliform concentration in waters used for human consumption (Class 2 definition, that is the class of the four rivers which had their water quality assessed) should not be higher than 1000 MPN per 100 mL. The results of analyses, for the most affected rivers (Irai and Palmital) presented values which varied from 12000 MNP/100 mL to 28000 MNP/100 mL^{-1} for point P1, and from 38000 MNP/100 mL^{-1} to 64000 MNP/100 mL for point P2, as indicated in Table 2. Rivers Irai and Palmital, in particular, are the most affected rivers by irregular land occupation in the CMR. Due to the unurbanised characteristics, these areas have no sewage collection, thus resulting in the discharge of raw sewage straight to the river. As a consequence of the presence of faecal coliform microorganisms in the raw water, the water company faces the problem of having to add higher chlorine concentrations in order to guarantee the quality of the product, that is, potable water, which increase the probability of THM formation, as it could be observed by the data previously presented.

6 FINAL REMARKS

The by-products formation in water distribution systems is a process that has been focus of intense research, especially aiming to establish its connection with the exposure of chlorine decay into the network. Despite that, issues related to its formation are basically denied in Brazilian water distribution systems. In such a context, the contribution of this study is to bring this issue into evidence and highlight the necessity of a systematic approach towards its protection, assuring reliable water in quantity and quality terms.

Comparing the data gathered by this study and the collection of data provided by the water company it is important to note that among 4606 results, from February 2000 to February 2001, only one point presented chlorine concentration higher than that established by the 1469/2000 Act, whereas in only one sampling campaign carried out by this study five out of ten points presented chlorine concentration higher than $2.0\,mg\,L^{-1}$.

This quite significant conceptual monitoring difference is a clear evidence that a more consistent research approach is required to understand by-products formation in Curitiba water distribution system. At the same time, comprises evidences that should not been denied. Another aspects should be accounted for further research activities. The hydraulic influence of the network was not considered in this work, has an important relevance to the fate of constituents into the system and must be reassured in the activities to come.

Finally, it should be emphasized when assessing the main contribution of this research that the Curitiba water distribution system has THMs formation clearly established in its bulk flow. The results herein comprised highlight the central point of a necessary research approach to better understand and model its fate for the benefit of guarantee its main objective: to distribute water with assured quality.

REFERENCES

Andreoli, C. (1999a) Os mananciais de abastecimento do sistema integrado da RMC. *Sanare*, 12(12): 19–30 (in portuguese).

Andreoli, C. (1999b) Limites ao desenvolvimento da RMC impostos pela escassez da água. *Sanare*, 12(12): 31–42 (in portuguese).

APHA. Standard Methods for the Examination of Water and Wastewater. 20th Edition, 1998.

Baribeau, H., Prevost, M. and Lafrance, P. 1994. Evolution of oxidants and disinfection by-products in distribution systems. *Proc. AWWA Annual Conference,* June 19–23. New York, 153–165.

Braga, M.C.B., Grassi, M.T. and Assis, H.C.S. 2001. Resultados parciais das análises para avaliação da qualidade da água de abastecimento de Curitiba. *Trabalho apresentado no II Seminário Água – Problemas e Soluções para o Século XXI*, Curitiba, 30 de novembro de 2001 (in portuguese).

Cancho, B., Ventura, F., Galceran, M. Diaz[a] and Ricart, S. 2000. Determination, synthesis and survey of iodinated trihalomethanes in water treatment processes. *Water Research*, 34(13): 3380–3390.

Clark, R., Grayman, W., Goodrich, J., Deininger, R. and Skov, K. (1994) Measuring and modelling chlorine propagation in water distribution systems. *Journal of Water Resoruces Planning and Management*, 120: 871–887.

Coelho, A.C.P. *Proposição de instrumento de aporte ao gerenciamento sustentável dos recursos hídricos da bacia do rio Piraquara.* CURITIBA, 2001. 30 p. Monografia submetida ao I Concurso de Monografias "Desenvolvimento Sustentável no Paraná: uma realidade possível?" Núcleo Interdisciplinar de Meio Ambiente e Desenvolvimento – NIMAD, Universidade Federal do Paraná (in portuguese).

Conselho Nacional de Meio Ambiente (CONAMA). Resolução n. 20 de 18 de junho de 1986. dispõe sobre a classificação das águas doces, salobras e salinas do Território Nacional, Brasília, 1986. (in portuguese)
Coordenação da Região Metropolitana de Curitba – COMEC. *Relatório de Impacto Ambiental da RMC*. Curitiba, 2000 (in portuguese).
Dalarmi, O. (abr./jun. 1995) Utilização futura dos recursos hídricos da RMC. *Sanare*, 4(4): 31–43. (in portuguese)
Fundação Nacional de Saúde (FUNASA). Portaria n. 36 de 19 de janeiro de 1990. Dispõe sobre o controle e vigilância da qualidade da água para consumo humano e seu padrão de potabilidade. Brasília, janeiro de 1990 (in portuguese).
Fundação Nacional de Saúde (FUNASA). Portaria n. 1469 de 19 de dezembro de 2000. Dispõe sobre o controle e vigilância da qualidade da água para consumo humano e seu padrão de potabilidade. Brasília, outubro de 2001 (in portuguese).
Hofmann, R. (2000). Using ammnonia to inhibit bromate formation during ozonation. *PhD. Thesis, Depat. of Civil Engineering*, University of Toronto, Toronto.
International Resources Government. *Issues and Regulations.* Disponível em: www.clo2.com/issues/issues.html. Acesso em: 26/04/2002
Macedo, J.A.B. Determinação de trihalometanos em águas de abastecimento público e de indústrias de alimentos. Viçosa, 1997. 90 f. Tese (doutorado em Ciência e Tecnologia de Alimentos), Universidade de Viçosa (in portuguese).
Macedo, J.A.B. (2000) *Águas e Águas.* Juiz de Fora: Ortofarma, (in portuguese).
ORGANIZAÇÃO PAN-AMERICANA DE LA SALUD (OPAS). *Guias para la calidad del agua*. Publicação científica n° 506, Washington (DC), 1987, 350 f (in portuguese).
ORGANIZAÇÃO PAN-AMERICANA DE SAÚDE (OPAS). *Água: a desinfecção da água*. Washington (DC), 1999, 20 p (in portuguese).
Salamuni, E., Luchesi, L.A.C., Nieweglowski F^{O}, M., Briski, S. J. and Hosokawa, R. T. *Avaliação ambiental da qualidade das águas superfiais na área do altíssimo Iguaçu.* Curitiba: UFPR-UTP, 2001, 84 p. Relatório técnico (in portuguese).
SANEPAR. *Sistema de abastecimento integrado de Curitiba e RMC.* 1 mapa: color, *demais informações não disponíveis* (in portuguese).
Solda, S. (2001). GIS para gestão de recursos hídricos. *InfoGEO*, 3(21): 40–43.
STATE OF NEW MEXICO. Department of Environment. *Drinking water – 20NMAC7.1.* Disponível em: www.nmenv.state.nm.us/NMED_regs/dwcsb/20nmac7_1.html. Acesso em: 25/04/2002.
Tominaga, M.Y. Trihalometanos em água de abastecimento: validação e correlação de dois métodos de análise por cromatografia em fase gasosa. São Paulo, 1998. Dissertação (mestrado em Farmácia, área de Análises Toxicológicas) – Faculdade de Ciências Farmacêuticas, Universidade de São Paulo. (in portuguese)
Tominaga, M.Y. and Mídio, A.F. Exposição humana a trihalometanos em água tratada. *Revista de Saúde Pública,* São Paulo, v. 33, n. 4, p. 413–421, 1999 (in portuguese).

The effect of transient flow on chlorine decay in water supply systems

D. Loureiro, S.T. Coelho, J. Menaia & A. Lopes
National Laboratory of Civil Engineering, Lisbon, Portugal

H. Ramos
Instituto Superior Técnico, Lisboa, Portugal

ABSTRACT: The study and modelling of chlorine decay in distribution systems has been the subject of intense research in the last decade. This paper reports on a series of experiments carried out on a lifescale test rig in order to quantify any possible effect of hydraulic transients on residual chlorine concentrations in drinking water, comparatively to the decay in steady-state flow conditions. The results relate to two types of tests: steady-state tests were run in order to investigate the decay of residual chlorine during steady-state flow, for varying values of the Reynolds number; transient tests comprised an initial steady-state period, followed by successive transient events and a new steady-state period. The transient events were induced by rapid valve closing manoeuvres. The series of results obtained provide evidence towards the existence of a slowing down effect of hydraulic transients on chlorine decay rates, for the conditions represented by the experiment.

1 INTRODUCTION

The decay of chlorine residuals throughout the networks is a prime concern of water supply managers. The disinfectant is important not only as a barrier against pathogenic microorganisms, but also as an overall indicator of water quality. After leaving the treatment plant, chlorine is subject to several chemical reactions, decaying along the distribution system. The decay can be influenced by several factors whose effects on chlorine decay are not completely characterized, namely physical characteristics of the distribution system (*e.g.*, material and age of tanks and pipes), chemical characteristics of the water (*e.g.*, organic matter, iron), system operation and maintenance (*e.g.*, intermittent operation, pressure fluctuation, water losses, pipe break) and hydraulic conditions (*e.g.*, retention times in tanks storage, flow conditions) (Clark *et al.*, 1993; Vieira and Coelho, 2003; Menaia *et al.*, 2002).

Several authors, however, have suggested that transient flow conditions may have a direct effect on water quality, beyond that caused by the associated structural deterioration of the pipes or the occurrence of negative pressures (LeChevalier, 1990; Brunone *et al.*, 2000). A transient flow regime is characterized by the temporal variation of pressure and flow throughout the water supply system, due to local disturbances, namely valve manoeuvres, pump trip-off and start-up or accidental pipe bursts, which induce the reversal of flow and the occurrence of velocity profiles completely different from the ones in steady-state flow conditions. Such flow conditions have been suggested as capable of promoting a more efficient flow mixing or cause the detachment of biofilm or scaling products (LeChevalier, 1990; Brunone *et al.*, 2000). Fernandes and Karney (2000, 2002) have suggested that the inertia and compressibility effects during transient events may lead to changes in chlorine concentrations.

Motivations for the present study of the effect of transients on chlorine decay included: (i) recent research about the influence of steady-state flow conditions on chlorine residual decay; (ii) possible relations between transient flow characteristics and water quality problems (iii) little experimental evidence availability for such relationships. Therefore, this paper aims to provide a set of experimental results that relate the decay of chlorine with the occurrence of transient events. The experimental set-up was developed in order to simulate transient pressure flow conditions, induced by fast closing manoeuvres. Some results relative to the decay of chlorine in steady-state flow conditions are also presented and a comparative analysis is carried out. The decay of chlorine residual was modelled using a parallel first order model (Powell *et al.*, 2000; Vieira and Coelho, 2003; Loureiro, 2003). The parallel first order model divides the reactions with chlorine in a first reaction phase, characterized by a fast reaction rate, and in a second reaction phase, where the reaction rate is slower. The model has three adjustable parameters and can be defined using following expression (Powell *et al.*, 2000):

$$C = C_0\, x\, e^{(-k_1 t)} + C_0 (1 - x) e^{(-k_2 t)} \tag{1}$$

where C = chlorine concentration at time t; C_0 = initial chlorine concentration; x = ratio of fast to slow reactions; t = time; k_1 = bulk decay constant for fast reactions and k_2 = bulk decay constant for slow reactions.

2 METHODOLOGY AND OBJECTIVES

The purpose of the experimental work was to analyse whether transient flow conditions can influence the decay of chlorine residual comparatively to the decay observed in steady-state flow conditions, for the same initial Reynolds numbers. A hydraulic test is presented in order to illustrate a typical tested transient event. The steady-state test, aimed to characterize the decay of chlorine considering steady-state flow conditions and provide database to evaluate the decay of chlorine during transient events. The transient test was developed to analyse if transient flow conditions could influence the decay of chlorine decay.

3 MATERIALS AND METHODS

3.1 *Experimental set-up*

The experimental set-up comprised a single pipe loop system with a total length of 200 m. The pipe material was polyethylene (HPDE), with an internal diameter of 43 mm and a wall thickness of 3.5 mm. The system was equipped with several isolation and check valves and a variable speed pump. A ball valve immediately upstream of the pump was used for generating fast transients. A peristaltic pump was used for adding chemicals to the system water. A flow meter and three pressure transducers were installed onto the system in order to monitor flow and piezometric head during steady-state and unsteady-state flow conditions, respectively. Water samples were collected throughout the system at three sample points: A1 – downstream of the variable speed pump; A2 – at the midway point along the loop; A3 – upstream of the manoeuvre valve. Sampling was carried out through Ø 13 mm ports sealed with butyl rubber stoppers, using a hypodermic syringe. Sampling was always conducted in triplicate throughout the study.

3.2 *Experimental procedure*

Each test comprised a preliminary procedure whose objective was to establish the hydraulic flow conditions, namely the steady-state flow regime, and to add chlorine and humates. Humates are the most common form of organic matter present in finished water and as such the main factor for chlorine decay. The compounds were added into the system at sampling point A1, using a peristaltic pump. After the preliminary procedure, steady-state tests and transient tests were carried out in order to monitor chlorine concentration during different flow regimes (i.e., steady-state and unsteady state flow conditions), based on water samples taken from the system. The steady-state flow conditions have been established based on typical Reynolds numbers in real water supply systems. To monitor the required chlorine concentrations, at each instant, water samples were analysed using the DPD ferrous titrimetric method (APHA, 1998).

3.2.1.1 Preliminary procedure

The system was filled with water from the public supply network. Before each test, any entrapped air was thoroughly eliminated and the required steady-state flow regime was established through the appropriate setting of the variable speed pump. Chlorine and organic matter were added into the system in order to adequately control initial experimental conditions. Chlorine was added to the test water as a sodium hypochlorite solution (5% active chlorine, Panreac), in order to start each test with a stable initial concentration of 1.5 mg/L. To simulate the organic content of the water, humates (humic acid, sodium salt, techn., Aldrich) were added to make up a total organic carbon (TOC) concentration of around 5 mg/L.

3.2.1.2 Steady-state test

To perform the steady-state test, a set of 3 water samples (corresponding to 1/3 of the flow cycle throughout the system) were taken at the same section (sampling point A3) and analysed periodically (10 min), during two hours (120 min). The goal was to have representative data about the concentration of chlorine into the test water along the system, during the test period. The steady-state tests have been carried out with the same flow velocity used during the preliminary procedure to add chlorine and organic matter.

3.2.1.3 Transient test

The transient test comprised three stages, in terms of flow conditions, in order to study the decay of chlorine residual: (i) a first stage with a total duration of one hour (60 min) under steady-state flow conditions; (ii) a second with a total duration of twenty minutes (20 min) under unsteady-state flow conditions; (iii) a third stage with a total duration of forty minutes (40 min) under steady-state flow conditions with the same flow velocity as the first stage (Figure 4). In the first stage, the experimental procedure was similar to that used in the steady-state test. The second stage consisted of successive transient flow events, caused by fast closing manoeuvres of the ball valve. Water samples have been taken at the sampling point A3 and analysed periodically (5 min).

The tests described have been replicated for a range of Reynolds numbers between 15000 and 35000. The range of Reynolds numbers tested was selected based on a study developed to analyze typical ranges of Reynolds in water supply systems, which led, for average flow conditions, to a range of Reynolds numbers between 10000 and 100000 and a flow regime correspondently to transition turbulent flow (Loureiro, 2003). The pressure under steady and unsteady-state flow conditions was measured using pressure transducers.

4 RESULTS AND DISCUSSION

4.1 *Hydraulic test*

Figure 1 illustrates the variation of piezometric head caused by a fast closure manoeuvre of the ball valve (average duration of 0.07 s), placed downstream of the pipe, at three sampling points (A1, A2 and A3). The steady-state flow discharge was 1.5 l/s, corresponding to $V = 1.03$ m/s and $Re \approx 35000$ and the maximum overpressure (Joukowsky overpressure) was $\Delta H_{max} = 30.5$ m. It is shown that the pressure response at the sampling point A1 is the reverse of that observed at sampling points A2 and A3, in terms of extreme pressure values. This behaviour is due to the occurrence of negative and positive pressure waves caused downstream and upstream of the manoeuvre valve respectively, in the loop-pipe system. The transient test was done by replicating successive

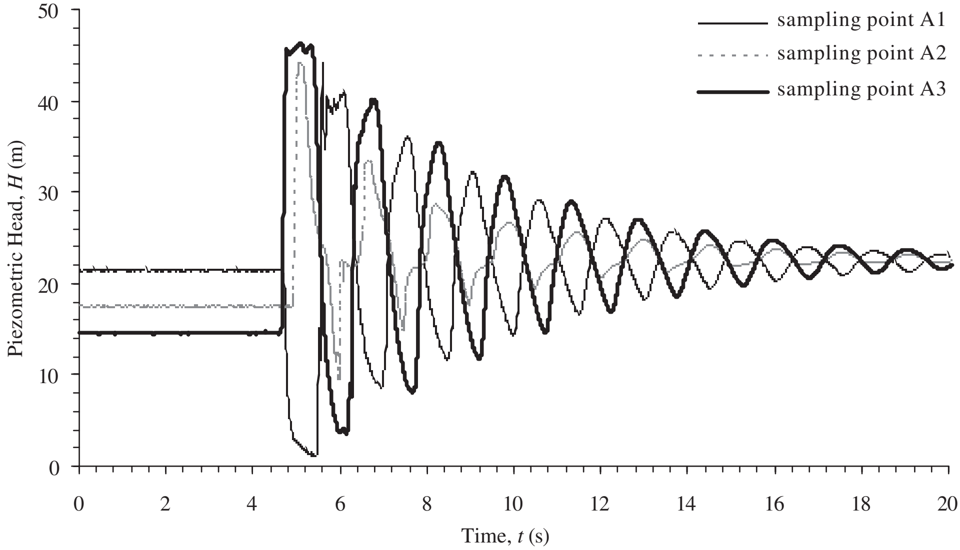

Figure 1. Experimental measurements of piezometric head time history at sampling points A1, A2 and A3.

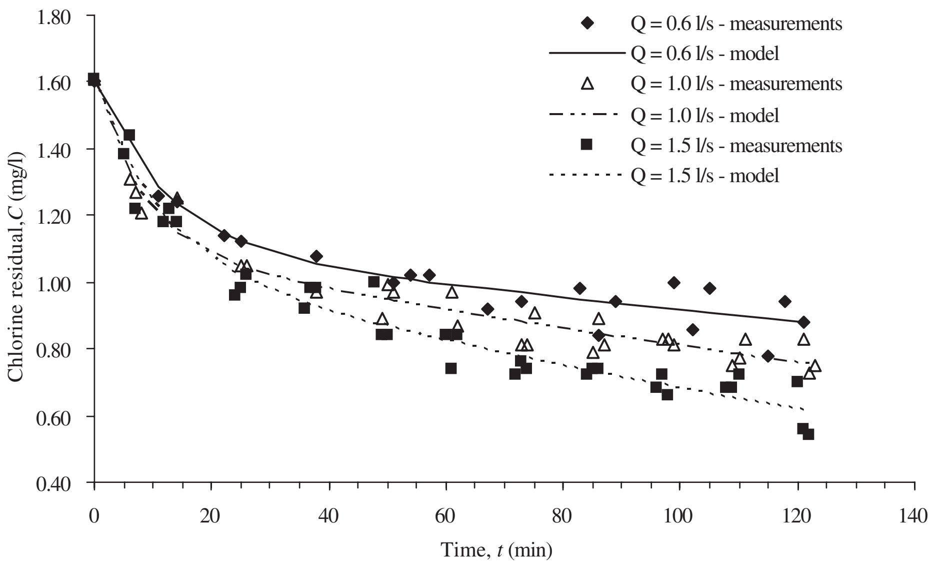

Figure 2. Chlorine decay tests at different steady-state flow velocities, with parallel 1st order model fitting.

transient events similar to the one illustrated in Figure 1 during twenty minutes, in between two steady-state flow regimes with the same flow discharge.

4.2 *Steady-state test*

Figure 2 illustrates the fitting between the experimental values of chlorine residual and a parallel first order model, for different steady-state flow values: Q = 0.6, 1.0 and 1.5 l/s. The decay of

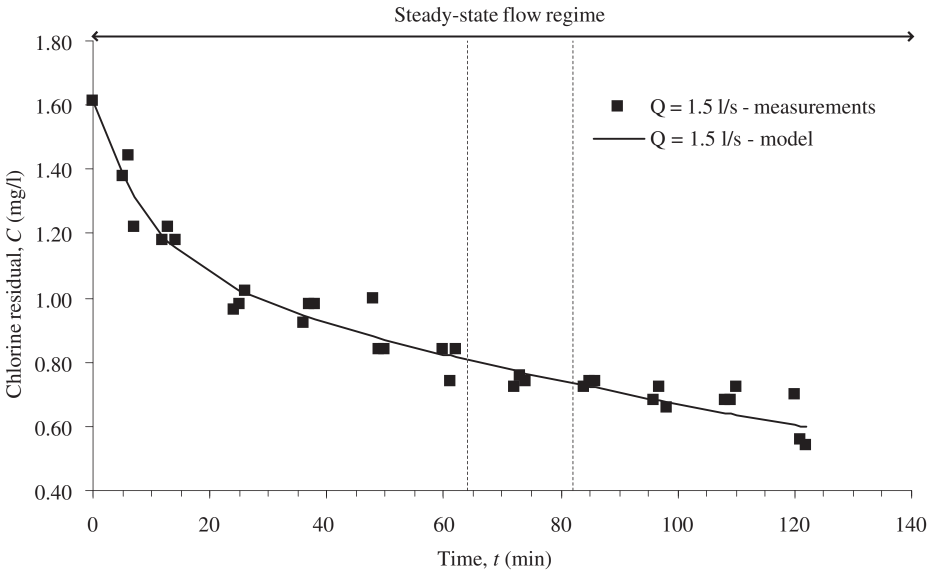

Figure 3. Chlorine decay for steady-state flow test with model fit to initial 60 minutes.

chlorine is characterized by a first phase, with a high decay rate and a short duration, followed by a second phase with a slower rate. The parallel first order model provided very good fit for the experimental data available, with average r^2 values above 0.95. Loureiro (2003) has shown that the first order model does not predict with enough accuracy the decay of chlorine observed, due to high decay observed during the initial phase. Furthermore, the results point out that the decay of chlorine residual depends on the Reynolds number, confirming recent research by Menaia *et al.* (2002), which has shown a direct influence of steady-state flow conditions on the chlorine decay rate. Under steady-state flow conditions, the increase of Reynolds number induces an additional turbulence, which will promote the reaction between the components.

Adjusting the parallel first order model to experimental data only to the first hour of test, it can be seen in Figure 3 that this model allows for reliable predictions of chlorine concentrations ($r^2 = 0.94$) during the second test hour, for steady-state test ($Q = 1.5$ l/s). This methodology used to predict the chlorine residual with the support of a kinetic model aims to compare the decay of chlorine during the second hour observed in a corresponding transient test (*i.e.*, with the same initial steady-state flow discharge).

4.3 *Transient test*

Figure 4 illustrates the concentration of chlorine measured during the transient test, as well as the curve fitting using the parallel first order model. Similarly to the steady-state test, the model was fitted to the experimental data obtained for the first hour of test. It can be seen that the model allows for a good fit to the experimental data during the first hour ($r^2 = 0.96$). Contrarily to the results obtained in Figure 3, the model underestimates the concentration of chlorine during the second hour. The occurrence of transient events indicates a slowing of the experimental decay rates as compared to the prediction given by the model. There is consistent statistical evidence throughout the experiments that the chlorine concentrations obtained during and after the transient events does not belong to the same population as those measured during the preceding steady-state period for a range of initial Reynolds numbers comprised between 15000 and 35000. However, for lower Reynolds numbers (Re $\approx$ 5000) the effect of transient events was not very significant.

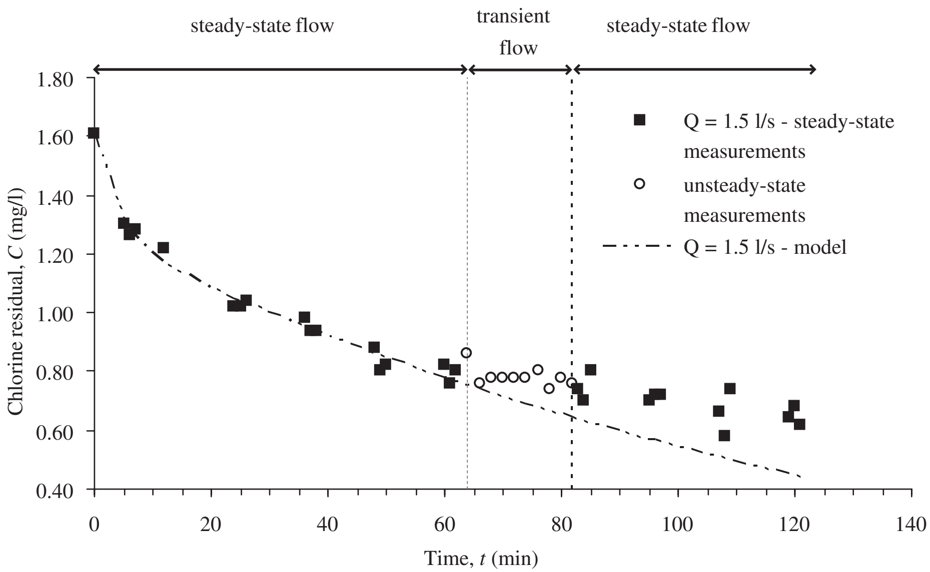

Figure 4. Chlorine decay for transient flow test with model fitted to initial 60 mins only.

Therefore, although hydraulic transients may promote some additional local mixing and consequently increase chemical reactions, particularly in regions of high shear stress (Brunone *et al.*, 1999, 2000), they occur in very short bursts and, in the case of valve closure manoeuvres, lead to significant decreases in the average flow turbulence (Reynolds number) along time, at each section of the system. Therefore, in contrast with the results observed for the similar steady-state flow test (Figure 3), a lowering of the decay rates (*i.e.*, slower decay) may be expected during the tested transient events, which was confirmed by the results obtained.

5 CONCLUSIONS

The results of the research described lead to the following main conclusions:

– The decay of chlorine during steady-state flow conditions is characterized by an initial phase with a high decay rate followed by a second phase with a slower decay rate.
– The parallel first order model is suitable for describing the decay of chlorine during steady-state flow conditions, without the occurrence of transient events.
– The occurrence of transient flow conditions, caused by rapid closing manoeuvres, has had an influence on chlorine decay rates. During transient flow conditions, the decay rate has decreased, comparatively to the results obtained during an equivalent period under steady-state flow conditions.
– The effect of transient events on chlorine decay, comparatively to the decay during steady-state flow conditions, is more relevant for higher initial Reynolds number.

REFERENCES

APHA, AWWA & WEF. 1998. Standard methods for the examination of water and wastewater. 20th ed. Washington DC. USA.
Brunone, B., Karney, B. & Ferrante, M. 1999. Velocity profiles, unsteady friction losses and transient modelling. *26th Water Resources Planning and Management Conference*. ASCE. Tempe. Arizona.

Brunone, B., Karney, B. W., Mecarelli, M. & Ferrante, M. 2000. Velocity profiles and unsteady pipe friction in transient flow. *Journal of Water Resources Planning and Management*. 126 (4): 236–244.
Clark, R. M., Goodrich, J. A. & Wymer, L. J. 1993. Effect of the distribution system on drinking water quality. *Journal of Water Supply: Research and Technology – AQUA*. 42 (1): 30–38.
Fernandes, C. & Karney, B. W. 2000. Numerical solution of the advection equation with reaction under transient conditions. *Proc. 8th Int. Conf. on Pressure Surges*. BHR Group Ltd. 12–14 April. Netherlands.
Fernandes, C. & Karney, B. W. 2002. Water quality modelling in pipe systems including the impact of inertia and compressibility effects. *World Water Congress 2002*. Melbourne (Australia). 7–12 April.
LeChevalier, M. W. 1990. Coliform regrowth in drinking water: a review. *Journal of American Water Works Association*. 82 (11): 74–86.
Loureiro, D. 2003. The influence of transients in characteristic hydraulic and water quality parameters. *MSc. Thesis*. IST. Lisbon (in Portuguese).
Menaia, J., Coelho, S.T., Lopes, A., Fonte, E. & Palma, J. 2002. Dependency of bulk chlorine decay rates on flow velocity in water distribution networks. *World Water Congress 2002*. Melbourne (Austrália). 7–12 April.
Powell, J. C., West, J. R., Hallam, N. B., Forster, C. F. & Simms, J. 2000. Performance of various kinetic models for chlorine decay. *Journal of Water Resources Planning and Management*. ASCE. 126 (1): 13–20.
Vieira, V. & Coelho, S. T. 2003. Practical conditions for the use of a first order chlorine model in water supply systems (submitted at CCWI – International Conference on Advances in Water Supply Management, 15–17 September 2003, Imperial College, UK).

An early warning contaminants detection system for water distribution networks

A. Ostfeld
Department of Civil and Environmental Engineering, Technion – Israel Institute of Technology, Haifa, Israel

A. Kessler & I. Goldberg
Kessler – Environmental and Water Resources Engineering (2002) Ltd., Haifa, Israel

ABSTRACT: An efficient water quality monitoring system is one of the most important tools to guarantee a reliable potable water supply. A methodology for finding the optimal layout of a detection system, taking explicitly into account the dilution and decay properties of the water quality constituents as distributed with flow, as well as the ability of the monitoring equipment to detect contaminants concentrations, are formulated and demonstrated. The detection system outcome is aimed at capturing contaminant entries within a pre-specified level of service, defined as the maximum volume of polluted water exposure to the public at a concentration higher than a minimum hazard level. The proposed methodology is demonstrated on an example application through a base run and sensitivity analysis.

1 INTRODUCTION

The quality of drinking water is a growing concern, and so is the list of substances of interest. Parts of them are non-conservative, increasing or decreasing in concentration as they flow through the network. Some may interact with others and with the pipe wall resulting changes in their forms.

The goal of a contaminants detection system for early warning in water distribution networks, is to reliably identify contamination events (accidental or deliberate) in source water or distribution systems in time to allow an effective response that reduces or avoids adverse impacts that may result from such an occurrence.

The objective of this paper is to develop a methodology for exploring the tradeoffs between the density of a detection system, and the level of public exposure to contamination intrusions. The density of the detection system is expressed by the number (and locations) of the monitoring stations, while the level of public exposure to contamination – by the maximum volume of consumed contaminated water, at a concentration higher than a Minimum Hazard Level (MHL).

The methodology developed is capable of dealing explicitly with the deterioration and dilution of water quality as distributed with flow, while providing an effective detection system for a given consumer level of service. Such a system provides early warning, against both external (accidental or deliberate) contamination intrusions, and internal water quality deteriorations.

2 LITERATURE REVIEW

Lee and Deininger (1992) and Kumar et al. (1997) faced the problem of optimal allocation of monitoring stations, dealing with detecting water quality deterioration in time as water flows downstream of the sources. Both works are limited to the case of an internal or gradual deterioration of water quality in the pipes. The case of a rapid deterioration of water quality due to an intrusion of an external source of contamination is not addressed [for instance, in case of a very long pipe elongated with consumption outlets, the models will result in a single monitoring station at the downstream pipe end].

Trying to expand and cope with this deficiency, Kessler et al. (1998) and Ostfeld and Kessler (1997, 2001) developed and applied a design methodology for detecting random accidental contamination intrusions in municipal water networks. The methodology developed is capable of identifying an optimal set of monitoring stations, for a given level of service, which allows capturing an external contamination intrusion into the system. The level of service is defined as the maximum volume of consumed contaminated water, prior to detection. The main shortcoming of Kessler et al. (1998) and Ostfeld and Kessler (1997, 2001), is in not taking into account the water dilution and water quality decay properties of the contaminants when distributed with flow. This limitation, together with those of Lee and Deininger (1992) and Kumar et al. (1997) are addressed in this paper.

3 CONCEPTS AND ASSUMPTIONS

The following concepts and assumptions are used in developing the proposed methodology, following Kessler et al. (1998): (1) a water quality detection system is a distributed set of monitoring stations that constantly monitor water quality parameters; (2) a level of service of a detection system is associated with the extent of damage to be prevented. The damage is related to the total consumed volume of contaminated water at a level higher than a specific concentration, prior to detection. A high level of service corresponds to a small volume of consumed contaminated water, and vice versa; (3) pollution due to an external intrusion is propagated by the immediate flow pattern and the flow patterns that follow; (4) a domain of detection is defined with respect to a particular node in the network. The domain of detection includes all nodes that are subject to contamination following pollution at that node; (5) every node in the network is a possible source of pollution, starting at any time; (6) all the water that passes through a node with a concentration higher than a MHL is considered contaminated; (7) the pollution particles are carried downstream with a velocity equal to the mean cross-sectional water velocity (i.e. a "piston flow", neglecting the pollutant dispersion); (8) the detection system is capable of providing real time monitoring data; (9) the probability of each node to become a source of external pollution is even; and (10) only one node acts at a time as a source of pollution.

4 METHODOLOGY

The methodology incorporates two main steps:

4.1 *Step 1: Pollution matrix construction*

The pollution matrix (Kessler et al., 1998) is an N by N matrix of 0–1 coefficients, where N is the number of nodes and "1" and "0" correspond to contaminated and non-contaminated nodes, respectively. The i-th row lists all contaminated nodes due to an accidental pollution at node i. The j-th column lists all polluting nodes (sources of pollution) that can contaminate node j.

From a monitoring point of view, the i-th row indicates the domain of detection due to an accidental pollution at node i. That is, a source of pollution at node i can be detected by all nodes having a "1" entry in the i-th row. The j-th column indicates the domain of coverage of node j. That is, a monitoring station at node j "covers" pollution intrusions at all nodes having a "1" entry at the j-th column.

In this paper a pollution matrix is constructed by "injecting" at each consumer node a given contaminant flow rate (mass/time), followed by a hydraulic Extend Period Simulation (EPS), with the demands at each of the nodes been the average consumptions over a typical demand cycle (e.g. a day); and by assigning a "1" entry, corresponding to each contaminant injection event, to all consumer nodes been polluted at an accumulated volume greater than the level of service, at a concentration higher than the MHL.

4.2 *Step 2: Minimum covering set*

Following Kessler et al. (1998), the minimum number of monitoring stations (comprising the detection system), is equivalent to the minimal set of columns, such that every row in the pollution matrix has an entry of "1" under at least one column of the minimal set. In other words, it is the minimum number of columns (monitoring stations) that "cover" all the rows (possible sources of pollution). Finding the above minimum set is known in graph theory as the Set Covering Problem (SCP). Solution of the Set Covering Problem is made using the algorithm suggested by Christofides (1975).

Following step 2 above, a minimum covering set of columns is selected in the pollution matrix. Each column corresponds to a particular node in the network for which a monitoring station is required.

An important case of interest is when the Minimum Detectable Level (MDL) (i.e. the ability of the monitoring equipment to identify contaminants concentrations) is higher than the MHL requirement (e.g. the equipment has a MDL of 0.1 mg/l, while only a 0.3 mg/l MHL is required). In such a case, a "1" entry at the pollution matrix is introduced each time the concentration at a node is detectable (i.e. equal or higher than the MDL), but the volume of contaminated water is accumulated only if the contaminant concentration is higher than the MHL. Using this procedure the pollution matrix is filled with "more 1 entries", thus fewer monitoring stations are finally required (i.e. as the equipment improves, less monitoring stations provide the same level of service). This will be demonstrated at the application part of the paper, allowing for trading-off the "no dilution assumption" of Kessler et al. (1998), with specific MDL and MHL concentration levels.

5 EXAMPLE APPLICATION

The methodology, casted in a software entitled QualMonNet (Kessler and Ostfeld, 2003), is demonstrated below on a small illustrative example [Example 1, EPANET (USEPA, 2001)].

The system consists of 12 pipes, eight consumers, a source, a pumping station, and an elevated storage tank (Fig. 1). The system is subject to a 24-hour demand pattern (Table 1).

The pump takes water from the source at node 9, which is at a constant water level of +800 ft. The characteristics of the pumping curve are a shutoff head of 332.5 ft, an intermediate point of 1500 gpm at 250 ft, and 3000 gpm at zero head. The operation of the pump is controlled by the water level in the tank: the pump is "on" below 110 ft, and "off" above 140 ft. The tank is 50.5 ft in diameter and +850 ft in elevation. The initial, minimum, and maximum water levels of the tank are 120, 100, and 150 ft, respectively.

Figure 1 describes the Base Run (BR) for Example 1. The BR data are a MDL of 0.1 mg/l (i.e. the sensitivity of the monitoring equipment to detect contaminant concentrations); a MHL of 0.1 mg/l (i.e. the maximum contaminant hazard concentration, above which the water is considered contaminated); a Level of Service (LOS) of 1000 m^3 (264172 gallons) (i.e. the maximum exposure to public of contaminated water, prior to detection); and a constant injection pollutant rate of 1000 mg/min. The BR results (Fig. 1 and Tables 2 and 3), show the monitoring stations selected and their coverage domains (Fig. 1); the BR pollution matrix (Table 2); and the BR polluted volume matrix (Table 3), indicating the volume of contaminated water, above a threshold of 0.1 mg/l at each of the nodes, prior to detection (i.e. prior to a total consumption of 1000 m^3 – the LOS).

The monitoring stations are located at nodes 23 and 32, which are Self Monitoring Stations [i.e. monitoring stations whose domain of detection is restricted to their own origin, thus only "1" entry appears at their corresponding rows (see Table 2 for nodes 23 and 32)]; and at nodes 11 and 22.

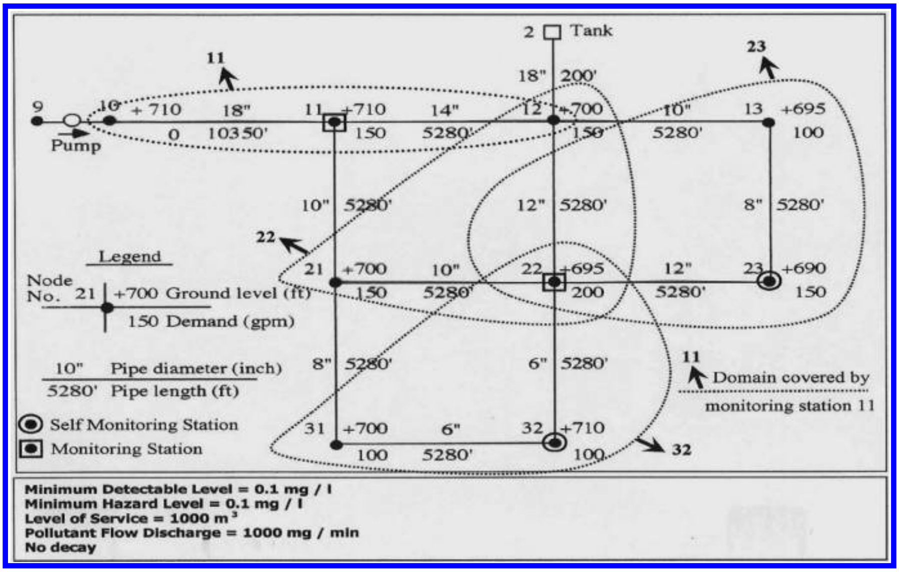

Figure 1. Base Run results.

Table 1. Demand pattern.

Time of day	Multiplier of average demand
00–02	1.0
02–04	1.2
04–06	1.4
06–08	1.6
08–10	1.4
10–12	1.2
12–14	1.0
14–16	0.8
16–18	0.6
18–20	0.4
20–22	0.6
22–24	0.8

Table 2. Base Run pollution matrix.

Source of pollution	Polluted nodes								
	10	11	12	13	21	22	23	31	32
10	1	1	0	0	0	0	0	0	0
11	0	1	1	0	1	0	0	1	0
12	0	1	1	1	0	1	0	0	0
13	0	0	0	1	0	0	1	0	0
21	0	0	0	0	1	1	0	1	0
22	0	0	0	0	0	1	1	0	1
23	0	0	0	0	0	0	1	0	0
31	0	0	0	0	0	0	0	1	1
32	0	0	0	0	0	0	0	0	1

Table 3. Base Run polluted volume matrix.

Source of pollution	Polluted nodes/Volume of polluted water (m^3)									Total volume of polluted water (m^3)
	10	11	12	13	21	22	23	31	32	
10	0	1022	0	0	0	0	0	0	0	1022
11	0	409	239	0	341	0	0	23	0	1011
12	0	170	409	159	0	273	0	0	0	1011
13	0	0	0	636	0	0	375	0	0	1011
21	0	0	0	0	511	227	0	295	0	1033
22	0	0	0	0	0	545	273	0	204	1022
23	0	0	0	0	0	0	1022	0	0	1022
31	0	0	0	0	0	0	0	545	477	1022
32	0	0	0	0	0	0	0	0	1022	1022

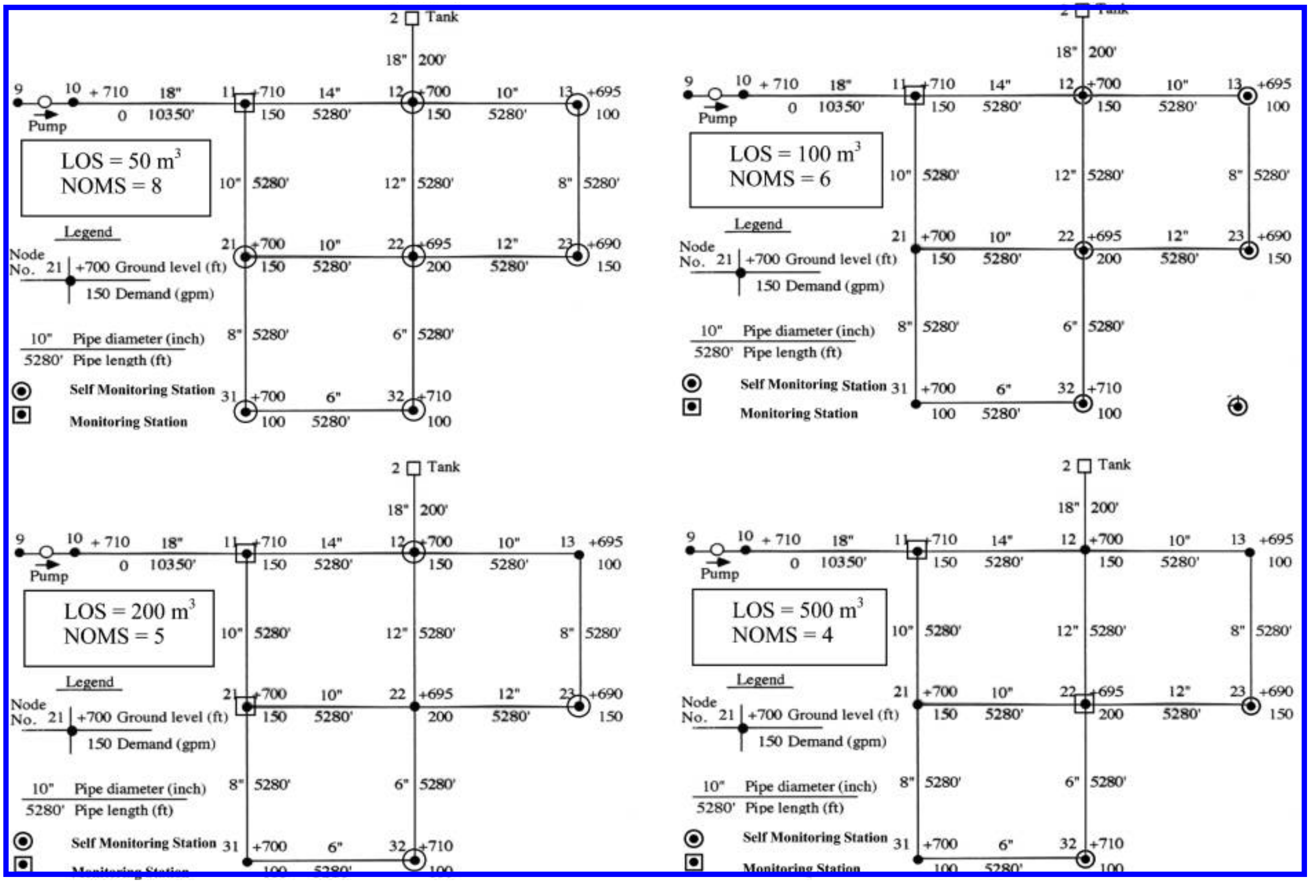

Figure 2. Tradeoff between the Level of Service and the Number of Monitoring Stations.

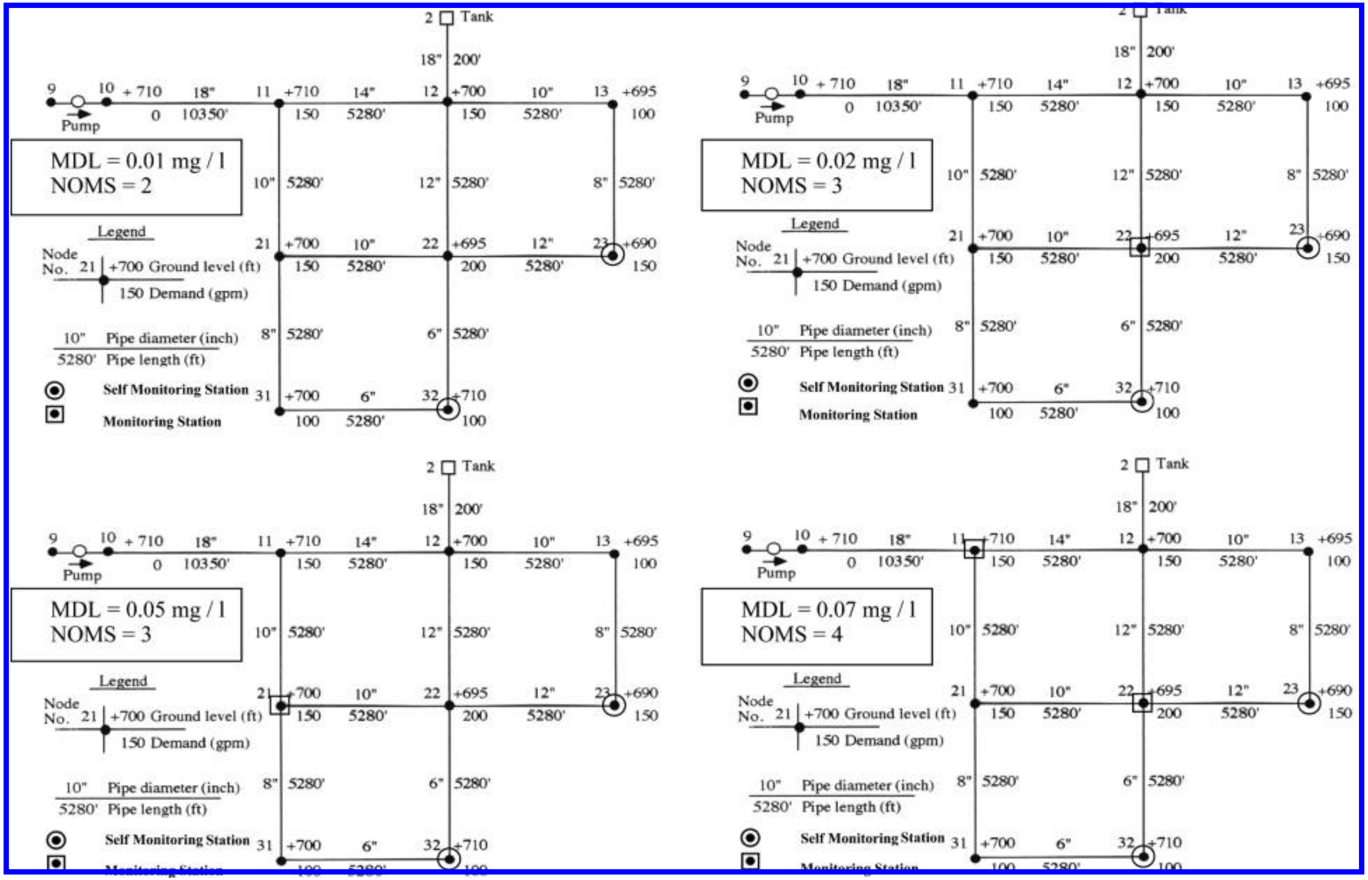

Figure 3. Tradeoff between the Minimum Detectable Level and the Number of Monitoring Stations.

Figures 2 and 3 describe Sensitivity Analysis (SA) to the BR. Figure 2 describes the tradeoff between the LOS and the Number of Monitoring Stations (NOMS): as expected, as the LOS decreases the NOMS increases [for $50\,m^3$, the NOMS is maximum (i.e. a monitoring station at each consumer node), where for $500\,m^3$ the solution coincides with that obtained at the BR]. Figure 3 describes the tradeoff between the MDL and the NOMS. As the MDL increases (i.e. as of better equipment), the number of the required monitoring stations decreases [e.g. only two monitoring stations if the MDL equals 0.01 mg/l (four at the BR, with a MDL of 0.1)].

6 CONCLUSIONS

A methodology and an example application for finding the optimal layout of a detection system in a drinking water distribution network, taking explicitly into account dilution and decay properties of water quality parameters, and the monitoring equipment performance capabilities, are formulated and demonstrated.

The detection system is aimed at capturing contaminant entries within a pre-specified level of service that is defined as the maximum volume of polluted water exposure to public at a concentration higher than a MHL. The proposed methodology, originating from Kessler et al. (1998), is a combination of hydraulic simulations with graph theory techniques, defining a minimum set of monitoring stations. The minimum set "covers" the entire network for a given level of service at a maximum degree of system invulnerability.

The model extends previous published work on this topic by addressing explicitly dilution and self decay/growth of the contaminants while distributed with flow, and by explicitly considering the monitoring stations capabilities to reveal contaminants concentrations. The "no dilution assumption" of a node been contaminated regardless of its polluted concentration (Kessler et al. 1998) becomes a private case of the proposed methodology.

Unequal nodes probabilities for becoming sources of pollution and explicit considerations of the unsteady flow conditions in the network need further research considerations.

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REFERENCES

Christofides N. 1975. Graph Theory – An Algorithmic Approach. *Academic Press*: 39–43.

Kessler A. and Ostfeld A. 2003. QualMonNet – a modeling software package for the design of water quality monitoring systems in water distribution networks.

Kessler A., Ostfeld A. and Sinai G. 1998. Detecting accidental contaminations in municipal water networks. *Journal of Water Resources Planning and Management Division, ASCE*, 124 (4): 192–198.

Kumar A., Kansal M.L. and Arora G. 1997. Identification of monitoring stations in water distribution systems. *Journal of Environmental Engineering, ASCE*, 123 (8): 746–752.

Lee B.H. and Deininger R.A. 1992. Optimal location of monitoring stations in water distribution systems. *Journal of Environmental Engineering, ASCE*, 118 (1): 4–16.

Ostfeld A. and Kessler A. 1997. Detecting accidental contaminations in municipal water networks: Application. *Proceedings of the WRPLM annual conference, ASCE, Illinois, USA*: 272–278.

Ostfeld A. and Kessler A. 2001. Protecting urban water distribution systems against accidental hazards intrusions. *Proceedings of the IWA second conference, berlin 2001*, published on CD.

USEPA (2001). EPANET 2.0. *http://www.epa.gov/ORD/NRMRL/wswrd/epanet.html*

Monitoring water quality in water supply and distribution systems

Y.A. Papadimitrakis
National Technical University of Athens, Department of Civil Engineering Hydraulics, Water Resources and Maritime Engineering, Division Iroon Polytechniou 9, Athens, Greece

ABSTRACT: Various reasons mandate nowadays the use of *automated* systems for monitoring *remotely*, and in a *continuous fashion*, the quality of (drinking) water in supply aqueducts (conveying water from reservoirs to treatment facilities) and in pipe distribution networks. Natural disasters and terrorist actions also provoke us with serious challenges regarding the vulnerability of these supply and distribution systems and have become an issue for further consideration by those in charge of their operation and maintenance.

The quality of drinking water maybe monitored by observing simultaneously several quality indices (from the parameters included in the E.E. regulations) at various sites of these water-conveying systems. This paper describes an integrated methodology for realizing such a monitoring. It also presents some criteria for selecting the observed indices and determining both the geographic distribution and density of the monitoring sites where collection, and transmission to a respective central control-and-command station, of the observed water quality characteristics is conducted. The monitoring systems are complemented by a small number of mobile units that perform occasional, *in situ*, quick chemical analyses for validating the performance of the continuously observing sensors at the various terminal or other site stations. At the respective central station, the data are *either* analyzed to produce index levels on a daily basis *or* used, via assimilation schemes, in conjunction with software that simulate the water quality in the entire supply and distribution networks to further provide a realistic picture of the water quality across the latter.

The advantages of this integrated monitoring methodology are enumerated and its capabilities to provide early warnings with regard to extreme events are compared with the (limited) capabilities of other systems that have appeared in the recent literature.

1 INTRODUCTION

Drinking water is of great importance to human health, particularly in large cities. The existence of aged water distribution networks, extending over large geographic areas, having frequently corroded pipes with inappropriate and inadequate joints, mandates the conduction of systematic water quality checks in local storage tanks, prior to water release into the distribution system, as well as within the latter prior to consumption. Similar checks are required in the supplying system of reservoirs and aqueducts, as the quality of raw water flowing into the treatment plants may affect the selection and application of a particular treatment process.

Catastrophic events, such as earthquakes, fires, flooding, etc., that usually lead to power outage may also create serious problems in the drinking water supply and distribution systems, and further disasters in the quality of water from pollution episodes, as contaminants and pathogens may enter the supply and particularly the distribution system and cause public health problems. Another recent concern is terrorism, especially in cities where the water supply system is relatively simple and vulnerable to contamination. To handle such extraordinary events particular information, regarding both the extent and the degree of water quality degradation imparted to the system, is required immediately after the episodes and/or the terrorists actions, for proper response of the authorities in charge to minimize the expected health risks and other detrimental effects associated with the polluted water flowing into the (supply and) distribution system. These events also stress the need to implement systematic monitoring in these water-supplying systems.

Yet, in most supply and distribution networks the current water quality-monitoring program is manual and expensive. *In situ* sampling of drinking water from the supply and particularly from the distribution system, by conventional means, shipment of the samples to a nearby-authorized chemical laboratory and subsequent analysis of the samples is done routinely. However, these checks are done infrequently, perhaps once a month or a few times per year and, therefore, do not guarantee that other changes in the quality of drinking water do not take place in between these infrequent checks. Such monitoring capabilities are primary the result of past philosophy prevailing in the design and operation of supply and distribution systems, without paying attention to automated monitoring of the quality of drinking water, perhaps due to the difficulties encountered in such complex dynamical systems. Thus, new ideas and approaches must be examined with respect to monitoring the quality of water in supply and distribution systems.

The widely spread practice, today, for a continuous and remotely (or tele-) controlled monitoring of the quality of both the atmospheric and oceanic environment appears sensible and necessary to be extended to the water supply and distribution systems. The theoretical concepts and scales (temporal and spatial) upon which the design (and subsequent operation) of automated and continuously monitoring systems of the water quality in coastal areas, deep seas and inland waters (rivers, lakes, etc.) is based, have been presented by several authors (Johannessen et al. 1993, 1994, Papadimitrakis & Nihoul 1996, etc.). Extended references for methodologies (or guidelines) of designing systems of monitoring the quality of inland waters maybe found in the *National Handbook of Water Quality Monitoring (1996).*

In this work, an integrated methodology for designing and operating an automated system to monitor (remotely and in a continuous fashion) the quality of drinking water in pipe distribution networks and in supply aqueducts is presented briefly (due to space limitation). The description focuses mainly on distribution systems, but a shorter reference for supply aqueducts can be found in a following section.

2 REASONING FOR REAL TIME MONITORING WATER QUALITY

There are several reasons for monitoring, in real time, the quality of drinking water in water distribution systems. The existence, for example, of corroded pipes in aged distribution systems may facilitate the presence of undesirable chemical substances in the network, either as a result of pipe wall oxidation-corrosion or through wall entrainment of various contaminants from the surrounding soil and/or the ground water table. Differential ground settling, pipe joint relaxation and accidental pipe breaking (during related construction or repair work) may also facilitate the entrainment of either contaminated water or contaminated soil or both into the distribution system. It is also possible that untreated water may enter accidentally the latter.

If, for any of the reasons cited above, contaminated or untreated water enters the distribution system, the contaminants contained in the water (and/or perhaps the products of pipe wall corrosion-oxidation) will be transported along the network by the flow and transformed by the various physical and biochemical processes that occur within the system.

There are many strict regulations that describe, nowadays, how monitoring and testing the quality of drinking water maybe done. Such EE (European Union) regulations (e.g., 80/778 and its 10/1997 modification) determine: the characteristics of water quality (in terms of several indices) that maybe monitored, their upper and lower bounds, the methods of testing and monitoring these characteristics, as well as the number and frequency of required tests. According to these regulations, water quality is checked by monitoring 61 parameters-indices which may be classified into five groups, such as: (1) physico-chemical parameters (e.g., temperature, conductivity, …),

(2) organoleptic parameters (e.g., color, turbidity, odor, taste, …), (3) parameters characterizing undesirable substances (e.g., nitrates, nitrites, ammonia, …), (4) parameters characterizing toxic substances (e.g., arsenic, mercury, …), and (5) microbial parameters (e.g., total coliforms, …).

Monitoring a judiciously selected small number of parameters, among the 61 regulated by EE, in the supply and distribution systems may suffice at present for the description of water quality in there.

3 THE MONITORING SYSTEM

The idea behind a system for remote and continuous monitoring of the water quality in distribution networks, and in supply systems as well, is based on the presence of a limited number of terminal or site stations and a respective central control-and-command station. It is noted that the expressions "terminal station" and "site station" relate to distribution networks and the supply aqueducts, respectively. Terminal stations are positioned at properly selected specific junction sites of the distribution network where water is pumped from the network to a small tank. There is a real time-on line-connection among the terminal and site stations and the corresponding central control-and-command station, in order to maintain a continuous picture of the monitored water quality indices. (For a further description of the monitoring system in supply aqueducts see section 8).

It has also been suggested, as an alternative, that part of the terminal or site stations may be formed as mobile (or vehicle-portable) units such that when needs arise (as, for example, from damage of network pipes, etc.) the system is served better by moving some of the mobile stations to the places where the extra needs exist. Furthermore, it should be clarified that it is also necessary to perform periodic cross-checks of water quality at a limited number of terminal or site stations by means of such mobile (or field-portable) units that will also serve as small chemical laboratories.

Thus, the remotely controlled monitoring systems will consist of: (1) a network of fixed terminal or site stations built at pre-selected junction or other sites of the distribution and/or supply system for *continuous* observations of several drinking water quality indices and transmission (wirelessly or otherwise) of the monitored information to a respective control center, (2) a control-and-command center where the received information is either analyzed statistically or embedded in a simulation software with aid of assimilation techniques, and (3) a limited number of mobile (vehicle-portable) biochemical units, necessary for both cross-checking the quality of monitored data from the continuously observing network instruments and the verification of their calibration maintenance, as well as for covering special situations where extra needs may arise from accidents or crisis events.

4 DISTRIBUTION AND DENSITY OF TERMINAL STATIONS

The selection of the sites, along a distribution network, where the small terminal stations are built, maybe assisted by the following criteria:

1. Ground morphology
2. Age of the distribution system
3. Hydraulic characteristics of the distribution network
4. The density of distribution network (km of conduits/km^2 network)
5. The spatial – temporal variability of contaminant concentrations in the network
6. Residence time (or renewal period) of water within particular areas of the distribution system.

As a rule of thump, it might be argued that when ground topography is irregular, the distribution system is aged, the density of network is high (many pipes/km^2 of network), the spatial and/or temporal variability of contaminant concentrations is intense, and the residence time of water is large, then the density of the terminal stations is expected to rise.

A few ideas for the selection of monitoring sites along a supply system are presented in section 8.

5 THE MONITORING SENSORS

During the last decade significant contributions have been made towards the improvement of the design and manufacturing of a variety of sensors for real time automated monitoring of the environment. During that period of time, various innovative aspects of monitoring the air and water quality were presented by research teams from Universities, small companies and the respective Industry that led, initially, to novel ideas for designing instruments and experimental techniques and later to the production of sensors and processing electronics covering, to some extent, the needs of environmental agencies.

With respect to aquatic environment, there are several sensors, nowadays, appropriate for automated and continuous monitoring of various quality parameters, as specified by the pertinent EE regulations. These parameters are associated with the physico-chemical properties of water and its organoleptic characteristics, the presence of organic pollutants, pesticides and their transformation products, the presence of toxic substances and heavy metals, trace elements, nutrients, the presence of bacteria and, perhaps, of other undesirable substances (as polychlorinated biphenyls in ground water, and some other substances in waste waters).

For sea water and inland waters, it is also possible to monitor (besides temperature, conductivity and pH) turbidity, salinity and dissolved oxygen concentrations, attenuation of light with depth at various wave lengths, chlorophyll ***a*** and algae concentrations, the radioactivity of surface waters (gamma radiation) and net solar radiation, some nutrient (e.g., orthophosphates, nitrites, nitrates) and hydrocarbon concentrations, suspended solids, sediment and particle concentrations. Extended references for the development of on-stream biosensors for pesticide detection, the "smart" sensors used for odor detection, new optical sensors and other sensors that may be used for *in situ* automatic monitoring of various water quality parameters, either from a fixed platform or a vehicle-portable (mobile) platform can be found in various sources as: Biosensors for Environmental Monitoring (1994), Instrumental Methods of Analysis. Modern Trends and Applications (1999), and in Bockreis & Jager (1999).

With regard to drinking water, it is feasible today to automatically monitor, in real time, the following parameters: the temperature of water, the water conductivity, pH and Redox potential, odor, the dissolved oxygen concentration, the concentrations of residual chlorine and of chlorine disinfection by-products (in the network), the concentrations of various heavy metals, of dissolved solids, nutrients, undesirable substances (nitrates and nitrites), and the presence of bacteria. The existing capabilities are less than those available for other aquatic environments, since the receiving sensors, accompanying transducers and other necessary electronics have to be of miniature or small size to occupy the limited space, usually, available inside the terminal stations built at some specific junction sites of the distribution systems. It is worth noting, however, that due to the interest shown by the industry of designing and producing automated sensors for monitoring (in real time) the water environment (in general, and the drinking water in particular), several new sensors appear in the market every year and older versions of existing instruments are redesigned and upgraded at fast rates. It is hoped that, within the next few years, all of the parameters required to be monitored in water supply and distribution systems, will be monitored remotely in a continuous and automated fashion. Space limitation is not usually an issue (as in distribution networks) for selecting proper sensors to monitor the quality of water in supply aqueducts.

6 DATA TRANSMISSION

The data collected at each terminal or site station, in the form of analog or digital time series, is (are) transmitted wirelessly or via leased lines or Internet Service Providers to the respective control-and-command station. It is clarified that the sampling rates of the monitoring quantities are usually determined by the encountered frequencies in the fluctuating signals, in a way that the Nyquist criterion is satisfied. Usually, a frequency twice the largest frequency encountered in all monitored signals is used to formulate a common sampling rate of all monitored data. The Data Acquisition

(DAQ) software, however, is flexible permitting variable sampling rates, depending on the needs of the monitoring program.

Wireless transmission frequently appears to be more flexible and attractive than the other alternatives. Hybrid solutions are also a possibility. Transmission cost, the quality of data transmission, and the possibility of existence of an independent and autonomous transmission network along the water supply and distribution systems, are some of the factors that may influence the final choice among the various alternatives.

The presence of a processor, at each terminal/site station, facilitates the communication of the data collected by the monitoring instruments, as the latter communicate with the external environment via a GPIB protocol (IEEE Reg., No. 488).

There are two communication alternatives among the monitoring instruments and the processor (computer) at each terminal station, either via direct incorporation of the monitoring configuration inside the terminal processor, or via a special BPIB card. That will depend on the specific methodology of monitoring the various chemical indices and the conversion of the monitored quantity to a presentable form.

In the first case, a fast, expandable, low cost and highly reliable system of data acquisition is created with direct plug-in of the monitoring instruments to each terminal computer unit. High transmission rates, and signals in digital form, are also obtained. Similar quality data is obtained when the second alternative is used, although the interconnection among the various units is different.

7 COMMUNICATION ALTERNATIVES

If a cable communication, among the terminal stations and the central control-and-command station, is selected, the following possibilities arise: (1) use of leased lines either from a local telecommunication organization or from ISP services (Internet Service Providers), and (2) use of an autonomous transmission network, in the area of interest, if such a network exists.

Receiving and transmitting data is accomplished via dial-up access or via simple modems. Such a cable interconnection among the various terminal stations has a low cost of data handling, but may present several deficiencies regarding its transmission capabilities. Reasons that may lead to the selection of either alternative (i.e., cable or wireless transmission) may depend on the specific application characteristics and requirements. Some of the factors that may directly influence the selection process are:

1. The distance between the most remote terminal station and the central control-and-command station
2. The number of terminal stations
3. The data volume that must be transmitted to the central control-and-command station
4. The ground morphology and the environment of H.M. wave transmission
5. The frequency and the way of possible transportation of monitoring (terminal) stations. The possible alternatives may be divided into two categories:
   a) Creation of a wide area wireless network among all of the terminal station and the central control-and-command station.
   b) Serial information transmission from-and-towards the terminal stations, treating the respective information by means of appropriate software.

8 WATER QUALITY SIMULATION

There are several techniques dealing with modelling of the water quality in a water distribution system. Such techniques are known as Eulerian and/or Lagrangian. The distinction depends on the volumetric control approximation used.

The software that simulates the quality of drinking water in a distribution system also simulates – in real time – the hydraulic characteristics of the flowing water in the network. More specifically, it tracks the flow of water in each pipe, the pressure at each pipe junction, the water elevation at each storage tank, and the concentrations of various substances through the network. Such substances may be total solids (T.S.), total suspended solids (T.S.S.), BOD-5, COD, total coliforms, NH_3-N (ammonium nitrogen), total phosphorus (T-PO_4-P), etc. It is also possible to simulate the concentrations of other toxic substances and heavy metals, as for example (As, Cd, Pb, Cu, Co, Ho, Hg, Zn, Cr, …), trace elements (Ca, Mg, K, Na, …), as well as salts, i.e., Cl^-, SO_4, SO_3, NO_3, NO_2, PO_4. The number and kind of simulated substances mainly depends on the availability of continuous monitoring of the particular substances.

As in the distribution system, a water quality model may also be used to predict the quality of water across the entire supply system. Yet, the simulations now are more complicated, as both the water motion in the (open or closed) channel and the physico-chemical and bio-geological processes there are more complex, at least in the open portion of the system where other environmental parameters (as, for e.g., sunlight, cloudiness, rainfall, etc.) may affect them, and ultimately the quality of water in there.

The simulation software incorporate an assimilation module that allow assimilation of the monitored data collected, in a continuous fashion or otherwise, from the specific junction (or other) sites where the terminal or site stations are located. The basic problems associated with the assimilation of data into mathematical models that simulate physical and/or biochemical processes have been understood, nowadays. Assimilation techniques have also been used with ecosystem models. Bayes Theorem and Monte Carlo Markov Chain algorithms have been used, for example, to estimate, in an optimal sense, several parameters in a multi-compartment ecosystem model dealing with flows of nitrogen amongst phytoplancton, zooplancton, nitrate, bacteria, ammonium, dissolved organic nitrogen and detritus (Fasham & Evans, 1995, Harmon & Challenor, 1997).

Typical methods, as the "sequence" or the "optimum interpolation" technique or schemes that utilize statistical errors depending on time and showing dynamic consistency may be used to maximize the precision of water quality simulation predictions, and the optimum interpolation technique is a particular choice.

The combination of simulation software and an assimilation module optimizes the simulation precision in the distribution and supply networks and the results become reliable enough, provided that in all mathematical models the problem of optimal calibration, validation and verification remains.

9 MONITORING WATER QUALITY IN SUPPLY SYSTEMS

Although certain aspects of monitoring water quality in supply systems were already addressed in previous sections some additional comments are now in order. The monitoring methodology along pre-selected sites of a water supply system, consisting of one or more aqueducts, follows closely the design philosophy of its counterpart used in the distribution networks, although the instruments and the structural details of site stations, along the open portion of the aqueduct(s), where local information is stored, processed and transmitted to a similar control-and-command station, may differ.

Apparently, space limitations are no longer a concern, as in distribution systems, and this allows modification of the practice used to monitor the various water quality indices, the conveyed discharges, and other pertinent quantities associated with the hydraulics of the open channel(s). Along these portions of a water supply system, water can be diverted or pumped towards one or more nearby external tanks placed at either side of the channel(s). There, sampling and chemical analysis is performed electronically. The information collected is stored locally in a computer where it is either processed or transmitted directly to a control-and-command station, as described previously.

The selection of local monitoring sites maybe guided by careful examination of the variation of several water quality indices along the aqueducts. Such examination maybe conducted by analyzing conventionally various water samples *in situ*, at least twice a year, during the high rainfall season and the dry summer period, taking into account the hydraulic characteristics of the open portion of the supply system and the conveyed discharges.

The water authorities (and/or utility companies), responsible for transporting drinking water from reservoirs, and other sources, to water treatment facilities, face today the need of having an early warning system installed along specific sites of the supplying aqueducts to further control the water quality variations caused by the presence of undesired substances of natural and/or anthropogenic origin or possibly associated with terrorist actions.

Water supply and distribution systems are critical elements of a nation's infrastructure and, until recently, were probably the most taken for granted among all infrastructures. They are facing now a growing spectrum of threats, including unsustainable practices, inefficient management, pollution, natural disasters and terrorism. Water is already the major factor in maintaining adequate public health and is rapidly becoming the major limiting factor for economic development worldwide. Because disruptions of water supply systems could potentially severely impact public health, public confidence, and regional economies, water supply systems could become attractive targets for future terrorist threats.

Hence it appears imperative, in a response to the emerging concerns, to develop a risk- and consequence-based vulnerability assessment technique to improve the safety and security of water supply and distribution systems against emerging physical, chemical, and biological threats. Such a risk-based method maybe based on similar techniques, developed in other scientific areas to help protect several high-consequence systems and facilities, and must address issues and concerns identified from recent evaluations.

Some countries have been living with the threat of terrorism for many decades. Two key elements of water security are process monitoring and the rapid detection of undesirable contaminants. The history of contamination of water supply systems suggests that the agents that might be used there for contamination are chemical or warfare agents, and biological agents – such as bacteria viruses and protozoa; toxins and a long list of industrial toxicants may also be used. In general, the biological agents of concern can be categorized as replicating agents (bacterial and viral), biotoxins, and waterborne pathogens. Since, however, for a contaminant to be effective it has to be tasteless, colorless, and odorless, many of the potential contaminants may not present in reality credible threats. Thus, the water supply industry may not be defenseless, and perhaps a number of small steps (and measures) can be taken that will make a supply system more secure. It is also interesting to note that several water treatment options maybe used effectively against the above agents.

There are several detection methods available today (and for some time) throughout the water treatment processes, from raw water intake to final distribution, but their presentation here falls beyond the scope of this work. However, it maybe stressed that monitoring and measurement technologies can be used to protect the process and the quality of the delivered water. It is interesting to name a few of the novel technologies used for the detection of chemical and biological contamination. Thus, in addition to "classical chemistry" methods, such as online measurements of temperature, pH, conductivity, and turbidity, immunological as well as highly advanced chemical screening techniques maybe employed, notably for measurements of polar compounds.

Knowledge about the identity and/or (possibly harmful) properties and fluctuations in concentrations of several undesired substances that may enter water supply systems is limited, to say the least. Furthermore, using existing chemical techniques, it is only possible to detect and identify a restricted suite of harmful substances. For added insight, notably regarding the occurrence of toxic effects, bio-alarming devices maybe used. These include online fish, daphnia, and algae monitoring as well as light-emitting bacteria.

A further important question that might (and must) be asked is: How can a water authority respond should it know or suspect that some contaminant has entered its water distribution system? Obviously, its consumers/customers as citizens of a particular geographic area should be warned, and the contaminant be flushed from the system as soon as possible. For any but the smallest system, identifying where the contaminant is and how it can best be flushed is a difficult problem. Should the contaminated pressure zone be isolated? Which hydrants should be opened to flush the system? Should any pumps be turned on or off in a response to this situation?

There are no easy answers to these questions due to the lack of information on the exact location of contamination. However, with a water quality model of the distribution system, the water authority/utility managers can run through a large number of "what if" scenarios to determine where the contaminant is likely to move and how the movement can be controlled by water authority/utility operations. Thus, with the aid of a calibrated (extended – period or real time operated) simulation model, the utility authority in charge can run alternative management scenarios with new initial conditions after probably adjusting the demands, if it feels that the demands will change as a result of public announcements.

11 MONITORING SYSTEM ADVANTAGES

From the previous description, it becomes evident that many practical benefits may arise from the daily use of the described monitoring system(s). These are: the reduction of human errors in the measurement chain, the continuous and simultaneous observation of several water quality indices in the networks, the low cost of monitoring resulting in a cost-effective operation of the system, and the ease of overall network maintenance; the latter is possible in conjunction with monitoring of other quantitative aspects of water supply and distribution. Furthermore, it is possible to eliminate stagnant water from the pipe network and upgrade the water quality, at least locally, by adjusting various remotely controlled valves. It is also possible, by proper monitoring, to adjust the chlorine levels in the entire distribution network. Last (but not least), the benefit list must include the ontime-diagnosis of undesirable water quality changes in the supply and distribution systems and the ability for a subsequent intervention to limit such undesirable quality deterioration, by coordinating maneuvers, particularly on the distribution network and directing the intervention squads.

Such monitoring systems increase the reliability of the networks (and of the utility organization that has the responsibility of the operational functionality of the entire supply and distribution system), especially during crisis events, and help to mitigate hazards and really maintain or even improve the quality of the supplied and distributed water, respectively.

Before summarizing, a few comments are in order for some of the methodologies (and systems) that have appeared in the recent literature and referenced in the previous section, particularly as regard their capability to provide an early warning for extreme events associated possibly with terrorist actions. The efficiency of such monitoring for water terrorism (e.g., by means of aquariums, etc.) has been the subject of extensive debates among various groups of scientists nowadays, particularly after the well-known events of the 11th of September of 2001.

It has been suggested that if terrorist events occur will most likely be the result of introduction of chemical contaminants rather than biological entities, as the quantities of material needed for a chemical assault would take less volume and be easier to obtain. Such a contaminant introduction is most likely to occur in the water supply aqueducts. While there is no proof that there is commercially available a method to monitor all of the possible chemicals that may be used to impact a water supply system, one may use the health of a bacteria culture (of own choice) to monitor the presence of toxic substances (and events). The method (uses a spectrophotometer or a color comparator disk and) can detect the presence of cyanide, mercury, heavy metals, organic and other substances that may impact the health of drinking water.

Another option for testing the water toxicity is the installation of a pass-through aquarium along specific sites of the supply system. In this case, finished or nearly finished water is routed through a series of aquariums housing various fish species. Yet, various issues have been raised as how this aquarium method would work in places where the water is chlorinated or fluoridated (or ozone purified) since these (former) methods may affect fresh water fish. Some remedies have been suggested to avoid the chlorination effects on fish, but then some bacteria and pathogens present regularly in the water may be lost and the aquatic life upset.

Some other tests have also been suggested when monitoring water for terrorism-via aquariums. These tests are run on water withdrawn from the supply channel and routed to a downstream intake (via a side channel). The tests are performed using several sensitive indicator species, as minnows, mussels, and smaller invertebrates (such as Daphnia or Copepods). The test species are exposed to the intake water, on a continuous basis, and are monitored for their condition/status. If there is a response, laboratory analysis and in stream sampling follows in order to determine what the toxin substance, and the extent of the toxic plume formed, may be. These tests are set up with threshold protocols establishing when a response occurs. The problem here is that the above tests were developed to anticipate a response to accidental spills in a riverine system used as a supply for raw water intake to water treatment, not for terrorist acts that are very difficult to anticipate.

Another technique to biomonitor for terrorist attacks (or other forms of toxicity) is to use zooplancton. Ceriodapnia culture might be coupled with an image analysis system to detect stressed swimming behaviors. Once stress is detected in the culture, the supply system can be tested and/or shut down, if necessary.

From the above brief descriptions, it may be concluded that biomonitoring in a water supply system may become efficient only in conjunction with other electronic devices (and software that may activate these devices in a response to a particular behavior of the organisms used to detect the presence of toxic substances) that can certainly be a part of the overall monitoring system of the water quality.

12 CONCLUSIONS AND FUTURE PERSPECTIVES

For all of the reasons presented and analyzed in the previous sections of this article, it is believed that the most complete and viable alternative for monitoring the quality of drinking water in supply and distribution systems appears to be that of a system operating in a remotely-controlled and continuous fashion. The supporting arguments may be summarized as follows: (a) the strict (EE) regulations and the high drinking water quality standards that have to be met in distribution networks, (b) the uncertainty that frequently exists in the quality of water within the distribution (and supply) system(s) caused by the insufficient knowledge of both the system detailed characteristics and of the bio-geo-chemical processes that govern the transport and fate of the various contaminants in the system(s), and (c) the need: (i) to have reliable and fast information for proper decision making in order to minimize health risks and save lives when extraordinary (or other) events occur, (ii) to avoid the adverse consequences of possible mall treatment practices that may produce undesirable substances in the distribution system, (iii) to reduce the operational cost, (iv) to upgrade the water quality in the distribution system by maintaining its disinfection capabilities at any time, and (v) to increase the systems reliability.

The instrumentation and computer technologies that may be used for monitoring and control of the water quality in supply and distribution systems are mature and well tested, today. The specific design of such automated systems, for a particular city, must take into account the geographic characteristics of the area, the systems characteristics, the regulatory constraints, and the existing socio-economic framework. The intensive research efforts made, nowadays, in the field of designing and production of multiparametric (apparatuses and) sensors, biosensors and mobile analytical instruments are expected to completely cover the existing needs of fully automated monitoring systems (including all of the specified 61 water quality parameters) shortly. The further enhancement of the software that will simulate completely the biochemical processes in the supply and distribution systems, and the perfection of assimilation schemes will provide unique capabilities for monitoring the quality of drinking water in these systems (in the near future).

REFERENCES

Biosensors for Environmental Monitoring. 1994. Second European Workshop, 17/18 February, Kings College, London, United Kingdom, Technologies for Environmental Protection, Report EUR 15622 EN.

Bockreis, A. & Jager, J. 1999. Odour Monitoring by the Combination of Sensors and Neural Networks, *J. Environmental Modelling and Software*, 14, 5, 421–426.

Fasham, M.J.R. & Evans, G.T. 1995. The use of Optimization Techniques to Model Marine Ecosystem Dynamics at the JGOFS Station at 470N 200W, *Phil. Trans. R. Soc. Lond. B*, 348, 203–209.

Harmon, R. & Challenor, P. 1997. A Markov Chain Monte Carlo Method for Estimation and Assimilation into Models, *Ecological Modelling* 101, 41–59.

Instrumental Methods of Analysis. Modern Trends and Applications (1999), International Conference Proceedings, Vol. I & II, M. Ochsenkuhn-Petropulu and K.M. Ochsenkuhn (Eds.), Chalkidiki, Greece, September 19–22.

Johannessen, J.A., Roed, L.P., Johannessen, O.M., Evensen, G., Hackett, B., Petterson, L.H., Haugan, P.M., Sanven, S. & Shuchman, R. 1993. Monitoring and modeling of the marine coastal environment, *J. Photogrammetric Engineering and Remote Sensing* 59, No. 3, 351–361.

Johannessen, J.A., Vachon, P.W. & Johannessen, O.M. 1994. ERS-1 SAR Imaging of marine boundary layer process, *ESA*, *Earth Observation Quarterly*, 46, 1–5.

National Handbook of Water Quality Monitoring. 1996. U.S. Depart. of Agriculture, National Resources Conservation Service.

Papadimitrakis, J.A. & Nihoul, J. 1996. Temporal (and spatial) scales and sampling requirements in environmental flows (with emphasis on inland and coastal waters). In N.B. Harmancioglu, M.N. Alpaslan, S.D. Ozkul & V.P. Singh (eds.), *Integrated Approach to Environmental Data Management Systems*, NATO ASI Series, Series 2: Environment- l31, Kluwer Academic Publishers, Dordrecht, pp. 117–132.

Role of operational changes in forming discoloured water in a water distribution system

R.A. Prince
Swinburne University of Technology, Melbourne, Victoria, Australia

G.L. Ryan
South East Water, Melbourne, Victoria, Australia

I. Goulter
Charles Sturt University, Bathurst, New South Wales, Australia

ABSTRACT: The majority of customer complaints received by water companies in Australia and in many parts of the world are of discoloured water. South East Water Ltd., in Melbourne Australia, has undertaken a study to understand discoloured water formation with the aim of reducing the incidences of discoloured water and associated customer complaints. The changes that occur to the water as it travels through a water distribution system may lead to the formation of discoloured water. This paper explores the extent to which known operational events such as burst mains, contributed to these water quality changes and thereby cause discoloured water and associated customer complaints. Continuous online monitoring of flow rate and turbidity at six locations in a water quality zone in the South East Water region were compared to recorded operational events and customer complaints over a 16-month period. It was found that operational changes were attributed to causing over a third of discoloured water events and a smaller proportion of customer complaints. Such results indicate the need for a review of the current operational practices, particularly flushing, and show that not all discoloured water is created by the resuspension of sediment by high velocities.

1 INTRODUCTION

1.1 *Background*

In Melbourne, Australia, discoloured water is usually a brown or yellow colour and has been attributed to the accumulation and subsequent resuspension of colloidal material (Prince et al. 2003). The colloidal material that is accumulated has been attributed to the turbid water entering from the unfiltered source water (0.7–2.3 NTU), as the system has few if any unlined mains. Preliminary evidence suggests that this material is predominately clay and silt (Prince et al. 2000) with no health issues due to the protected catchments from where the water originates. The water is dosed with chlorine for disinfection, lime for pH correction, and fluoride for public health considerations.

The primary method of addressing discoloured water has been to counteract the accumulation of sediment in water mains through cleaning activities. These activities include routine planned flushing from service reservoirs to the extremities of the system, plus isolated flushing following aesthetic related incidents. The cost of these activities is in the order of hundreds of thousands of dollars. Thus it would be beneficial to more precisely target cleaning programs or prevent conditions conducive to creating problems, based on improved understanding of actual conditions of discoloured water formation within the system.

Particles causing problems in the water distribution system can arise from a number of locations: from source water (Yarra Valley Water, 1999; Linn & Coller, 1997; Gauthier et al. 1999) and treatment (Gauthier et al. 2001); or generated within the water distribution system itself from pipe and fitting corrosion, erosion (Stephenson, 1989), biological growth (Stephenson 1989; Clark et al. 1993; Gauthier et al. 1996; Brunone et al. 2000; Gauthier et al. 2001), external contamination such as during pipe repairs (Gauthier et al. 1996), and chemical reactions from the formation of iron or manganese oxides (Gauthier et al. 1996; Lin & Coller 1997; Walski, 1991; Sly et al. 1989; Stephenson, 1989). Results to date indicate that the majority of material in Melbourne's water mains arises from source water. The Melbourne system receives unfiltered water of turbidity typically less than 2 NTU, which is only likely to cause an aesthetic issue if concentration of particles occurs (most likely due to accumulation in the water mains). Whilst this paper draws on observations from an unfiltered distribution system, it provides approaches that are applicable to many reticulated water supplies. Discoloured water formation and discoloured water complaints are an issue for both filtered and unfiltered systems.

1.2 *Discoloured water formation*

It is generally accepted in literature and in industry that any particles that are deposited at the pipe wall are resuspended into the bulk flow when a critical minimum shear stress is exceeded (Prince et al. 2003; Boxall et al. 2001; Walski & Draus, 1996; Hoven & Vreeburg, 1992; Walski, 1991). The frequency of these events is related to the frequency at which this minimum shear stress is exceeded, and the severity is due to the amount of material that has accumulated i.e. the time between critical sheer stress exceedences. Thus the worst discoloured water events will occur when conditions are favourable for accumulation of material and there is a significant time interval between critical shear stress exceedences. This includes transfer mains that are oversized or during winter flows where the sheer stress only infrequently exceeds the critical minimum. In such circumstances operational events such as flushing procedures, bursts, and fire fighting may result in a sudden increase in demand greater than normal daily flow, could cause a discoloured water event.

A literature survey was able to locate only a few articles that directly referred to operational changes, with all other articles classifying them as "abnormal events". Chambers (2000), also commented on the lack of data and reports on operational events, and reported on a collation of experience of selected staff from UK water companies. Activities within the water distribution system that may lead to discoloured water reported by Chambers (2000) included: (high risk) burst and mains repairs; (medium risk) operation and maintenance of valves, step tests for leakage control, operation of hydrants (for fire fighting, sample collection or bowser filling), recommissioning of out-of-service mains, mains recharging, change of supply; (low risk) pump switching and poor service reservoir maintenance. Other papers that mention operational events causing discoloured water events include Hoven & Vreeburg (1992) – hydrant operations, Gray (1994) – repair and maintenance work.

2 METHOD

2.1 *Study Zone*

To measure the role of operational changes in causing discoloured water, Wantirna Water Quality Zone within the South East Water region was chosen as the Study Zone. Wantirna Water Quality Zone shown in Figure 1. It was chosen because it was a predominately residential, small, self-contained, hydraulically simple system, typically supplied with water from one direction through a service reservoir comprising of two separate tanks. In 2001/2002 it supplied 8890 properties and had above average number of customer complaints. A more detailed explanation of the Study Zone can be found in Prince et al. (2003, 2001, 2000).

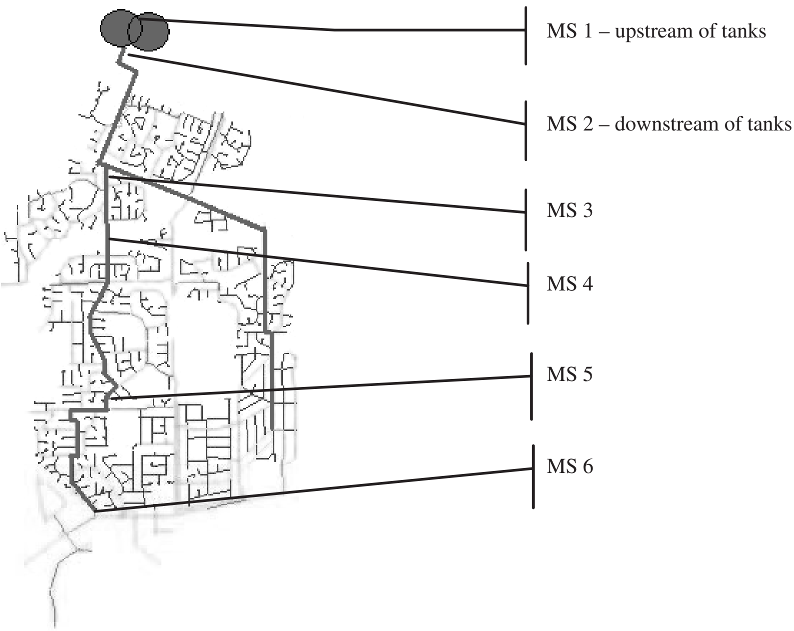


Figure 1. Schematic of the Wantirna Study Zone showing the six monitoring sites (MS).

2.2 *Data sources*

Six monitoring sites were installed on the west 450 mm (ID) transfer main (Figure 1) and monitoring was conducted from February 2001 to June 2002. Each monitoring site measured flow rate and turbidity with equipment housed in a cabinets placed in the nature strip or public land beside the water main. Site locations were chosen to maximize the coverage of the Study Zone while maintaining the ability to monitor the movement of the discoloured water through the system. Monitoring Sites 1, 2 and 3 were used to indicate if a discoloured water event had come directly from source water, while Monitoring Sites 3, 4, 5 and 6 were used to identify the location and measure the intensity of discoloured water events.

To study the formation of a discoloured water event and determine its probable cause the following were used: continuous online data for turbidity and flow rate; a monitoring site visitation and event log; discoloured water customer complaint data; operational event data; fire fighting location information; operator's and field contractor's journals; and expert advice. A hydraulic model was used to determine the areas downstream of each monitoring site that would receive discoloured water if a turbidity spike was detected. All operational events and customer complaints that occurred within these areas could thus be linked to an upstream and downstream Monitoring Site (MS).

2.3 *Preparation of continuous online turbidity results.*

It was found that the turbidimeters required cleaning every two weeks due to algae growth and sediment accumulation (after discoloured water events). To remove any additional error to results that may have occurred, a background datum from median measured results was calculated between each two week cleaning cycle to establish a baseline for analysis. The median turbidity values were used for analysis plus the standard error of the turbidimeter, because a median value would be less affected by spikes of maximum or minimum readings than a mean and therefore give a better indication of normal background readings. In this way results between two week cleaning cycles could be used as a continuous measurement. Turbidity minus the background datum was called Spike Turbidity (SNTU). The average background turbidity for the system was 1.4 NTU.

2.4 *Identification of discoloured water spikes*

For a turbidity event to become a customer complaint, the recorded turbidity would need to reach levels where the customer could detect the turbidity. Australian Drinking Water Guidelines (1996) recommend that turbidity in drinking water be kept below 5 NTU for aesthetic considerations, because this is the level at which a customer will see turbidity in a glass of water (NHMRC, 1996). Therefore a case study of discoloured water events captured with continuous online testing should at least include all events where the turbidity exceeded 5 NTU. To allow for the testing of this turbidity complaint threshold, major turbidity events of less than 5 NTU were also observed. As the average background datum was 1.4 NTU, a major discoloured water event was defined as having a maximum turbidity exceeding 2.6 NTU.

3 RESULTS

Table 1 shows the major turbidity events that occurred during the period of investigation, categorised by the attributed cause. Some 15 of these spikes where concluded to be due to disturbances to the turbidimeter during meter cleaning or verification of calibration, blocked valves, ant infestation and algae growth (i.e. not real), as determined from maintenance logs and site visits. Disturbances of the turbidimeter were more common at the beginning of the sampling period than at the end due to improved technique, ant nest extermination and replacement of valves. It is advised that for anyone installing online turbidimeters in the field that turbidimeters be checked regularly (at least fortnightly) and a site visitation log kept. These spikes could have confused results if not identified correctly. The next largest categories were major spikes that were attributed to mains flushing (5), spikes that entered the system from above the service reservoir (5) and spikes that were formed in the water distribution system but with no obvious cause (5).

The detailed categories listed in Table 1 can be grouped together to form causation groups. This grouping (Figure 2) indicated that, the largest proportion of turbidity spikes occurred during events that created abnormally high water velocities (57%) however a significant percent of spikes did not seem to be a result of high velocity but were created in the water distribution system (27%). It was possible that they were created by at shear force through instantaneous pressure variance such as water hammer or as a result of vibration of the main, both of which have been noted

Table 1. Count of major turbidity spike events by attributed cause for the Wantirna Water Quality Zone between 12 February 2001 and 4 June 2002.

Attributed cause	Total
Not real	15
Cause unknown, not flow related	5
Flushing	5
From above the service reservoir	5
Cause unknown, flow related	3
Normal demand	3
Burst main	2
Mains repair, not flow related	2
Burst hydrant	1
Hydrant repair	1
Authorised hydrant use	1
Fire sprinkler system malfunction	1
Change in supply, not flow related	1
Grand total	45

as contributors. 11% of the major spike events arose from outside the study area. The most significant of these events was not a discoloured water event but white water, which arises from air entering the main due to an upstream incident.

Figure 3 shows that the largest proportion of the real discoloured water events that were created within the water distribution system were caused by operational changes: 40% of turbidity events could be attributed to operational events within the distribution system. 27% were caused within the distribution system but the cause was attributable.

Table 2 shows 20 discoloured water customer complaints (from 19 events) were recorded from February 2001 to June 2002. Of these, only 10 complaints were recorded within three days of a Major Turbidity Event (to allow for the fact that the date recorded against complaints is the day the customer rang the water company, not the day the customer saw discoloured water). The shaded area on the table represents the location where the complaint was recorded. It shows that the spike turbidity recorded in the transfer main in the study area was not always above the complaint threshold when a complaint occurred: in fact, only 50% of complaints could be explained by major turbidity events observed in the transfer main. Four discoloured water customer complaints occurred when major turbidity events were caused by higher than average velocity events; three were attributable to events that were not flow related.

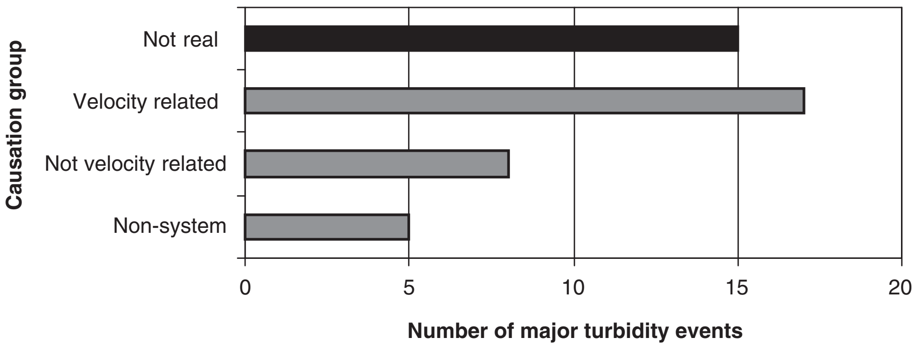


Figure 2. Number of major turbidity events grouped by causation group that occurred in the 16 month sampling period in the study zone.

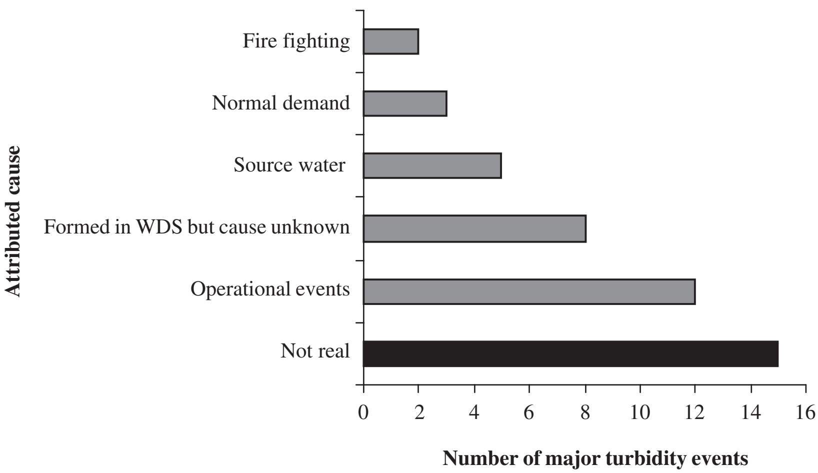


Figure 3. Number of major turbidity events that occurred in the 16 month sampling period in the study zone grouped by type of attributed cause.

Table 2. All Discoloured Water Customer Complaints (DWCC) between Feb. 2001 and June 2002 showing maximum recorded SNTU in proceeding 3 days and operational event attributed to causing major turbidity event. Shaded area indicates the Monitoring Site between where the complaint occurred.

Date of DWCC	Type	Spike Turbidity (NTU) MS 1	MS 2	MS 3	MS 4	MS 5	MS 6	Major spike attributed cause
2-Mar-01	Brown	0.2		0.3		4.9		Cause unknown, not flow related
4-Mar-01	Brown	0.2		0.0		4.9		Cause unknown, not flow related
7-Mar-01	Brown	0.6		0.2		0.3		
9-Mar-01	Brown	0.4		0.2		0.2		
20-Mar-01	Black	0.8		0.0		0.1		
5-Apr-01	Brown	0.3		0.1		0.1		Not Real
11-Apr-01	Brown	0.1		6.9		1.6		Cause unknown, not flow related
24-Apr-01	Brown	0.4		2.4		0.0		Burst main
26-Jun-01	Brown	0.2				0.4	1.3	
17-Aug-01	Brown	0.0		0.1	0.0	0.0	0.8	
23-Aug-01	Brown	0.3		0.5	2.1	8.8	0.0	Burst hydrant
9-Sep-01	Brown			10.0	10.0	10.0	10.0	Above Service Res. main repair
20-Sep-01	Brown	0.0		0.1	0.0	0.1	0.2	
25-Sep-01	Brown	0.1	3.2	1.4	1.8	1.4	0.5	Mains repair
8-Nov-01	Brown	0.3	0.0	0.0	0.0	0.0	0.4	
4-Jan-02	Brown	0.0	0.5	0.0	0.0	0.0	0.0	
30-Jan-02	Brown	0.1	0.2	0.1	8.6	0.0	0.3	Spike through tank
22-Feb-02	Brown	0.2		0.4	0.0	0.4	2.7	Normal demand
18-Mar-02	Brown			0.3	0.1	0.7	0.1	
22-May-02	Brown			0.0	0.0	0.0	0.0	

4 DISCUSSION

In this paper results of analysis of continuous online testing (COLT) of turbidity and flow, collected from six monitoring sites on a transfer main in the Wantirna Water Quality Zone were reported. The transfer main had distribution system off-takes along its length, allowing for variations in flow conditions in sections of the zone. Continuous online testing of turbidity allowed for the testing of the episodic and transient nature of the discoloured water events. Recorded operational events and other recorded events (such as fire fighting) within the water distribution system were linked to discoloured water events, as defined by major turbidity events, to determine the type of phenomenon that create conditions for discoloured water formation.

It was apparent that median turbidity values could lead to the conclusion that the transfer main was effectively clear of any particulate material, however during times of abnormal flow, discoloured water spikes generated within the water distribution system exceeding 10 NTU, indicated the likely accumulation of settled material at the pipe wall. Thus samples taken under normal flow conditions do not necessarily represent the water quality at times of abnormal flow.

Flushing of isolated areas without ensuring that upstream water mains are free from material (or that this material will not be suspended at the rate of velocity generated) may lead to discoloured water being formed further up the system. This is particularly poignant within a transfer main, as this could potentially lead to discoloured water reaching a large area.

There was not a strong link between the occurrence of a discoloured water event and discoloured water customer complaints. Customers do not respond to all discoloured water events. The response of customers is a complex issue. Not all customers would see an event, not all customers are inclined to complain.

5 CONCLUSIONS

Most discoloured water was formed from velocity in the main exceeding normal peak velocity conditions. Of these events, most were associated with flushing of a water main somewhere else in the water distribution system. Most discoloured water events that were not caused by higher velocities were attributable to high concentrations of turbidity entering from outside the Wantirna zone.

Not all turbidity events were noticed by customers, of those that were observed, four discoloured water customer complaints could be attributed to higher than normal velocity events and three were attributable to non-flow related events.

REFERENCES

Boxall, J.B., Skipworth, P.J. & Saul, A.J. 2001. A novel approach to modelling sediment movement in distribution mains based on particle characteristics. *Proceedings of the Computing and Control in the Water Industry Conference UK: De Monfort University, 3–5th September 2001*. UK.

Brunone, B., Karney, B.W., Mecarelli, M. & Ferrante, M. 2000. Velocity Profiles and Unsteady Pipe Friction In Transient Flow. *Journal of Water Resources Planning and Management* 126(4): 236–44.

Clark, R.M., Grayman, W.M., Males, R.M. & Hess. A.F. 1993. Modeling Contamination Propagation in Drinking Water Distribution Systems. *Journal of Environmental Engineering*, 119(2): 349–54.

Chambers, K. 2000. Drinking Water Quality and Health – Distribution Systems – DW-03. *Operational and maintenance strategies for maintaining water quality in distribution systems*, 00/DW/03/12. WRc plc, London, UK.

Gauthier, V., Barbeau, B., Millette, R. & Prévost. M. 2001. Suspended Particles in the drinking Water of two Distribution Systems. *Water Science and Technology: Water Supply* 1(4): 237–45.

Gauthier, V., Gérard, B., Portal, J.M., Block, J.C. & Gatel, D. 1999. Organic Matter as Loose Deposits in a Drinking Water Distribution System. *Water Research* 33(4): 1014–26.

Gauthier, V., Rosin, C., Mathieu, L., Portal, J.M., Block, J.C., Chaix, P., & Gatel, D. 1996. Characterization of the deposits in drinking water distribution systems. *Proceedings of Water Quality Technology Conference of the American Water Works Association.*

Gray, N.F. 1994. *Drinking Water Quality: problems and solutions*. Chichester, New York: John Wiley and Sons.

Hoven, T.J.J, & Vreeburg, J.H.G. 1992. Distribution system analysis by continuous monitoring and network calculation. *Water Supply* 10: 117–24.

Lin, J. & Coller, B.A. 1997. Aluminium in a water supply, Part 3: Domestic Tap Waters. *Water: Journal of Australian Water and Wastewater Association*. 11–13.

National Water Quality Management Strategy. 1996. Australian Drinking Water Guidelines – 1996. *National Water Quality Management Strategy, Australian Drinking Water Guidelines – 1996*, National Water Quality Management Strategy, Australia.

Prince, R., Goulter, I. & Ryan, G. 2001. Relationship between Velocity Profiles and Turbidity Problems in Distribution Systems. *World Water and Environmental Resources Congress Orlando, Florida*: American Water Works Association.

Prince, R.A., Goulter, I. & Ryan, G. 2003. What causes customer complaints about discoloured drinking water? *WATER, Journal of the Australian Water Association* 30(2): 62–68.

Prince, R., McManus, K. & Goulter, I. 2000. Colour, Turbidity Levels and Dirty Water Customer Complaints – Water System Performance Indicators? *Xth World Water Congress – Sharing the Waters of the World, Melbourne, Australia*: IWA.

Sly, L.I., Hodgkinson, M.C., & Arunpairojana, V. 1989. Deposition of Manganese in Drinking Water Distribution System. *Journal ClO2:* Web page, [accessed November 1999]. Available at www.clo2.com/reading/drinking/system.html.

Stephenson, G. 1989. *Removing Loose Deposits From Water Mains: Operational Guidelines, A Source Document For The Water Mains Rehabilitation Manual*. Water Research Centre, Swadon. UK.

Walski, T.M. 1991. Understanding Solids Transport in Water Distribution Systems. *Proceedings Water Quality Modelling In Distribution Systems*, pp. 305–9.

Walski, T.M. & Draus, S.J. 1996. Predicting Water Quality Changes During Flushing. *American Water and Wastewater Association Annual Conference*, pp. 121–29.

Yarra Valley Water. 1999. *Discussion Paper: Dirty Water Complaints in Courts and Dead End Streets* [Unpublished Draft]. Yarra Valley Water, Melbourne, Australia.

Multi-objective genetic algorithm for optimal scheduling of chlorine dosing in water distribution systems

C.J. Rouhiainen & M.O. Tade
Department of Chemical Engineering, Curtin University of Technology, Perth, Australia

G. West
Department of Computing, Curtin University of Technology, Perth, Australia

ABSTRACT: This paper presents two new multi-objective genetic algorithm models using a Pareto-based selection technique for determining the optimal schedule of chlorine dosing within a water distribution system with multiple, competing objectives: primarily disinfection control and aesthetic control. Formulating the optimal dosing problem using a Pareto-based multi-objective genetic algorithm offers water utilities an innovative solution technique, with each optimisation simulation returning a Pareto-optimal set of candidate solutions. An overview of the object-oriented application architecture, encapsulating each model, is also given, including the independent linked design with the water network simulation program. Both models were applied to a hypothetical distribution system, including comparison with existing non-Pareto techniques. Results are discussed, including the advantages of using the new Pareto-based multi-objective models.

1 INTRODUCTION

Chlorine is a common disinfectant used in drinking water distribution systems to ensure customers receive safe, pathogen free drinking water. However, the disinfection potential of chlorine is reduced as water travels through the distribution system interacting with the physical, chemical and biological environment (Powell et al., 2000). To ensure satisfactory chlorine levels at the extremities of the distribution system, dosing of chlorine at the start of the system may need to be high. However, research has shown that high chlorine levels can result in taste and odour problems (Tansley & Brammer, 1993) or possible health problems (Chlorine Chemistry Council 1997), caused by excessive disinfection by-products. Therefore, monitoring and controlling the levels of chlorine within the distribution system is an important area for the water industry. To achieve the necessary disinfection control as the water flows from the outlet of the water treatment plant (usual location for post-treatment chlorine dosing) to the customer taps, whilst also satisfying aesthetic control and minimising disinfection by-products, advanced techniques are necessary in order to predict, monitor and control the chlorine levels within the distribution system. One such technique consists of using optimal scheduling models.

Several studies clearly demonstrate the need for optimal scheduling models to help maintain chlorine residuals within the distribution system within prescribed limits (Levi & Mallevialle, 1995; Uber et al., 1996). This problem has been partially addressed by developing optimal chlorine booster disinfection scheduling models (Tryby et al., 1997; Boccelli et al., 1998; Tryby et al., 1999; Nace et al., 2001). However, these scheduling models assume a first-order chlorine decay algorithm, simplifying the complex nonlinear optimisation problem. Studies have shown that first-order chlorine decay algorithms do not adequately represent the system, due to the complex physical, chemical and biological reactions that occur in water as it travels from treatment plant to customer taps. To overcome this limitation Rouhiainen & Tade (2003) developed an optimal scheduling model using a single objective genetic algorithm. This model is capable of handling improved nonlinear chlorine decay algorithms by separating the genetic algorithm code from the network simulation code.

Existing optimal scheduling models solve the problem of disinfection control and aesthetic control by essentially combining these two competing objectives to create a "quasi" single objective problem, for solution using single objective optimisation techniques. For example, the most recent scheduling model (Rouhiainen & Tade, 2003) uses the classic weighted-sum method, where each objective is assigned a weight (the sum of weights equals one). However, the disadvantages of this approach and other "quasi" single objective formulations are that the trade-off relationships between the two objectives (in this case disinfection control and aesthetic control) are often unknown *a priori* and in the case of the weighted-sum method the model is often sensitive to the weighting factors used.

To address these limitations, this paper presents two new optimal scheduling models using a Pareto-based multi-objective genetic algorithm. The models do not require prior knowledge of objective priorities and each aims to produce a Pareto-optimal set of candidate solutions, where a decision maker, typically a water utility operator or manager, can select the best solution (from the Pareto-optimal set) given new knowledge (post optimisation) of the trade-off relationship between each objective. The models are also capable of handling improved nonlinear chlorine decay algorithms by separating the multi-objective genetic algorithm code from the network simulation code, EPANET (Rossman 2000). An overview of the application architecture encapsulating each model is given. Both models were applied to a hypothetical distribution system, including comparison with existing non-Pareto techniques.

2 MULTI-OBJECTIVE OPTIMISATION

Most real-world optimisation problems involve multiple and conflicting objectives that need to be tackled separately while respecting various constraints, leading to an overwhelming problem complexity. If there is more than one objective function and it is preferred that they be treated separately, such as maximising disinfection control whilst minimising aesthetic concerns, the problem is defined as a multi-objective optimisation problem. The general (minimisation) form of a multi-objective optimisation problem given Z objective functions and n decision variables follows:

$$\boxed{\left.\begin{array}{lll} \text{Minimise} & f_Z(\mathbf{x}), & z = 1, 2, \ldots, Z \\ \text{subject to} & g_J(\mathbf{x}) \geq 0, & j = 1, 2, \ldots, J \\ & h_K(\mathbf{x}) = 0, & k = 1, 2, \ldots, K \\ & x_i^L \leq x_i \leq x_i^U, & i = 1, 2, \ldots, n \end{array}\right\}} \quad (1)$$

Equation 1 can also be stated as a maximisation problem by multiplying all Z objective functions by -1. A solution $\mathbf{x}$ is defined as a vector of n decision variables with lower and upper bounds x^L and x^U respectively. The problem can also be subject to J inequality and K equality constraints.

With multi-objective optimisation problems there does not necessarily exist a solution that is best with respect to all objectives. A solution may be best for one objective but worst for another. However, there usually exists a set of solutions, called *non-dominated* solutions, where no improvement is possible in any objective function without sacrificing at least one of the other objective functions (refer to Fig. 1 for an example). This non-dominated set is also referred to as the Pareto-optimal set, the admissible set, the efficient points, the non-dominated frontier, or the *Pareto Front* (PF_{TRUE}). Refer to Deb 2001 & Coello Coello et al., 2002 for definitions of domination, non-domination and Pareto-optimal.

Multi-objective optimisation problems have received increased interest from researchers with various backgrounds since early 1960 (Gen & Cheng, 2000). In the past few years, there has been

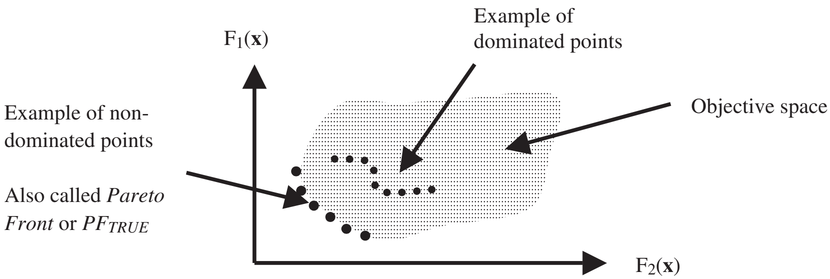


Figure 1. Example of PF_{TRUE} for two-objective (minimisation) problem.

an increase in applying genetic algorithms (due to their success) to solving multi-objective optimisation problems, also referred to as evolutionary multi-objective optimisation. The genetic algorithm is a highly suitable technique for solving multi-objective optimisation problems, which solves the optimisation problem in parallel, using a "population" of potential candidate solutions.

3 PROBLEM FORMULATION

The problem presented in this paper has two primary competing objectives: (1) disinfection control and (2) aesthetic control.

Disinfection control is achieved by ensuring adequate chlorine residual, nominally greater than 0.1 mg/L, is maintained at all customer demand points within the distribution system. This is formulated as a minimisation problem as shown in Equation 2.

$$\text{Minimise } F_d(\mathbf{x}) = \sum_{m=1}^{M} \left(C_{dr} \sum_{t=TS_S}^{TS_e} \left(\left| \min\left[\left(u - u_{\min} \right), 0 \right] \right| \right) S_m f_T \right) \tag{2}$$

where: m = monitor node, M = number of nodes monitored for disinfection control, C_{dr} = cost rate ($ per consumer service per day), t = time interval (nominally 60 minutes), TS_s = start time for monitor node m, TS_e = end time for monitor node m, S_m = number of consumer services at node m, u = model predicted chlorine residual (mg/L) at node m at time interval t, u_{min} = minimum chlorine residual (mg/L) to achieve maximum disinfection control (nominally 0.1 mg/L), f_T = fraction of day (nominally 1/24). The cost rate per service per day C_{dr} reflects the cost to the community associated with the risk of drinking water containing pathogens. The start time interval TS_s is the time of the first non-zero chlorine residual at monitoring node m calculated by running the network simulator with a short pulse of chlorine at $t = 0$ at the chlorine dosing station. The end time interval TS_e is calculated as the sum of the start time interval plus the duration of chlorine dosing (nominally 24 hours).

The goal of aesthetic control is to minimise taste and odour problems associated with high chlorine residuals, nominally greater than 0.6 mg/L. This is formulated as a minimisation problem as shown in Equation 3.

$$\text{Minimise } F_a(\mathbf{x}) = \sum_{n=1}^{N} \left(C_{ar} \sum_{t=TS_S}^{TS_e} \left(\max\left[\left(u - u_{\max} \right), 0 \right] \right) S_n f_T \right) \tag{3}$$

where: n = monitor node, N = number of nodes monitored for aesthetic control, C_{ar} = cost rate ($ per consumer service per day), t = time interval (nominally 60 minutes), TS_s = start time for monitor node n, TS_e = end time for monitor node n, S_n = number of consumer services at node n,

u = model predicted chlorine residual (mg/L) at node n at time interval t, u_{max} = maximum chlorine residual (mg/L) for aesthetic control (nominally 0.6mg/L). The cost rate per service per day C_{ar} reflects the cost to the community associated with complaints for drinking water with taste and odour problems. TS_s, TS_e, and f_T are calculated as per Equation 2.

To solve the above multi-objective problem, to produce an optimal 24 hour dosing schedule, a solution **x** represents 24 decision variables, where each decision variable represents a dose rate for each one hour time interval.

4 SOLUTION PROCEDURE

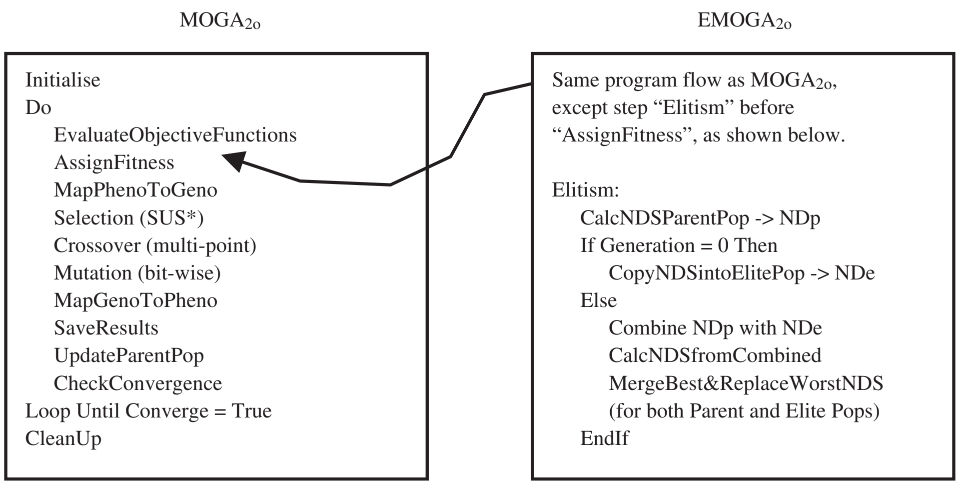
The multi-objective optimisation problem is solved using a new application OGAT (Object-oriented Genetic Algorithm Toolkit), incorporating two new Pareto-based multi-objective genetic algorithm models: (1) a multi-object genetic algorithm ($MOGA_{2o}$) and (2) an elitist multi-object genetic algorithm ($EMOGA_{2o}$). Program flows for each of these models are shown in Figure 2. The structure of OGAT, including an example (using $EMOGA_{2o}$) with typical program flow, is shown in Figure 3.

4.1 *Model development*

Although $MOGA_{2o}$ and $EMOGA_{2o}$ were designed and developed using all new object-oriented programming code, the fitness assignment algorithm within each model is based on the fitness assignment procedure from the existing Pareto-based multi-objective genetic algorithm (called MOGA) developed by Fonseca and Fleming (1993). Future reference to the MOGA model, within this paper, will be referred to as Fonseca and Fleming's MOGA model. All other citing of the abbreviation MOGA will be short for "multi-objective genetic algorithm".

Coello Coello et al. (2002), presents a comprehensive review of all known multi-objective "evolutionary" algorithms (MOEAs) and specifically suggests using algorithms, such as Fonseca and Fleming's MOGA model, which use known MOEA theory, including Pareto-based selection, niching and fitness sharing.

The main difference between $MOGA_{2o}$ and $EMOGA_{2o}$ is that the latter incorporates elitism. In this case a secondary population is used to store the current non-dominated solutions (NDS) from



*Stochastic Universal Sampling

Figure 2. Program flow diagrams for $MOGA_{2o}$ and $EMOGA_{2o}$.

the set of solutions found so far. In summary this secondary population is combined with the current population of the next generation and better NDS are updated in both populations. This elitism mechanism is initially only a simple algorithm (still under development) and does not include all the necessary features to prevent premature convergence or guarantee diversity along the Pareto Front. However, as will be shown later, preliminary results show the addition of this simple elitism component still proved beneficial.

Both models support binary-parameter or gray-parameter representation, use proportionate fitness selection (stochastic universal sampling), multi-point crossover and bit-wise mutation.

4.2 *Software architecture for model implementation*

The new application (OGAT) was designed and developed using several new object-oriented code components linked to an existing network simulation code EPANET (Rossman 2000). New components were written using the Microsoft Visual Basic (VB) programming language and are compiled as either stand-alone executables (VB EXE) or dynamic link libraries (VB DLL). All VB objects support Microsoft's Component Object Model (COM) architecture. EPANET, a free open source Windows™ based application, is used to calculate the hydraulic and water quality system dynamics for the water distribution system. The core network analysis algorithms for EPANET

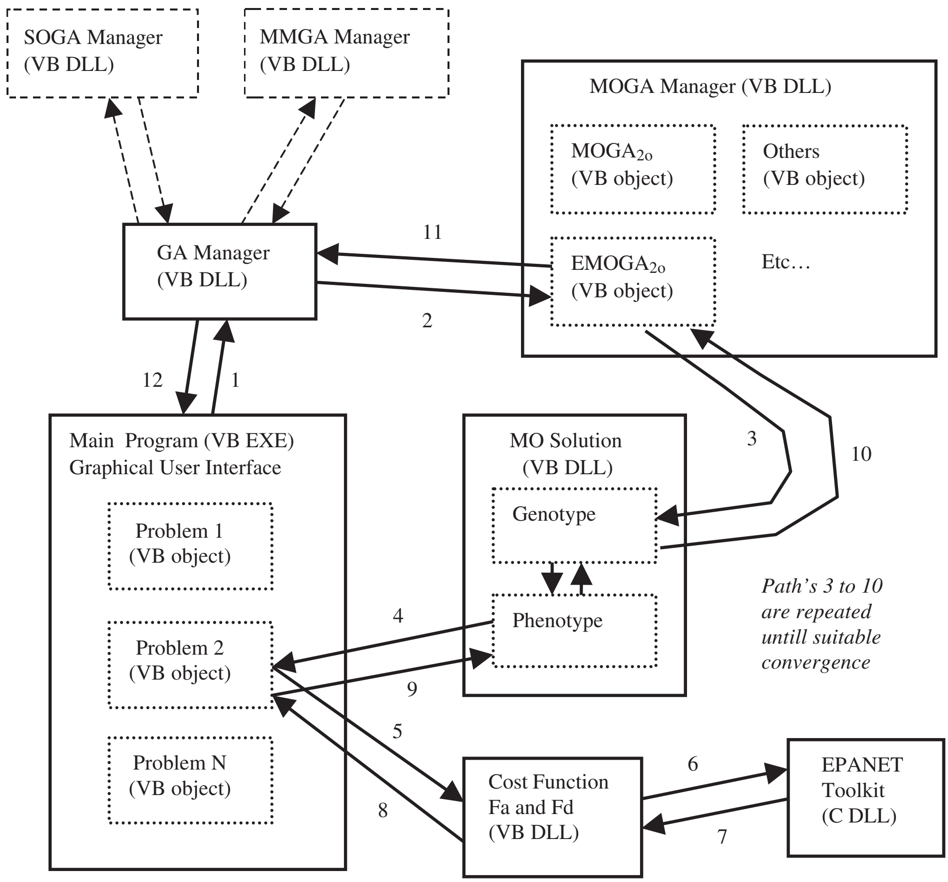


Figure 3. Program flow diagram for OGAT (with example showing program flow for $EMOGA_{2o}$).

were designed and developed using the C programming language and are packaged as a dynamic linked library (C DLL). EPANET is dynamically linked to OGAT objects using the EPANET Programmer's Toolkit (Rossman, 1999).

The overall object-oriented design ensures each major piece of program code is encapsulated from other code and provides a stable framework for future improvements. For example, changes to program code within $MOGA_{2o}$ will not cause unnecessary changes (e.g. errors) to program code within other model components. This ensures the progressive development of a robust genetic algorithm optimisation toolkit.

OGAT has six major components as shown in Figure 3: (1) Main Program, (2) GA Manager, (3) SOGA Manager, (4) MMGA Manager, (5) MOGA Manager, and (6) MO Solution.

The main program provides the graphical user interface and can host multiple single objective (SO), multi-modal (MM) or multi-objective (MO) problems and their associated data. For example, Figure 3 shows the flow paths for a particular multi-objective optimisation problem (called Problem 2) where the Problem 2 "object" contains all relevant (problem specific) optimisation details, such as the number of objective functions (including pointers to any custom function solver) and their definitions, the number of decision variables including minimum precision and lower and upper bounds, and custom load data and save data functions.

The GA Manager hosts three main genetic algorithm model managers, SOGA Manager, MMGA Manager, and MOGA Manager. Due to space limitations, the details of SOGA and MMGA models are not discussed in this paper. The MOGA Manager currently hosts two object models: $MOGA_{2o}$ and $EMOGA_{2o}$. The last major component, MO Solution, contains the code for each solutions phenotype (e.g. decision variables) and genotype (e.g. binary string) structure, including objective function values and associate fitness values. MO Solution also contains the mapping code between the phenotype and genotype structures.

5 MODEL APPLICATION

Both $MOGA_{2o}$ and $EMOGA_{2o}$ models were applied to a hypothetic distribution system (Fig. 4). For ease of presentation all network details are omitted. The goal was to produce the Pareto-optimal set of solutions (PF_{TRUE}) by solving Equations 2 and 3. The decision variable bounds, shown in Figure 5, were calculated using the previously developed hybrid elitist (single-objective) genetic algorithm model HEGA (Rouhiainen & Tade, 2003). HEGA uses the weighted-sum method to convert the two objective functions (Equations 2 and 3) into a "quasi" single objective problem, for solution using a single-objective genetic algorithm. The lower bound curve was produced using a weighting factor of 1 for 100% aesthetic control (all monitoring nodes below 0.6 mg/L). The upper bound curve was produced using a weighting factor of 1 for 100% disinfection control (all monitoring nodes have a chlorine residuals greater or equal to 0.1 mg/L).

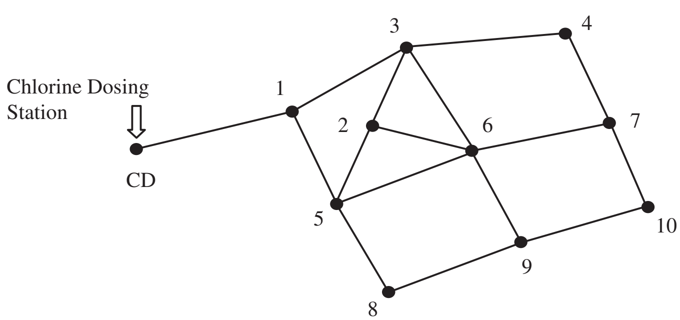


Figure 4. Hypothetical distribution system (showing dosing at node CD and demand nodes 1 to 10).

5.1 *Finding* PF_{TRUE} *using non-Pareto model (HEGA)*

To evaluate the effectiveness of the two new models ($MOGA_{2o}$ and $EMOGA_{2o}$) it is often beneficial to know the true Pareto-optimal front (PF_{TRUE}). One way of doing this was by using HEGA (a non-Pareto model), previously developed by Rouhiainen & Tade (2003), which is based on the weighted-sum method. Using a set of seven uniformly spaced weighting factors for aesthetic control, ranging from 0 to 1, seven solutions (which may represent PF_{TRUE}) are compared. As can be seen by Figure 6, uniformly spaced weighting factors do not always equate to uniformly spaced solutions. Five additional solution points had to be found using trial and error weighting factors, between 0.01 and 0.1245, to determine the approximate curve for PF_{TRUE}.

5.2 *Finding* PF_{TRUE} *using enumeration*

Another way to determine PF_{TRUE} is using enumeration. An attempt was made to find PF_{TRUE} by solving all possible solutions Fx_{ALL} and calculating the non-dominate set PF_{TRUE} from Fx_{ALL}.

Using the lower and upper bounds shown in Figure 5 and assuming a minimum desired precision of 0.1 mg/L per decision variable, the total search space equates to 2.02E + 31 possible solutions. Considering each solution requires running the EPANET network simulation (to evaluate the objective functions defined in Equations 2 and 3) and estimating the simulation time as 0.05 seconds per solution, the total time to do full enumeration is estimated as 1.17E + 25 days. Therefore, finding PF_{TRUE} using total enumeration is not feasible.

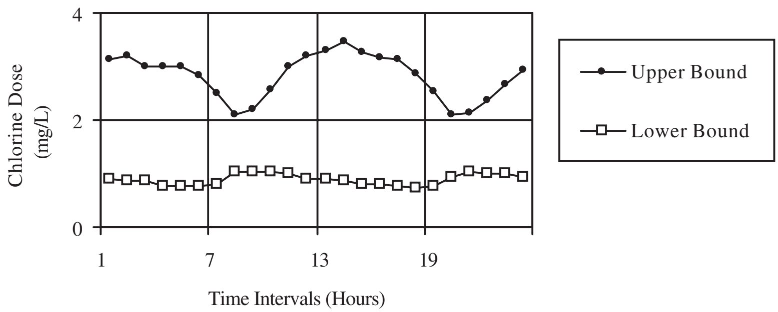


Figure 5. Decision variable bounds calculated using HEGA.

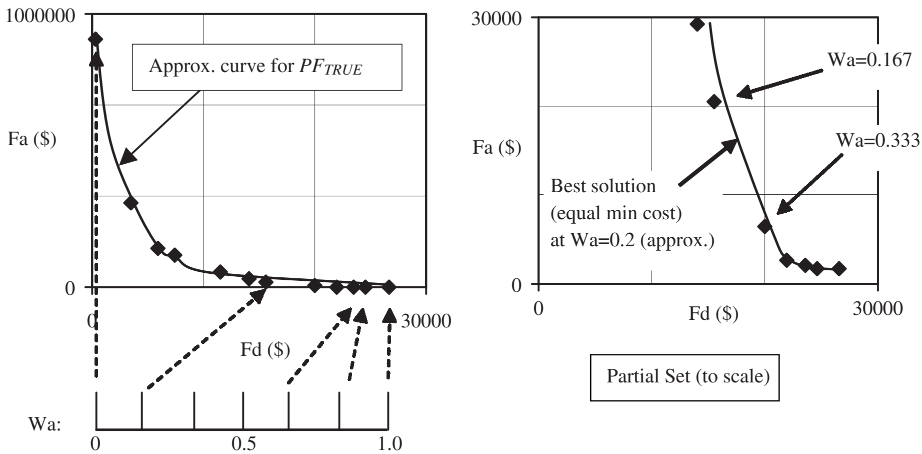


Figure 6. Approximate PF_{TRUE} using HEGA (single-objective weighted-sum method).

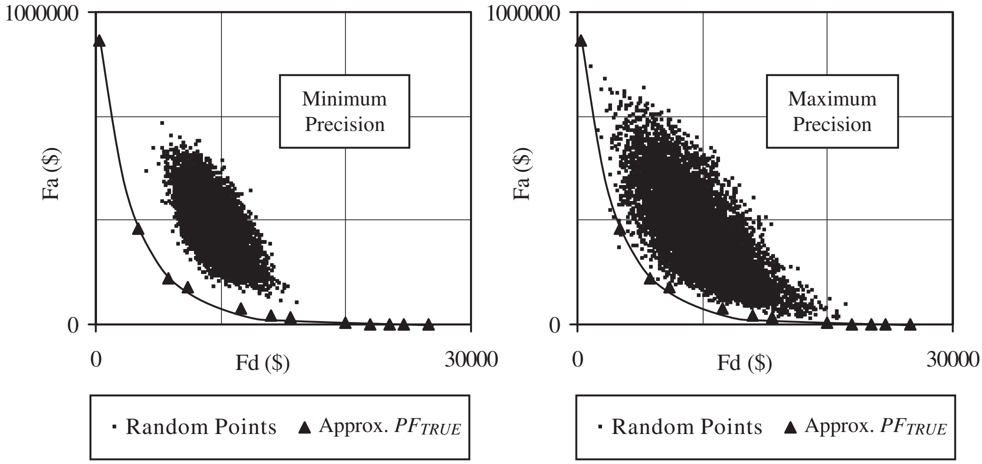


Figure 7. Density of objective space using partial enumeration (10,000 random points).

Although it was not possible to use enumeration to find PF_{TRUE}, enumeration can still provide valuable information about the density of the objective space and possibly the "landscape" of the search space. Using partial enumeration 10,000 random points were evaluated (from within the search space) and 10,000 solutions in objective space were plotted alongside the approximate curve for PF_{TRUE} found previously using HEGA. Figure 7 shows two examples graphing the density of objective space using two different decision variable precisions.

The left graph assumes a minimum decision variable precision of 0.1 mg/L. The total number of solutions plotted equates to only 4.95E-26% of the total possible search space. The right graph assumes a maximum possible precision, calculated by assuming two dose values per decision variable: the lower bound and the upper bound (see Fig. 5). The total number of solutions plotted equates to 5.96E-02% of the total search space. The results shown in Figure 7 indicate that the majority of solutions lie near the centroid defined by: Fa = \$300,000 and Fd = \$10,000. As will be shown later, the density of solutions within the object space hinders the ability of the new Pareto-based models to find all points within PF_{TRUE}.

5.3 *Finding* $\mathrm{PF}_{\mathrm{TRUE}}$ *using a Pareto-based model* ($MOGA_{2o}$)

A good Pareto-based MOGA model should find all points within PF_{TRUE} as well as maintain diversity along the Pareto-optimal front. As with single-objective genetic algorithm models most MOGA models also require "tuning" of parameters, such as population size (N), probability of crossover (Pc) and probability of mutation (Pm). Without prior knowledge of algorithm performance the following nominal starting parameters are commonly used: $N = 100$, $Pc = 0.9$, and $Pm = 0.01$. Figure 8 shows the last generation of non-dominated solutions, using a population size $N = 100$ (left graph) and $N = 500$, for example, (right graph).

In this example, increasing the population size improves the model's ability to find more points in PF_{TRUE}. However, it was observed that (without an elitism preserving strategy) some Pareto-optimal solutions found in previous generations were lost from the final generation. Hence, the motive for developing an elitist version of MOGA_{2o}, called EMOGA_{2o}.

5.4 *Finding* $\mathrm{PF}_{\mathrm{TRUE}}$ *using an elitist Pareto-based model* ($EMOGA_{2o}$)

The elitist version of MOGA_{2o} (EMOGA_{2o}) can find more points along the Pareto-optimal (near optimal) front. We say near optimal as we do not know the true Pareto-optimal front (PF_{TRUE}).

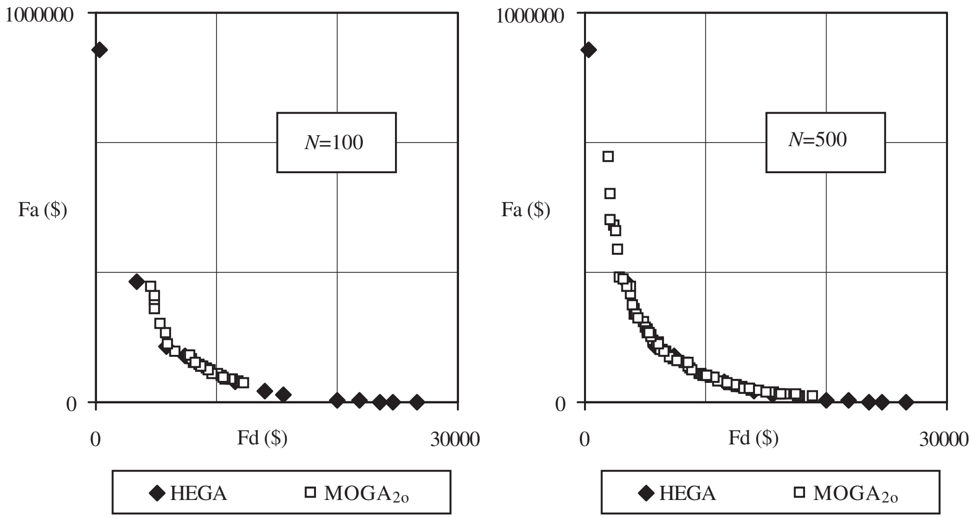


Figure 8. Non-dominated solutions (approximate PF_{TRUE}) using $MOGA_{2o}$ ($N = 100$ and 500).

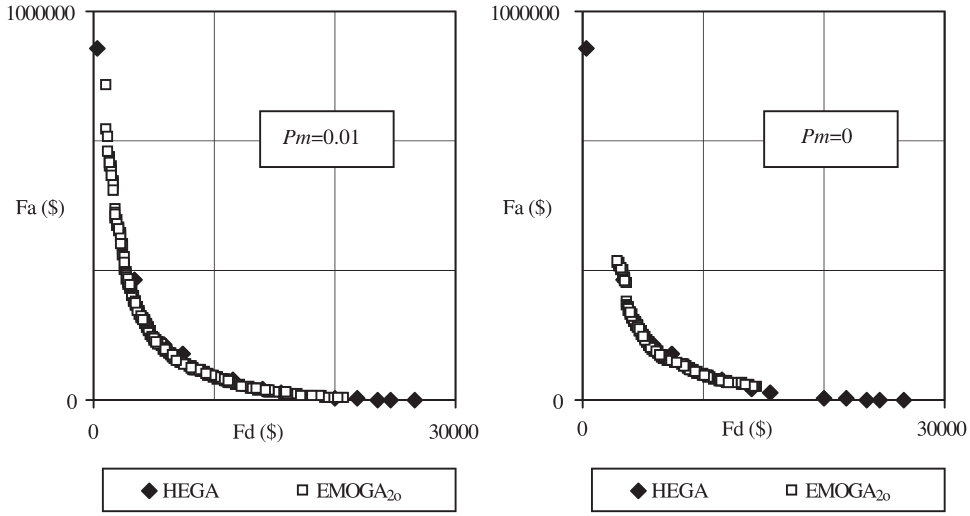


Figure 9. Non-dominated solutions (approximate PF_{TRUE}) using $EMOGA_{2o}$ ($Pm = 0.01$ and 0).

However, as can be shown by Figure 9, the current elitism technique is sensitive to the probability of mutation (Pm). To overcome this, the addition of a niching technique is required to preserve diversity along the Pareto-optimal front. Nonetheless, with a little tuning, $EMOGA_{2o}$ still provides significantly better results than $MOGA_{2o}$ or other non-Pareto methods.

6 CONCLUSIONS

This paper presents two new multi-objective genetic algorithm models ($MOGA_{2o}$ and $EMOGA_{2o}$) which use a Pareto-based selection technique for determining the optimal schedule of chlorine dosing within a water distribution system considering multiple, competing objectives: primarily disinfection control and aesthetic control. An overview of the model structures were given, including the new object-oriented application framework.

To evaluate the effectiveness of each new model's ability to find the true Pareto-optimal front (PF_{TRUE}) existing non-Pareto based techniques were used in an attempt to find PF_{TRUE}. The first attempt used an existing weighted-sum model (HEGA). Although several points along the Pareto-optimal (near optimal) front are found, the weighted-sum method has the disadvantage of requiring prior knowledge from a decision maker about the trade-off relationships between the varying objective functions. In most real world optimisation problems this is not known *a priori*. Furthermore, although an approximate PF_{TRUE} can be found using several different weighting factors, this is a trial and error process and requires new model runs for every new weighting factor. It was concluded that, for the purpose of finding PF_{TRUE}, HEGA is not an effective model.

Given the enormous size of the search space it was concluded early in the investigation that enumeration was not a practical method of finding PF_{TRUE}. However, partial enumeration provides an insight into the "landscape" of the objective space, in particular the density of this space. It was concluded that the dense region within the objective space hindered both the $MOGA_{2o}$ and $EMOGA_{2o}$ model's ability to find certain points within PF_{TRUE} that lie towards the two extreme ends of the front, where the objective function space is sparse. Upon further review of literature it was found that the fitness assignment algorithm adopted from Fonseca and Fleming's (1993) MOGA model is sensitive to the density of the objective space (Deb, 2001). Therefore, it is recommended that both $MOGA_{2o}$ and $EMOGA_{2o}$ be evaluated using a modified fitness assignment method, based on other proven Pareto-based MOGA models, such as that used by Horn and Nafpliotis (1993) or Srinivas and Deb (1994), for example.

Finally, it is concluded that $EMOGA_{2o}$ performed better overall as compared to existing non-Pareto methods, including $EMOGA_{2o}$'s companion model $MOGA_{2o}$.

REFERENCES

Boccelli, D.L., Tryby, M.E., Uber, J.G., Rossman, L.A., Zierolf, M.L. & Polycarpou, M.M. 1998. Optimal scheduling of booster disinfection in water distribution systems. *Journal of Water Resources Planning and Management*. 124(2): 99–111.

Chlorine Chemistry Council, Ed. 1997. Drinking Water Chlorination White Paper: A Review of Disinfection Practices and Issues, Chlorine Chemistry Council.

Coello Coello, C.A., Van Veldhuizen D.A. & Lamont, G.B. 2002. *Evolutionary Algorithms for Solving Multi-Objective Problems*. New York: Kluwer Academic.

Deb, K. 2001. Multi-Objective Optimization Using Evolutionary Algorithms. John Wiley & Sons: Chichester

Fonseca, C.M. and Fleming, P.J. 1993. Genetic algorithms for multi-objective optimisation: formulation, discussion and generalisation. In S. Forrest (ed), *Proceedings of the 5th International Conference on Genetic Algorithms, Urbana, Illinois*. 416–423.

Gen, M. & Cheng, R. 2000. *Genetic Algorithms & Engineering Optimization*. New York: John Wiley & Sons, Ltd.

Horn, J. & Nafpliotis, N. 1993. *Multiobjective optimisation using the Niched Pareto Genetic Algorithm*. Technical Report IlliGAl Report 93005, University of Illinois at Urbana Champaign, Urbana, Illinois, USA.

Levi, Y. & Mallevialle, J. 1995. Global strategy and new tools for maintaining water quality in distribution system. *Proceedings of the China Water Supply Association – Safety of Water Supply in Transmission and Distribution Systems, November 1995*, Shanghai, China.

Nace, A., Harmant, P. & Villon, P. 2001. Optimization of location and chlorine dosage of the booster chlorination in water distribution network. *Proceedings of the World Water & Environmental Resource Congress, May 20–24, Orlando, Florida*. In D. Phelps and G. Sehlke (eds), Bridging the Gap: Meeting the World's Water and Environmental Resources Challenges.

Powell, J.C., Hallam, N.B., West, J.R., Forster, C.F. & Simms, J. 2000. Factors which control bulk chlorine decay rates. *Water Research*. 34(1): 117–126.

Rossman, L.A. 1999. The EPANET programmer's toolkit for analysis of water distribution systems. *Proceedings of the 26th Annual Water Resources Planning and Management Conference, June 6–9, Tempe, Arizona*. Vol 4E: 1–10.

Rossman, L.A. 2000. *EPANET Users Manual, EPA/600/R-00/057*, United Stated Environmental Protection Agency, Version 2.
Rouhiainen, C.J. & Tade, M.O. 2003. Genetic Algorithms for Optimal Scheduling of Chlorine Dosing in Water Distribution Systems. *Proceedings of the 20th Convention of the Australian Water Association, 6–10 April 2003*. Perth, Australia. 1–16
Srinivas, N. & Deb, K. 1994. Multiobjective Optimization Using Nondominated Sorting in Genetic Algorithms, *Evolutionary Computation*. 2(3): 221–248.
Tansley, N.S. & Brammer, L.F. 1993. Chlorine residual modelling in distribution – the importance of taste and the maintenance of effective disinfection, In B. Coulbeck (ed), *Integrated Computer Applications in Water Supply – Applications and Implementations for Systems Operation and Management (vol 2)*. Research Studies Press Ltd: 111–126.
Tryby, M.E., Boccelli, D.L., Uber, J.G., Rossman, L.A., Zierolf, M.L. & Polycarpou, M.M. 1997. Optimal scheduling of booster disinfection, *Proceedings of the AWWA Annual Conference, June, Atlanta, GA*. Vol C: 375–384.
Tryby, M.E., Boccelli, D.C., Koechling, M.T., Uber, J.G., Summers, R.S. & Rossman, L.A. 1999. Booster disinfection for managing disinfectant residuals. *Journal of American Water Works Association*. 91(1): 95–108.
Uber, J.G., Summers, R.S., Boccelli, D.C., Koechling, M.T., Tryby, M.E. & Rossman, L.A. 1996. Booster Chlorination. *Proceedings of the AWWA Annual Conference, June 23–27, Toronto, Ontario, Canada.* Vol D: 197–202.

Optimal location of water quality monitoring sites in water distribution systems

Hyoungmin Woo & Ju-Hwan Kim
Water Resources Research Institute, Korea Water Resources Corporation, Taejon, South Korea

ABSTRACT: Water quality monitoring of the distribution system is important in controlling the water quality in the system and also distributing the water to customer's taps. In the previous research, only the water quantity was considered as a factor of monitoring importance. Even if the water quality as well as the water quantity was considered, the retention times in the system was used to estimate the water quality at the network nodes. However the retention times cannot well represent the water quality because of their nonlinear relationship. In this paper, we present a new method of determining the optimal monitoring sites, in which the water quality is incorporated into the optimization model with a different method for water distribution system from the previous researches. In the method, the water quality at the nodes is determined by using a simulation model EPANET rather than retention times. In this paper, list processing algorithm is applied to track the shortest path of flow in a junction from water source. In the case of multi-water source system, network model is used for tracking the regional boundary of water distribution as followings; which source takes charge of a supply region, whether the source is changed by the pass of time or not, tracking of water source, the regional bounds by the amount of transmitted flow. So is applied to constructing the coverage matrix and determining all the flow paths. With the coverage matrix, an integer-programming problem is formulated and solved. Both the cases of steady-state and extended period simulations are analysed by the newly developed method. For the extended period simulation, minimal set of covering algorithm is used to locate the optimal monitoring sites. Our method provides more reasonable monitoring sites than those of the previous methods for two sample distribution networks.

1 INTRODUCTION

1.1 *Background*

The Safe Drinking Water Act(SDWA) describes the sampling frequency and the substances to be monitored, but not the sampling locations, only limited guidelines are provided. It only suggests that the water sampling sites should be representatives in the water distribution system. Hence, the determination of optimal water-quality monitoring sites in the distribution system has been the subject of several research papers. A previous study that is directly related to the location of monitoring sites is that of Lee and Deininger (1992). Their solution is based on the general feature that water quality deteriorates with time and distance from the source. Thus, all covered nodes are selected as the monitoring station. The term "covered node" was used to denote that the water quality at a particular node can be inferred by the water sampling at some downstream nodes. The methodology proposed by Lee and Deininger (1992) just considers not water quality but water quantities.

Avner Kessler et al. (1998) suggested the methodology of detecting accidental contaminations in municipal water networks. Its objective was identifying the best selection of monitoring stations, which allows capturing an accidental intrusion of contamination within a given level of service.

The term "level of service" was the maximum volume of contaminated water that was consumed prior to detection. They considered water quality by adapting the retention time as shortest paths on auxiliary network. Even though the water quality as well as the water quantity was considered, the retention times cannot well represent the water quality because of their nonlinear relationship.

1.2 *Sampling and coverage*

By adapting previous research concepts, we assume that sampling occurs at the nodes, and that the aim of the sampling is to cover most of the water demand in the network. In order to cover the entire demand, practically every node would have to be sampled. However, we assume that the water quality at other nodes can be inferred by taking flow patterns into account. If water sampled passed in its entirety through another node, then we can assume that the water quality is also known at that node. A covered node means that the water quality of the node is assumed to be known. If 75% of the water sampled passed through another node, then the water quality at this node can be inferred. Similarly, 50% and 25% can be assumed. Associated with each node there is a demand. If the water quality at a node i is known, and node i has a demand of d_i, and D is the total demand, then the fraction d_i/D of the total demand is known. This can also be expressed as percentage of the total demand by multiplying the fraction by 100. Thus, for example, if node i is sampled with a demand of 5 units out of a total of 100 units, the monitoring station is representative of 5% of the demand. Another station located at another demand point may represent 10% of the demand, and would therefore be more representative in terms of flow. If one had to locate only one station, one would choose the one, which has the highest representatives. The term "representative" requires definition. When a sample is taken at a tap and the water quality is analyzed, one strictly knows the quality of one single tap. The quality at the closest node of the distribution network is then assumed to be known. Another term "covered" or "coverage" will be used to convey the notion that by placing a monitoring station at a certain node the water quality is known and the demand at the node is covered. Thus placing a monitoring station at node i with a demand of 5 out of a total of 100 leads to a coverage of 5%.

However, there is more known about a water distribution system. An analysis of the hydraulics of the network shows where the water that was sampled came from and where it is going. Thus the water quality at some of the closest nodes can be inferred. The water fractions calculated can be summarized in a matrix form. This matrix will be called the water fraction matrix W and shows the fractions of water contributed to the sampling station by the nodes of network. From this water-fraction matrix, several knowledge-carrying matrices can be derived that are based on a decision in which fraction of water is acceptable to call a node covered. A knowledge-carrying matrix is obtained from a water-fraction matrix W by deciding if the knowledge can be carried to upstream nodes or not.

$$W(i,j) = \frac{\text{flow contribution from node j}}{\sum \text{inflow into sampling station i}} = \frac{Q_j}{\sum Q_i} \tag{1}$$

2 SHORTEST PATH SEARCH TECHNIQUE

2.1 *List processing approach*

The list processing approach is a method to enumerate minimal paths in a network. The method is based on a combinatorial approach to the problem of detecting minimal paths an loops rather than a purely mathematical one. This is appropriate for computing node-pair reliabilities at all the nodes rather than one node in a network because the minimal paths for all the nodes, except for a source node, can be enumerated simultaneously. Basically, the minimal path is one of the possible permutations of all the pipes in a network. If it were possible to determine combinations of all the pipes, the combination which is properly ordered could be selected as a valid path. Unnecessary

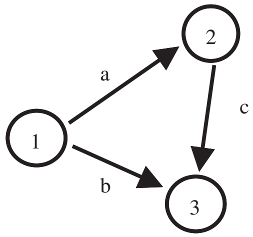


Figure 1. Bridge network.

Table 1. List processing approach.

Pipe		Combination	Valid Path	Specific Path
A	a	1,2	1–2	
B	b	1,3	1–3	1–3
	ab	1,2 1,3		
C	c	2,3	2–3	
	ac	1,2 2,3	1–2–3	1–2–3
	bc	1,3 2,3		
	abc	1,2 1,3 2,3		

paths could be deleted from the list of the valid paths to obtain specific paths. These specific paths become the minimal paths of interest (Yoon, 1990).

In order to enumerate all possible pipe combinations for the simple network shown in figure 1, all the pipes in the network are identified and written on the first column of table 1 where, * denotes a null pipe and the first entry in the list of the combinations. New combinations are generated as each pipe is appended successively to every combination generated already. The newly generated combinations are listed at the bottom of the list. When a given pipe is appended to the null pipe, the combination is simply the given pipe alone. The null arc appears in the list to get the process started and to insure that the combinations of all the pipes taken one at a time will appear in the list of combinations. For example, pipe *a* is, first, combined only with the null pipe; therefore, pipe *a* itself is the unique combination generated by the pipe *a*. Second, pipe *b* is appended to all the previously listed combinations which are null pipe * and combinations **a*. The obtained combinations by the appending are **b* and **ab*. Finally, pipe *c* is also appended to each of the four combinations such as *, **a*, **b* and **ab*. This appending process results in four combinations such as **c*, **ac*, *bc*, **abc* as given in table 1. Thus, all possible combinations for the three pipes are enumerated completely.

With the obtained pipe combinations, the combinations which do not form logically valid paths should be deleted. These can be achieved simply by never appending an pipe to the combinations in the list unless the terminal node number of the combination in question matches the initial node number of the arc to be appended. For example, in the case of combination **ab*, it is not a valid path because the ending node of pipe *a* is not identical to the starting node of pipe *b*. As shown in table 1, four valid paths exist in the simple network. If the paths initiating form node 1 are required in estimating the values, path (2–3) is not needed any more even though it is a valid path. As a result, the specific paths whose source node is identical to node 1 are enumerated completely in table 1 in addition, in computing the values for all the nodes, the specific paths obtained should be distinguished into any node of interest since the sink node can be any node except for the source node. For example, we are interested in the value between the source and node 3 as a sink, the specific paths whose terminal node number is equal to 3 are identified. Consequently, two paths such as (1–3), (1–2–3) are obtained as the minimal paths between the two terminal nodes.

2.2 *Numbering convention*

The Terminal Numbering Convention (TNC) is a well known and convenient method in numbering the nodes and pipes of a network to enumerate the minimal paths. In this method, all the pipes and nodes of a network should be numbered in such a way that the terminal node of each pipe is assigned to a number greater that the number used for its initial node. The list processing approach, however, requires a more convenient numbering scheme, which is only constrained in the pipe number rather than the node number. The numbering procedure for the subroutine is as follows:

- Assign successive numbers to all the nodes. The source is assigned with number 1.
- Assign successive numbers in such a way that the number of the pipe originating from a specific node is greater than the number of pipes terminating at the corresponding node.

3 INTEGER PROGRAMMING ALGORITHM

3.1 *Additive algorithm*

Any integer variable x with a finite upper bound u (i.e., $0 \leqslant x \leqslant u$) can be expressed in terms of 0–1 (binary) variables using the substitution $x = 2^0 y_0 + 2^1 y_1 + 2^2 y_2 + \cdots + 2^k y_k$, where k is the smallest integer satisfying $2^{k+1} - 1 \geqslant u$ and $y_0, y_1, \ldots$, and y_k are 0–1 variables.

Requirements of Additive Algorithm:

- The objective function is of minimization type with all nonnegative coefficients.
- All the constraints must be of the type (=<).

Solution Procedure:

- Convert objective function by introducing replacing variables.
- Convert constraint equations by using slack variables.

$$I_j = \sum_{all\ i} \min\{0,\ s_i - a_{ij}\}$$

- Using initial all-zero binary variables, the associated slack solution is determined.
- Select the branching variables based on the use of measure of slack infeasibility, where, s_i = current value of slack i; a_{ij} = constraint coefficient of the variable x_j in constraint i.
- Compute the objective function value.
- Repeat 4)~5), replacing the branching variables.

4 FORMULATION AS OPTIMIZATION PROBLEM

4.1 *Object function*

Maximum coverage of the network by considering both water quantity and water quality together can be formulated as optimization problem. The formulation where the sampling stations should be located such that the largest amount of the water demand is covered and the severe condition of water quality (i.e. contamination or residual chlorine concentration). Two sets of variables are introduced, x_i and y_i. Both are 0–1 integer variables. y_i is used to determine the coverage of demand of node i. If node i is covered, y_i equals one, otherwise y_i equals zero. x_i is used to describe whether node i is on the path of any specific sampling station or not under a given water fraction criterion. If node i is on the path, *xi* equals one otherwise x_i equals zero.

$$Maximize\ \ f(x_j) = \sum_{i=1}^{n} \frac{1}{c_i} d_i y_i \tag{2}$$

where, n = total number of nodes;

$$\boxed{\begin{aligned} c_i &= \left(\frac{\text{concentration of node } i}{\text{concentration of source}}\right) \times 100\ (\%); \\ d_i &= \left(\frac{\text{demand of node } i}{\text{total demand}}\right) \times 100\ (\%); \end{aligned}}$$

x_j = integer variable , selection of monitoring station (if selected: 1, if not selected: 0);

y_i = integer variable, node contribution of selected monitoring site j (if contributed: 1, if not contributed: 0).

4.2 *Constraints*

The first constraint means the number of monitoring stations, which you want to select. The meaning of the remaining constraints is that the demands should be counted only once even if they are covered more than once, and that demands should not be counted if they are not covered. As representative is defined as percent coverage of the demands, the demand of any node should be counted only once even if it is covered more than once; otherwise the representative would exceed 100 percent. Maximizing the representative in a given network prevents redundant sampling, or more than one sampling on the same path. The 0–1 integer programming code will assign 1 or 0 to satisfy the constraints.

$$\sum_{j=1}^{n} x_j \le NS \tag{3}$$

$$\boxed{\sum_{j=1}^{n} a(j,i)\,x_j - y_i \ge 0 \qquad (i = 1,2,\ldots,n)} \tag{4}$$

where, NS: A given number of monitoring stations; $\bar{a}$ $(j,i) = A^T$:Transpose of matrix A; x_j: Describe whether node j is on the pathway of any specific sampling station or not under a given water fraction criterion.

5 METHODOLOGY

Step 1: Hydraulic Simulation
An extended-period simulation of the water distribution network is carried out using a hydraulic simulation model of EPANET. The output of the hydraulic simulation would be used to the input data for optimization problem including the water quality data for each node in the network.

Step 2: Pipe Renumbering
To get the minimal path of the network, the entire flow path is found and then pipes are renumbered according to flow paths. It is the essential process for finding minimal paths.

Step 3: Minimal Path Enumeration
By the list processing approach, all shortest paths of the network can be found.

Step 4: Coverage Matrix
A matrix, termed the coverage matrix, is constructed to represent the domains of detection and coverage of each node of the network. It is a $N \times N$ matrix of 0–1 coefficients, where N is the number of nodes and "1" and "0" correspond to contributed or covered nodes, respectively. The i th row lists all covered nodes due to a sampling station of the node i. The j th column lists all covered nodes (monitoring sites) that can cover node j. Thus, the quality at node i can be detected by all nodes having a "1" entry in the i th row. The j th column indicates the domain of coverage of node j.

Step 5: Zero-One Integer Programming
To solve the Zero-One integer-programming problem, the additive algorithm is used by changing the type ($\leqslant$), with negative right-hand sides, if necessary. These constraints are then converted to equation by using (continuous) slack variables to the left-hand side of the constraints.
Step 6: Repeat Step 1 ~ Step 5

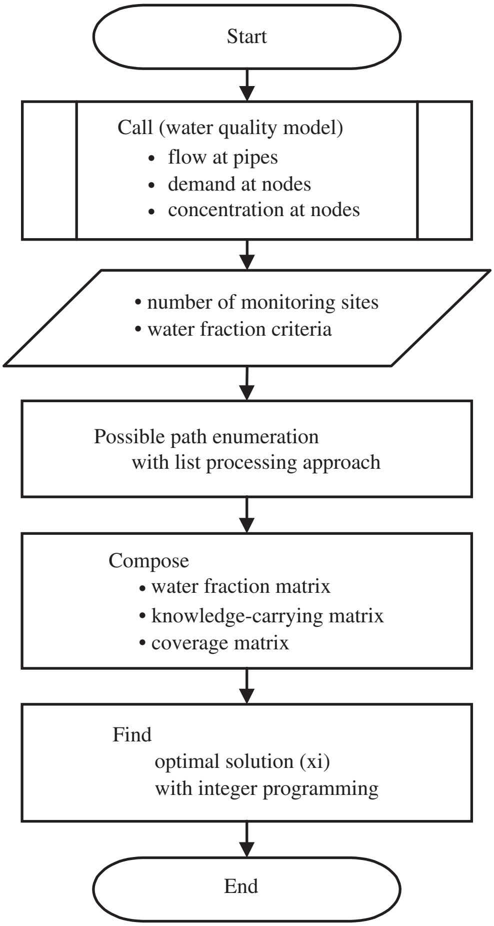


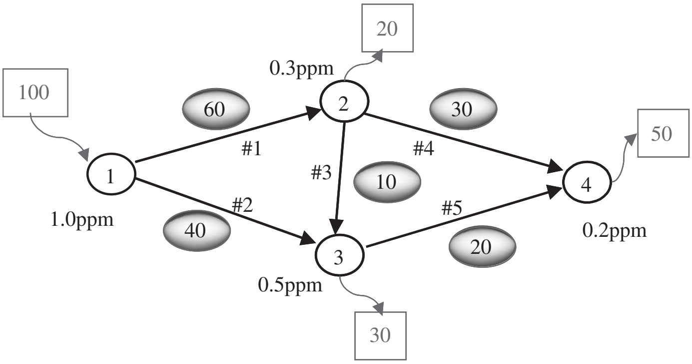
Figure 2. Solution procedure.

In case of extended-period simulation, the variation of water quality would be considered as the concentration of state of each hydraulic time step. The results of each calculation show multiple monitoring sites which was selected in each time step.

Step 7: Minimal Covering Set
To select the optimal monitoring stations, another 0–1 integer-programming problem is needed to be formulated as the set covering problem. The optimum solution of this problem corresponds to a particular node in the network for which a monitoring station is assigned.

6 ILLUSTRATIVE CASE

The proposed methodology is demonstrated in a small bridge network model. It can express most of cases, which could happen in real networks. The number in rectangular box is the percentage of node demand fraction. The number located in each node is the chlorine concentration of node. Table 2 shows water fraction matrix of bridge network. 70% of representative is applied in table 3 of coverage matrix. Table 4 is used to compose the object function by the coverage transpose matrix. Figure 4 is the captured screen of program execution.



$$W(3,1) = 0.8 + 0.2 = 1.0$$

$$W(4,2) = \frac{30}{50} + \frac{20}{50} \times \frac{10}{50} = 0.68$$

$$W(4,3) = \frac{20}{50} = 0.4$$

Figure 3. Bridge network.

Table 2. Water fraction matrix.

	1	2	3	4
1	1	0	0	0
2	1	1	0	0
3	1	0.2	1	0
4	1	0.68	0.4	1

Table 3. Coverage matrix (for 70% criterion).

	1	2	3	4
1	1	0	0	0
2	1	1	0	0
3	1	0	1	0
4	1	1	0	1

Table 4. Transpose matrix.

	1	2	3	4
1	1	1	1	1
2	0	1	0	1
3	0	0	1	0
4	0	0	0	1

6.1 *Object Function*

$$\mathit{Maximize}\ \ f(x_j) = \sum_{i=1}^{n} \frac{1}{c_i} d_i y_i = \frac{1}{100} \cdot 0 \cdot y_1 + \frac{1}{30} \cdot 20 \cdot y_2 + \frac{1}{50} \cdot 30 \cdot y_3 + \frac{1}{20} \cdot 50 \cdot y_4 \tag{5}$$

6.2 *Constraints*

$$\boxed{\begin{aligned} x_1 + x_2 + x_3 + x_4 &\le 2 \\ x_1 + x_2 + x_3 + x_4 &\ge y_1 \\ x_2 + x_3 + x_4 &\ge y_2 \\ x_3 + x_4 &\ge y_3 \\ x_4 &\ge y_4 \end{aligned}} \tag{6}$$

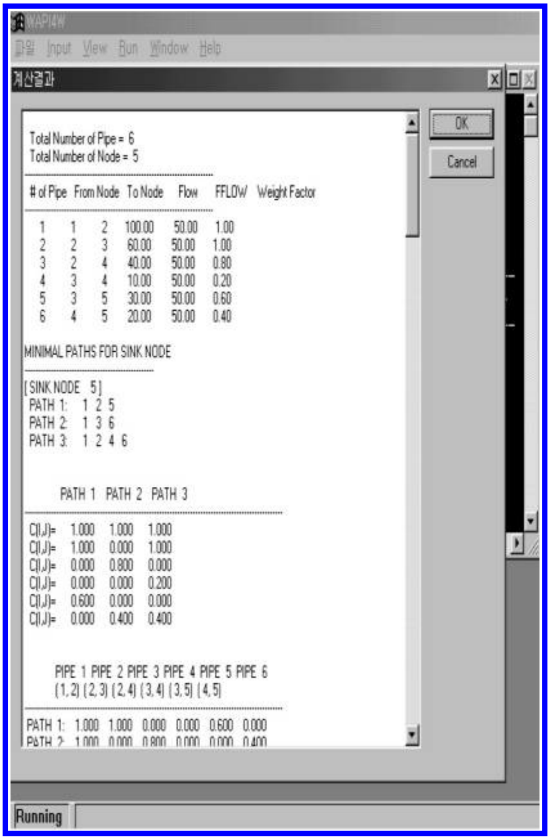


Figure 4. Execution screen.

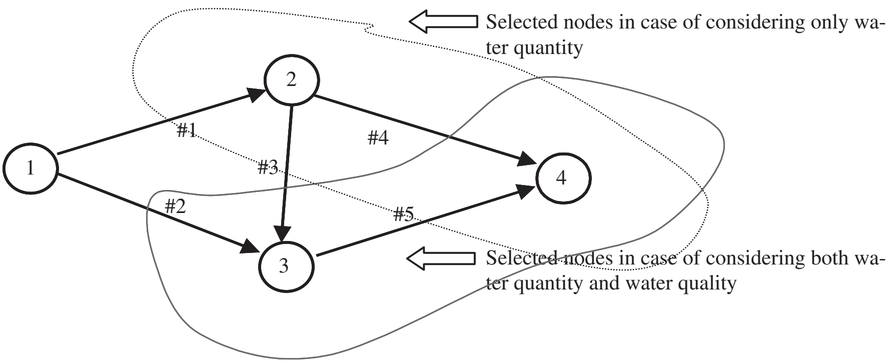


Figure 5. Result of bridge network.

6.3 *Results*

The output of simulation result shows that node 3, 4 are selected instead of node 2, 4. By considering water quality, it could give more rational results for monitoring stations. The results are shown in Figure 5.

7 CONCLUSIONS

In this paper, we present a new improved method of determining the optimal monitoring sites, in which the water quality is incorporated into the optimization model with a different method for water distribution system from the previous researches. In this method, the water quality at the nodes is determined by using a simulation model EPANET.

Monitoring stations in water distribution systems should be located such that they provide maximum information about the current states of the drinking water. The combination of minimal path enumeration, coverage matrices, and integer programming provides a first step towards rational algorithms for locating monitoring stations. The results of an analysis of only two examples show that improvements in the sampling schemes are possible.

In this paper, we present a new method of determining the optimal monitoring sites, in which the water quality is incorporated into the optimization model with a different method for water distribution system from the previous researches. In the method, the water quality at the nodes is determined by using a simulation model EPANET rather than retention times. Also, some previous research paper suggests that the way to flow of the network is founded by the method of minimal path enumeration with list processing, but it is not efficient and does not consider the actual flow of networks. In this paper, Dijkstra's algorithm is applied to track the shortest path of flow in a junction from water source. In the case of multi-water source system, network model is used for tracking the regional boundary of water distribution as followings; which source takes charge of a supply region, whether the source is changed by the pass of time or not, tracking of water source, the regional bounds by the amount of transmitted flow. So is applied to constructing the coverage matrix and determining all the flow paths. With the coverage matrix, an integer-programming problem is formulated and solved. Both the cases of steady-state and extended period simulations are analysed by the newly developed method. For the extended period simulation, minimal set of covering algorithm is used to locate the optimal monitoring sites. Our method provides more reasonable monitoring sites than those of the previous methods for two sample distribution networks.

REFERENCES

Avner Kessler et al. (1998), "Detecting Accidental Contaminations in Municipal Water Networks.", *J. of Water Resources Planning and Management*, Vol 124, No. 4

Clark R. M., Grayman, W. M. (1998), *Modeling Water Quality in Drinking Water Distribution Systems*, AWWA.

Hamdy A. T. (1997). *Operations Research,* Prentice Hall International, Inc.

Lee, B. H., and Deininger, R. A. (1992). "Optimal location of monitoring stations in water distribution systems.", *J. Envir. Engrg.*, ASCE, Vol 118(1), 4–16.

Neukrug, H. M., Burlingame, G. A., Wankoff, W., and Picker, M. J. (1995). "Water-quality regs: Staying ahead." *Civ. Engrg.*, (January), 66–69.

Rossman, L. A. (1994). *EPANET user's manual*. Risk Reduction Engrg. Lab., U.S. Envir. Protection Agency, Cincinnati, Ohio.

Rossman, L. A., Clark R. M., and Grayman, W. M. (1995). "Modeling chlorine residuals in drinking water distribution systems." *J. Envir. Engrg.*, 120(44), 803–820.

Yoon, J. (1990). *Reliability Analysis of the Water Distribution System with the Performance Index*, M.S. Thesis, School of Engineering, University of Pittsburgh, Pittsburgh, PA.

HAAs formation kinetics in water distribution system in Shen Zhen

Zhao Hongbin, Zhao Zhiling & Ren Yueming
Department of Municipal and Environmental Engineering, Harbin Institute of Technology, Harbin, China

ABSTRACT: HAAs is one of the by-products of chlorination disinfection and harmful to the health. The tests were carried out in the City of Shen Zhen, China on the HAAs formation kinetics in water distribution system. A new method was developed based on EPA 6251.B and 552 and proved to be more time saving and accurate. Many factors, such as organic substances, NH_4-N, temperature, bromate, reaction time, chlorine consume, PH, etc. have effect on HAAs formation and organic substances, NH_4-N, temperature, bromate and reaction time are the main factors. Experiments were done respectively and mathematics equations were concluded. Based on the research, dynamic monitoring model on HAAs in water distribution system may be set up in the future.

1 INTRODUCTION

In recent years attention has been drawn to various aspects related to the influence of disinfection by-products in water distribution systems. For a long time, the microbiological quality remained the only criterion for evaluation water quality. In 1970's, a relation was established between the formation of thihalomethanes (THMs) and chlorination of water (Rock,1974). In the past few years, increasing attention has been given to the study on haloacetic acids (HAAs).

There are nine kinds of HAAs (Table 1), some of which may cause cancer. And HAAs is much more poisonous than THMs. The US Environmental Protection Agency (USEPA) has established a maximal contaminant level of 60 μg/l for the sum of the five HAAs: MCA, DCA, TCA, MBA and DBA. In China, only the concentration of DCA and TCA are limited in 50 μg/l and 100 μg/l.

Because of the water pollution, usually HAAs has a high concentration in the water distribution systems, which damages people's health much. Research on how to monitor and control HAAs is important.

To develop a monitoring and control model of HAAs in water distribution system, HAAs formation kinetics is essential. The objective of the work described here was to measure the rates of

Table 1. List of haloacetic acids.

Name	Abbreviation
Monobromoacetic acid	MBA
Dibromoacetic acid	DBA
Tribromoacetic acid	TBA
Monochloroacetic acid	MCA
Dichloroacetic acid	DCA
Trichloroacetic acid	TCA
Bromochloroacetic acid	BCA
Bromodichloroacetic acid	BDCA
Dibromochloroacetic acid	DBCA

formation HAAs in water distribution system in Shen Zhen, China based on the rates observed for the same water held in glass bottles.

2 EXPERIMENTAL PROGRAM

All water were performed using water in the distribution system in Shen Zhen. Firstly, we did the experiment in the lab. Each experiment began by adding chlorine, bromide, NH_4-N, and PH control chemicals to glass bottles. Then we check the results in the water distribution system.

Most of the methods used for determining HAAs are based on the principle that they are derivatized into esters in order to de detected by GC/ECD. The US Environmental Protection Agency (USEPA) has established two methods to determine HAAs: 6251.B using and 552. Some authors have used water-water extraction (Katherine S. Brophy, 1999) and ion exchange (David Benanou,1998) to extract HAAs.

We developed a new method to determine HAAs based on EPA 6251.B and EPA552 and made two changes.

2.1 *A new process to heat up the chromatogram*

$37^{\circ}C$ (21min) $\xrightarrow{11^{\circ}C/min}$ $136^{\circ}C$ (3min) $\xrightarrow{20^{\circ}C/min}$ $236^{\circ}C$ (3min) t:41 min (6251.B)

$40^{\circ}C$ (5min) $\xrightarrow{10^{\circ}C/min}$ $140^{\circ}C$ $\xrightarrow{25^{\circ}C/min}$ $240^{\circ}C$ (3min) t:22 min (new method)

Compared with EPA6251.B, the new method reduced the time to heat up the chromatogram greatly with the same precision. The time that temperature keeps 37°C was reduced, because during this section, the main goal is to separate the solvent and carbinol from the water. During the section the temperature varies between 40°C∼140°C the temperature was prompt slowly because the main goal is to separate the HAAs from the water and the section temperature vary between 140°C∼240°C, the temperature was prompt rapidly because the main goal is to separate the impurity from the water.

2.2 *A new dechloriner*

Both EPA552 and 6251.B use NH_4Cl as the dechloriner, which can combine with chlorine in the water. In our experiment , ascorbate was used as the dechloriner. And it was proved that ascorbate was better than NH_4Cl and $Na_2S_2O_3$, which can be seen in Table 1(PH = 7) and Table 2(PH = 10) below. The experiment was done with HAAs of 20.8 μg/l and chlorine of 2.5 mg/l.

As showed above, ascorbate is better as a dechloriner than NH_4Cl and $Na_2S_2O_3$. It is more accurate.

Table 2. Chlorine determination with dechloriners (PH = 7).

None	Dechloriner	Ascorbate	NH_4Cl	$Na_2S_2O_3$
0d	20.8	20	17.6	17.7
2d	20.8	19.3	17.4	16.4
5d	20.8	19.6	16.5	13.2

Table 3. Chlorine determination with dechloriners (PH = 10).

None	Dechloriner	Ascorbate	NH_4Cl	$Na_2S_2O_4$
0d	20.8	18.5	16.8	15.8
2d	20.8	18.7	16	13.5
5d	20.8	18.3	15.1	10.2

3 RESULTS AND DISCUSSION

Many factors have effect on HAAs formation such as organic substances, NH_4-N, temperature, bromate, reaction time, chlorine consume, PH, etc. But we got the conclusion through static experiment that organic substances (UV254), NH_4-N, temperature, bromate and chlorine consume are the main factors. The results are showed below.

3.1 *Effect of organic substances*

Attempts were made to determine the effect of the organic substances on HAAs formation. By glass bottle experiment a linear correlation between UV254 and HAAs was observed:(Fig. 1).

The relationship an be expressed as the following mathematical equation:

$$\text{HAAs}(\mu g/l) = 5.149\text{UV254}\ (\mu g/l)$$
$$R^2 = 0.9865$$

3.2 *Effect of NH_4-N*

Attempts were made to determine the effect of the NH_4-N on HAAs formation. By glass bottle experiment a linear correlation between NH_4-N and HAAs was observed:(Fig. 2).

The relationship can be expressed as the following equation:

$$\text{HAAs} = 34.951\ NH_4\text{-N}^{-0.5154}$$
$$R^2 = 0.8997$$

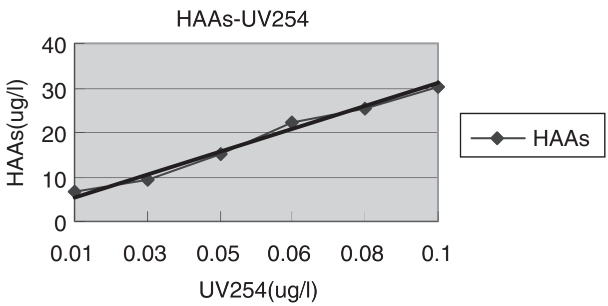


Figure 1. HAAs vs organic substances (UV254).

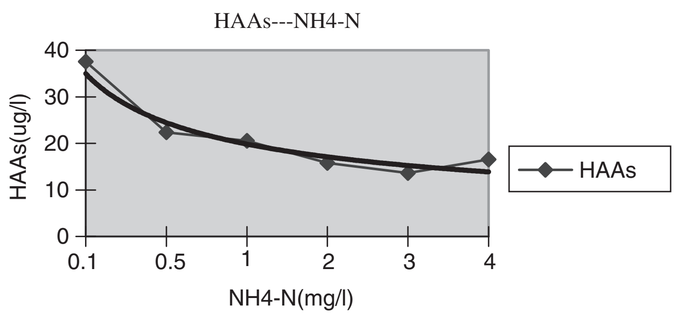


Figure 2. HAAs vs NH_4-N.

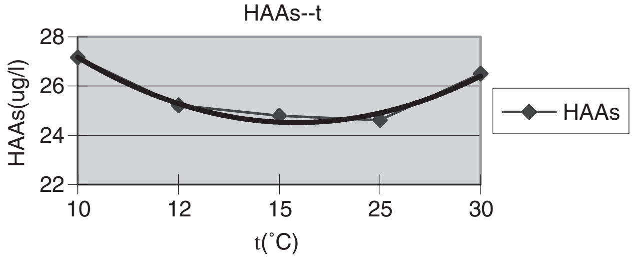


Figure 3. HAAs vs t.

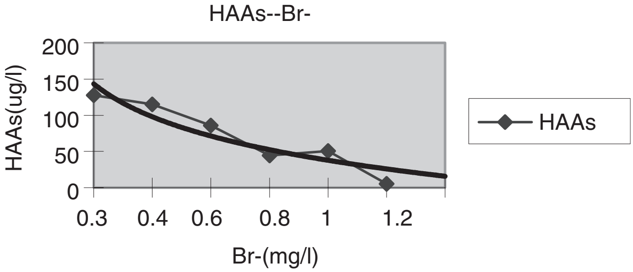


Figure 4. HAAs vs bromate.

3.3 *Effect of temperature*

Experiment were done and the correlation between HAAs and temperature is shown in Fig. 3.

The relationship between HAAs and temperature can be expressed as the following equation:

$$\mathrm{HAAs} = 0.562\,t^2 - 3.581\,t + 30.188$$
$$R^2 = 0.9652$$

3.4 *Effect of bromate*

Experiment were done and the correlation between HAAs and temperature is shown in Fig. 4.

The relationship between HAAs and bromate can be expressed as the following equation:

$$\mathrm{HAAs} = -65.562\,\mathrm{Ln}(\mathrm{Br}^-) + 143.33$$
$$R^2 = 0.8708$$

The mathematical expression described above is based on the experiment hold in Shen Zhen, and these are not necessarily the same in other water distribution systems. There are still lot of work to do make the result more exact.

4 CONCLUSIONS

The experiments were done in the lab using the water brought from the water distribution system in Shen Zhen. A new method to determine HAAs was developed and proved to be more efficient and accurate. The factors effect the formation of HAAs were studied respectively. The following findings and conclusions can be drawn from the results of these experiments.

1. The new process to heat up the chromatogram reduced the time with the same precision.
2. As a dechloriner, ascorbate is better as a dechloriner than NH_4Cl and $Na_2S_2O_3$.
3. HAAs formation is mainly effected by organic substances, NH_4-N, temperature, bromate, and mathematics equation can be concluded.

4. Based on the HAAs formation kinetics, intensive research may be done and finally a mathematics model of HAAs in water distribution system will be developed to monitor and control the HAAs in water distributions.

REFERENCES

Clark, R.M. et al. 1998. Control of Microbial Contaminants and DBPs For Drinking Water in the US: Cost and performance. J Water SRTAqua 47(6): 255–265.

Staff, I.M. 1985. Chloramines a Practical Alternative. J. AWWA. 77: 33.

Abdul, et al. 2000. DBP Formation in Desalination Processes from Disinfectants. Desalination 129: 261–271.

Rook, J.J. 1974. Formation of Halforms During Chlorination of Natural Waters. Water Treatment. Examin 23: 234–243.

Bellar, T.A. et al. 1974.The occurrence of Organohalides in Chlorinated Drinking Waters. J. AWWA 66: 703–706.

Philip, C., Singer, et al. 1998. Control of DBPs in Drinking Water. Water Environment. Research 70(4): 727–734.

Bull, R.J., Ram, N.M., Calabrese, E. 1986. Organic Carcinogens in Drinking Water. New York: John Wiley and Sons. Inc. 353–371.

Rook, J.J. 1976. The Mechanism of DBP formation in Chlorine Water. J. AWWWA. 68: 279.

Krasner, S.W. et al. 1998. Theoccurrence of DBPs in US Drinking Water. J. AWWA 81(8): 41–53.

US EPA. Natural Primary Drinking Water Regulations: Disinfectants and Disinfection By products. Federal Register. 63(241), 69390–69476.

Yuefeng, Xie, et al. 1998. Analyzing HAAs and Ketoacids Without Diazomethane. J. AWWA. 90(4): 131–138.

David, W Hand et al. 1995. Destruction of DBP Precursors with Catalytic Oxidation. Jour AWWA 7(6): 84–96.

Lind, C, et al. 1996. Reducing Total and Dissolved Organic Carbon: Comparing Coagulants. Environ. Technol 6(3): 54–59.

Jie chen, Wei & Clifford, P. 1998. Weisel. Halogenated DBPs Concentrations in a Distribution System. Journal of AWWA 90(4): 151–163.

Nieminiski, E.C., Chaudhuri, S. 1993. The occurrence of DBPs in Utah Drinking Waters. J. AWWA 85(9): 98–105.

Artificial neural network for prediction of water quality in pipeline systems

Ju-Hwan Kim, Hyoungmin Woo, Hyo-Won Ahn & Woo-Gu Kim
Water Resources Research Institute, Korea Water Resources Corporation, Taejon, South Korea

ABSTRACT: The applicabilities and validities of two methodologies for the prediction of THM (trihalomethane) formation in a water pipeline system were proposed and discussed. One is the multiple regression technique and the other is an artificial neural network technique. There are many factors that influence water quality, especially THMs formations in water pipeline systems. In this study, the prediction models of THM formation in water pipeline systems are developed based on the independent variables proposed by American Water Works Association (AWWA). Multiple linear/nonlinear regression models are estimated and three layer feed-forward artificial neural networks have been used to predict the THM formation in a water pipeline system. Input parameters of the models consist of organic compounds measured in water pipeline systems such as TOC, DOC and UV254. Also, the reaction time to each measuring site along pipeline is used as input parameter calculated by a hydraulic analysis. Using these variables as model parameters, four models are developed. And the predicted results from the four developed models are compared statistically to the measured THMs data set. It is shown that the artificial neural network approaches are much superior to the conventional regression approaches and that the developed models by neural network can be used more efficiently and reproduce more accurately the THMs formation in water pipeline systems, than the conventional regression methods proposed by AWWA.

1 INTRODUCTION

New restrictive rules for filtration of surface water and maximum levels of total trihalomethane in pipeline systems are being imposed by the drinking water standards. The new disinfection/disinfectants by-products (DBP) rule addresses the possibility of specifying maximum contaminant levels for each individual THM species since their health risks differ significantly. The study by US national cancer institute in 1976 indicated that chloroform, a major component of THMs, is an animal carcinogen and eventually a suspected human carcinogen. Bromoform and bromodichloromethane were reported carcinogenic later. It is clear that properly developed water quality models to simulate the temporal and spatial variations of different substances in pipeline system can potentially assist the water utilities' operators in abiding with the drinking water quality standards. A number of such kind of models have emerged during last decade. They were mainly developed to model the chlorine under different dynamic conditions. However, the appearance and monitoring of trihalomethane in water pipeline systems is practically difficult due to the complexities of analysis.

The THM compounds develop in chlorinated water containing organic precursors, such as humic and fulvic acids. It has been reported that the relative contribution to the formation of THMs by the humic acids react more readily with the chlorine. Even though it is well known that TTHM increases with time, information about the reaction of the mechanism of the formation of THM and its species is still limited. This paper presented modelling techniques for the prediction of, multiple linear and nonlinear regression and artificial neural networks (ANN) and application results for the prediction of THM formation in water distribution system based on statistical analysis of the observed water quality data in water distribution system. The model parameters are selected as used in formula proposed by American Water Works Association (1993), Urano (1983) and Engerholm-Amy (1987). A steady state hydraulic analysis program (KYPIPE model) was applied to get hydraulic characteristics of the water distribution system under investigation.

2 BACKGROUND INFORMATION

2.1 *Multiple linear and nonlinear regression model*

This approach makes a description by using a set of equation. Information required to construct this model can be obtained in variety ways, such as observations and experiments. Instead of kinetics of THM formation, other water quality parameters in distribution systems have relative contribution to the formation of THM. Regression equation can be generated by parameters information. Scatter diagrams of each of the predictor variables were individually analyzed against the response variable. Inclusion of more predictor variables in a multiple linear regression model is worth testing in this study. A multiple linear regression model that expresses the relationship between THM formation and water quality parameters as independent variables, is given by Equation (1). And nonlinear regression model can be described by Equation (2).

$$y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_n x_n \tag{1}$$

$$y = \beta_0 \cdot {x_1}^{\beta_1} \cdot {x_2}^{\beta_2} \cdots {x_n}^{\beta_n} \tag{2}$$

where β_0, β_1, $\beta_2, \ldots, \beta_n$ denote model parameters, x_1, $x_2, \ldots$, x_n and y denote independent and dependent variables, respectively.

2.2 *Artificial neural network*

An artificial neural network (ANN) is a network of parallel, distributed information processing systems that relate an input vector to an output vector. It consists of a number of information processing elements called neurons or nodes, which are grouped in layers. The input layer processing elements receive the input vector and transmit the values to the next layer of processing elements across connections where this process is continued. This type of network, where data flow on way (forward), is known as a feed-forward network. A feed-forward ANN has an input layer, an output layer, and one or more hidden layers between the input and output layers. Each of the neurons in a layer is connected to all the neurons of the next layer, and the neurons in one layer are connected only to the neurons of the immediate next layer. The strength of the signal passing from one neuron to the other depends on the weight of the interconnections. The hidden layer enhances the network's ability to model complex functions. The data passing through the connections from one neuron to another is manipulated by weights that control the strength of a passing signal.

When these weights are modified, the data transferred through the network change and the network output alters. The neurons in a layer share the same input and output connections, but do not interconnect among themselves. All the nodes within a layer act synchronously. Hence, at any point of time, they will be at the same stage of processing. The activation levels of the hidden nodes are transmitted across connections with the nodes in the output layer. The level of activity generated at the output nodes is the network's solution to the problem presented at the input nodes. Each node multiplies every input by its weight, sums the product, and then passes the sum through a transfer function to produce its result. At the beginning of training the network weights are initialized, either with a set of random values or based on some previous experiences. The weights are optimized to get a specific response from an ANN. When these weights are modified, the data transfer through the ANN changes and the overall network performance alters. The learning algorithm adjusts the weights such that for a given input, the difference between the network output and the actual output is small.

In this paper, the generalized Delta rule is used to train a multi-layer perceptron for THM formation by other water quality data. THM concentration is produced as an output pattern by presenting water quality data to the network as an input pattern. According to the difference between the produced output and the observed, the parameters of network are adjusted to reduce the output error. The error at the output layer propagates backward to hidden layer, until it reaches the input layer. Because of feedback propagation of error, the generalized Delta rule is also called by error back propagation algorithm. The output from node i, O_i, is connected to the input node j through the interconnection weight W_{ij}. Unless node k is one of the input nodes, the state of node k is:

$$O_k = f(\sum W_{ik} O_i) \tag{3}$$

where $f(x) = 1/(1 + e^{-x})$, called transfer function, and the sum is the total of all nodes in the adjacent layer. This transfer function is usually a steadily increasing S-shaped curve, called a sigmoid function. The transfer function also introduces a nonlinearity that further enhances the network's ability to model complex function. The sigmoid function is continuous, differentiable everywhere, and monotonically increasing. The output is always bounded between 0 and 1, and the input to the function can vary between plus or minus infinity.

Let the resulting target (output) state node be t. Thus, the error at the output node can be defined as

$$E = \frac{1}{2}\sum_k (t_k - O_k) \tag{4}$$

where node k is the output node. The gradient descent algorithm adapts the weights according to the gradient error, i.e.,

$$\Delta W_{ij} \propto \frac{E}{W_{ii}} = \frac{E}{O_i}\frac{O_j}{W_{ii}} \tag{5}$$

Specially, we define the error signal as

$$\delta_J = \frac{E}{O_J} \tag{6}$$

With some manipulation, we can get the following generalized Delta rule:

$$\Delta W_{ij} = \eta \delta_j O_j \tag{7}$$

where η is an adaptation gain. The δ_j is computed based on whether or not node j is in the output layer. If node j is one of the output nodes,

$$\delta_j = (t - O_j)O_j(1 - O_j) \tag{8}$$

If node j is not in the output layer,

$$\delta_j = (t - O_j)O_j \sum k\delta_j W_{kj}) \tag{9}$$

In order to improve the convergence characteristics, we can introduce a momentum term with momentum gain α to Equation (7).

$$\Delta W_{ij}(n+1) = \eta \delta_j O_i + \alpha \Delta W_{ij}(n) \tag{10}$$

where n represents the iteration index. Once the neural network is trained, it produces very fast output for a given input data. It only requires a few multiplications and calculations of a transfer function.

3 DESIGN OF MODELS

3.1 *Description of models architecture*

Three techniques adopted in this study are applied to develop prediction models. The models are evaluated by comparing the results in terms of the accuracy, convenience, and ease of use. The difference among models is mainly in the input structure for the purpose of investigating its impact on the output accuracy and determining the most appropriate one for the case of the THM formation in water distribution systems.

Model I is consisted by seven independent variables that are used as inputs with the measured data. This model is expressed in the multiple nonlinear form as follows,

$$DBP = k \cdot T^a \cdot pH^b \cdot (Cl_2)_0{}^c \cdot Br^d \cdot UV254^e \cdot t^f \tag{11}$$

where k, a, b, c, d, e, f denote regression coefficients; T = temperature (°C.) TOC = total organic carbon (mg/L), $(Cl_2)_0$ = initial concentration of chlorination (mg/L), Br = bromide level (mg/L), UV254 = UV light absorbance (mg/L), t = reaction time (hr).
Model II is similar to model I, but it is expressed by linear form of each term. Six independent variables are used as inputs and k′, l, m, n, o, p denote regression coefficients.

$$DBP = k' + l \times T + m \times pH + n \times (Cl_2)_0 + o \times UV254 + p \times t \tag{12}$$

The ANN is a computing paradigm that may have more than one mode. The feed forward neural networks with back-propagation learning algorithm are the most widely used neural networks. This study employs three-layer networks. The configuration of a neural network includes determining the number of hidden layers, the number of nodes in each of the hidden layers, and the connection weights.

The ANNs are trained with a set of input and known output pairs called training set. Many learning examples are repeatedly presented to a network, and the process is terminated when the difference is less than a specific value. The final weight matrix of the trained network represents its knowledge about the problem. Model III and model IV are constructed and applied to the prediction to THM formation in pipeline system. The system equations of models structure are expressed as follows;

$$\text{Model III} \quad DBP = ANN[T, pH, (Cl_2)_0, TOC, DOC, UV254, t] \tag{13}$$

$$\text{Model IV} \quad DBP = ANN[T, pH, (Cl_2)_0, UV254, t] \tag{14}$$

The architecture of model III and model IV can be seen in Figure 1 and Figure 2, respectively.

3.2 *Indicator of model performance*

A determination coefficient (R-square) is one of the most commonly used performance measures for model evaluation. This provides information about model predictive capabilities. Equation for determination coefficient is given by,

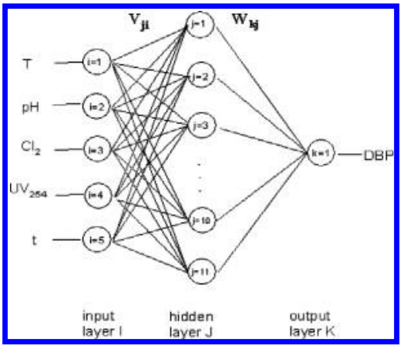


Figure 1. Neural network model architecture of Model III.

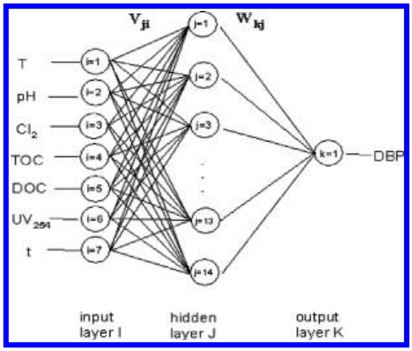


Figure 2. Neural network model architecture of Model IV.

$$\boxed{R^2 = \frac{\sum_{i=1}^{N}(x_i - \hat{x}_i)^2}{\sum_{i=1}^{N}(x_i - \overline{x}_i)^2}} \tag{15}$$

where $\bar{x}_i$ denotes the mean of measured value x_i, and $\hat{x}_i$ stands for the estimated value by each proposed model, and N is number of observations.

4 APPLICATION AND RESULTS

To apply the THM prediction modelling in pipeline system, water quality data are measured and collected. Data describing the pipeline system, which consists of 44 pipes and 45 nodes, and sampling sites characteristics are listed in Table 1. Ten sampling sites (W1–W10) are selected in the system where the measuring works of water quality data are available such as pump stations, water tank and public facilities etc. Also, the reaction time and hydraulic properties in Table 1 are calculated easily by using KYPIPE model.

Correlation analysis is performed using collected water quality data. Input variables are selected by the consideration of the results from correlation analysis. The linear relationships between THM formation and other water quality parameters (temperature, pH, Cl_2, TOC, DOC, UV254) can be shown in Figure 3 through Figure 9. From the analysis, it is found that THM formation is correlated in the order temperature, DOC, TOC, pH, Cl_2, UV254. It is important factor to predict THM formation in pipeline system although the correlation coefficient of reaction time shows lowest value among water quality data as 0.006.

From the analysis of water quality data, the model equations of multi-regression methods are developed as follow:

$$DBP = 6.188 \cdot T^{0.4794} \cdot pH^{-2.869} \cdot (Cl_2)_0^{\;0.5107} \cdot Br^{e} \cdot UV254^{0.3598} \cdot t^{0.06916} \tag{16}$$

$$DBP = 0.05867 + 0.000687T - 0.009643pH + 0.01095(Cl_2)_0 + 0.4876(UV254) + 0.0002916t \tag{17}$$

The correlation and determination coefficients of the results for each model are presented in Table 2 and the water pipeline system characteristics under consideration are shown in Table 2. In the prediction results of proposed four models, model III shows the excellent prediction capability. The determination coefficient of model III is 0.972 as shown in Table 2.

Table 1. Characteristics of water pipeline system and sampling sites.

Upper node	Down node	Diameter (mm)	Distance (m)	Discharge (CMD)	Velocity (m/sec)	Pipe No.	Travel time (hr)	Sampling sites
–	–	–	–	–	–	–	–	W1 (CJ Water
0	1	1650	954	247500	2.8646	1	0.1978	Treatment Plant)
1	2	1200	7126	122500	1.4178	2	1.5790	
2	3	1200	626	120500	1.3947	3	0.1410	
3	4	1200	2952	119500	1.3831	4	0.6705	
3	46	150	1360	500	0.0058	46	1.1536	W2 (G Water
3	47	250	1900	500	0.0058	47	4.4768	Tank)
4	5	1200	400	118500	1.3715	5	0.0916	
5	6	1200	148	112200	1.2986	6	0.0358	
5	49	350	1910	6300	0.0729	49	0.7001	
49	50	200	850	800	0.0093	50	0.8011	W3 (K Water
49	51	350	2774	5500	0.0637	51	1.1646	Tank)
6	7	1100	2456	112200	1.2986	7	0.4993	
7	8	2000	695	112200	1.2986	8	0.4670	
8	9	1100	3191	112200	1.2986	9	0.6487	
9	10	1100	2751	112000	1.2963	10	0.5602	
10	11	1100	46	112000	1.2963	11	0.0094	W4 (J Pump
11	12	1100	58	112070	1.2971	12	0.0118	Station)
12	13	1100	325	112000	1.2963	13	0.0662	
13	14	2000	665	112000	1.2963	14	0.4477	
14	15	1100	3232	112000	1.2963	15	0.6582	
15	16	1100	633	111000	1.2847	16	0.1301	
16	17	1100	2877	110000	1.2731	17	0.5965	
17	18	2000	1260	110000	1.2731	18	0.8637	
18	19	1100	1560	110000	1.2731	19	0.3235	
19	20	1100	3874	109500	1.2674	20	0.8069	
19	55	100	1420	500	0.0058	55	0.5353	W5 (S Assembly
20	21	1100	854	107600	1.2454	21	0.1810	House)
20	60	300	1693	1900	0.0220	60	1.5116	
60	61	300	52	1900	0.0220	61	0.0464	W6 (M Pump
61	62	300	73	1900	0.0220	62	0.0652	Station)
21	22	1100	5100	107600	1.2454	22	1.0810	W7 (CA Water
22	23	1100	26	107600	1.2454	23	0.0055	Treatment Plant)
23	24	700	5266	28300	0.3275	24	1.7187	
24	67	250	300	2000	0.0231	67	0.1767	W8 (D Industrial
24	25	700	273	26300	0.3044	25	0.0959	Zone)
25	26	700	2444	25300	0.2928	26	0.8922	
26	27	700	100	20300	0.2350	27	0.0455	
27	28	700	4184	19000	0.2199	28	2.0339	
27	70	300	1278	1300	0.0150	70	1.6678	W9 (B Water
28	29	700	1155	16000	0.1852	29	0.6667	Tank)
28	71	300	3263	1200	0.0139	71	4.6130	
71	72	250	420	1200	0.0139	72	0.4123	
72	73	300	810	1200	0.0139	73	1.1451	
73	74	200	1501	1200	0.0139	74	0.9431	W10 (D Pump
28	80	250	7600	1800	0.0208	80	4.9742	Station)

The variation of learning error of model III by iteration number can be seen in Figure 10. It can be found that the learning error is continuously decreased as the training of the model is progressing.

The relationship between measured and predicted THM are shown in Figure 11 through Figure 14 to investigate the applicability and performance of models. Four models are compared with linear

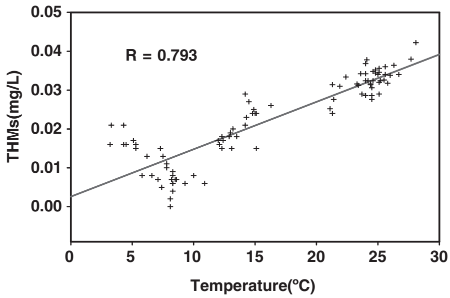


Figure 3. Relationship between THMs and T.

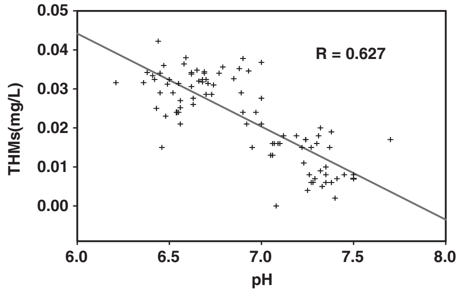


Figure 4. Relationship between THMs and pH.

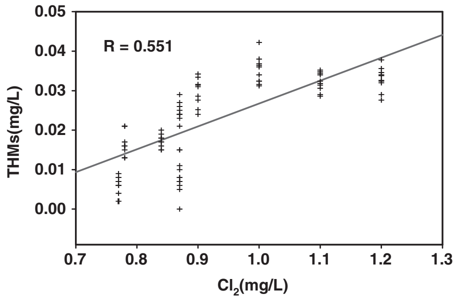


Figure 5. Relationship between THMs and Cl_2.

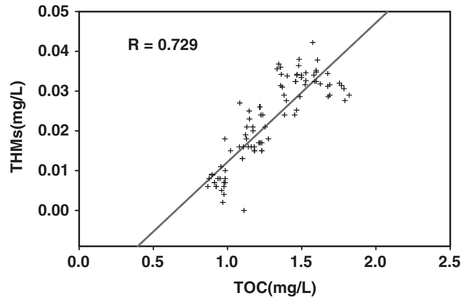


Figure 6. Relationship between THMs and TOC.

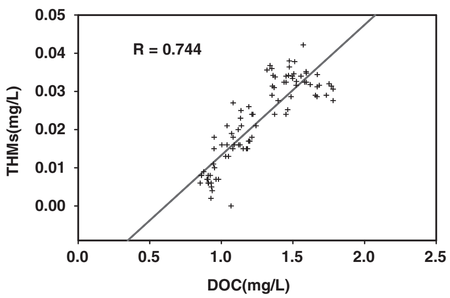


Figure 7. Relationship between THMs and DOC.

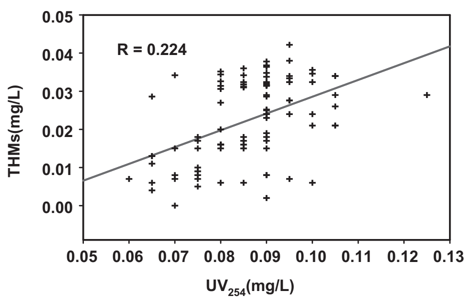


Figure 8. Relationship between THMs and UV254.

Table 2. The correlation and determination coefficients of results for each model.

Index	Correlation coeff.	Determination coeff.	Remarks
Model I	0.917	0.841	Multiple nonlinear regression
Model II	0.937	0.878	Multiple linear regression
Model III	0.986	0.972	ANN
Model IV	0.977	0.955	ANN

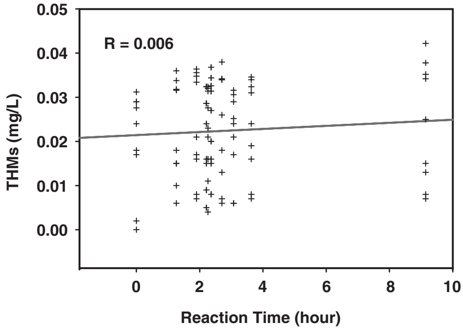


Figure 9. Relationship between THMs and reaction time.

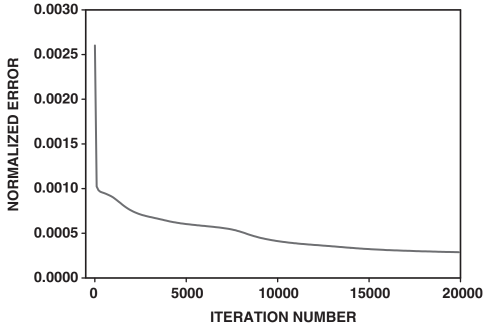


Figure 10. Variation of learning error.

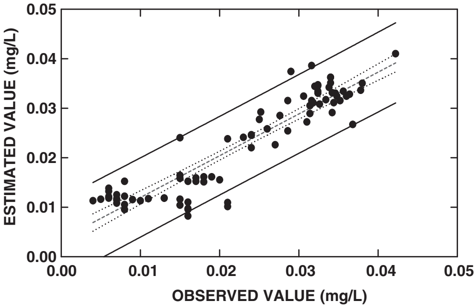


Figure 11. Comparison of result by Model I.

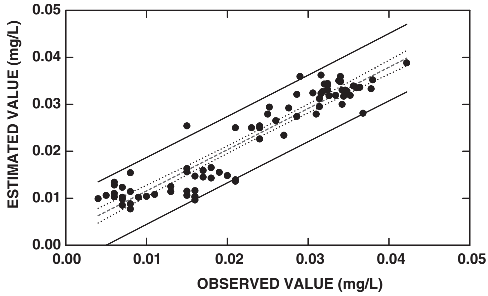


Figure 12. Comparison of result by Model II.

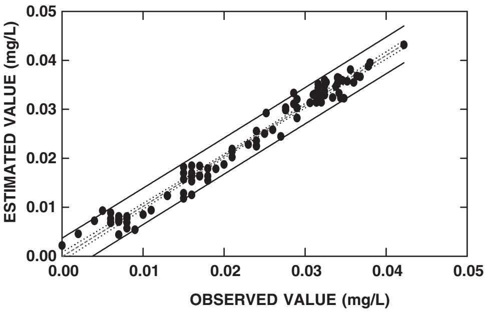


Figure 13. Comparison of result by Model III.

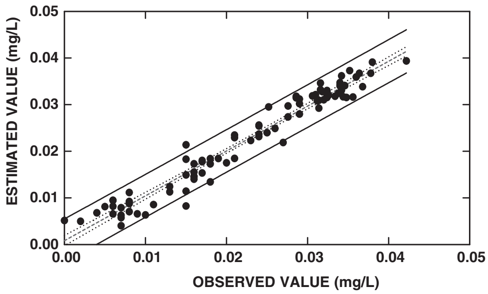


Figure 14. Comparison of result by Model IV.

regression line, prediction limit and confidence limit. The results can be found in Figure 11 through Figure 14. Compared with those figures, it can be concluded that linear relationship of model III in Figure 13 has a best fitting to measured values.

5 CONCLUSIONS

Water quality prediction models in pipeline systems based on multiple linear/nonlinear and neural network are developed and presented. Especially, the relations between THM formation and other water quality parameters are shown and analyzed to formulate the process.

In the prediction results of proposed four models, model III based on neural network theory shows the excellent prediction capability. The ANN methodology has been reported to provide reasonably good solutions for circumstances where there are complex systems that may be poorly defined or understood using mathematical equations, problems that deal with uncertainty like water quality in pipeline systems. Especially, the reaction in pipeline system complicates defining and estimating the variation of water qualities. In this respect, neural network can be an effective and viable tool for not only the prediction of THMs formation but also other water quality factors in pipeline system.

Although the neural network method shows the usefulness in this study for modelling water quality in pipeline systems, the process of applications and data management (pre-processing and post-processing) are well-defined for the purposes of application areas. However, it has to be noted that further research is needed to fully understand its modelling capability.

REFERENCES

Amy, G.L., Chadik, Z.K. and Chowdhury, Z.L. (1987). "A developing model for predicting trihalomethane formation potential kinetics," *J. of AWWA* 79(9), 89.

Bryant, E.A., Fulton, G.P. and Budd, G.C. (1992). "*Disinfection alternatives for safe drinking water.*" Van Nostrand Reinhold, New York.

Clark, R.M., Smalley, G., Godrich, J., Tull, R., Rossman, L.A., Vasconcelos, J.J. and Boulus, P.F. (1994). "Managing water quality in distribution systems: simulating TTHM and chlorine residual propagation," *J. Water SRT-AQUA* 43(4), pp. 182–191.

John Hertz, Andes Krogh, Richard G. Palmer, (1991). "*Introduction to the theory of neural computation.*" Addison-Wesley Publishing Company.

Urano, K. et al. (1983). "Empirical rate equation for trihalomethans formation with chlorination of humic substances in water," Water Research, Vol. 12.

Kim, J.H., Kang, K.W. and Park, C.Y. (1992). "Nonlinear forecasting of streamflows by pattern recognition method," *Korean J. of Hydroscience*, Korean Ed., Vol. 25, No. 3.

Lisboa, P.G.J. (1992). "*Neural networks.*" Chapman & hall, London, pp. 5–6.

Montgomery Watson, (1993). "*Mathematical modelling of the formation of THMs and HAAs in chlorinated natural water.*" Final Report, AWWA.

Reckhow, D.A., Singer, P.C. and Malcom, R.L. (1990). "Chlorination og humic materials: Byproduct formation and chemical interpretation; Envir." Sci. & Technol., 24, 11.

Rossman, L.A., Clark, R.M. and Grayman, W.M. (1994). "Modelling chlorine residuals in drinking water distribution systems," *J. Envir. Engrg*., ASCE 120(4), pp. 803–829.

Singer, P.C., Barry, J.J. III., Palen, G.M. and Scrivner, A.E. (1981). "*Trihalomethane formation in north Carolina drinking waters.*" AWWA, Aug.

Wood Don, J. (1981). "*Computer analysis of flow in pipe networks including extended period simulation.*" User's Manual, Office of engineering, Continuing education and extension, Univ. of Kentucky, Lexington, Ky.

Chapter 7 – Effectiveness of water conservation options

Alternative water management options in a developed catchment

C. Diaper & S. Maheepala
CSIRO Urban Water, Melbourne, Australia

ABSTRACT: This paper describes a preliminary feasibility study of the development of alternative water management options for a fully urbanized, residential, stormwater catchment. The study is a collaborative project carried out by CSIRO Urban Water (CUW) and Sydney Water Corporation (SWC). The methodology used was adapted from one developed for Greenfield sites, the established catchment requiring a more iterative process to incorporate detailed information on current catchment condition and practices and community attitudes. Additional factors to be considered when selecting suitable technologies and techniques (tools) for developed urban areas included; the condition and evolution of existing infrastructure, development trends in the catchment, open space availability and recreational and amenity values, current policies and practices and socio-economic status of catchment residents. This preliminary study highlighted the need to extend and improve the tools available so that they can be applied to existing sites and utilize existing infrastructure and promote behavioural change in established communities.

1 BACKGROUND

Greenfield sites provide a blank canvas for developers and the application of new and innovative water management processes. Conversely in developed sites the potential for immediate technological change is limited as the costs and social impacts of removal and replacement of existing infrastructure are high. Researchers have generally focused on Greenfield sites, due to the flexibility available. However, the majority of the population lives in established communities and improvements in water management in existing areas have the potential to reap far greater benefits. Our attention to these areas is warranted.

Currently in Australia, a number of recent and planned urban infill and redevelopment sites incorporate technologies such as rainwater harvesting, small-scale treatment and water efficient devices. It is likely that the uptake of these approaches will increase with time. Initiatives are generally confined to small-scale sites, such as unit or townhouse developments and do not consider their relationship with the broader water servicing infrastructure beyond the site that is being developed.

The water authorities that own and operate the water servicing infrastructure beyond property boundaries have also been investigating alternative technologies and techniques to improve the performance of existing infrastructure. Improvements are sought in level of service and environmental outcomes, as well as the management of assets to minimise currently large renewal and replacement costs. Developed areas present particular challenges with regard to reconfiguration and integration of water, wastewater and stormwater systems.

Finally, even when problems requiring a new response (such as stormwater pollution) are identified there are few frameworks available for making decisions as to whether an end-of-pipe response, or a more systemic integrated approach provides the better solution. The frameworks that have been developed do not consider the total water cycle, nor do they have a comprehensive view of sustainability factors.

A systematic approach to identifying site-specific optimum approaches at precinct, catchment and regional scales to improve water servicing of existing urban areas, based on consideration of the total water cycle and in accordance with sustainability principles, would benefit the community, the water authorities and the environment. Such an approach should coordinate strategic planning and delivery of improved water servicing. It should facilitate involvement of the many stakeholders and provide a clear basis for decision making, taking into account the ecological, social and economic benefits and drawbacks of a range of options. Options should be carefully selected to meet the needs of the particular precinct, catchment or region. There have been a number of studies investigating the feasibility of using alternative water management options in Greenfield sites (Lloyd, 2001). The main outcome of one study was a User Manual for the assessment of Greenfield sites (Mitchell et al., 2002; The Institute for Sustainable Futures and CSIRO Urban Water, 2002) and it is this methodology that is the basis for development of a similar tool for existing urban areas.

The current project provides a preliminary assessment of factors, features and issues to be considered in the development of such an urban water management tool. The assessment supports best planning practices by gaining an in-depth understanding of the current processes and the community aspirations within the catchment. The study has four main steps:

1. Site data collection.
2. Community consultation.
3. Identification of objectives.
4. Assessment of alternative water cycle management options.

Unlike the methodology developed for Greenfield sites the four steps were not carried out sequentially but simultaneously. For example community consultation continued and additional data was collected as specific alternatives and their requirements were identified.

2 THE STUDY

2.1 *Site description*

The study stormwater catchment is relatively small, only 1.125 km^2. The site is mainly residential as can be seen from an aerial photograph (Figure 1) and there is a small commercial centre. There are two primary schools and also a large social club. The majority of the roads are residential and

Figure 1. Aerial photo of stormwater catchment. (Public open space highlighted)

there are no highways or arterial roads. Most of the open space is located along the beach front and at a small park situated at the middle of the catchment, the rest is dispersed throughout the catchment (Figure 1). The beach is used for swimming and surfing and is especially popular with families.

The catchment and its surrounds were developed during the first half of last century, and development continues, providing accommodation for an increasing population. Stormwater from the catchment discharges from outfalls at either end of beach and flows directly across the sand and into the ocean. At times of heavy rain this discharge is seen as highly turbid and discoloured.

The soils in the catchment generally have low fertility, high permeability and a high erosion hazard, being susceptible to both sheet and gully erosion. The climate is mild and sunny but with a mean annual rainfall of 1354 mm per year and the possibility of sub tropical thunderstorms in the summer months.

There are three main types of residential housing in the catchment, detached houses, semi-detached houses and flats. In terms of land use 80% of the catchment is used for detached houses and although the single detached house average lot size is small for the Sydney region (496 m^2), the range of lot sizes is large, between 133 m^2 and 1520 m^2. As expected from a densely populated urban area the percentage of total impervious surfaces within the catchment is high, at 51.9%, the majority located in the residential sector (30%). Extrapolation of demographic trends in the catchment suggests that the population will continue to increase. As the catchment is already fully urbanised the options for increasing the number of dwellings include; adding storeys to houses, converting to dual occupancy or demolishing and building a number of dwellings on existing sites.

The water mains in the area were installed between 1910 and 1920 and the water reaches the catchment through two 450 mm mains and then flows through to individual properties in 100 mm pipes. The water pressure varies from 100 m head (max) to 50 m at the highest point in the catchment and is within the statutory 15 m required for domestic use. Although the pressure is adequate for domestic requirements, both now and in the foreseeable future, recent development has seen single family homes replaced with low rise blocks of units. This building classification (BCA Class 2) has much more stringent requirements for water pressures for firefighting, and often the street mains are inadequate for this purpose. The quality of the mains infrastructure is unknown and so the water company takes a conservative view when assessing supply capability. Thus, developers are required to either replace the street main, or install on-site retention tanks to meet firefighting needs.

The stormwater system consists of three main drains, two of which are located at the centre and northern end of the catchment and the third at the southern end. The two central and northern main drains are 1.8 m in diameter and collect most of the stormwater from the catchment before draining to the northern end of the beach. During periods of heavy rainfall, stormwater flow in the pipes is at times increased by overflow from the sewage system. The sewage system is connected to the stormwater pipes via emergency relief structures at three locations within the catchment. This overflow is caused by infiltration of surface water into the sewers through cracks in the sewer pipes, exacerbated by the low soil permeability in the area. The EPA advises people not to swim in the area for three days after heavy rainfall.

The separate sewer collection system was constructed around 1929 with a sewer pumping station installed in 1940. No scheduled works on the wastewater system are identified in a water company five year plan and the system is in average condition for its age.

2.2 *Community consultation*

A specified feature of this project was communication with local residents and businesses, local decision makers, state agencies and the general community. A communication plan was developed to raise awareness and to provide opportunities for feedback and comment by affected and interested parties. Communication, consultation and participation plans for the preliminary phase of the project included:

- Introduction of the concept to relevant groups and individuals in the area
- Introduction of the concept to other stakeholders

- Provision of a mechanism for feedback from the community and specific groups and individuals to the project team
- Circulation of explanatory brochure to all residents within the catchment
- Establishing an interactive website
- Using available avenues for presentations and informal communication

In addition, formal discussion with the Local Council, State and Federal members for the area and additional representatives from the local water company, Road Traffic Authority (RTA), Department of Land and Water Conservation (DLWC), Environment Protection Authority (EPA), Coastal Councils and a local Environment Centre was undertaken. This ensured that support at the local and political level was gained. Press releases in two local papers gave wide publicity to the project and a brochure outlining the perceived benefits of the project was prepared and distributed to all households within the catchment.

2.3 *Identify objectives*

Detailed discussion with catchment stakeholders and data collection identified the following water cycle management objectives as being of prime importance:

- Minimise ecological and human health risks from stormwater
- Minimise aesthetic impacts of stormwater on beach
- Minimise sewer exfiltration and overflow
- Maximise water supply system capacity

The above objectives encompass site-specific issues. In addition, an overarching principle of the study was to assess the alternative water management options in terms of a sustainability objective. The performance indicators selected for this assessment were; financial cost, health, social engagement and acceptance, water and other resources (Maheepala S. et al., 2002; The Institute for Sustainable Futures and CSIRO Urban Water, 2002).

2.4 *Scenario development*

The methodology for selection of combinations of tools for improving water cycle management is a generic process which has been previously applied to Greenfield developments (Mitchell et al., 2002; The Institute for Sustainable Futures and CSIRO Urban Water, 2002). In this preliminary study, the first step of initial scenario identification was completed. In brief, site specific data is collected and assessed (Section 2 and 2.4.1) at the same time a toolbox of techniques and technologies is created or updated (Section 2.4.2). The objectives for the site which incorporate the views of all key stakeholders and any outcomes of the data survey are then used as a basis for generation of initial alternative water management themes. These themes range from the base case to the ultimate case in which all objectives are met. Site specific structural and non-structural tools are then used to fulfill the objectives highlighted by each theme. The structural tools used in this preliminary study were selected from a toolbox developed for Greenfield sites. Non-structural tools were extended to include market based instruments, a tool specifically developed for this study.

2.4.1 *Site specific factors*

The development of alternative water management options required detailed knowledge of catchment characteristics such as topography, soil characteristics, climate, housing types, sizes and density, development trends, water usage, open space availability and amenity value, condition and routing of existing infrastructure, socio-economic status of residents and current catchment policies and practices. Less detailed information in all the above areas was also required for areas adjoining the catchment.

From the catchment data, the following site specific factors were identified as having a possible influence on the selection of water cycle management options:

- Amphitheatre shape of the catchment will affect location of treatment technologies and possible pumping requirements.
- Low soil permeability will mean low stormwater retention which may affect selection and performance of treatment techniques and capacity to utilise groundwater or retain water in wetland type devices.
- Low soil fertility will affect selection of plant types and natural treatment systems.
- High soil erosion hazard will affect selection of stormwater treatment techniques.
- High soil acidity may be a corrosion hazard for subsurface tanks and selection of plant types.
- Seasonal/storm rainfall patterns will affect reliability of rainwater if used as a supply.
- The wastewater and stormwater outputs from different density housing types will affect water reuse options.
- The small average lot size may affect the selection of individual household treatment or collection technologies.
- The small amount of public open space in the catchment creates a conflict between amenity value and space available for treatment. For example the area around the beach front and foreshore has high recreational and amenity value and so utilisation of this area as water storage or treatment will need to complement this social value.
- The increase in housing density in the catchment may decrease the pervious area available and will also change the firefighting water requirements.
- Range of socio-economic status may affect affordability of some options (range is from poverty line to very wealthy).
- Alternative policies for improving the sustainability of the water cycle in the catchment will need to align with or replace existing policies and practices.

In addition, when assessing alternative water cycle management options, the presence and condition of existing infrastructure will need to be considered and options may need to evolve from this existing infrastructure.

2.4.2 *Structural and non structural tools*

There are many options available to change the way in which the water cycle is managed and different tools will have different effects on potable water, stormwater and wastewater volumes and quality. Structural tools involve physical construction and can be applied on different scales, from application to a single allotment to a whole catchment or region. Non structural methods are very diverse and include fees, rebates and subsidies, through to green offsets, regulation, monitoring, education programmes and the creation of new markets (Table 1).

The use of non-structural tools is especially important in an existing urban area, where the introduction of structural alternatives could cause substantial disruption and the established community might need additional persuasion to accommodate change. The tool box in this current study was developed from one applicable to Greenfield sites, with the addition of a new group market based instruments (MBIs). These MBIs were developed for this study, and are designed to address specific problems of an urban catchment.

2.4.3 *Water and contaminant balance*

Following data collection and analysis and the development of a tool kit of available improvement options, analysis of the water and contaminant flows in the catchment was undertaken. The water balance provides a clear, broad picture of the resources available to meet the demands placed on the supply system () and also provides a framework within which optimal development of the available resources and evaluation of alternatives can occur (Bell, 1972; Hayes et al., 1980). The contaminant balance seeks to identify sources of water contaminants within the urban environment, and to quantify contaminant loads associated with water flows (Mitchell et al., 2000).

Table 1. Structural and non-structural tools selected for their applicability to catchment.

Structural tools	Non-structural tools
Rainwater tanks Infiltration systems Wetlands	Education programmes – Water audits, leaflets and local advertisement and awareness campaigns
Road re-design Gross pollutant traps Recycling neighbourhood carwash facility Low water use garden Greywater reuse	Regulations and building codes – Regulation for pervious surfaces in new build, fines for littering/dog excrement, fines for illegal connections
Wastewater reuse Composting toilets Sewer mining Water saving fixtures and appliances Sewer relining/repair Other stormwater treatment technology Reduce distribution system leakage Reticulation systems pressure management	Economic or Market Based Instruments – Seasonal pricing, two-tier pricing structure for reticulated potable water, taxing pollutants picked up by stormwater, environmental levies with rebates, penalties, sewerage charge rebate, subsidies and provision of green offsets

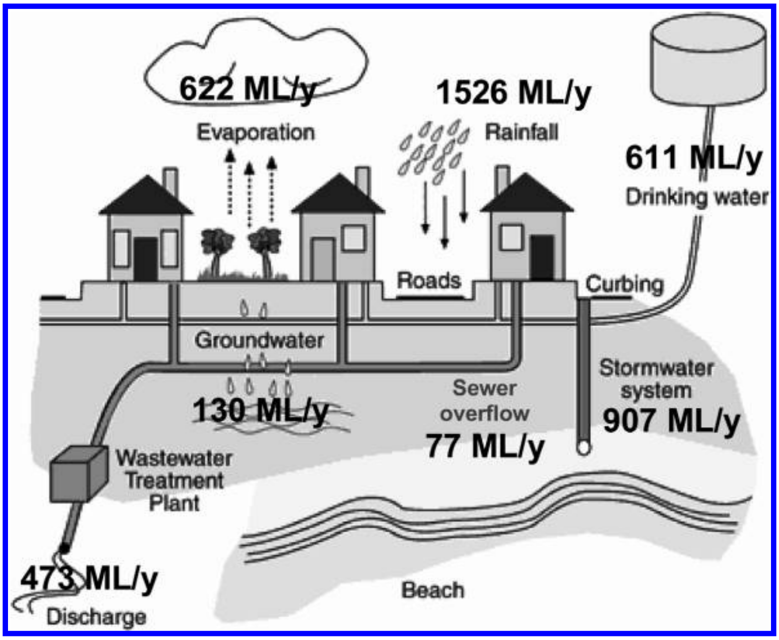


Figure 2. Catchment average annual water balance.

The preliminary water balance indicated that on an annual basis the majority of water entering the catchment is via rainfall and the dominant outflow from the catchment is stormwater (Figure 2). Although these results appear to indicate potable water could be replaced by rainwater as a supply, more detailed analysis on a daily basis indicted the reliability of this supply was poor due to the extreme nature of rainfall events. The initial contaminant balance indicated that the major proportion of stormwater contaminants leading to health risks, originates from sewer overflows and that the major source of suspended solids is the impervious surfaces.

2.4.4 *Preliminary options*

A "broadlist" of alternative water cycle management options was developed utilising the generic methodology described in Section 2.4. These options have potential to meet the site specific objectives and move towards a more sustainable system and are outlined as follows:

Scenario 1 – Maximise water system capacity using water saving devices and subsidies, information and education.

Scenario 2 – Restore natural drainage ecosystems using raintanks, wastewater pipe maintenance, infiltration systems, taxing, subsidies, information and education.

Scenario 3 – Minimise supply, maximise use of stormwater and wastewater using raintanks, water pipe maintenance, greywater, stormwater collection and taxing, penalties, subsidies, information and education.

Scenario 4 – Decentralised improvements to complement current system using water saving devices, stormwater collection, wastewater recycling and taxing, green offset, two tier pricing, regulation, information and education.

Scenario 5 – Immediate autonomous catchment using all of structural and non structural tools.

These options were selected to represent a range of possibilities, from minimal change to existing infrastructure (Scenario 1), to an ultimate case in which no potable water flows to the catchment, no wastewater leaves the catchment and stormwater flows mimic those predevelopment (Scenario 25. The further assessment of these options in terms of sustainability objectives, will provide insight into what is possible within the catchment, in terms of financial cost, health, social engagement and acceptance, water and other resource perspectives.

3 CONCLUSIONS AND RECOMMENDATIONS

Some general conclusions from this project apply not only to this preliminary case study but also to other urban catchments. Firstly, the scenario selection methodology used in this project is more iterative than for Greenfield sites, requiring more detailed information and data on the current practices and condition of the catchment. The water and contaminant balance modelling is also more iterative, as the model can be refined when more detailed information on the site becomes available.

The role of non structural tools is more important for urban catchments, as a primary issue is motivation to change established behavioural patterns. In this project, catchment specific market based instruments were developed and these were tailored to meet specific objectives. This is a new, emerging research area and an important aspect of the next phase of this project will be to develop these tools further and to assess their effectiveness. Also, community participation and involvement in a study of an existing urban area is key to successful outcomes. Many alternative water management system design choices will need the approval of the community. For example, "is the community willing to give up open space within the catchment in order to provide treatment of the stormwater?"

The toolbox of techniques and technologies for an urban catchment needs to be developed further to include more innovative tools. For example, components of the existing infrastructure could be used for applications other than that for which they were originally designed i.e. leakage from existing stormwater systems could be utilized as an infiltration technique. Water cycle management options should consider the possibility of exporting waters to end users outside the catchment or importing water from an adjacent catchment. This may provide a revenue stream if the exported water can be treated to a suitable quality, or reduce requirements for water supply within the catchment if water of a suitable quantity and quality can be imported.

This preliminary feasibility study provides the basis for an assessment methodology for alternative water management options in existing urban areas. The second stage of the study will comprise of the following:

- Confirmation of scenarios for analysis
- Continuation of community consultation/awareness

- Additional data collection
- Detailed analysis of options
- Selection of the optimum solution

As a precursor to this basic methodology, further innovation in scenario development and selection of tools is required.

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REFERENCES

Bell F.C. (1972) The Acquisition, Consumption and elimination of water by the Sydney Urban System. Proceedings of the Ecology Society, Australia, Sydney 161–176

Hayes R.J., Popko K.A. and Johnson W.K. (1980) *Guide Manual For Preparation Of Water Balances*. Hydrologic Engineering Center, Davis, CA. AD-A102014/8

The Institute for Sustainable Futures & CSIRO Urban Water (2002) *Greenfield Manual Version 1, New approaches to Water Services*

Lloyd S.D. (2001) *Water sensitive urban design in the Australian context*: Synthesis of a conference held 30–31 August 2000, Melbourne, Australia. Cooperative Research Centre for Catchment Hydrology, Report number 01/7

Maheepala S., Speers A., Booker N. and Syme G. (2002) Recommendations for developing a framework for assessing sustainability of urban water systems. CSIRO Urban Water BCE Doc. 02/084

Mitchell V.G., Gray S. and Farley T. (2000) *Accounting for water and contaminants in urban areas*, In Xth World Water Congress, Melbourne, 12–17 March, IWA

Mitchell V.G. (2002) Informing the decision process to improve the sustainability of an urban Greenfield development water system, IEMSS Biennial meeting of the International Environmental Modelling and Software Society, 24–27 June, Lugano Switzerland

Blackwater separation in a residential house by vacuum toilets

Thilo Herrmann
ITH Water Technology, Bayreuth

ABSTRACT: The water pollution by potassium mining and the fertilizer value of sewage sludge are reflected. The separation of blackwater in households to produce fertiliser is examined. A new method to divert blackwater and greywater separately by one single vacuum pipe system is presented. An existing urban housing building with 32 flats has been retrofitted. Vacuum toilets and a vacuum system for the separate collection of blackwater has been installed. As result the water demand for flushing toilets was determined to 9 litres per person and day. The energy demand for the vacuum system was measured to be 27 kWh per person and year. The investment costs of the system are presented.

1 MOTIVATION AND BACKGROUND

1.1 *A look back on fertiliser and agriculture*

Travellers in Northern Germany will recognize the shape of bleak barren hills at the horizon: The residues of ancient and recent potassium mining in Germany, Figure 1. A main component is kitchen salt, easily washed out by rainwater, Figure 2. Even decades after the mines have been given up, the heaps are a continuous source of salt polluting surface and groundwater (Spiegel 1998). The chloride concentration in the river Werra may increase to 9 g/l in a dry summer, sometimes leading to fish kill (HAZ 1997). The reason for potassium mining is the demand of agriculture for fertiliser besides the use in the chemical industry. Up to the middle of the 19th century the

Figure 1. Potassium mining residues, consisting mainly of salt.

Figure 2. Salt is washed out and cristallized at the foot.

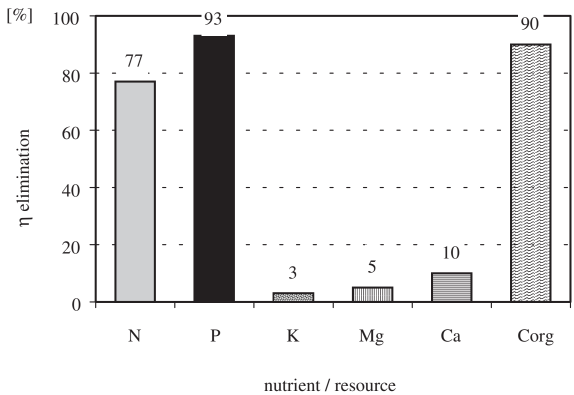

Figure 3. Elimination efficieny of an advanced large scale sewage treatment plant for the nutrients Nitrogen (N), Phosphorous (P), Potassium (K), Magnesium (Mg), Calcium (Ca) and organic carbon (Corg). The values have been calculated using the following references: Nowak (1993), Siegfried (1995), Lindert (1995), RAwVwV (1992), Koppe & Stozek (1998).

faeces and the waste of the households was brought back from the cities to the farmland. By the widespread introduction of the gravity sewer in the 20th century the faeces have been diverted from the arable land to the rivers. In the beginning without any treatment, now advanced with 3-step treatment. Modern sewage treatment plants eliminate nitrogen and phosphorous to a high extent, Figure 3. But sewage sludge contains only a little portion of the original nutrient content of the faeces.

Most of the energy demand of wastewater treatment works is used for aeration, needed to oxidise ammonia and subsequently to release the nitrogen to atmosphere. On the other hand the fertiliser factories use energy to transform air nitrogen to ammonia for the production of fertiliser. This is a kind of nitrogen recycling, but the question is, if this way is energy efficient and sustainable. For potassium (K) the balance is quite different: The elimination efficiency of advanced wastewater treatment is less than 4% (Rodewyk 1979). The potassium passes the treatment works unaffected, Figure 3. Assuming advanced treatment, the recovering of phosphorus by sewage sludge seems to

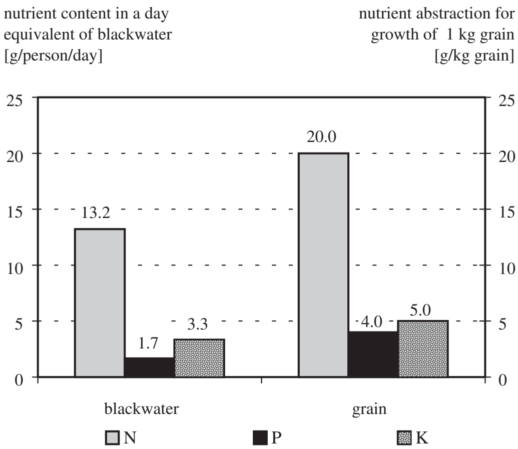

Figure 4. Nutrient content of a persons day equivalent of blackwater and fertilizer demand (nutrient abstraction off the soil) to grow 1 kg of grain.

be efficient: About 90%. But recent research results show that the plant availability of the phosphorus in the sludge may is only 25 to 50% of the total phosphorus content. Even worse is the effect of sludge application when there is a high dosing of iron based precipitation agents in the treatment works: the application of an iron-rich sludge may is lowering the plant availability of the phosphorus in the soil. Thus, according to the German EPA the use of sewage sludge in agriculture is not justified (UBA 2000). In general German rivers does not provide bathing water quality. This is a consequence of the introduction of treated sewage and stormwater overflows in almost every water course. Facing these problems and the pollution by potassium mining, the idea arose to collect the toilet effluent, what is called blackwater, and the rest of the household water, what is defined as greywater, separately. The toilets contribute 80% of the nitrogen, 70% of the Phosphorous and more than 90% of the Potassium of the total mass flow in urban sewage. Therefore the blackwater should be recycled after treatment as fertiliser in agriculture. Figure 4 shows what is the fertiliser value of the blackwater one human produces per day. It's enough to grow about half a kilogram of grain per day. This is not surprising: All nutrients going into a human by food, are leaving the human in form of excreta. There are no losses of elements, it is a flow state equilibrium.

1.2 *The introduction of vacuum toilets*

Up to now the use of vacuum toilets is known from ships, aircrafts and modern trains. Some hospitals provide vacuum toilets to collect the excreta of people treated by radioactive substances to store it for radioactive decay. Vacuum toilets are so far applied under conditions, where there are special requirements for the transport or the necessity of storage of the toilets effluent. In 1991 it was the first time, when 12 private flats in Norderstedt near Hamburg have been equipped by vacuum toilets. The objective there was to investigate the saving of flushing water.

2 A NEW TECHNICAL APPROACH

2.1 *Sewage separation by vacuum drainage*

Since the elimination of organic carbon and nitrogen is not desired, the separately collected blackwater should be treated anaerobically. To make this process effective, the dilution of the excreta by water should be as low as possible. The best technique to meet this requirement would be a dry toilet.

But composting toilets are not accepted by the general public for some reasons. Vacuum toilets need between 0.3 and 1.4 litres per flush. They are as comfortable as conventional toilets and therefore they should be acceptable for the dwellers in an ordinary urban building. A vacuum toilet allows the collection of the excreta in a more concentrated form compared to a conventional flushing toilet. The anaerobic treatment of such a toilet effluent is feasible and biogas is gained. The addition of organic household waste to the blackwater is possible when crushed. This could be done by a decentral mill, which is placed near the waste bins and what is connected to the vacuum sewer system.

Now the question arises, what to do with the greywater. A diversion in a separate sewer outside of the buildings seems to be too costly. The treatment can be done aerobically by a planted soil filter, by membrane filtration, or any other biofilm treatment. But all this requires at least a semi-centralized treatment works. The problem is to avoid separate pipe systems but to avoid mixing. The idea is to use one vacuum pipe system and to divert the different qualities of sewage in different time intervals to the system. That means to divert greywater all over the day, and to divert blackwater only during a definite time span in the early morning. Small vacuum buffer tanks collect the blackwater of a building during the day, and are to be emptied automatically by a central control unit into the vacuum sewer system. The greywater is diverted by gravity to the connection manhole between building and public vacuum sewer. In this manhole a storage volume for greywater is provided for the case of a vacuum break down. The vacuum sewer system outside the buildings is quite conventional like the vacuum systems nowadays applied in rural areas.

2.2 *Project design*

To realize the first step of this sanitation concept, an urban housing building in the city of Hannover was retrofitted with vacuum toilets and a separate blackwater and greywater pipe system. Since the building was already connected to the public sewer, the vacuum system is limited to the building itself. So far the blackwater and the greywater are diverted to the public sewer. It is foreseen to digest the blackwater on an experimental scale later on. This will be useful to investigate the production of biogas and to optimize the digesting process. The first step was realized for evaluation and optimization of the vacuum sanitation within the building.

2.3 *Project building*

The building was constructed in 1962 and a complete retrofit was necessary in 1997. It is a four-storey building comprising 32 rented flats. Most of the apartments are rented out under welfare status. The building was insulated, the windows and the plumbing and heating installation were renewed. A combined heat-power station in the basement driven by natural gas produces electricity and heat for the building. Peak demands of heat are supplied by a gas boiler, the peak demand and the surplus of electricity is drawn by or delivered to the public net. The residents remained living in the building during the constructions. In April 1998 the vacuum toilets have been set in operation. Figure 5 shows a vacuum toilet in a flat, Figure 6 the building after renovation. Table 1 gives the data of the building.

The building is situated in a densely populated district of Hannover, built between 1960 and 1970, where some other innovative sanitation systems have been installed during the last 8 years. Under initiative of the local water supplier, the Stadtwerke Hannover AG there have been built various different systems for greywater treatment and reuse, urine separating toilets, a willow plantation for waste water evaporation, a soil filter for rainwater reuse and infiltration. The area is called OekoTechnikParkHannover and you can visit all the installations under regular operation. Under http.//www.oeko-technik-park.de you will find an illustrated description of all the energy and water related projects.

2.4 *Vacuum system*

Figure 7 shows the scheme of the vacuum system. The blackwater pipes are kept under subatmospheric pressure by a vacuum station. The vacuum station consists of a buffer tank, vacuum and

Figure 5. Vacuum toilet.

Figure 6. The project building after renovation.

Table 1. Parameters of the project building.

Ground area	671 m^2	60.4 m × 11.1 m
Housing area	1900 m^2	
Number of toilets	32 + 1	1 toilet per flat
Number of residents	73 to 80	average 78.4
Fluctuation	9 changes of tenants	during 40 months

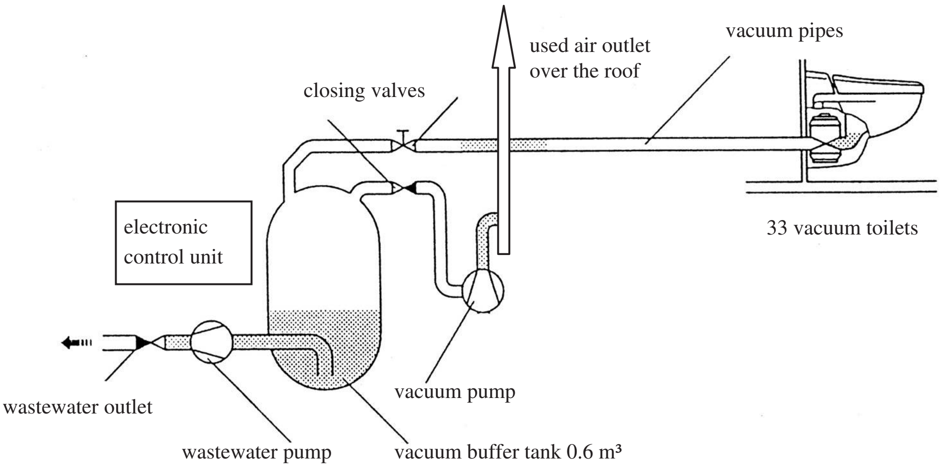

Figure 7. Scheme of the vacuum system.

wastewater pumps and an electronic control unit. The toilets are connected via the vacuum pipes with the buffer tank. Within this tank air and water are separated. The pressure is hold steadily between 450 and 550 hPa by running two vacuum pumps. The wastewater pumps empty the tank when a defined level of water is reached. The water level is detected by electrodes, diving at an

Figure 8. The central vacuum station in the basement; right hand side: vacuum pumps; middle: vacuum buffer tank; left hand side sewage pumps.

angle of 45° into the tank. Excessive air is exhausted over the roof of the building. Up to now the blackwater is diverted in the existing public sewer.

3 RESULTS

3.1 *Water consumption*

The water and energy consumption has been measured in the period from April 1999 to August 2001. Figure 9 gives the daily consumption of flushing water and electricity for the vacuum station. The daily flushing water consumption varied between 580 and 740 litres, the mean is 697 l/d. The specific consumption per person was 7.5 to 9.4 litres per day with a mean of 8.9 l/p/d. There is no explanation known for this variation.

The number of inhabitants was determined according to the data of the residents registration office. In about every third flat the tenants changed during the evaluated period of 28 months. May be the flushing habits of different tenants were different. The water consumption for flushing toilets prior to the retrofit is not known. Assuming a flushing water demand of 40 litres per person and day using conventional toilets, the consumption of flushing water would be 3140 litres per day for the building. Thus the saving by the vacuum toilets is calculated to 890 m^3 per year or 11.4 m^3 per person and year. The water price in Hannover is 3.21/m^3, for drinking water including wastewater charges. The yearly saving in water costs for the building is determined to 2853 per year or 36.39 per person and year.

3.2 *Energy consumption*

In contrast to conventional gravity toilets, where the energy for the transport through the pipes is provided by the potential energy of flushing water, the vacuum station is consuming electricity for the vacuum and wastewater pumps. The total energy consumption was measured to 2008 kWh per year or 26.6 kWh per person and year. The specific costs of public electricity are 0.18/kWh what means yearly costs of 361 for the building or 4.58 per person and year for operating the vacuum toilets. Regarding the electricity consumption versus time in Figure 10, there is a strongly decreasing trend. The reason is, that the vacuum pumps are running part of the time against closed valve

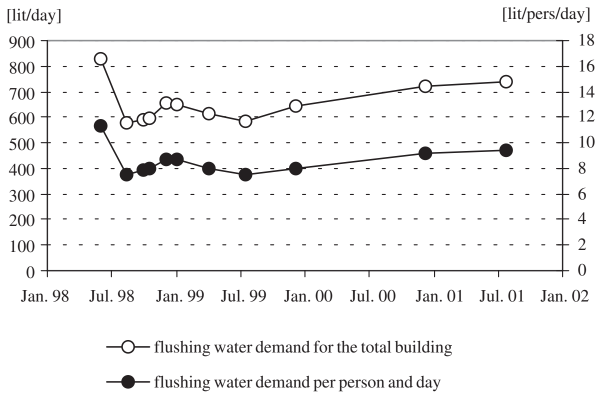

Figure 9. Water consumption.

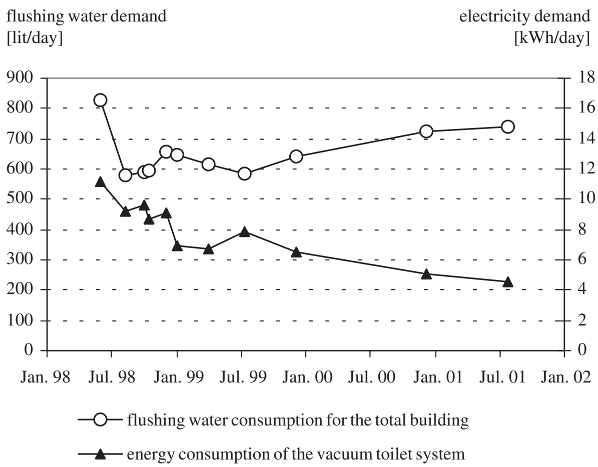

Figure 10. Energy and water demand of the vacuum toilet system.

to heat up. The heating up is necessary to evaporate the water out of the pump oil. The times the pumps are idling have been reduced several times, so the energy consumption decreased over time. Nevertheless there is still a potential to reduce the running time of the vacuum pumps by 80%, when the engine oil would be heated up by an external heating system. This might be done by an oil-water heat exchanger run by hot water from the heating system of the building. The costs of this optimization will be calculated in the near future.

3.3 *Costs*

The necessary investments for the vacuum system are shown in Table 2. The investments have been about three times the costs of conventional toilets. Some supplemental expenses were necessary for the scientific survey, Table 3. A cost balance is given in Table 4. A quarter of the investments was financed by subsidies of the regional government. These subsidies are financed by revenues from groundwater abstraction taxes, the so called “Wasserpfennig” (water penny).

The current costs for maintenance and repair were published by Herrmann and Hesse (2002). Up to now it can be summarized that the saving in water costs can pay back the deduction, the

Table 2. Necessary investment costs.

Investments	Number	Price in Euro incl. taxes
Vacuum toilets	32	20,744.13
Vacuum pipe net	1	10,633.51
Vacuum station, mounted	1	27,288.41
Divers	1	5,556.46
Installation, plumbing	1	9,396.05
Planning	1	8,452.29
Sum of investments		82,070.87

Table 3. Supplemental investments.

Extraordinary costs	Price in Euro incl. taxes
Scientific evaluation	3,821.91
1 vacuum toilet for demonstration purposes	1,250.91

Table 4. Cost balance, savings and subsidies.

Source of crediting	Price in Euro incl. taxes
Costs of conventional toilets incl. pipes, estimated	−28,223.31
Subsidies from the regional government, sponsored by the "Wasserpfennig"	−29,433.54
Additional costs for the housing company	24,414.02

interest for the capital and the costs of ordinary maintenance. Due to several blockings in the ground pipes, caused by misuse of the toilets as waste bin by some residents, the system up to now could not be operated cost efficiently. Although, after a technical optimization of the system, we expect the system to run neutral in costs.

4 DISCUSSION AND CONCLUSION

At a flushing water consumption of 9 litres per person and day the concentration of organic carbon in the blackwater can be calculated to 1.5 g/l TOC (total organic carbon), when the TOC of the urine is not considered (Calculations according to data from Geigy 1995 and Koppe/Stozek 1998). The organic carbon in urine derives mainly from urea and therefore does not contribute to the production of methane. For an economic production of biogas the concentration should be higher. The addition of organic and food waste may double the carbon content. These could be taken in the system via the toilet or bigger items via a public grinder/crusher, which is connected to the public vacuum sewer. A significant increase of the gas production could be achieved by the addition of used cooking fat. A separate collection in the households should be considered. The fat has to be inserted directly into the digester via a mixer, to prevent deposits in the pipes. This is a well proofed method of agricultural biogas operators. The installed vacuum toilets demand 1.4 litres of water per flush according to manufacturers data. Meanwhile there are vacuum toilets available demanding only 0.3 litres per flush, but not yet in ceramic quality. The implementation of the suggested vacuum-based blackwater recycling system is more effective, when there is no public sewer already existing. The investments for vacuum sewers are lower than for gravity sewers. This cost benefit may compensate the additional costs in the private households. The rearrangement of the existing gravity sewer system will take a long period of time, since there is a lot of capital invested which has to be deducted during the lifetime of the system. The contamination of the blackwater with excreted drugs and their metabolites needs further research, since these substances have been detected in drinking water which was abstracted from bank filtrate.

ACKNOWLEDGEMENT

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REFERENCES

Geigy 1985. *Wissenschaftliche Tabellen Geigy*, Teilband Körperflüssigkeiten, 8. Aufl. 1977, 4. Nachdruck 1985, Ciba-Geigy AG, Basel.

HAZ 1997. Zuviel Salz in der Werra, *Hannoversche Allgemeine Zeitung*, 11 October 1997, Hannover.

Herrmann T. & Klaus U. 1997. Fluxes of nutrients in urban drainage systems: Assessment of sources, pathways and treatment techniques, *Wat. Sci. Techn.* Vol. 36, No. 8–9, p. 167–172.

Herrmann T. & Hesse T. 2002. Vakuumtoiletten im Wohnungsbau, *gwf Wasser Abwasser*, April 2002, Oldenbourg Vlg. München, 296–313.

Koppe P. & Stozek A. 1998. *Kommunales Abwasser*, 4. Auflage, Vulkan-Verlag, Essen, Germany.

Lindert, Mark (1995). Zur Kohlenstoff- und Energiebilanz von Schwemmkanalisation und aerober Abwasserbehandlung. *Proceed. Jahrestagung der Fachgruppe Wasserchemie in der GdCh*, 22. – 24.5.1995, auch KA 2/1995 S. 242ff.

Nowak 1993. Der Einfluss der Klärschlammbehandlung und Klärschlammentsorgung auf das Verfahrenskonzept der Kläranlage. In: *Wiener Mitteilungen, Wasser, Abwasser, Gewässer*, Bd. 110, Wien, cited according to Seyfried (1995).

RAwVwV 1992. *Rahmenabwasser-Verwaltungsvorschrift* vom 1.1.1992, Anhang 1, Gemeinden Größenklasse 5, Bundesgesetzblatt, FRG.

Rodewyk 1979. *Informationen über Kali, Ratgeber für die Landwirtschaft*, Heft Nr. 4, 2. Aufl., Kali + Salz AG (ed.), Kassel, p. 23.

Seyfried C.F. 1995. ATV-Fortbildungskurs H/3 am 10.3.1995: *Getrennte Schlammwasserbehandlung*, ATV, Bad Honnef, Germany (ed.).

Spiegel 1998. *Der Spiegel* 7/1998, Hamburg, p. 68.

TWJ 1993. *Trinkwasserjournal* 1/93, p. 6.

UBA 2000. *UBA Jahresbericht 2000*, Umweltbundesamt (German EPA), Berlin, p. 167.

Modelling of water discharges for houses with conventional and low water use appliances

C.S. Lauchlan
HR Wallingford, Wallingford, UK

A. Dixon
Sheffield Hallam University, Sheffield, UK

ABSTRACT: There is increasing pressure from regulatory authorities, water companies and domestic users for improved efficiency of new domestic appliances to reduce water consumption. Improved water efficiency will result in modified practices in the use of water and therefore it is important to ensure drainage systems are designed to continue to adequately transport solids as water usage patterns change. The present project was aimed at investigating the effect of reduced water use on domestic site drainage, and from the results propose changes to current practice to ensure future site drainage designs can adequately cope with reduced flows. This paper describes the development of typical discharge outflow hydrographs from domestic dwellings using stochastic techniques. Ten different types of domestic house were modeled for both a conventional water use scenario (circa 1985–1995) and a future water use scenario. A house type corresponds to a particular configuration of appliances. The flow volumes, outflow rates and frequency of usage for each scenario were based on an extensive review of available literature. The stochastic model was then developed to generate estimates of appliance use event frequency and appliance discharge volumes for single dwellings over a 24-hour period. The model used a Monte Carlo simulation method to generate the patterns of water usage. Probability distributions for each appliance volume were generated based on study data by Butler (1991). This stochastic modelling process produced household outflow hydrographs that were then used as inflow hydrographs for numerical models of small drainage networks.

1 INTRODUCTION

Conservation of water as a valuable natural resource has become one of the central issues in achieving sustainability. The 150l/hd/day currently used in the UK is predicted to rise in the near future, mainly as a result of an increase in single occupancy households, as well as an increase in shower ownership and frequency of use. In order to reduce this trend in consumption or at least prevent further rise, water saving practices, procedures and techniques have therefore been encouraged in recent years. However, the environmental and economic case for reduced water usage is often made on the basis of little information on the implications it may have on drainage and sewer systems.

The UK Department of Trade and Industry commissioned this study to assess the effect of reduced water usage in domestic buildings on the upper parts of the drainage system. This section of the drainage system is most critically affected by variations in the flow volumes and therefore most at risk of inadequately coping with the trend towards reduced water use.

The study involved a number of stages, from assessing the water discharge rates for both present and future reduced water use appliances, the stochastic modelling of discharges from different houses, through to numerical simulation of flows in small drainage networks.

This paper describes in detail the stochastic modelling aspect of the project, where water discharges for houses with conventional and low water usage appliances were simulated for a variety of different types of households. The outflow hydrographs that were produced as results of the stochastic modelling were then used as inflow hydrographs for numerical modelling of small drainage networks.

2 MODEL DESCRIPTION

2.1 *Background*

The stochastic model used in this project generates estimates of appliance use event frequency and appliance discharge volumes for domestic households. The program is based on previous work by Butler (1991) and Dixon (2000). Dixon (2000) describes the development and application of the original stochastic model.

The original stochastic model was used to generate water quantity and quality data in the form of extended, statistically valid time series of domestic appliance discharges. Estimates of appliance event frequency, then appliance discharge volume and finally appliance discharge water quality were then calculated. The data collected by Butler (1991) was used as the basis for the appliance usage cumulative probability distributions in the stochastic model.

A number of modifications were incorporated into the original program as part of this project, and are described in the following section.

2.2 *Modifications in present model*

For the present study a stochastic model was required that generated statistically valid time series of domestic appliance discharges for a range of different household types, for differing levels of occupancy and for differing appliance discharge characteristics.

The original model was therefore modified in a number of ways:

- Water quality information was not included,
- The original hourly time interval was reduced to sub-hourly intervals, as specified by the user,
- Modification parameters for frequency of use and flow rate probability distributions were introduced.

It should be noted that the original model produced a mixture of inflow and outflow discharges depending on the supply and demand of the greywater system.

An additional modification was also made to the model to simulate the use of an in-house siphon system. The siphon system is based on the concept of WISA drainage siphons. The system works by collecting 14–18 litres of wastewater discharged from the appliance(s) and once filled the collected wastewater is emptied by the principle of siphoning.

To account for this in the model the total discharge from all appliances at each time step was summed cumulatively. When the cumulative volume discharged reaches 18 litres the siphon begins to discharge. The discharge continues until the siphon is empty. The output time series is therefore made up of a time series of volumes discharged from the siphon only.

2.3 *Stochastic method description*

The model uses Monte Carlo simulation methods to generate hourly patterns of water usage from a range of different household appliances. Different probability distributions are used for weekday and weekend periods and household occupancies of 1 to 5 people can be selected. Study data by Butler (1991) was used to generate cumulative probability distributions for each appliance volume.

The Monte Carlo method is applied to generate further data estimates from limited data available. This method uses random numbers to index cumulative probability distributions made up from the appliance frequency of use data, thus generating time series of appliance events that have the same statistical properties as the parent data set. One important constraint of the Monte Carlo method is that it assumes independence between events in the series.

2.4 *Appliance types simulated*

The types of appliances included in the model were the WC, kitchen sink, bath, hand basin, washing machine, dishwasher and shower. No information was available on usage patterns for dishwashers as in the original study by Butler (1991) only 3 of the 28 houses surveyed had a dishwasher present. To overcome this restriction a dishwasher usage pattern based on washing machine information was developed.

It should be noted that to develop the model Dixon (2000) assumed that each house in the study had at least a bath, shower, washbasin and WC. In reality, only the bath, washbasin and WC were present in all 28 households. The shower was present in 19 (68% of households).

For both the washing machine and dishwasher the discharges were split into four equal component discharges. This is in order to simulate the filling-emptying routine that these type of appliances use.

2.5 *Appliance volumes and discharge rates*

The appliance volumes and discharge characteristics used in the present study are based on the results of an extensive literature review and a small-scale field test. Discharge rates and volumes were obtained from previous studies (e.g. Keating & Lawson (2000), Environment Agency Fact Cards (2001)), product tests (e.g. *Which? Online*, 1998 as summarized on the Environment Agency Fact Card 1), National Standards (BS8301:1985, BS6700:1987, BS EN 752-4:1997, BS EN 12056-2:2000) and information provided from manufacturers. Standards that have been superceded were included as the 'conventional' scenario is based on typical volumes and flow rates for household appliances circa 1985–1995.

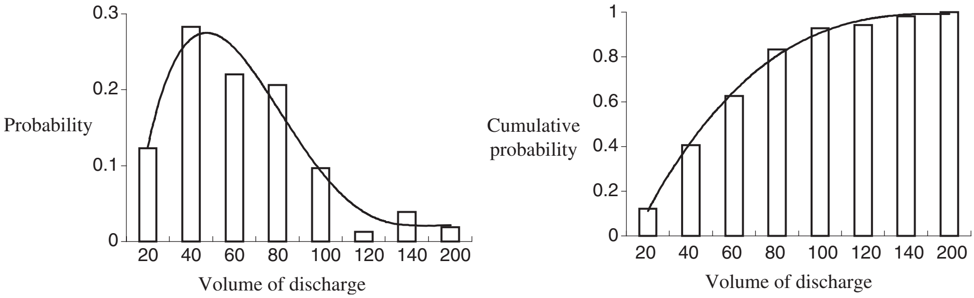
A summary of the appliance characteristics used in the present model is provided in Table 1.

The future use characteristics for 'frequency of use' were based on information provided for the future scenario options of the Market Transformation Program (MTP, 2002). This program identifies the key areas of likely change in the future up to the year 2025. The key areas of change with respect to water use are likely to be: (a) increased number of dishwashers per household; (b) substitution of baths for showers, and an increase in shower use frequency; (c) a decrease in the frequency of use of the kitchen sink as more dishwashers are being used.

Table 1. Summary of appliance discharge volumes and rates used in the present study.

	Conventional use (households circa 1985–1995)			Future use (new build) – beyond 2010		
Appliance	Volume (l)	Flow rate (l/s)	Frequency of use (use/head/day)*	Volume (l)	Flow rate (l/s)	Change of frequency of use **** (use/head/day)
WC	8	0.8	3.7	4	1.2	Constant
Shower	150	0.5	0.31	30	0.5	Increased by 35%
Bath	180	0.5	0.24	40	1.3	Decreased by 35%
Wash basin	6	0.5	3.4	3	0.15	Constant
Kitchen sink	22.5	0.5	2.0	20	1.2	Decreased by 50%
Washing machine	80**	0.7	0.17	25**	0.7	Constant
Dishwasher	25**	0.7	0.17	13**	0.2	Increased by 100%

* From Butler (1991).
** In the stochastic model this is discharged as four separate events of equal volume (each event = 25% of the total volume).
**** Based on Market Transformation Program (MTP, 2002).



Figures 1 and 2. Probability distribution and cumulative probability distribution for shower discharge volume frequency based on diary study data (Dixon, 2000).

The changes in appliance volume and flow rate for the future scenario are based on information provided by manufacturers and from national standards. For the project these change in volume, flow rate and 'frequency of use' result in an overall decrease in water consumption of approximately 40% for the future scenario compared to the conventional situation.

The discharge volumes for wash basin, shower and bath were modified to form frequency distributions. The range of each class was estimated to give a sufficient number of classes to maintain the distribution characteristic for each appliance but not too many as to take up excessive computation time. Subsequently, these frequency distributions were modified to form cumulative probability distributions similar to those used in the estimation of appliance event frequency (see Figures 1 and 2).

2.6 *Time interval*

The time interval of the original model was 1 hour. This had to be reduced to provide meaningful hydrographs for the drainage model. Appliance discharges generated by the original model at 1 hourly intervals were input to a further module. This module processed the hourly flow rate to be a flow rate in a unit of time specified by the user (e.g. 5 seconds). Each discharge event was assigned a random start time within the hour and was allowed to continue beyond the original hour if required.

2.7 *Simulation parameters*

A variety of different configurations of appliances and hence typical daily discharges were required for the study. To obtain these results a number of typical household types were developed. In total 10 different house types, each with a different appliance configuration, were determined. They are summarized in Table 2.

House types 6 and 7 are considered to be representative of possible future house types and are not presently common house types. House type 6 was chosen to represent a high water recycling level whereby the majority of water is recycled within the house. For testing of the siphon system the same appliance configuration as applied to Type 4a was used.

There are two occupancy levels given for each house type in Table 2. They were chosen to be representative of the likely range of occupancy levels that could reasonably be expected for a particular house type.

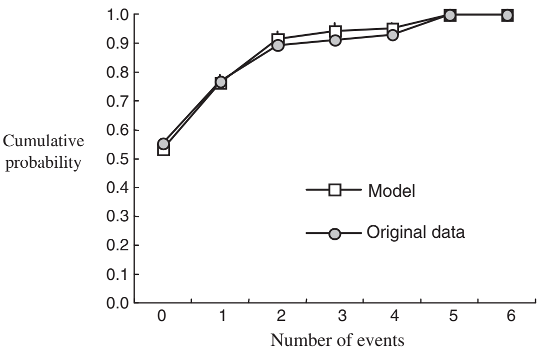
2.8 *Stochastic model validation*

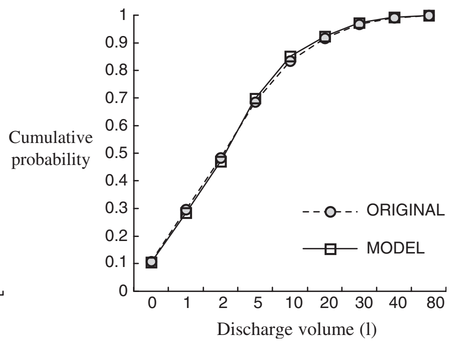
Dixon (2000) undertook a comparison of the model simulation results for frequency of appliance usage with the original data set and data for appliance usage (Butler, 1991). The model was found to generate data that was statistically similar to the original data even after a small number of trials.

Table 2. Typical household and appliance types.

House type no.	Appliances each house type contains	Occupancy no. of people	Typical house type
1	1B, 1WC, 1WB, 1KS	1, 2	1 bedroom terrace/flat
2a	1B, 1WC, 1WB, 1KS, 1WM	1, 4	1–2 bedroom terrace/flat
2b	1B, 2WC, 2WB, 1KS, 1WM	1, 4	1–2 bedroom terrace/flat
3a	1B, 1WC, 1WB, 1KS, 1WM, 1S	2, 4	Semi detached
3b	1B, 2WC, 2WB, 1KS, 1WM, 1S	2, 4	Semi detached
4a	1B, 1WC, 1WB, 1KS, 1WM, 1S, 1DW	2, 5	Semi detached
4b	1B, 2WC, 2WB, 1KS, 1WM, 1S, 1DW	2, 5	Semi detached
5	2B, 3WC, 3WB, 1KS, 1WM, 1S, 1DW	2, 5	Detached house
6	1KS, 1WC	1, 4	Water recycling scenario
7	Siphon system	2, 5	Siphon system

Key: B = bath, DW = dishwasher, KS = kitchen sink, S = shower, WB = washbasin, WC = toilet, WM = washing machine.





Figures 3 and 4. Comparisons of estimated discharge volume of the shower to the original data (Dixon, 2000).

The following figures show a comparison of probability distributions derived from model data and the original data. The 'number of events' plot shows the data produced over 249 trials generating shower events for the period 8–9 am. The 'discharge volume' plots in Figures 3 and 4, show the data produced over 1000 trials generating shower discharge volumes. Typically in the model tests the number of comparable trials is in excess of 1000.

The model validation for the present study therefore focussed on the time step modifications implemented for this project. It was necessary to verify that the model still reproduced the statistical characteristics of the original data set for all time resolutions.

A number of verification runs were performed and the model output was compared with frequency of domestic discharge data from Shouler et al, 1994. The general trend of a morning and an evening peak was present in both the study data and the model results. The frequency of appliance use was generally higher in the study data. This is likely due to differences in techniques used to gather the data. Butler (1991) used a diary study of 28 houses, while Shouler et al used flow meters in houses. Both studies do however fall within the wide spectrum of domestic appliance usage patterns.

The rounding errors resulting from model conversion of the 1-hour interval time series for a sub hourly series were also assessed. The errors are related to the relationship between the average flow rate for each appliance and the typical discharge volumes for each appliance. It was found that a time step of 5 seconds or less resulted in a difference in per capita consumption of 6% or less. It was therefore decided to use a time step of 5 seconds in all the simulations.

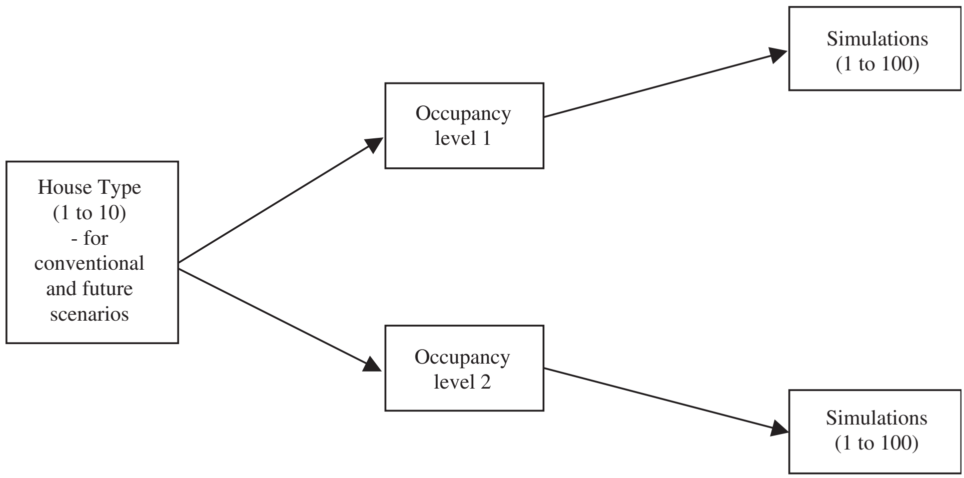


Figure 5. Outline of test procedure used for the stochastic modelling.

The impact of the length of the simulation was also tested. Simulations were run with both a one day and a five day simulation period, both using the same appliance configuration and occupancy level. The general characteristics of both tests were found to be the same.

Although the stochastic model is a powerful tool to generate time series of appliance discharges there are limits to what the model can simulate. One limit is the Monte Carlo method of estimation. It can simulate time series of data that have the same statistical properties as the parent data, although to do this it is assumed that each individual event is independent of another. For the case of water discharges from household appliances this is erroneous. For example, a WC use is commonly followed by washbasin use. The other limitation is in the assumption that volume of appliance discharge is independent of the time of day that it occurs.

For the present study it was concluded that these model limitations would not have a significant effect on the overall results and hence the method was appropriate.

3 METHODOLGY

3.1 *Simulation time and number of simulations*

The original model produced hourly patterns of water usage. This was modified for the present study to produce time series flow volume outputs at sub-hourly intervals down to a minimum of one second. All of the present study simulations used a time interval of five seconds in order to minimize rounding errors in the model.

An overall simulation period of 24 hours (weekday only) was chosen for each run. Each run was then repeated 100 times. Therefore for each scenario 100 daily outflow hydrographs were produced. Each run was also performed using both conventional appliance data and future appliance data. In total 3600 stochastic model simulations were undertaken. See Figure 5 for an outline of the test procedure.

The tests for the recycling scenario and the siphon system were performed using future appliance data only. It was decided not to test the conventional appliance models as both the recycling and siphon system scenarios are unlikely for this situation.

For the recycling tests only a kitchen sink and WC were used to produce discharges to the drainage system. For the siphon system the arrangement of household appliances was the same as for House Type 4a, as given in Table 2. A siphon capacity of 18 litres was used.

The time series results from the stochastic model tests were produced at 5-second intervals. These results were modified to 15-second intervals before being incorporated in the numerical

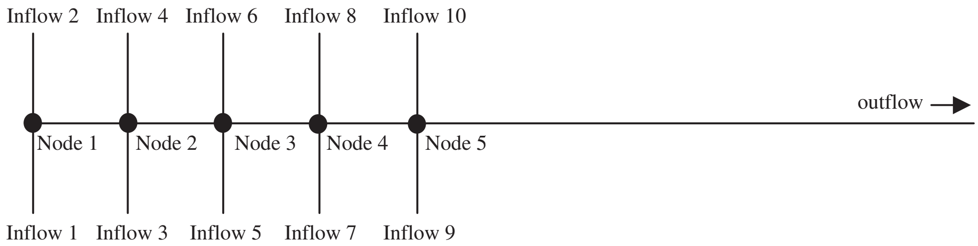


Figure 6. Example drainage network model.

models. The 15-second time step was the minimum allowable time step in InfoWorks CS. Tests using InfoWorks CS were performed to ensure the impact of this time step limitation was minimized. It was found that the peak flow rate of each discharge was slightly reduced but the volume discharge remained the same. These are described in more detail in the project report.

4 NUMERICAL DRAINAGE MODELLING

The results of the stochastic modelling described in this paper were used in the next stage of the overall project. This involved the development of numerical models of small drainage networks (1 to 10 houses) using InfoWorks CS. An example drainage network is given in Figure 6. The length of pipe between the inflow and the main drainage pipe was incorporated in order to simulate the attenuation of the discharges as they pass through the household drainage system out to the edge of the property. A limited field test, conducted as part of the project, confirmed that there is significant attenuation of the flow in this region.

Each inflow hydrograph produced using the stochastic model was used to represent an inflow into the drainage system. Ten inflows were therefore required per network. As shown in Figure 1, inflows were connected to the main drainage system in pairs. The distance between these pairs of flows was varied from 5 to 12.5 meters. The distance between the inflow points was chosen based on an analysis of a range of recent domestic housing developments. The shorter distances are more likely to be typical of terrace or semi detached housing, while the other end of the scale is suitable for the large detached homes.

As each stochastic model was repeated 100 times this allowed each numerical model to be tested ten times using different inflow hydrographs for each test. This approach was chosen to be consistent with the stochastic nature of the inflow results.

5 CONCLUSIONS

The stochastic modelling described in this paper has been undertaken as part of a larger project to assess the impact of reductions in water usage on drainage systems. The purpose of the modelling was to be able to generate a large number of outflow hydrographs from domestic houses that could be used in numerical modelling of small domestic drainage networks.

The stochastic model used was a modified version of the model described by Dixon (2000). This modified model was able to produce a statistically valid time series of results for appliance discharges from domestic dwellings at any time interval from 1 second to 60 seconds. In addition, the ability to model an in-house drainage siphon system was also incorporated.

To simulate domestic appliance discharges a total 10 different types of houses were defined, depending on the arrangement of appliances required. The appliance volumes and discharge rates were chosen to represent a 'conventional' scenario (circa 1985–1995) and a 'future' scenario (beyond 2010). The occupancy level of each house type was also varied so as to represent the range of likely occupants.

A large number of tests were performed and the results were used as inflow hydrographs for numerical modelling of small drainage networks.

Overall, the project has shown that stochastic modelling of household discharges using a Monte Carlo approach is applicable and can produce statistically valid time series of discharges from domestic households. It is a useful tool in assessing the future impact of reduced water use on small drainage networks.

REFERENCES

Butler D (1991). A Small-Scale Study of Wastewater Discharges from Domestic Appliances. *J. IWEM*, Vol. 5, p179–185.

Dixon, A (2000). Simulation of Domestic Water Re-Use Systems: Greywater and Rainwater in Combination, PhD Thesis, Department of Civil and Environmental Engineering, Imperial College, London.

Shouler, M.C., Ferret, F.H. and Pitts, N., (1994) Analysis of domestic appliance discharge patterns, *CIB W62 International Symposium on Water Supply and Drainage for Buildings*, PD143/94. Building Research Establishment. BRE, Garston, UK.

Market Transformation Programme (2002) Briefing Notes, http://www.mtprog.com/briefnotes/Briefing_Note_-_BN_WET_1.htm, updated 11/03/02.

Keating, T. and Lawson, R (2000). *The Water Efficiency of Retrofit Dual Flush Toilets.* Prepared for the Environment Agency and Southern Water.

Environment Agency (2001). *Conserving water in buildings – Fact Cards*.

National Standards (BS8301:1985, BS6700:1987, BS EN 752-4: 1997, BS EN 12056-2: 2000).

British Standards Institution (1984). BS 6340: 1984. Shower units. Specification for shower heads and related equipment. BSI.

British Standards Institution (1985). BS 8301: 1985 Code of Practice for Building Drainage (*superseded by BS EN 752*). BSI.

British Standards Institution (1987). BS 6700: 1987. Specification for the design, installation, testing and maintenance of services supplying water fir domestic use within buildings and their curtilages. BSI.

British Standards Institution (1994). BS 572: 1994 Code of Practice for sanitary pipework (*superseded by BS EN 12056*). BSI.

British Standards Institution (1995). BS 6465: Sanitary installations Part 1: 1995 Code of practice for scale of provision, selection and installation of sanitary appliances Part 2: Code of practice for space requirements for sanitary appliances.

British Standards Institution (2000). BS EN 12056-2:2000 Gravity drainage systems inside buildings. Part 2 Sanitary pipework, layout and calculation. BSI.

British Standards Institution (1998). BS EN 752-4:1997 Drain and Sewer Systems outside buildings. BSI.

The comprehensive monitoring of a rainwater recycling system and its potential impact on future water supply demand

M. Day
Severn Trent Water Limited

ABSTRACT: This paper reports on the comprehensive monitoring of a rainwater recycling system installed at domestic properties in the UK. The unit evaluated is a direct feed system, which uses rainwater for toilet flushing, washing machine usage and supplying an outside tap. By using state of the art metering, data logging and remote retrieval technology, the rainwater and water supply systems at two homes were closely monitored. The data collected enabled an accurate and daily water balance to be produced, including estimates of rainwater harvested. Rainfall data collected from a nearby rain gauge station were used to compare the rainwater harvested with volumes that could have potentially been collected during the exercise. With such detailed knowledge of the whole process available, it has been possible to model the performance of the recycling system for all years between 1995 and 2000, inclusive. The forecasts have been used to estimate typical annual savings, pay back periods and the potential impact on future demand.

1 INTRODUCTION

This project was a joint partnership exercise, set up to examine the performance of 'Freerain', the rainwater recycling system designed by Gusto Products. The partners in the project were, Gusto Construction Ltd, the project sponsors, Severn Trent Water Ltd., who provided the monitoring equipment and data analysis service, and the Environment Agency, who provided the shaft encoders and loggers for the tank measurements and gave advice and guidance on water management issues.

Gusto Construction Ltd. is a small privately owned, innovative homebuilder, producing quality, environmental and sustainable homes in Nottinghamshire. In order to achieve sustainable housing, each property they produce is designed to reduce consumption and recycle waste; wherever possible using recycled materials or products that are manufactured by environmentally friendly processes. Every home built incorporates a rainwater recycling system, together with many other environmental features; these include solar water heating, extra insulation, low energy appliances, heat recovery ventilation and garden composting units.

The product 'Freerain' is an advanced water recycling system, designed by their sister company Gusto Products, and is installed at each property on their award winning Millennium Green Development, in Collingham, near Newark. The company's policy of continuous improvement, both in design and construction was a key driver for the project. It was initiated so they could learn about the practical use of rainwater systems in the UK, with a view to improving the design, to ensure they have both a reliable and efficient working product.

With good measurement being an essential part of any research and development project, Severn Trent were asked to provide the logging and remote retrieval of consumption data, using the technology employed by their Domestic Consumption Monitor. Planning of the exercise started during the summer of 2000 and the first monitoring commenced a few months later, at the beginning of September.

2 THE 'FREERAIN' INSTALLATION

Rainwater is collected from all available roof surfaces, including garages and conservatories and diverted to a 3.3 m^3 rainwater tank located underground in the back garden of the property. Before entering the tank all rainwater passes through a filter, where the manufacturer claims about 10 percent of the collected volume is lost. At the Millennium Green properties there is no storm water run-off into local drains; at times when the tank is full, any excess over spills to a soak away. Each system is designed so that the tank overflows a minimum of three times a year.

When required the rainwater is pumped around the house, to the various toilets, washing machine and external tap. There is an additional 'top-up' supply, fed directly off the main supply to the home, which is used during times of dry weather. When the water in the tank drops to a critical level, the 'top-up' pump cuts in and adds to the tank a volume equivalent to whatever has just been removed, plus an additional 25 litres. This ensures that a sufficient supply of water remains in the tank at all times.

Germany was the source for most of the technology used to develop the system, as they have been using rainwater recycling systems for more than 20 years and currently have greater than half a million similar installations.

3 COMPREHENSIVE MONITORING PROCESS

3.1 *The properties*

Two different homes on the development were chosen for evaluating the project although the properties chosen to monitor were effectively self-selecting. It was desirable to install the equipment during the building of the homes in order for the meters, loggers and wiring to be installed discreetly and with a minimum of disruption to the customers. This meant if the monitoring was to commence as soon as possible, the properties that were nearing completion had to be chosen. With an average of 10 houses only being built each year, the selection was limited to a small number of homes. The first chosen was plot 7, a four bedroom, detached property which was purchased by a middle-aged couple. The second, plot 19, was a larger, six bedroom, detached home which was occupied by a family of five; a young couple with three young children. Monitoring started at plot 7 on 11th September 2000 and at plot 19 on 13th December 2000 and continued through until the end of August 2001.

3.2 *The monitoring equipment*

High accuracy, pulse output meters were fitted on the gas, electricity and water supplies; one each for the gas and electricity and three for the water. These were:

- one on the main supply, used for drinking water, cooking and personal hygiene.
- one on the outlet from the rainwater tank, to measure the use for toilet flushing, washing machine usage and external taps, and
- one on the 'top-up' supply, to monitor how much mains water was used to supplement any shortfall in rainwater.

The consumption through all five meters was logged at 15-minute intervals.

The Environment Agency provided shaft encoders and the relevant loggers for measuring the levels in the rainwater tanks. This allowed a complete balance of water in the system and home to be calculated.

3.3 *Data retrieval*

All of the data were retrieved remotely, every week, to a PC located at Severn Trent Water's Headquarters in Birmingham, by using the Vodafone data network and PMAC software developed by Technolog Ltd. This is the same technology currently employed by Severn Trent Water's Domestic Consumption Monitor, which has proven to be both reliable and effective.

3.4 *Tank calculations*

The rainwater storage tanks in the system are cylindrical in shape and are installed on their sides. This means that there is not a linear relationship between the level in the tank and the actual volume. The levels of rainwater in the tank were converted to estimated volumes by applying basic geometry calculations.

3.5 *Rainwater estimates*

The only input or output of the system that could not be measured directly was the rainwater entering the tank. However, a simple equation incorporating all other known values allowed an estimate of rainwater harvested to be produced.

3.6 *Water quality sampling*

Severn Trent Water's Quality and Environmental Services (QES) department took 25 water samples from the outside tap at Plot 7, between 16th October 2000 and 30th July 2001. Each sample was analysed for a variety of chemical and bacteriological parameters as advised by QES.

Since the completion of the monitoring exercise they have taken three much larger samples from each tank to analyse for *Campylobacter, Giardia* and *Cryptosporidium*.

4 THE ANALYSIS

4.1 *Rainwater usage*

In addition to splitting consumption between rainwater and mains supply used, it was possible to estimate the daily volumes of rainwater used solely for toilet flushing, by examining the fifteen-minute interval consumption. It was relatively easy to identify an individual or group of flushes during each time period, enabling an average number of flushes and a total volume per day to be estimated.

4.2 *Rainwater collection and system performance*

The performance of the rainwater collection system was evaluated by comparing estimates of rainwater harvested with the volumes of rainwater that potentially could have been collected. The 'potential' volumes were calculated from local rainfall data and the surface area of the properties. The nearest reliable rain gauge station to the Millennium Green development was Calverton, for which the Environment Agency provided 15-minute rainfall figures (in millimetres) for the 12 months period of the trial. The total collection surface areas for plot 7 and plot 19 were 150 m^2 and 168 m^2 respectively. Multiplying the rainfall (mm) in a given period of time by the surface area and dividing by 1,000 gave a volume of rainwater falling on the roof, in metres cubed, during that time. It was possible to compare the two sets of figures, potential and harvested, during time intervals as small as 30 minutes.

Daily comparisons were combined in order to summarise the performance of the system at each of the two properties. For each system it was possible to estimate the average amount of rainwater collected in the tank each day and the average amount of rainwater not collected, which includes water lost:

- via the overflow and 'soak away' at times when the tank was full.
- by adsorption and evaporation on the roof tiles.
- during the filtration process.

It was possible to extend this analysis further to provide estimates of rainwater lost during the collection process. At times when the tank was full it is not possible to separate out the volumes lost via the overflow and those lost by the collection system. In order to estimate this latter component accurately, only those days when the ‘potential’ volume was less than the spare capacity in the tank were included in the calculations. i.e. when there was room in the tank to collect all of the rain that fell on the roof during the day.

5 RESULTS

5.1 *Monitoring*

5.1.1 *Daily consumption*

The daily consumption of each property was broken down into water taken from the mains supply and rainwater used. The latter volumes are not those taken from the tank each day; any ‘top-up’ water used was subtracted from the volume taken from the tank, as it is already included in the mains supply figure.

Plot 7 used on average 285.5 litres/day, of which 161.5 litres was provided from the mains supply and 124.0 litres from rainwater, which is 43.4 percent of the average daily usage. If no ‘top-up’ water had been used to supply the tank, the figure would have been 46.5 percent.

Plot 19 used on average 579.0 litres/day, of which 362.3 litres was provided from the mains supply and 216.7 litres from rainwater. This latter figure equates to 37.4 percent of the average daily usage. This percentage figure is substantially less than for plot 7 because there were more occasions during the trial when ‘top-up’ water was used. If no ‘top-up’ water had been used the figure would have been 44.7 percent of the daily usage.

Water consumption during the exercise was around the level forecast. When taking into account the size of the family at each property, the usage at both plots was typical of average results obtained from Severn Trent Water’s Domestic Consumption Monitor.

The analysis of the toilet flushes in plot 7, showed there was an estimated 31.5 percent of the daily household usage attributed to toilet flushing. This comprised an average of 13.8 flushes a day, with a volume of 6.5 litres per flush. A similar analysis at plot 19 estimated toilet flushing at 26.4 percent of the daily household usage.

The remainder of the water taken from the tank can be attributed to washing machine usage and external use. e.g. garden watering and car washing. The residual values were 15.0 percent and 18.3 percent, for plots 7 and 19 respectively. There was no evidence from their consumption pattern, that the former used any water for external use. However, at plot 19 there were signs that above average quantities of water were being used from the tank during the month of June. The additional quantity was estimated at 3.93 m^3, which equates to 1.9 percent of total daily usage.

5.1.2 *Rainwater tank volumes*

An analysis of the volumes of rainwater in the tanks each day showed that in the autumn and winter months there were many days when, particularly at plot 7, the tanks were full to overflowing. However this was not the case during the spring and summer when, as mentioned previously, plot 19 used ‘top-up’ water on numerous occasions. The amount of ‘top-up’ water used was 15.3 m^3, which was in contrast to plot 7, where there was only 3.2 m^3 of ‘top-up’ supply required.

5.2 *Rainwater collection*

The results from the rainwater collection analysis showed that 62.8 percent of the rainwater falling on plot 7 was lost during the 12 months duration of the project. The corresponding figure for plot 19 was about 44.4 percent; substantially lower because consumption at this property was much higher. More rainwater was taken out of the tank each day, thus allowing more room in the tank for rainwater to be collected. These figures may seem high, but the period of the measurement exercise was extremely wet. The system modelling described later, forecasts that during the six years, 1995 to 2000, the average losses would have been 39.7 percent for plot 7 and 22.8 percent for plot 19. The corresponding figures for 1996 would have been just 21.7 percent and 7.9 percent respectively. Not surprisingly the biggest proportion of the losses occur during the wetter winter months.

5.3 *System performance*

The results from the system performance analysis showed that 12.4 percent of the rain falling on both of the properties was lost by the collection system. This compares favourably with the manufacturers claim that 10 percent of the rainwater is lost during the filtration process. Although it was not possible to measure, it seems reasonable to assume the other 2.4 percent were lost by adsorption on to the tiles or by evaporation by the sun and wind. This explanation is supported by the fact that on many days when rainfall was less than 1 mm, no rainwater collection was recorded.

Systems installed at properties completed since the start of the project have been fitted with a new integrated cross flow filter, which has reduced the collection losses to less than 5 percent.

5.4 *Water quality analysis*

The results of the 25 samples suggested that there could be low levels of faecal contamination possibly associated with birds. Eight of the samples showed presumptive *E. coli* ranging from 1 to 20 per 100 ml, with an average value of five. However the bacteriological quality appeared to improve as the system settled down; only one of the last 15 samples gave a positive result. All but two of the samples taken exhibited odours described as grassy, earthy or musty and in the samples taken after the project there was no evidence of *Campylobacter, Giardia* or *Cryptosporidium*.

Currently there are no regulations regarding quality standards for the use of rainwater, although there are some general recommendations. Severn Trent Water's Quality and Environmental Services department put their interpretation on the results obtained, based upon their knowledge and experience of potable supplies only. They felt that given the significant odours and relatively poor bacteriological quality compared to potable supplies, the water was suitable for toilet flushing and garden watering, but questioned its suitability for use in washing machines. This view was based upon the facts that although the rainwater supply is not covered by the Water Supply (Water Quality) Regulations 1989 they do cover washing and therefore washing machines would be included. In situations where 'Boil water advice' has been given (e.g. following *E. coli*. contamination of part of the distribution system), they would recommend the use of a 'Hot wash cycle' in washing machines.

5.4.1 *Feedback from customers*

All twenty of the properties on the Millennium Green development have been actively encouraged to comment upon their experiences of using rainwater in their washing machines. In addition they have all been given the option of having the washing machine feed switched to the mains supply. There have been no negative comments from any customer and no one has taken up the offer to revert to a mains water feed.

6 SYSTEM MODELLING

From the results of the monitoring and rainwater collection and system performance work, it was clear that during the trial period, the tank at plot 19 was not large enough. More than 40 percent of the rainwater that could have been collected was lost, while 15.3 m^3 of 'top-up' water was used to keep the system going during the drier periods. With such detailed knowledge produced about the system, it was possible to build a reliable model, to establish the optimum tank size. This also allowed other different factors in the model to be changed, so their effect could be estimated, thus providing forecasts of optimum savings. The factors in the model were:

- Rainfall (variable 1995–2000)
- Roof surface area (variable)
- Average daily household consumption (variable)
- Percentage of daily household consumption used from tank (constant = 45%)
- Percentage of 'potential' volume collected (constant = 87.5%)
- Level in tank, at which 'top-up' supply cuts in (constant = 19%)
- Tank size (variable)

For any combination of the factors, it was possible to forecast:

- The volume of rainwater used.
- The volume of 'top-up' water used.
- The number of days the tank would over flow.

Initially, the modelling was carried out in three stages:

- The 12 months period of the trial, a wetter than average year with 862 mm of rain.
- 1995 a drier than average year with 447 mm of rain.
- 1995–2000 a long term evaluation, with an average of 581 mm of rain.

For each of the time periods the following situations were modelled

- Plot 7, with an average consumption of 285 l/d and a roof surface area of 150 m^2.
- Plot 19, with an average consumption of 579 l/d and a roof surface area of 168 m^2.
- An average household within the Severn Trent area, with a consumption of 350 l/d and a roof surface area of 100 m^2.

Note: An assumption was made that, at the start of the period modelled, all of the tanks were filled to the level at which the 'top-up' supply cuts-in.

7 MODELLED RESULTS

Table 1. September 2000–August 2001 (rainfall = 862 mm).

	3.3 m^3 tank		4.7 m^3 tank		6.5 m^3 tank	
Situations modelled	Rainwater used (m^3)	'Top-up' used (m^3)	Rainwater used (m^3)	'Top-up' used (m^3)	Rainwater used (m^3)	'Top-up' used (m^3)
Plot 7	45.9	0.9	46.2	0.6	46.2	0.6
Plot 19	80.1	15.0	86.3	8.8	89.2	5.9
Average	53.3	4.2	54.5	3.0	56.0	1.5

Table 2. 1995 (rainfall = 447 mm).

	3.3 m^3 tank		4.7 m^3 tank		6.5 m^3 tank	
Situations modelled	Rainwater used (m^3)	'Top-up' used (m^3)	Rainwater used (m^3)	'Top-up' used (m^3)	Rainwater used (m^3)	'Top-up' used (m^3)
Plot 7	35.0	11.8	36.9	9.9	38.4	8.4
Plot 19	50.8	44.3	53.9	41.2	56.8	38.3
Average	33.1	24.4	35.4	22.1	37.0	20.5

Table 3. 1995–2000 (average rainfall = 581 mm/annum).

	3.3 m^3 tank		4.7 m^3 tank		6.5 m^3 tank	
Situations modelled	Rainwater used (m^3)	'Top-up' used (m^3)	Rainwater used (m^3)	'Top-up' used (m^3)	Rainwater used (m^3)	'Top-up' used (m^3)
Plot 7	42.4	4.4	44.1	2.8	44.9	1.9
Plot 19	63.5	31.6	68.2	26.9	72.1	23.0
Average	42.1	15.4	44.4	13.1	45.8	11.7

8 DISCUSSION OF MODELLED RESULTS

8.1 *Trial period*

The results for this particularly wet period, with an above average amount of rain falling in the 12 months (862 mm) are given in Table 1. In line with expectations, increasing the size of the tank for plot 19 increased the volume of water saved. By roughly doubling the size of the tank to 6.5 m^3, an extra 9.1 m^3 (11.1%) of rainwater used was forecast. In contrast Plot 7 made only 0.3 m^3 (0.6%) extra savings and an average household would have saved an extra 2.7 m^3 (5.1%).

Not unexpectedly the results of the modelling compared very favourably with what actually happened during the trials.

8.2 *1995*

Having monitored the rainwater system during one of the wettest periods in recent years, it was decided to forecast what would have happened in 1995. This was one of the driest 12 months during the last 10 to 20 years, when only 447 mm of rain fell. Moreover, there were only 90 mm of rain during the six months, March to August, and only 17 mm during the three summer months of June, July and August.

The forecasted savings in water usage given in Table 2, were greatly reduced from the trial period; Plot 7 was down 23.7 percent and Plot 19 and the average household were both down by around 37 percent. Furthermore, extra savings as a result of a larger collection tank were also reduced; for Plot 19 this was from 9.1 m^3 to 6.0 m^3 (down 34.1%).

Despite the very dry weather even plot 19 with a 6.5 m^3 tank would have overflowed on six occasions; all during November and December.

8.3 *1995–2000*

The results for this six-year period, given in Table 3, were obtained in order to have a more realistic forecast of the longer-term savings at properties where rainwater systems are fitted.

The largest savings were in 1998, which was not the wettest year; that was 2000. The reason for this is January, March and May of 2000 were relatively dry, with only 15 mm or less of rain per month, whereas in 1998, February was the only dry month at 8 mm. All the others had at least 20 mm recorded.

The smallest savings were in 1995, again not the driest year that was 1996. In 1996 there was a steady, if not particularly high rainfall each month, whereas in 1995, as previously stated the summer months were very dry. (less than 17 mm in total).

The effect of increasing the size of the collection tank depends upon the household consumption level and the collection potential. Table 4 below shows the extra savings in m^3 by increasing the size from 3.3 m^3 to 6.5 m^3.

Only at Plot 19 was there any significant advantage gained by increasing the tank size. At Plot 7 there was very little benefit from a larger tank, because there was nearly always more than enough rain collected to meet the demand. i.e. the collection potential is far greater than the level of consumption. For example between 1995 and 2000 the collection potential averaged at 87.2 m^3/annum, while the consumption was only 46.8 m^3 (53.7%). This is in contrast to Plot 19 and the average household where the two figure are approximately equal. The volumes for all three situations are given in Table 5.

Table 4. Extra savings by increasing tank size during 1995 to 2000.

Situation modelled	Average (m^3)	Minimum (m^3)	Maximum (m^3)
Plot 7	2.5	1.1	4.1
Plot 19	8.6	2.2	13.3
Average	3.7	0.0	5.7

Table 5. A comparison of rainwater tank consumption and potential volumes.

Situation modelled	Water use per annum (m^3)	Average for 1995–2000		Driest year 1996		Wettest year 2000	
		Potential (m^3)	Use as % of potential	Potential (m^3)	Use as % of potential	Potential (m^3)	Use as % of potential
Plot 7	46.8	87.2	53.7	60.0	78.0	114.3	40.9
Plot 19	95.1	97.6	97.4	67.2	141.5	128.0	74.3
Average	57.5	58.1	99.0	40.0	143.8	76.2	75.5

Even in the driest year, at Plot 7 the consumption was still only 78.0 percent of the collection potential, which was very similar to the equivalent figures for Plot 19 and the average household, in the wettest year. In 1996 the consumption for these two properties was more than 40 percent greater than what could be collected.

8.4 *General*

Clearly the biggest limiting factor within the process is the volume of rainwater that can be collected. This is calculated by rainfall in mm, times the surface area of the property's roof. Of the two elements in the calculation the surface area is perhaps the most limiting. In the absence of any data, a figure of 100 m^2 was used for the roof surface area of an average household. This is perhaps generous, as there are likely to be many houses much smaller than this. The properties included in the trial are very much larger than typical housing stock, which means savings in the trial will be larger than those in reality. The effect of reducing the roof surface area by half is exactly the same as halving the rainfall. This means that typically, even in a wet year, the savings for a property with a roof surface area of 75 m^2 will only equate to those forecasted at Gusto for 1995.

9 COST AND SAVINGS

9.1 *Typical saving*

For the long-term average (1995–2000), the modelled savings for a 3.3 m^3 tank, made on the annual water charges are:

- Plot 7 – £55.16/annum
- Plot 19 – £82.61/annum
- Average – £54.77/annum

* These figures are based upon water that has not gone through the mains supply meter, because rainwater was used.
The equivalent savings for a 6.5 m^3 tank are:

- Plot 7 – £58.41/annum
- Plot 19 – £93.80/annum
- Average – £59.59/annum

The current annual charge for 2003/04 is 130.1 pence/cubic metre, about 50 p of which is charged for taking away and treating the wastewater produced. At present domestic customers with rainwater systems are not charged for this on their rainwater usage.

9.2 *'Payback' periods*

When taking in to account the cost of a complete rainwater system, which includes tank, pump, pump control, top-up valve and micro-processing unit, average payback periods are likely to be in excess of 40 years. This does not allow for any maintenance or running costs in the estimate. Furthermore, if rainwater recycling 'takes-off' in a big way, it is highly likely that Water Companies would have to start charging customers for treating the waste water produced as a result of using rainwater. At current prices it would increase the estimated 'payback' period by almost 60 percent.

10 CONCLUSIONS

The 'Freerain' system is a reliable and effective way of providing rainwater to be used for toilet flushing, washing machines and outside taps. This generally accounts for 45 percent of a domestic household's daily water usage. However, typical annual rainfall and the roof surface area of the average domestic home means it is unlikely that sufficient rainwater will be harvested to support all three functions. Realistically the actual figure would average out at about 35 percent. Given the concerns about the suitability of rainwater for washing clothes, it may be advisable to restrict its use to just the two functions. In an average year using rainwater for just toilet flushing and outside taps would not affect the savings made.

From section 10 the savings and 'payback' periods forecast suggest that domestic rainwater systems are not viable, based purely upon the economics for the customer. System costs will have to be reduced significantly if costs/savings are important in selling the technology. This may not be too much of an issue as several thousand pounds on the cost of a new home may only represent one or two percent of the overall purchase price.

Although it may not be cost beneficial from a customer's viewpoint, perhaps the industry should be considering much wider issues. With the threat of global warming and pressure from the Environment Agency to reduce groundwater abstraction in certain areas of the country, rainwater recycling could be part of the answer. For example, if next year each new property built in the Severn Trent region were to have a rainwater recycling system installed, it could reduce demand by 3Ml/day; and in 10 years this could become 30Ml/day. Additionally there are also likely to be benefits for Sustainable Drainage Schemes (SuDS); rainwater harvesting is already considered an effective method of helping to reduce sewer flooding. Perhaps before too many more rainwater recycling systems are installed, the Water Industry should be looking at modelling the impact of installing large numbers of such systems on the general environment, including all aspects of the industry's business.

The views in this paper are those of the author and not necessarily those of Severn Trent Water Limited.

Domestic greywater characterisation and its implication on treatment and reuse potential

E. Friedler* & N.I. Galil
Faculty of Civil and Environmental Engineering, Technion, Haifa, Israel

ABSTRACT: In regions suffering from water scarcity, domestic greywater can serve as an alternative water source especially for toilet flushing and gardening. However, greywater may be highly polluted posing health and aesthetic risks. Eighteen scenarios were explored, in each at least one greywater stream was excluded and the effects on flow and pollutants loads were studied. Excluding the combined stream of the first 2 stages of the washing machine and dishwasher, and the kitchen sink reduced more than 50% of the organic matter, nutrients and boron. Excluding the bath and shower reduced the faecal coliforms significantly. The realistic penetration ratio in Israel was assessed by two different methods reaching 18–33% nationally in 2023 (equivalent water saving of 30–54 MCM/Y). This water saving is of the same order of magnitude of a medium size seawater desalination plant, but obviously a more sustainable option.

1 INTRODUCTION

Domestic water consumption in industrialised countries ranges between 100 to 180 [l/c/d]. This consists 30–60% of the urban water demand. Due to increasing water scarcity in many countries, new non-conventional water sources are developed (mainly seawater desalination). However, these "non conventional" sources entail relatively high production costs, as well as negative environmental effects, such as increased emissions to the atmosphere (CO_2 and other pollutants), disturbance to the adjacent marine environment (for each 1 m^3 of desalinated water produced about 1.7 m^3 is withdrawn from the sea), etc. Therefore, at the same time a thorough revision of domestic water uses is needed, in order to enhance utilisation efficiency, to lower the overall consumption and consequently minimise the desalination needs.

Besides minor quantities, most of the domestic in-house water consumption is transformed into domestic wastewater, which can be classified into two major categories:

i. *Greywater* originating from all domestic "water generating" appliances except toilets, comprising 60–70% of the in-house water demand.
ii. *Blackwater* originating from toilets, comprising 30–40% of the in-house water demand.

Toilet flushing and garden irrigation are the main possibilities of on-site domestic greywater reuse. Greywater reuse can reduce the specific domestic water demand by 45–65 lit./*capita*/day (l/c/d) or 16–23 m^3/c/y. This could become significant if considerable implementation is obtained (may reach 10–20% reduction of the urban water demand). The concept of in-house greywater reuse has been investigated lately especially in the EU and Japan, where natural resources conservation is the main motive for this initiative. However, since this concept is relatively new, full-scale systems are not common (Nolde, 1999; Ogoshi *et al.*, 2001; UK Environment Agency, 2000). Most of the research

* Corresponding author: eranf@tx.technion.ac.il

to date focused on the single-house scale, while the high-rise-buildings/neighbourhood scales are hardly reported.

Although Greywater is conceptualised to be "clean" it may be highly polluted and thus pose health risk and negative aesthetic and environmental effects (Almeida *et al.*, 1999; Diaper *et al.*, 2001; Dixon *et al.*, 1999; Rose *et al.*, 1991). It also exhibits high variability, which is reflected in high fluctuations of discharge volumes and pollutants loads, for example: discharge volume of the shower ranges from 2 to 120 l/use and its COD load ranges from 8,000 to 36,000 mg/use (Friedler and Butler, 1996). As a result of the above, direct in-house reuse requires highly efficient and reliable conveyance, storage and treatment systems, in order to prevent the use of water that may cause health risk, negative aesthetic effects (i.e. offensive odour & colour) and adversely effect the environmental. This is especially true in warm countries were higher ambient temperatures may increase organic matter degradation rate and enhance regrowth of pathogens.

Various treatment processes are suggested in the literature. However, since in-house greywater recycling is in its infancy only few of-the-shelf systems are commercially available and even less were tested on a full scale for long periods of time (UK Environment Agency, 2000; Diaper *et al.*, 2001). Initially, physical processes were preferred. Today a combination of physical, biological and chemical treatment processes are reported, which are usually followed by a disinfection unit (Hills *et al.*, 2001; Jefferson *et al.*, 2001; Ogoshi *et al.*, 2001; Shin *et al.*, 1998). Due to space constraints, the treatment technologies selected are ones who have small footprint.

The research carried out in the Technion is divided into four main stages:

i. Characterisation of quantity and quality of greywater streams generated from individual greywater generating appliances.
ii. Assessment of the water saving potential in different sectors (i.e. domestic; education; industry, commerce and public service; tourism), in order to assess the national water conservation potential and to set priorities for implementation.
iii. Construction and operation of a pilot plant to examine the performance of treatment technologies. The plant includes RBC base system and MBR based system.
iv. An engineering & techno-economic study, where engineering aspects will be examined and a feasibility analysis will be conducted. This will be done in order to assess the minimal size of the greywater recycling scheme which is economically feasible. Broader issues on the neighbourhood – city – nation levels will be also considered.

The first two stages were completed during the first year of the research; a pilot plant treating the greywater of 14 flats was constructed in the Technion campus and is at present in its start-up phase; and the framework for the techno-economic feasibility study was set. This paper describes the results of the greywater sampling campaign, discusses the implication of the findings on treatment and reuse options while exploring various scenarios of inclusion or exclusion of different greywater streams, presents a systematic approach for the assessment of the realistic national water saving potential.

2 GREYWATER CHARACTERISTICS AND THEIR IMPLICATION ON TREATMENT AND REUSE OPTIONS

2.1 *Characterisation survey*

A sampling campaign was performed in order to characterise the quantity and quality of greywater generated from individual household appliances. A total of 140 samples were taken, with the following distribution: wash basin (WB) 33, bath (BT) 10, shower (SH) 19, kitchen sink (KS) 19, dishwasher (DW) 24, and washing machine (WM) 35. The discharge volume of each sampled event was measured. 20 parameters were analysed in the laboratory in accordance with the methods of Standard Methods (APHA *et al.*, 1998): pH, electrical conductivity (EC), chlorides, sodium, boron, ammonia, phosphate, total solids, volatile total solids, suspended solids (total & volatile), COD (total & dissolved), BOD (total & dissolved), TOC (total & dissolved), total oil, MBAS and faecal coliforms (FC). All information was stored in a database which contains more than 2,000 data entries. Table 1 presents the average values, the standard deviation (Std) and the coefficient of variation (CV) obtained for each parameter-appliance combination, with the exception of the washing machine and the dishwasher or which only the average values are presented (their variability is discussed separately). Table 2 presents the calculated total specific daily load of domestic greywater as calculated by summation of each appliance pollutant load. The specific daily discharge volume was found to be 104 l/c/d (agreeing well with the national average of 98 l/c/d; Shisha, 2003), TSS load of 27 g/c/d and BODt of 36 g/c/d, the latter two comprising 50–60% of the "common" specific load of municipal sewage.

- As expected, the washing machine, dishwasher and kitchen sink are the major pollutants generators, with COD concentration of about 1,300 mg/l, BOD of up to 700 mg/l, phosphate up to 500 mg/l and 600–700 mg/l chlorides and sodium.
- The dishwasher was found to be the most significant boron contributor with an average concentration close to 4 mg/l (one order of magnitude that all other appliances). WM effluents in Israel (in contrast to other countries) do not contain high boron concentration, since all manufacturers in Israel are required to produce low boron content powders/liquids (Israel Ministry of Environment, 1999).
- In contrast to the above, the bath and shower were signaled as the major sources of faecal coliforms with average concentrations of $4 * 10^6$ CFU/100 ml.

All the 4 appliances for which the variability is depicted in Table 1, pose high variability both in the quantity and quality of effluents as expressed by high CV. The wash basin greywater stream presented the highest volume variability (CV 0.9), while for most quality parameters the bath and kitchen sink greywater had the highest variability (with many CV values well over 1).

2.2 *Source separation*

In order to develop a proper treatment and reuse scheme of greywater, each source daily discharge volume and pollutants load was evaluated. Figure 1 depicts the absolute and relative contribution of each appliance, were the high variability between appliances is explicitly presented. For example:

- The dishwasher and the washing machine were signaled out as the major sources of most pollutants with 24% and 16% of the daily discharge volume, 49% and 12% of the TSS, 36% and 25% of the CODt, 49% and 22% of the BODt, 51% and 19% of the total oil respectively.
- The wash basin in contrast was found to be the least polluting appliance contributing less than 10% to the total load of all pollutants.

The washing machine and dishwasher, being major pollutants contributors, deserve special attention. Whereas their discharge volumes are quite constant, their pollutants emission shows distinct stepwise pattern. The cleaning cycles of the WM and the dishwasher are divided to 4–5 stages, the firsts of which contribute most of the pollutants loads Figure 2. The first stage ("wash") of the washing machine contributes only 18% of the discharge volume, but as much as 64%, 40%, 30% and 34% of the total load of COD, NH_4-N, PO_4-P and detergents (MBAS) respectively. For the dishwasher it is the second stage ("rinse 2") which is the major pollutants contributor with 33% of the volume and 50%, 70%, 90%, 27%, 64% and 84% of the total load of COD, NH_4-N, PO_4-P, detergents (MBAS), chlorides and boron respectively.

Considering the findings presented in Figure 1 and Figure 2 and the fact that the daily flow for potential domestic reuse (toilet flushing and garden irrigation) is substantially less than greywater daily discharge (about 65% of the generated greywater or less, see below), it may be possible not to treat and recycle all greywater streams, but rather to chose the less polluted ones and thus need less treatment and have less potential negative health, environmental and aesthetic effects.

In order to explore the consequences of source separation, 18 different scenarios Table 3 were studied in each, at least one greywater stream was excluded and its effect on the daily discharge volume and pollutants loads was calculated. It should be noted that although this type of separation may seem impractical today, the technological ability does exist and the only shift to be performed is a conceptual one to be taken by engineers and decision makers.

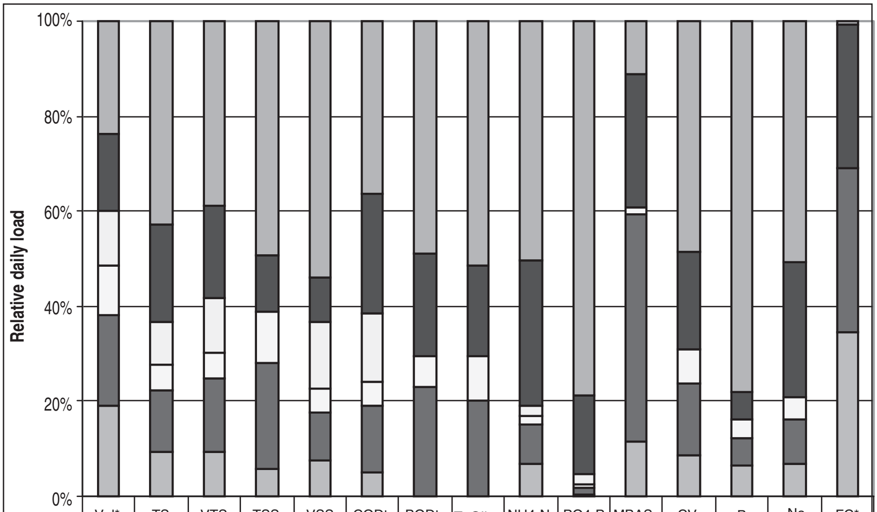
Figure 3 shows that scenario 18 resulted in the most significant improvement of the main greywater stream. The total daily flow was reduced by 35%, the loads of CODt, BODt, NH_4-N and PO_4-P reduced

Table 1. Pollutants concentration for each appliance.

		BT			SH			WB			KS			WM	DW
Parameter	Units	Avg	Std	CV	Avg	Std	CV	Avg	Std	CV	Avg	Std	CV	Avg	Avg
Volume	l/use	53	27.5	0.52	28	18.8	0.66	1.9	1.7	0.9	12	5.96	0.51	85	22.4
PH	–	7	0.04	0.01	7	0.36	0.05	7.0	0.3	0.04	–	–	–	7.5	8.2
EC	μmho/cm	1,200	409	0.34	1,565	485	0.31	1,200	401	0.33	–	–	–		2,721
TS	mg/l	777	303	0.39	1,090	440	0.40	835	263	0.31	1,272	1,020	0.80	2,021	2,819
VTS	mg/l	318	244	0.77	533	281	0.53	316	194	0.61	661	593	0.90	765	1,045
TSS	mg/l	78	105	1.34	303	205	0.68	259	130	0.50	–	–	–	188	525
VSS	mg/l	76	97.7	1.28	102	83.6	0.82	86	51.5	0.60	250	329	1.31	106	424
COD t	mg/l	230	195	0.85	645	289	0.45	386	230	0.60	1,083	1,040	0.96	1,339	1,296
COD d	mg/l	165	105	0.63	319	218	0.68	270	173	0.64	634	620	0.98	996	547
BOD t	mg/l	173	218	1.25	424	219	0.52	205	42.5	0.21	–	–	–	462	699
BOD d	mg/l	75	65.6	0.88	237	125	0.53	93	57.3	0.61	–	–	–	381	262
TOC t	mg/l	91	89.1	0.98	120	69.6	0.58	119	44.3	0.37	–	–	–	361	234
TOC d	mg/l	47	27.6	0.59	59	31.5	0.53	74	26.3	0.35	–	–	–	281	150
Tot. Oil	mg/l	77	114	1.48	164	150	0.91	135	177	1.31	–	–	–	181	328
NH_4-N	mg/l	1	1.49	1.67	1.3	0.83	0.66	0.4	0.29	0.74	0.5	0.85	1.71	4.9	5.4
PO^4-P	mg/l	5	5.34	1.17	12	13.7	1.18	15	13.8	0.95	29	29.4	1.00	169	537
MBAS	mg/l	15	14.9	1.02	61	46.8	0.77	3.3	31.1	0.93	–	–	–	42	11.1
Cl^-	mg/l	166	128	0.77	284	167	0.59	237	118	0.49	–	–	–	450	716
B	mg/l	0.4	0.09	0.21	0.35	0.12	0.35	0.4	0.2	0.49	–	–	–	0.4	3.8
Na	mg/l	112	44.2	0.39	151	82.9	0.55	131	56.8	0.43	–	–	–	530	641
FC	cfu/100 ml	4E6	6.9E6	1.73	4E6	8.5E6	2.12	390	–	–	–	–	–	4E6	6E4

Table 2. Total daily pollutants' load of domestic greywater.

Vol l/c/d	VTS mg/c/d	TSS mg/c/d	VSS mg/c/d	CODt mg/c/d	CODd mg/c/d	BODt mg/c/d	TOCt mg/c/d	T. Oil mg/c/d	NH_4-N mg/c/d	PO_4-P mg/c/d	MBAS mg/c/d	Cl^- mg/c/d	B mg/c/d	Na mg/c/d	FC CFU/c/d
104	66,961	26,673	19,626	89,138	50,390	35,862	15,610	15,982	270	17,049	2,491	36,915	123	31,566	2.3E + 08



	Vol*	TS	VTS	TSS	VSS	CODt	BODt	T. Oil	NH4-N	PO4-P	MBAS	CV	B	Na	FC*
DW	25	70,503	26,130	13,134	10,616	32,425	17,491	8,211	136	13,433	277	17,897	96	16,035	1.4E+6
WM	17	33,950	12,844	3,163	1,785	22,500	7,769	3,048	82	2,817	700	7,556	7	8,910	7.0E+7
KS	12	14,964	7,773		2,783	12,739			6	345					
WB	11	9,348	3,537	2,903	962	4,320	2,292	1,510	4.3	162	37	2,656	5	1,463	4.4E+3
SH	20	21,370	10,452	5,942	1,991	12,649	8,309	3,213	23	203	1,189	5,557	7	2,956	8.0E+7
BT	20	15,237	6,225	1,531	1,490	4,505			18	89	288	3,248	8	2,202	8.0E+7

Figure 1. Relative pollutant contribution of each greywater generating appliance (All values in mg/c/d except: vol – l/c/d, FC – CFU/c/d; Vertical internal order of each column follow appliances vertical order in the legend).

to 50–60% of their original levels, the load of boron reduced to less than 20% of its original level, while a milder effects were observed on other pollutants. Scenarios 3 and 15 were also significant having somewhat less distinct positive effect on pollutants loads, but reducing the daily flow merely by 27% and 17% respectively. The above is true for organic matter and nutrients, however as to pathogens (as indicated by *E. Coli.*), the bath and shower are the major sources – with reduction to 65% of the original FC concentration when either one is excluded from the general stream. This posses a dilemma as high organic matter and nutrients concentrations may lead to negative aesthetic effects and to greater regrowth potential of pathogens on one hand; high concentrations of chlorides, sodium and boron may negatively effect soil and plants, while high pathogens concentrations may pose higher health risk.

3 ASSESSMENT OF THE WATER SAVING POTENTIAL

Most greywater reuse schemes up to date focused on the single house – small scale systems, that do not contribute significantly to the regional/national water budget. This section analyses the national water saving potential of large scale implementation of on-site domestic greywater reuse schemes in the urban sector. Israel was chosen as a case study as it represents semi-arid countries suffering from water shortage. A shortage that is the reason for the introduction of a national desalination scheme of hundreds of million cubic meters per year (MCM/y), by 2008. The specific domestic water demand in Israel averages 161 l/c/d (59 m^3/c/y), totalising 390 MCM/y nationally (2003) – the second major water consuming sector, after agriculture. Table 4 depicts the distribution of the domestic water demand for various uses.

The following data served as a basis for assessing the national water saving potential:

i. Israel's population was 6.7 Million people in 2003 and its growth rate approximates 2% *per* year (Israel Central Bureau of Statistics, 2002).

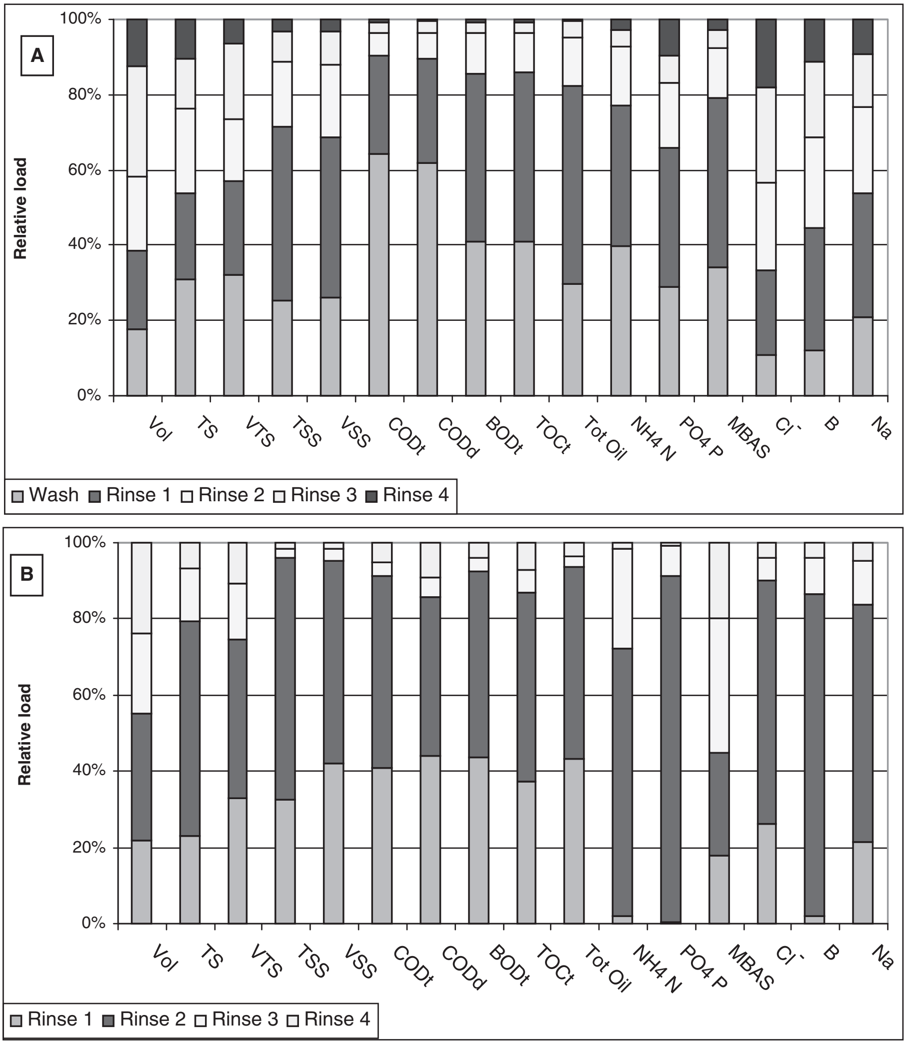


Figure 2. WM (A) and Dishwasher (B) – Relative pollutant loads of different stages of the cleaning cycle.

Table 3. Excluding various greywater streams – The 18 scenarios studied.

Scenario	Excluded appliance	Scenario	Excluded appliance
1	BT	10	DW Rinse 1
2	KS	11	DW Rinse 2
3	SH	12	DW Rinse 3
4	WB	13	DW Rinse 4
5	WM Wash	14	WM Wash 1 + DW Rinse 1
6	WM Rinse 1	15	WM Wash 1 + DW Rinse 1 + KS
7	WM Rinse 2	16	DW Rinse 1 + DW Rinse 2
8	WM Rinse 3	17	WM Wash + WM Rinse 1
9	WM Rinse 4	18	WM Wash + WM Rinse 1 + DW Rinse 1 + DW Rinse 2 + KS sink

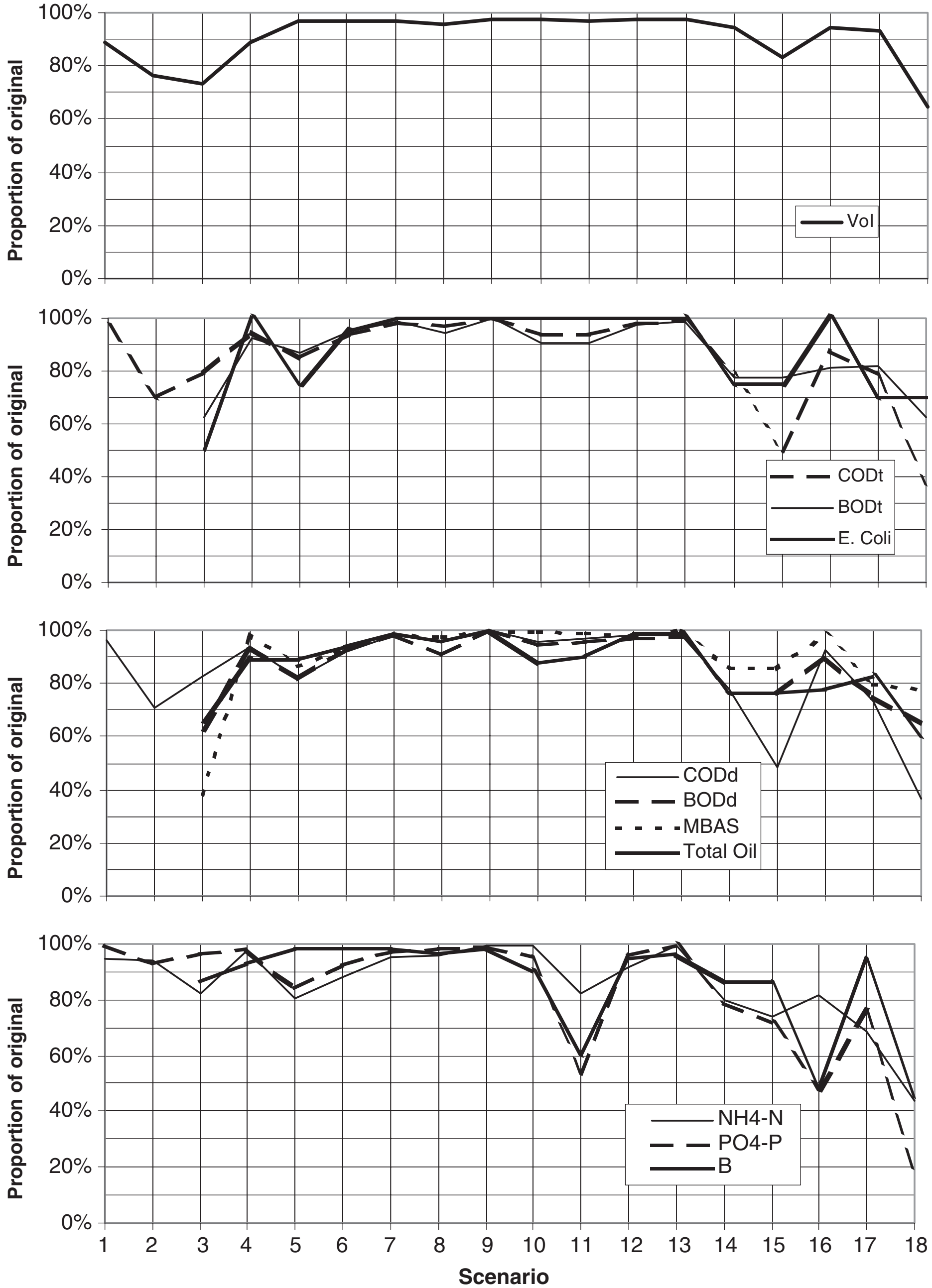


Figure 3. Effects of excluding various greywater streams on domestic greywater quantity and quality.

ii. Present daily WC flushing volume is 55 l/c/d (Shisha, 2002), with most houses having dual flush toilets of 9 and 6 l/flush for full and half flushes respectively. As lower volume dual flush toilets were introduced during 2003 (flush volumes of 6 and 3 l/flush) the future daily WC flushing volume is projected to reduce to 37 l/c/d.

Table 4. Average domestic water demand characteristics in Israel (Shisha, 2002).

		Appliance	Daily demand (l/c/d)	Proportion	
Indoor		Bath/shower	40	25%	
		Wash basin	15	9%	Total
	Greywater	Kitchen	30	19%	Greywater
		(incl. D. washer & K. sink)			98 l/c/d 61%
		Washing machine	13	8%	
	Blackwater	Toilet flushing	55	34%	Reuse Potential
Outdoor		Garden watering	8	5%	63 l/c/d
Total			161		

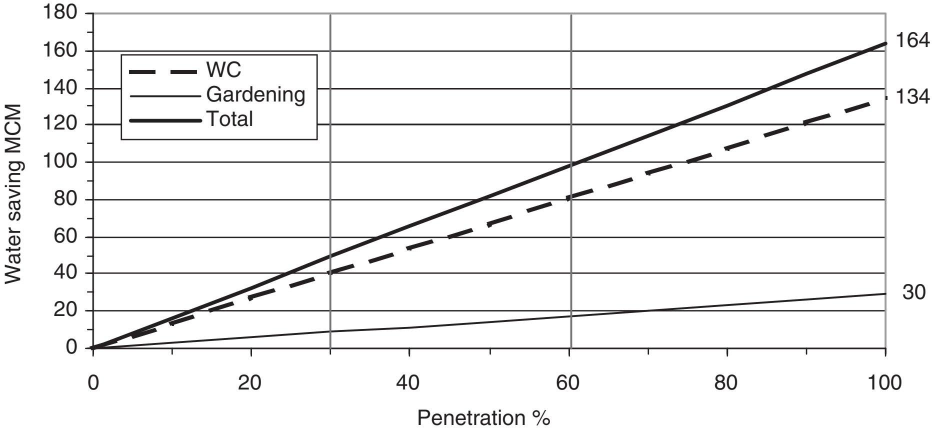


Figure 4. The water saving potential in domestic sector in Israel in 2023 (10 million residents).

Figure 4 depicts the water saving potential as a function of the relative penetration of greywater reuse schemes in the domestic sector. 3 possible scenarios were analysed for 2023:

i. *Full penetration* – i.e. all houses domestic sector will contain a greywater reuse unit. This is actually the maximum potential of water saving (which is unrealistic), with 164 MCM/Y.
ii. *60% penetration,* a maximal scenario with water saving of 98 MCM/Y. It is unrealistic in 20 years time, but may become such realistic in the further future.
iii. *30% penetration* – This may be realistic in 20 years time. Here the water saving sums to 49 MCM/Y of which 40 and 9 MCM/Y for toilet flushing and garden irrigation respectively.

3.1 *Assessment of the likelihood of 30% national water saving in 2023*

In order to assess the realistic penetration ratio of on-site greywater reuse in the domestic sector in 2023 and the consequent water saving potential, the following assumptions were made:

- 3 storey high building containing 12 flats is the smallest building that can finance construction, operation and maintenance of greywater reuse system by a professional certified firms. In the last 20 years about 55% of newly constructed residential flats are built in buildings of at least 3 storeys high (Israel Central Bureau of Statistics, 2002). It was assumed that the same ratio will remain in the future (a very conservative assumption as the country is becoming heavily populated).
- Average "core" family size is 3.36 person (Israel Central Bureau of Statistics, 2002). It was assumed that one flat is occupied by one family and that the family size will not change in the future.

The realistic water saving potential in 2023 was assessed by two methods:

i. *Building design life* – The design life of buildings is 50 years, e.g. each year 2% of the existing buildings are replaced by new ones (or totally refurbished). Thus in 20 years time:
   - 40% of today's population will live in new buildings.
   - The population will increase by 3.2 million people.

   The above values sum to 5.9 million people who will live in flats constructed from 2003 onwards. At least 55% of them will reside in buildings of 3 storeys or more. Calculating the above yields water saving of about 54 MCM/Y (33% penetration), of which 44 and 10 MCM/Y results from reuse for toilet flushing and garden irrigation respectively.
ii. *Newly constructed flats* – In 2002 47,100 newly constructed residential flats were handed to residents. In the last 40 years this number increased by 450 flats/year (Israel Central Bureau of Statistics, 2002), thus in the next 20 years 998,000 new residential flats will be constructed (55% of which in buildings of 3 storeys or more). Calculating the above, while considering family size, yields water saving of 30 MCM/Y (18% penetration) of which 25 and 5 MCM/Y results from reuse for toilet flushing and garden irrigation respectively.

Thus, the projected realistic penetration ratio for 2023 ranges from 18% to 33% which corresponds to water saving of 30–54 MCM/Y nationally. This water saving is of the same order of magnitude of a medium size seawater desalination plant.

3.1.1 *Other sectors*

The discussion above concentrated on the domestic sector. The same methodology could be implemented for assessing the water saving potential in other sectors (i.e. business, commercial, education, industrial etc.). The water saving potential of these sectors differs from the one of the domestic sector, in accordance with:

- Sources of greywater, and the relative contribution of each source.
- The specific quantity of greywater generated (l/c/d).
- The specific number of toilet flushes (number of toilet flushes/person/day).
- The specific area of garden (if at all) (m^2/c).
- The possibility to construct and operate greywater reuse schemes.
- The health risk that the recycled greywater may pose (especially important in the health care sector).

4 SUMMARY AND CONCLUSIONS

Due to increasing water scarcity in many countries, non-conventional water sources are developed (mainly seawater desalination). These sources entail relatively high production costs and cause negative environmental and aesthetic effects. Therefore, simultaneously a thorough revision of domestic water uses is needed, in order to enhance utilisation efficiency, lower the overall consumption and consequently minimise the desalination needs. Domestic greywater has a high potential for on-site reuse mainly for toilet flushing and garden irrigation.

The research conducted in the Technion comprises of 4 steps: domestic greywater sources characterisation, assessment of the national water saving potential, pilot scale study of greywater treatment and reuse, and a feasibility study. The sampling campaign which comprises more than 2000 data entries showed that domestic greywater, may be highly polluted and thus pose health and environmental risks and cause aesthetic nuisance. The washing machine, dishwasher and the kitchen sink were signalled as the major contributors of organic matter, phosphate and salts (chlorides and sodium). The dishwasher was found to be the most significant contributor of boron. In contrast to these, the bath and shower were the major contributors of faecal coliforms. This posses a dilemma when treatment and reuse is considered as high organic matter, nutrients and salts concentrations may cause negative aesthetic and environmental effects and greater regrowth potential of pathogens, while potentially high pathogens concentrations (as indicated by high FC concentrations) pose elevated health risk.

In order to develop a proper greywater treatment and reuse scheme, 18 different scenarios were explored, in each a different combination of greywater streams was excluded from the "main greywater stream", and the effects on the quality and quantity of this "main stream" were studied. This analysis showed that excluding the combined stream of the washing machine wash and the first rinse, together with the first 2 stages of the dishwasher cleaning cycle and the kitchen sink effluent resulted in a 35%, 60%, 40% and 80% reduction of the daily greywater volume, organic matter, nutrients, and boron loads respectively.

The realistic penetration ratio of on-site greywater reuse in the domestic sector in Israel was assessed by 2 different methods reaching 18–33% penetration in 2023. This corresponds to 30–54 MCM/Y water saving nationally (pop. of about 10 million people in 2023), of which about 80% results from reuse for toilet flushing and the rest from reuse for garden irrigation. This is a substantial water saving which is in the same order of magnitude of a medium size seawater desalination plant. Greywater reuse in the domestic sector is a step towards more sustainable urban water utilisation, that considers not only human needs, but also broad environmental aspects.

ACKNOWLEDGEMENTS

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REFERENCES

Almeida M.C., Butler D. & Friedler E., 1999, At-source domestic wastewater quality, *Urban Wat.,* **1(1)**: 49–55.

APHA, AWWA, WEF, 1998, Standard methods for the examination of water and wastewater. 20th Ed.

Diaper C., Dixon A., Butler D., Fewkes A., Parsons S.A., Stephenson T., Strathern M. & Strutt J., 2001, Small scale water recycling systems – risk assessment and modelling. *Wat. Sci. Tech*., **43(10)**: 83–90.

Dixon A.M., Butler D. & Fewkes A., 1999, Guidelines for greywater reuse: Health issues. *J. IWEM*, **13**: 322–326.

Friedler, E. & Butler, D.,1996, Quantifying the inherent uncertainty in the quantity and quality of domestic wastewater. *WST*, **33 (2)**: 63–78.

Hills S., Smith A., Hardy P. & Birks R., 2001, Water recycling at the Millennium Dome. *WST.,* **43(10)**: 287–294.

Israel Central Bureau of Statistics, 2002, *Data from Internet site*: www.cbs.gov.il

Israel Ministry of Environment, 1999, *Water regulations: prevention of water pollution – laundering powders.*

Jefferson B., Laine A.L., Stephenson T. & Judd S.J., 2001, Advanced biological unit processes for domestic water recycling. *WST.*, **43(10)**: 211–218.

Nolde E., 1999, Greywater reuse systems for toilet flushing in multi-storey buildings: over ten years experience in Berlin. *Urban Wat.*, **1**: 275–284.

Ogoshi M., Suzuki Y. & Asano T., 2001, Water reuse in Japan. *WST*, **43(10)**: 17–23.

Rose J.B., Sun G.S., Gerba C.P. & Sinclair N.A., 1991, Microbial quality and persistence of enteric pathogens in greywater from various household sources. *Wat. Res*., **25(1)**: 37–42.

Shin H.S., Lee S.M., Seo I.S., Kim G.O., Lim K.H. & Song J.S., 1998, Pilot-scale SBR and MF operation for the removal of organic and nitrogen compounds from greywater. *WST*, **38(6)**: 79–88.

Shisha A., (personal communication) 2002 Division of water saving, Water Commission, Israel.

UK Environment Agency, 2000, *A study of domestic greywater recycling*. 60 p.

The design and operation of a grey water treatment plant

A.D. Wheatley
Department of Civil & Building Engineering, Loughborough University, Loughborough, Leicestershire, UK

S. Surendran
Environment Agency, Kings Meadow House, Kings Meadow Road, Reading, UK

ABSTRACT: Example results are presented from a five year study of combined grey water and rainwater reuse for lavatory flushing (40 population student hall). Treatment (75 litres/hr) was a sequence of balancing, bioreactor and filtration. The project also included metering of all water using appliances and the data shows increased showering compared to domestic households but otherwise similar use and peaking factors. There was always surplus reclaimed water but washing machine effluent was added during year 2 to challenge the treatment plant. Average treated water qualities were BOD 3 mg/L turbidity 2NTU and faecal coliforms 40 cfu/L. UV disinfection was added and eliminated the indicator bacteria.

1 INTRODUCTION

Research to improve the sustainability of water supply and urban runoff have been widely reported. The incentives for these changes namely economic growth, environmental impact, climate change and more stringent standards for actual drinking water are also well known. This paper reports on the most widely applied process combined rainwater and grey water reuse. The difference between this project and others producing near potable water (Hill et al 1998 for example) was that it used a combination of biological and deep bed filtration in a package plant.

2 SYSTEM DESIGN

Details of the evolution of the design have been previously published (Surendran and Wheatley 1998, Surendran 2001). The approach, based on traditional wastewater treatment, provided multiple barriers to potential pathogens and the removal of both dissolved and particulate contaminants. It is a four stage plant (Figure 1).

Stage one was a septic tank (1.4 m^3) for flow balancing and buffering shock loads from detergents and cleaners. Stage two was a submerged aerated bioreactor for removing soluble organic load (0.25 m^3), protected by a screening and roughing filter. The biological filter included two sizes of biological media coarse and fine. Problems with solids blockage in the bioreactor were anticipated and performance of a second moving bed bioreactor was also investigated. The results were not significantly different and the results omitted from this paper (see Surendran 2001 for details). The third stage was gravity filtration to remove turbidity generated by the bioreactor. This was designed as two parallel units (0.25 m^3) but with the head space on one set 20 mm above the other. This allowed for the cascading of a portion of the flow as head loss increased on the first filter. The media in the bioreactors and the gravity filters were replaceable reticulated foam cartridges. The fourth stage was a UV lamp on the pump riser from the final low level treated water tank (700 litres) to the roof

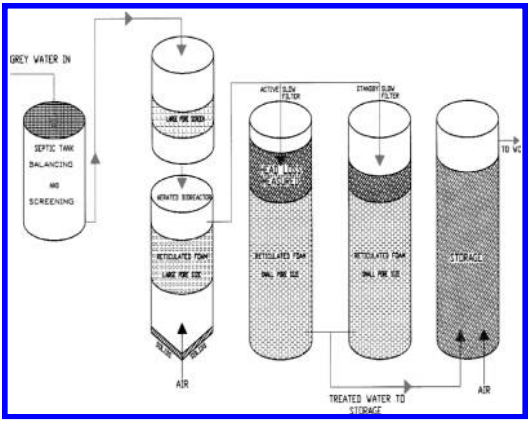


Figure 1. Schematic process diagram of the fixed bed grey water treatment plan.

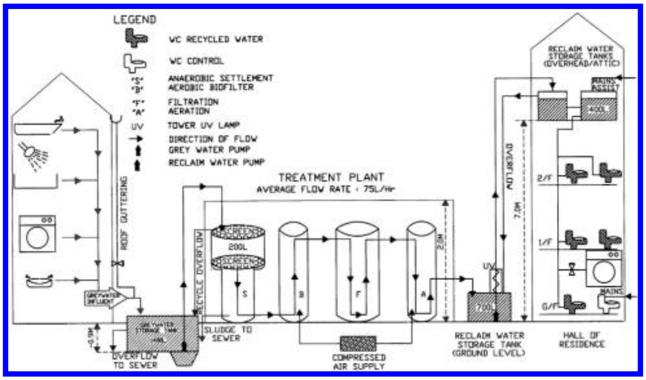


Figure 2. Schematic diagram of the royce recycling system.

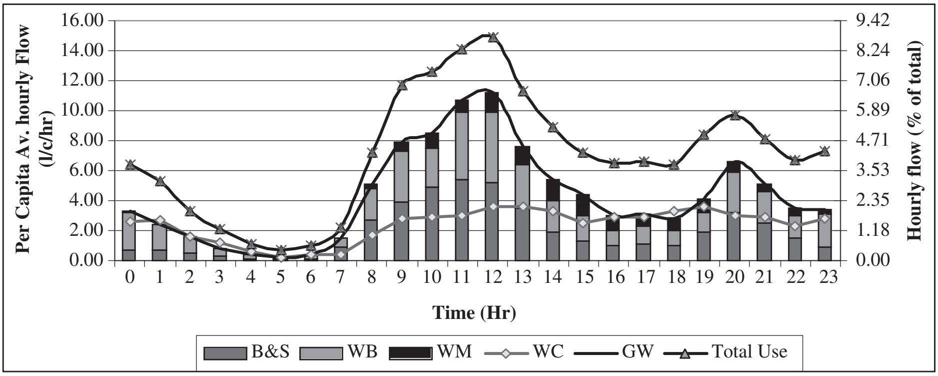
cistern (400 litres with mains standby WRAS 2000 (Figure 2). The UV lamp was added for the final year of the project (WEDCO Model 15/3P Sudbury).

The plant was made from four identical commercially available glass fibre tubes (0.45 m diameter × 2 m in height) bolted to a standard pallet (1.0 m × 2.0 m). Two pumps were required, one to lift grey water from the septic tank into the bioreactor and the second from the final low level tank through the UV lamp to the roof cistern. The pumps were operated by timer with overflow loops to allow the use of oversized pumps and to avoid solenoid valves and electrodes. The pumps operated five minutes every 30 minutes with integral low level overrides. Forward feed flow was controlled by restriction valves to a maximum of 3 L/minute the bioreactor was continuously aerated.

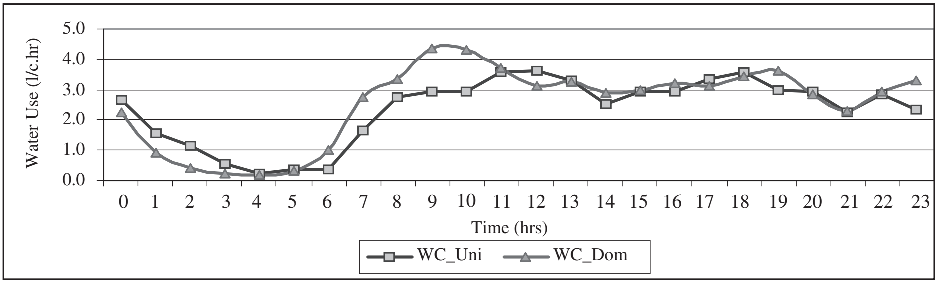
The hall was built in 1965 and all the drainage to the building was external. The block has 43 occupants on three floors, none of the rooms had separate plumbing. Each floor has a bathroom including two showers, three lavatories, six hand basins and a bath. The downstairs kitchen included a washing machine. The grey water was taken from the hand basins, showers, baths and washing machine. The kitchen and cleaners sinks were avoided.

3 RESULTS

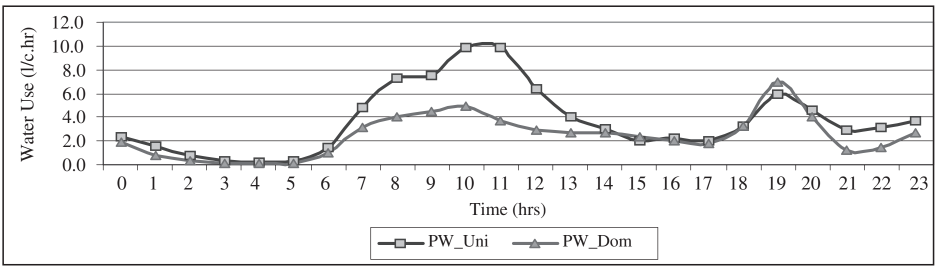
Water usage patterns were of interest to assess the peak demands and size buffering tanks. Averages of these are shown in Figure 3. Notable points were the similar patterns of grey water production and



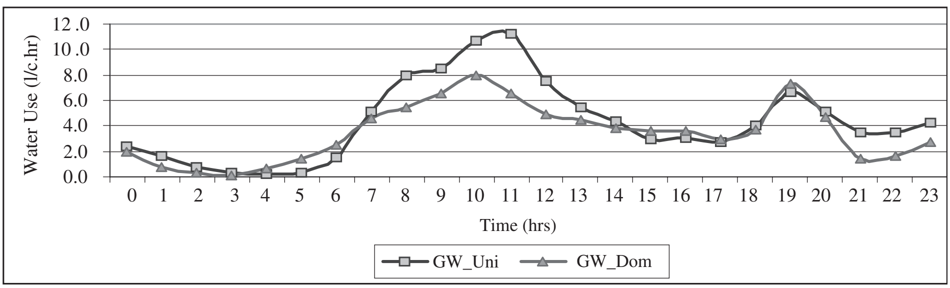
a) types of water use



b) comparison of university and domestic lavatory use



c) comparison of university and domestic bathroom flows



d) University and domestic grey water flows

Figure 3. Water usage.

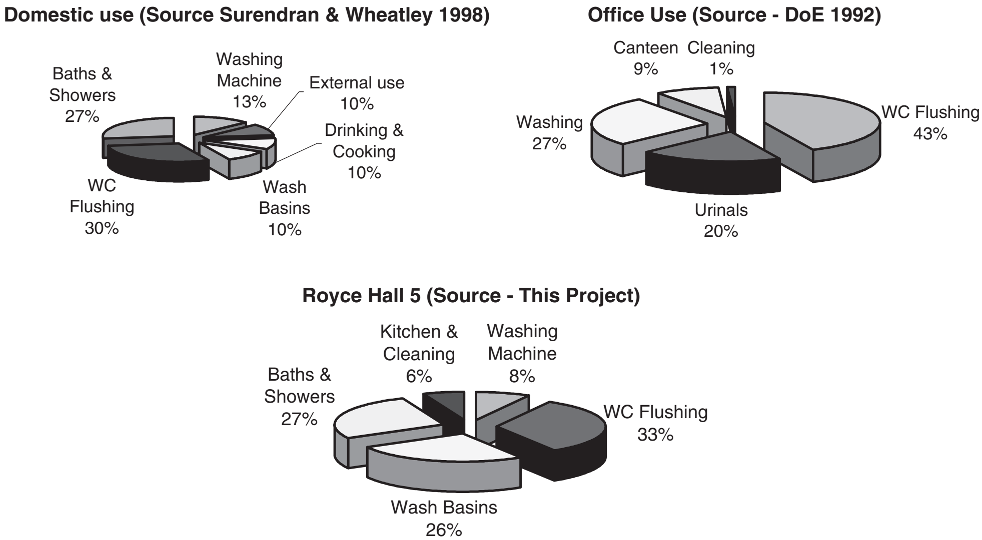


Figure 4. Comparison of water use in the domestic, office and university environments.

water use dominated by washing and showering which was greater than the ordinary domestic use (Figure 4). Average daily water consumption (144 L per head) was consequently greater than average domestic consumption. There is also a balance between lavatory flushing and grey water production in the afternoons and evenings (3.5 L/capita or 14 flushes an hour). Thus the buffering cistern was oversized (peak use to production ratio 2:1).

Analysis of coliforms, TOC, suspended solids and turbidity were carried out 2–3 times a week using standard methods (Eaton et al 1995) on the incoming grey water and final treated water. Interstage and additional analysis of volatile solids, conductivity, BOD and nutrients less frequently typically monthly. Average data is shown in (Table 1) and during out of term time (Table 2).

The typical water quality indicators are presented but the broader range of parameters can be found in the thesis (Surendran 2001). Removal of microbial indicators occurs at every stage in the process, but the flocculation in the bioreactors was the most important stage. The filtration stage reduced counts to less than 10 cfu per 100 mls. The septic tank anaerobically digests settleable solids, turbidity, suspended solids and organic carbon are all increased by the septic tank. Previous work (Dixon et al 2001) has also noted this. Dixon et al noted microbial growth on storage but at much shorter retention times and anoxic conditions. The septic tanks used here was the smallest commercially available. Conductivity was also increased showing solubilisation but BOD is reduced implying some biomass incorporation or gasification. No sludge was removed from the septic tank after five years use and there is also a proportionally greater increase in suspended solids than turbidity suggesting the formation of biomass rather than filterable particles. These results would benefit from further work to model mass balance and to design filter and membrane pore sizes. Both suspended solids and turbidity are removed in the bioreactor and filter.

The immediate risk from recycled water is from pathogens. The thermo-tolerant coliform (formally faecal coliform) counts in the final treated water were lower than the "UK/EU bathing water standards" the "WHO" guidelines for irrigation and un-restricted recreational water use" and also the Florida, Texas and Arizona standards for WC flushing. The total coliform counts were also lower than the "UK and EU bathing water" guideline values but higher than BSRIA Hawaii, California and Japanese standards for reclaimed water use for WC flushing (Mustow et al 1997). Incorporation of the disinfection system would be needed to meet there guidelines consistently. The low turbidity, suspended solids and TOC results allowed the UV lamp to eliminate the microbial counts.

Table 1. Term time performance (means and standard deviations mg/L except where noted).

	Septic tank		Bioreactor			Final storage	
	Inlet	SD	Inlet	Outlet	Filter		SD
Thermotolerant (faecal) coliforms cfu/100 mls	1005	1026	620	67	7	4	5
Total coliforms cfu/100 mls	16,000					52	47
TOC	163	40	184	23	6.5	8.5	8.5
BOD	70		41	7	4	3	2
NTU	36	21	67	19	2	2	2.5
SS	41	30	108	33	3	2	3
Conductivity μS	550	107	750	740	730	700	132
NH_3N	1.6	0.9					

Table 2. Out of term performance (UV off).

	Septic tank	Final storage	Lavatory cistern	Control with mains cistern
Thermotolerant coliforms cfu/100 mls	511	15	17	2
TOC	16	4	4	3
NTU	13	2	2	2

The out of term results indicated microbial counts at the detection limits (Table 2), the standard deviations in the results are high.

Comparisons with other treated grey water results show similar limits to performance. Maeda et al (1977) for example published data on recycled conventionally treated and disinfected sewage, turbidity values were 1.7 NTU, BOD 5.5 mg/L and TOC 12.0 mg/L. Sayers (1998) reports on the Environment Agency study on performance of simple sieve filters followed by disinfection. Total coliform counts were 10^3 cfu/100 mls, suspended solids 25 mg/L and BOD $>$ 100 mg/L. Results are also better than the wetland approach, Marland (1999) used reed beds and other wetland systems, thermo tolerant coliforms (faecals) were 500 cfu/100 mls, suspended solids 10 mg/L and BOD 5 mg/L.

Membrane filtration on the other hand generates better water quality, Hill et al (2001) for example reports on the Dome project where microbial counts were 0, and suspended solids and TOC's less than 5 mg/L. Thus the results show near potable water quality standards can be achieved using the conventional package plant approach. More extensive long term comparative data are necessary to aid the design choice in different situations but this seems unlikely without a stronger planning incentive.

4 CONCLUSIONS AND RECOMMENDATIONS

- A package plant ($2\,m^2 \times 2\,m$) was used over a 5 year study period to reclaim grey and rainwater from a 40 population residential building for lavatory flushing.
- The design used traditional principles of balancing, bioreactor and deep bed filtration. Operation relied on pumps, compressed air, geotextiles and UV disinfection to generate a water quality which met all international standards for non potable reuse.
- Further work on life cycle and economic analysis at commercial scale are necessary to produce a guidance note on designs suitable to meet the international planning standards on dual plumbing.

REFERENCES

Department of the Environment. 1994. Bathing Waters (Classification) regulations 1991, *SI 1991/1597 (Water, England Wales), SI 1991/1609 (Public Health, Scotland). London: HMSO.*

Department of the Environment 1992. Using Water Wisely consultation paper. *London: HMSO.*

Dixon, A.M., Butler, D. & Fewkes, A. 1999. *Guidelines for grey water reuse health issue*. J. Chartered Institute of Water Environmental Management *13(5): 322–326.*

Eaton, A.D., Clesceri, L.S. & Greenberg, A.E. 1995. Standard Methods for the Examination of Water and Wastewater. *Washington: APHA AWWA & WEF.*

EU. 1995. European Council Directive concerning the quality of Bathing Water. *76/160 EEC 91/692/EEC and Council Decision 95/1. Brussels: European Commission.*

EPA/USAID. 1992. Guidelines for Water Reuse. *Manual EPA/625/R-92/004. Washington: Environmental Protection Agency.*

Hill, S., Smith, A., Hardy, P. & Birks, R. 2001. *Water Recycling at the Millennium Dome*. Water Science and Technology *43(10): 287–294.*

Maeda, M., Nakada, K., Kawamoto, K. & Ikeda, M. 1995. *Area-wide use of reclaimed water in Tokyo, Japan.* Water Science and Technology *33(10): 51–57*.

Marland, A. 1999. *Design & operation of natural treatment systems to produce re-use quality effluent for large scale applications:* Proceedings Water Recycling and Effluent Re-use, Gatwick 26–27 April 1999. *London IQPC Conferences, Copthorne,*

Mustow, S., Grey, R., Smerdon, T., Pinney, C. & Wagget, R. 1997. Water conservation – Implications of using recycled grey water and stored rainwater *report* 13034/1. *Bracknell: Building Services Research and Information Association.*

Sayers, D. (1998). A study of domestic grey water recycling interim report. *National water demand management center, Worthing: Environment Agency.*

Surendran, S. & Wheatley, A.D. 1998. *Grey water reclamation for non-potable re-use.* J Chartered Institute of Water Environmental Management *12(6): 406–413.*

Surendran, S. 2001. Development of In-house grey water and roof water reclamation system for large institutions during 1994 to 1998. *Loughborough University PhD Thesis.*

WHO/UNDP 1977. Developing human health-related chemical guidelines for reclaimed wastewater and sewage sludge applications in agriculture. *Geneva: World Health Organization.*

Water Regulations Advisory Scheme. 2000. Water Regulations Guide. *Newport: WR*

Chapter 8 – Water economics

Setting the right water tariff for targeted standards of service

E. Cabrera
Institute for Water Technology, Universidad Politécnica de Valencia, Spain

M. Dubois
Departamento de Ingeniería Hidráulica, Universidad Central de Venezuela

R. Cobacho, E. Cabrera Jr. & V. Espert
Institute for Water Technology, Universidad Politécnica de Valencia, Spain

ABSTRACT: As has been recently recognised by the European Water Framework Directive (WFD), an adequate urban water management demands a correct water-pricing policy. In particular, the article 9 of the WFD suggests a full cost recovery, mainly as far as urban and industrial uses are concerned. This is a request that, due to historical reasons and vested interests, most of the south European countries (like Spain) are far from being able to fulfil. Developed countries certainly find themselves in an even worse situation, and with complicated socio-economic circumstances, they find serious difficulties to even recover partially the costs. Politicians most frequently establish urban water pricing policies. Since they are usually more concerned about their short-term popularity than for the long-term technical results, urban water supply systems are poorly managed. The real fact is that water prices respond mainly to socio-economic and political factors than to any other rational criteria. However a sustainable water supply management requires the inverse approach: the standards to be fulfilled must be initially defined while the water tariff should be such that it would allow satisfying the established goals.

1 INTRODUCTION

The water tariff in a water undertaking influences the available economic resources, and is the most influential factor in its technical management in the medium–long term. The European Framework Directive acknowledges this fact (UE, 2000) and consequently specifies the principle of full cost recovery that should be taken into account no later than 2010. This will probably lead to a water policy change in many countries, especially in those that have been subsidising this natural resource (such as the Mediterranean countries).

Starting from the hypotheses that the quality of service in a water undertaking depends on the available economic resources (for given technical and professional capabilities) the relationship between the quality of service and the costs recovered through the water tariff seems evident. Generally speaking a higher tariff will result in a better quality of service. Consequently, once the targets for the standards of service have been established, so will the costs to be recovered and the average water tariff.

A good example of this may be found in what is bound to be one of the most costly issues in the near future in most water undertakings: the rehabilitation and renovation of the network. Nowadays in the literature (Skarda, 1996) this item is linked to the degree of environmental awareness and the water management culture of the country. In the most developed countries, the degree of rehabilitation/renovation reaches up to an annual 2% (in length), a value that decreases to even 0 in countries with limited resources and/or inefficient management. The target value for this concept should be determined taking into account the physical state of the network and the level of quality of service targeted. However, most of the existing models used to calculate the network renovation rate (Eisenbeis et al., 2002; Le Gauffre, 2003; Poinard, 2003; Saegrov, 2003) take into account the network's history (for instance the number of breaks per kilometre and year) while ignoring the quality of service to be delivered, and consequently the repercussion in the water tariff of the costs to be recovered.

In this paper, the principle of costs recovery will be used as the starting point, relating the standards of quality to be reached and the average tariff that will allow reaching them. Once the relationship standards-tariff is established (a relationship also dependant on the state of the system and its characteristics) it is possible to determine the increase in the tariffs needed to reach the targeted quality of service. The idea is consequently to proceed in the logical order, giving priority to technical criteria instead of giving it to the political ones that so often have conditioned the water tariffs. A transparent and rational procedure can be explained to the public opinion and consequently assumed by the responsible politician.

2 QUALITY STANDARDS

A complex service such as the water supply can consider many quality standards. External standards are clearly perceived by the user, either on an everyday basis, or in the case of extreme situations such as droughts. On the other hand, internal standards (of a more technical nature) are especially useful for the professionals in charge of the service. In any case, and regardless of the direct or indirect perception of the users, there alternative criteria to classify the quality standards. For instance, and attending to the institution setting them, in Europe up to 5 levels can be identified (Hirner, 2001). From the communitarian standards to the local ones, going through the national and regional levels and without forgetting a complementary level set by the standards promoted by the technical associations. In this sense, the role played in Germany by the DVGW is a clear example.

This paper will deal with external standards, which will be classified in five categories. In order of importance:

- *Quality standard*. The ultimate purpose of any water supply system is to offer drinking water. The water should be consumable at the user's tap, and this is the first condition to be considered. The regulation covering this point is the European Directive regarding water for human consumption 80/778/CEE, later modified by the 98/83/CE.
- *Quantity standard*. Water must be supplied 24 hours a day, 365 days a year. From a startpoint of having enough resources to satisfy the yearly demand, the treatment plants, the service reservoirs and the distribution network should be dimensioned accordingly in order to satisfy peak demands. This would imply avoiding the unhealthy user tanks or any sort of service interruptions.
- *Pressure standard*. Even at peak demands, a minimum level of service pressure must be maintained, while at low consumption times, it should be kept under a maximum value. The simplest rule is the German one (DVGW Planning Rule W 403). It stablishes a higher limit of 80 mH_2O and a lower limit of $15 + 3$ N mH_2O, being N the number of stories allowed by municipal ordinances in the area, with $N \leq 5$. In other words, the pressure must be right, nothing more, nothing less.
- *Leakage standard*. The distribution network should be able to deliver the users most of the water injected in the system. Consequently the level of real losses (or leakage) should be very low. It is convenient to remind that leakage is closely linked to the three previous standards. Trying to deliver a continuous supply of water in low efficiency networks would demand a larger dimensioning of the system. Additionally, a leaking network may also have quality problems due to pathogen intrusions that may appear in low pressure conditions (AWWARF, 2001). This situation also often leads to service interruptions.
- *Metering standard*. All consumptions by users must be metered. Even though in some developed countries metering is not compulsory (the United Kingdom being the best known case),

from a technical point of view there is no doubt about the convenience of universal metering. It is equally important to outline the necessity of accurate metering, since the accumulated error for the whole network considerable depending on the quality, class and age of the meter.

These two standards are especially perceived by the user in drought periods, for they may even lead the utility to service interruptions when their levels are not adequate. In the case of a need to ration water, first leakage should be reduced to the minimum and metering should be done as intensely and accurately as possible, all before to proceed with service interruptions that might affect the quality of water (Lund and Reed, 1995). Only through these actions (AWWA, 1992) the system may efficiently be managed, saving and storing the necessary water to guarantee the demand during drought periods.

In order to better explain the last two standards, it is convenient to remind the existence of urban utilities that provide (with the existence of user tanks or not) a continuous supply of water in normal conditions, and consequently fulfil the first three standards. However, during drought periods, in which the consumption must be reduced, the high levels of leakage and poor metering lead to service interruptions. That was the case in the region of Andalucia (Southern Spain) during the 1991–95 drought (Cabrera and Garcia-Serra, 1997). Up to ten million inhabitants suffered daily service interruptions of over 10 hours, because leakage in the networks exceeded 30% and it was necessary to save water. Additionally, this practice leads to a number of breaks and bursts in the network up to ten times higher than those generated in normal conditions (Lambert et al., 1997).

To conclude this point, it is important to mention that other external standards, like those regarding the service to users, can also be established. However, and since their influence on the water tariff is not as direct, they will not be considered for this paper.

3 EVALUATING THE UTILITY'S CURRENT STATUS

Every utility reaches with time a certain equilibrium that depends on the existing water tariff and the capabilities of the technical staff in charge. The equilibrium point can be expressed by means of the levels reached for each of the standards of quality mentioned:

- *Quality standard*. Either a numerical value or a descriptive assessment can be assigned taking into account the local drinking water regulations (in Europe the directive 98/83/CE). If the regulations are not satisfied in a satisfactory manner, the reasons need to be identified, and the investments needed to correct the situation estimated.
- *Quantity standard*. The daily average of hours of service interruptions is a good indicator. In many developing countries this should be the first item to be considered. If there is continuous service without interruptions, it will be necessary to determine if this is possible due to user tanks. In the case of insufficiency, and like in the previous case, it will be necessary to determine the upgrades needed in the system to overcome it, and the necessary investment to deliver continuous service without the aid of intermediate storage.
- *Pressure standard*. Evaluating the pressure standard does not make any sense unless the requirements of quantity have been fulfilled. In such case, it is necessary to evaluate the pressure levels in the network by means of an extended period simulation (Rossman, 2000) verifying that the values lie within the established minimum and maximum values. In the event of pressure being inadequate, it will be necessary to determine the necessary changes (redimensioning the system, or operational changes) to correct the situation, and evaluate the corresponding necessary investments.
- *Leakage standard*. A typical indicator for leakage is the efficiency of the network η_n, resulting from the ratio between the flow rate delivered to users[1] Q_s and the metered flow rate. A necessary

[1] Throughout this paper the supplied flow rate Q_s will represent the total flow rate consumed by users, regardless of whether it is metered or not. In accordance with the balance that will be presented later, this flow rate is equal to the sum of the registered and uncontrolled consumed flow rates ($Q_s = Q_r + Q_{uc}$)

remark needs to be made with respect to using the volumetric efficiency as a leakage indicator. This indicator is not appropriate for any kind of comparison, and consequently the target levels will probably be different for different systems[2]. Should the leakage level be below the established target, the necessary investment may be calculated by means of the management and maintenance curve that will be presented later.

- *Metering standard*. It can be established by using the ratio between the registered flow rate Q_r and the supplied flow rate Q_s. It is also important to determine which percentage of the metered flow rate is billed, since it will affect the final value of the tariff, something that has been stressed in the balance proposed by IWA (IWA, 2000).

In order to precisely define the previous flow rates, a water balance is presented next. This balance will also be useful to determine the costs to be recovered through the tariff based on the billed water. It is also convenient to point out that volumes and flow rates will be addressed indistinctively, although volumes will always be referred to a certain period of time, and hence could be considered as average flow rates.

4 WATER BALANCE

The proposed water balance (Almandoz et al., 2003) is very similar to the IWA one (IWA, 2000) although it is based on the simple principle that all volumes need to be metered. The intervening terms and their relationships are presented in Table 1.

As mentioned before, the balance considers universal metering as a starting point. In such case, and in the absence of illegal connections, the apparent losses correspond in fact to the aggregated error of all meters in the utility. These apparent losses Q_{uc} when added to the registered flor rate Q_r provide the supplied flow rate Q_s, which is the flow rate delivered to the users. Additional conditions make the audit more complex (Almandoz, 2003).

Once the previous flow rates are obtained from the audit, it is possible to obtain the values for the indicators corresponding to the leakage and metering standards. These are the network efficiency η_n and the metering efficiency η_m, defined as the percentage of metered water with respect to the total delivered flow rate to the users:

$$\eta_n = \frac{Q_s}{Q} \tag{1}$$

$$\eta_m = \frac{Q_r}{Q_s} \tag{2}$$

Table 1.

Injected system's flow rate, Q (metered by meters located on the injection point.)	Registered flow rate, Q_r (metered by domestic meters)		Customer's supplied flow rate, Q_s (also called delivered flow rate, it is compounded by Q_r and Q_{uc})
	Uncontrolled flow rate, Q_u	Uncontrolled consumed flow rate, Q_{uc} (apparent losses)	
		Uncontrolled leakage flow rate Q_{ul} (real losses)	

[2] As recommended by IWA, a more appropriate way to express leakage would be losses/service connection/time, or losses/(length of network)/time. The target efficiency may be established by setting a common target with these indicatros and translating them into efficiency values of every system.

This would conclude the evaluation of the status of the utility with regards to the selected standards. The obtained indicators, along with the current water tariffs, represent the starting point for the determination of the new water tariffs needed to deliver the targeted standards of service.

5 COSTS STRUCTURE

In order to evaluate the economic influence of the variations of the preceding standards (both in the total costs in the system, as well as in the value of the associated uncontrolled water) it is necessary to establish a cost structure. Cobacho and Cabrera (2001) propose a specific structure, in which the different process variables are identified and related. A brief description of that structure follows:

5.1 *Fixed Costs (C_F)*

These are system costs that are basically not dependant on the volume "V" of produced and injected water. They may be divided in the following categories:

- *Fixed administration and personnel costs*, C_{Fp}: Costs related with the staff and the administration structure.
- *Fixed capital costs*, C_{Fc}: These include the costs associated with the system's infrastructure.
- *Fixed meters maintenance costs,* C_{Fmm}: Costs derived from the necessary maintenance of all meters in the system, which influence the uncontrolled volume due to metering errors. The relationships developed in Arregui (1999) will be used to include these costs.
- *Fixed network operation and maintenance costs*, C_{Fmn}: These are costs generated by operation and maintenance activities in the network (in a wide sense they include repairs, active leakage control, pressure management and infrastructure). The costs are an exponential function of the volume lost through leaks V_{UL}, a function that depends on the actual characteristics of each utility.

All of the above leads to:

$$C_F = C_F(V_{UL}) = C_{Fp} + C_{Fc} + C_{Fmm}(V_{UC}) + C_{Fmn}(V_{UL}) = k_1 + C_{Fmm}(V_{UC}) + C_{Fmn}(V_{UL}) \tag{3}$$

where, C_{Fp}, C_{Fc}, and C_{Fmm} are considered constants for being independent of V_{UL} , and will be from now onwards referred as the constant $k_1 + C_{Fmm}(V_{UC})$.

It is important to highlight the fact that the term $C_{Fmm}(V_{UC})$ has been considered constant because on one hand, the maintenance costs of the network and the uncontrolled volume are much larger than those related with meters. On the other hand, it has been assumed that the management of the domestic meters will be such that through replacement policies, the maintenance costs of meters and the associated uncontrolled volume will remain constant.

5.2 *Variable costs (C_V)*

These costs are directly dependant on the produced volume and injected volume "V". For this same reason, they can be expressed as a function of a unit cost c_u and V, as expression 4 shows.

$$C_V = C_V(V_{UL}) = c_u\,V = c_u\,V_R + c_u\,V_{UC} + c_u\,V_{UL} = k_2 + c_u\,V_{UL} \tag{4}$$

where $c_u\,V_R$, $c_u\,V_{UC}$ have been considered constants, since V_{UC} is constant and very small compared to V_{UL} and both terms are independent of V_{UL}. As a result, their sum will be referred from here onwards as the constant k_2.

It is also convenient to point out, that the variable cost per m^3 produced may be broken into two elements from the management point of view:

- *Internal unit costs*, c_{ui}. It is the one resulting from the processes developed by the water distribution company. They basically include electricity and the chemicals used for production.
- *External unit costs*, c_{ue}. These are the costs imposed to the distribution company from the administration. It usually takes the form of a tax per m^3 produced and the money is usually employed in the amortization of the bulk supply infrastructure. It somehow also represents the political cost of water, and in the future could include its ecological value or some form of the opportunity costs.

5.3 *Total cost to recover (C_{TR})*

The sum of all fixed and variable costs will result in the total cost to recover, C_{TR}. This amount will have to be compensated from the income derived from an adequate billing of the consumed water.

$$C_{TR} = C_F + C_V \tag{5}$$

It is then quite simple to graphically represent the Curve of total costs to recover (CCTR) as a function of the lost uncontrolled water, and it can be done by adding the graphs of the fixed and variable costs mentioned before (Figure 1).

5.4 *Water tariff*

Independently of the point in which the system is operating, it is convenient to point out how to calculate the final price p_v to be billed to the users. Following the full cost recovery principle, it can be summarized in expression 6:

$$C_{TR}(V_{UL}) = p_v\, V_R \tag{6}$$

where V_R is the registered volume and p_v is the average tariff. This average tariff will not be directly applied to the user, but it is one of the main variables for the tariff structure to be developed later. Additionally and developing the expression obtained for it (expression 7) it is possible

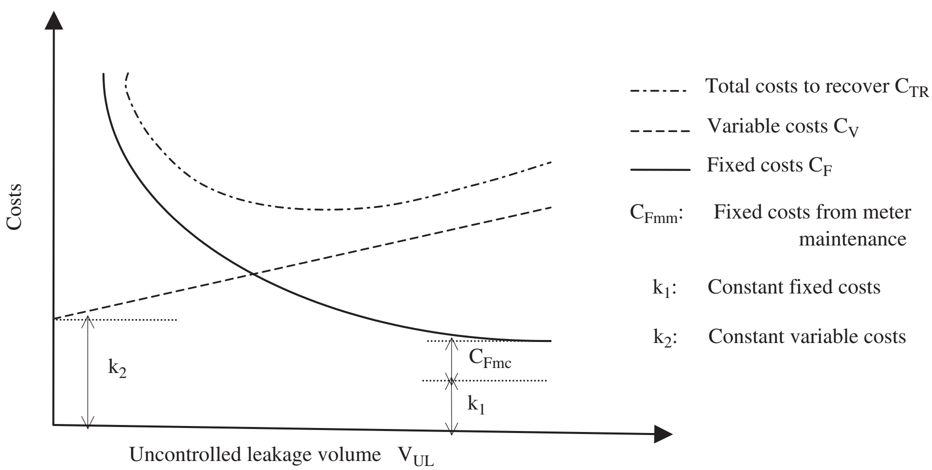


Figure 1. Curve of total costs to recover CCTR.

to determine how the system's management affects the final price of water through the existing uncontrolled lost water.

$$p_v = \frac{C_{TR}}{V_R} = \frac{c_u(V_R + V_{UC} + V_{UL}) + C_{Fp} + C_{Fc} + C_{Fmm}(V_{UC}) + C_{Fmn}(V_{UL})}{V_R} \tag{7}$$

It is convenient to point out that the registered volume as considered in this paper cannot be completely billed and only part of it will constitute revenue water (V_B), while most usually public uses will end up as non revenue water. However, and as it has been shown, the C_{TR} are only charged to those users that are billed, and consequently, the larger the non revenue water portion is, the more unbalanced the situation will be. The following expressions show this fact.

$$C_{TR}(V_{UL}) = p_v' \, V_B \tag{8}$$

$$p_v' = \frac{C_{TR}}{V_B} = \frac{c_u(V_R + V_{IC} + V_{UL}) + C_{Fp} + C_{Fc} + C_{Fmm}(V_{IC}) + C_{Fmn}(V_{UL})}{V_B} \tag{9}$$

Later, by interpreting expressions (7) and (9) it is possible to deduce that the price of water would be increased if the V_R is substituted by the V_F. It would also be possible to determine the influence of the non revenue water on the tariff.

6 INFLUENCE OF THE QUALITY OF SERVICE ON THE WATER TARIFF

Regardless of the operation point of the network[3], it will always be possible to determine if it is operating within the quality standards specified. Additionally, it will be interesting to determine how sensitive are both the uncontrolled volume and the projected tariff to the different management alternatives that may be chosen in order to improve the quality of service and the fulfilment of the standards. This analysis may be done by means of the cost structure described above, and the total costs to recover (since the projected water tariff is just the consequence of dividing this figure by the revenue water). Additionally, and in order to simplify the presented figures, the constants k_1 and k_2 have been included. It is important to mention that although some management actions may contribute to improvements in all the standards categories, the influences here presented will be independent for clarity purposes.

- *Quality of water*. Although providing a proper quality of the water at the consumption point is already a demanding standard, this quality may always be subject to improvement (or for instance the effects of potential contamination may be reduced) which would lead to an increase in the costs. For instance, if the chemicals used in a treatment plant are changed to improve the water quality, or the equipment in the plant is changed for a more efficient one, the increase in costs will increase the slope of C_V, and the value of k_2 displacing C_V upwards. This will also result in an increase on C_{TR}, maintaining a constant V_{UL}. Figure 2a shows this case, where the total costs to recover raise from point (1) to point (2), maintaining V_{UL} constant. If on the other hand, the actions were aimed to reduced leakage in order to avoid pathogen intrusions through a better network maintenance, the C_{TR} will be increased with a reduction of the real losses. This situation is shown in Figure 2b where point (3) is displaced to point (4).
- *Quantity of water*. The impossibility to deliver the amount of water demanded by users in a network may be due to several reasons. Depending on the actions taken to correct this situation, there will be a different repercussion on the costs and the water tariff. If the problem occurs in

[3] As a function of the total costs to recover and the lost uncontrolled volume.

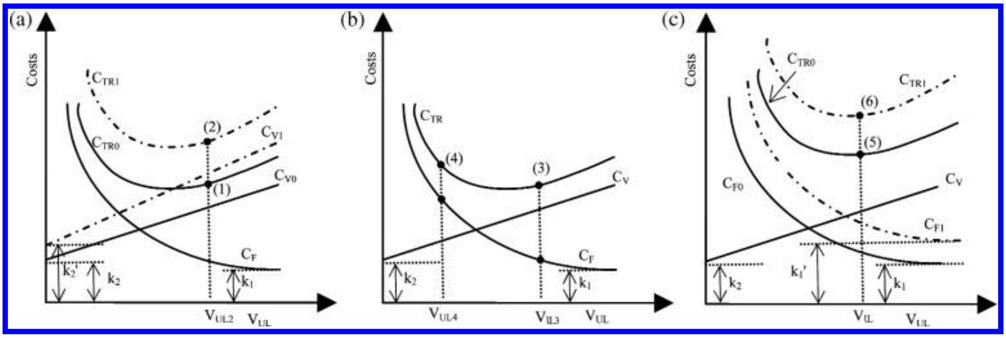


Figure 2.

a well maintained network (where demand simply exceeds the delivered water) it may be necessary to incorporate new supply sources, increasing the production costs and the slope of c_u as well as a upwards displacement of constant k_2, as shown in Figure 2a where the initial point (1) is moved to point (2), increasing C_{TR} and maintaining the initial V_{UL}.

In networks with a high level of leakage, in order to increase the supply it would be necessary to invest in network maintenance, with the consequent increase in C_{TR} and a reduction in V_{UL}, displacing the point in the C_{TR} curve from (3) to (4) – see figure 2b.

If the problems were due to the hydraulic insufficiency of the network (as a result for instance of the increase in friction in old pipes) the situation would be identical to the one described in 2b. However, if the problem was due to an increase in demand resulting of additional users in the network, the solution would probably lie in an increase of the capital costs to adequate the infrastructure, increasing constant k_1, displacing the maintenance curve upwards as shown in Figure 2c, and increasing C_{TR} from (5) to (6), with V_{UL} remaining the same.

- *Pressure*. There are two main problems that may arise regarding pressure in the network, to have insufficient pressure or an excess of it.

  In the case of low pressures, there are several possible causes. If the pressure head at the admittance point was insufficient, it would be necessary to increase the pumping or the reservoirs head. This would mean increasing the capital costs, and consequently C_{TR}. This situation is also shown in Figure 2c, where k_1 is displaced to k_1', maintaining the shape of the maintenance curve of the network.

  On the other hand, the low pressure could be due to an increase in the pipe friction, with a need for rehabilitation or replacement. This could increase the maintenance costs, and consequently C_{TR}, with a decrease in V_{UL} due to a lower level of leakage. This situation would have a similar representation to the one shown in figure 2b.

  In cases of high pressure (an excessive level of pressure may favour leakage and cause problems in accessory elements) it would be necessary to regulate it, either through an improvement in operation of the system, or incorporating the necessary equipment. This would mean an increase in C_{Fmn} and C_{TR} while V_{UL} decreases. This situation is also similar to the one shown in Figure 2b, there in order to improve the network maintenance, the operation point would move upwards and to the left.

- Leakage. The leakage management basically influences two aspects: the volume of water lost and a potential pathogen intrusion that would also affect the quality of water. The only way to minimise both aspects is through an effective network maintenance. If the network maintenance leads to a certain volume lost through leaks, it generates a C_{TR} as shown in point (3) in Figure 2b. In the case of a need to reduce the leakage volume and consequently reduce the risks of contaminating the water, an improvement in the maintenance will be needed, displacing point (3) to point (4) with the consequent increase in C_{TR} and the water tariff.

  It is necessary to mention, that according to the network's characteristics, the maintenance curve will be different for every network and either displaced to the right or left, but its shape will be similar in appearance to the one shown in any of the three graphs in Figure 2 (a, b and c).

- *Metering.* The importance of metering cannot be stressed enough, for all the four aspects of the quality of service mentioned before will only reflect the true situation of the utility if metering is accurate (with regards to C_{TR} and V_{UL}). Furthermore, readings in domestic meters are probably the most influential measurement in the value of C_{TR}. As mentioned before, this could be represented as a fixed maintenance cost that when increased to improve the metering, will displace the curve of maintenance costs upwards, increasing the C_{TR}, but maintaining constant the V_{UL}, as represented in Figure 2c. It is also important to mention that the consequences on the water tariff for an improved metering are not direct. The C_{TR} increases due to the meter maintenance costs, but on the other hand improved metering reduces the apparent losses (which also need to be recovered). Additionally, the average water tariff will be calculated with the revenue water which will also increase due to more accurate metering[4]. In any case the aggregated influence of all these factors will vary from one utility to the other.

7 CONCLUSION

The relationship between the quality of the service and the water tariffs is hardly ever considered in the management of urban water utilities and these two issues are often dealt with separately.

Taking into account such situation, this paper has presented guidelines to qualitatively evaluate such relationship, as well as the factors that may influence it. The model presented pretends to offer a unique structure to consider all the different factors, and should represent a useful tool during the decision making processes that affect the utility's policies.

Some further comments are needed in reference to the water tariffs. The average price or tariff mentioned throughout the paper, is nothing but an initial reference for the target price of the m^3. This average value should represent the starting point in the development of a more complex tariff structure, depending on the particular circumstances of each utility – an aspect that lies outside of the scope of this paper, the water tariff has been presented exclusively as the means to recover the costs of the service. Other factors such as the promotion of a more efficient use of water through pricing or the need for benefits in private companies that may be managing the service, should also be taken into account depending on the circumstances.

Finally, it is important to point out the fact that this paper is just a first step in a larger task to be fulfilled. The authors intend in future contributions to develop and quantify the basic guidelines and main relationships presented here.

ACKNOWLEDGEMENTS

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REFERENCES

Alegre, H., Hirner, W., Baptista, J. M., Parena, R. (2000). Performance Indicators for Water Supply Services. *Manual of de Best Practice Series.* London.IWA Publishing.

Almandoz, J., Cabrera, E., Arregui, F., Cabrera, E. Jr., Cobacho, R. (2003). Leakage assessment through water networks simulation**.** *Journal of Water Resources Planning and Management. ASCE.* Accepted to be Published during 2003.

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[4] Metering errors are in average always on the negative side, and consequently meters usually register a smaller volume than the consumed one.

Almandoz, J. (2003). *Directrices y Herramientas de Mejora del Rendimiento Hídrico en Redes de Aguas Urbanas.* Tesis Doctoral. Departamento de Ingeniería Hidráulica y Medio Ambiente. Universidad Politécnica de Valencia. España.

Arregui de la Cruz, Francisco (1999). Propuesta de una Metodología para el Análisis y Gestión del Parque de Contadores de Agua en un Abastecimiento. Tesis Doctoral. Universidad Politécnica de Valencia. España.

AWWA (1992). Drought Management Planning.USA. AWWA.

AWWARF (2001). *Pathogen Intrusion into the Distribution System*.USA. AWWA Research Foudation and AWWA.

Cabrera, E., Garcia, Serra, J. (1997). *Problemática de los Abastecimientos Urbanos. Necesidad de su Modernización*. España. Unidad Docente Mecánica de Fluidos. Universidad Politécnica de Valencia

Cobacho, R., Cabrera, E. (2001).Valoración del Agua Incontrolada (I). Modelación Básica del Rendimiento Óptimo de la Red. Mejora del Rendimiento Hidráulico de las Redes de Distribución de Agua a través del Control de Fugas, Mantenimiento y Rehabilitación.España. Instituto Tecnológico del Agua. Universidad Politécnica de Valencia.

Eisenbeis, P., Le Gauffre, P., Saegrov, S. (2002). Water Infraestructure Management: An Overview of European Models and Databases.*Urban Water Supply Handbook.* USA. McGraw-Hill.

Hirner, W. (2001). Regulation of Urban Water Supply: Standards of Quality and Control.*International Seminar, Valencia, June 26–28, 2001: The Urban Water Supply Debate.*

IWA. Operations and Maintenance Specialist Group: Task Force on Water Losses. (2000). *Losses from water Supply Systems: Standard Terminology and Recommended performance measures.* London. IWA Publishing.

Lambert, A., Myers, S., Trow, S. (1997). *Managing Water Leakage*. *Economic and Technical Issues*.London. Financial Times Energy.

Le Gauffre, P. (2003). Decision Support for prioritisation of Water Networks Rehabilitation Projects-data Needs and Data Avialability. *Proc. PEDS 2003, Valencia (Spain) 22–25 April*. Rotterdam: Balkema.

Lund, J.R., Reed, R.U. (1995). Drought water rationing and transferable rations. *Journal of Water Resources Planning and Management. Nov/Dec.*

Poinard, D. (2003). Rehabilitation of Water Networks: Data Collection and Data adquisition at Compagnie Generale des Eaux-Lyon. *Proc. PEDS 2003, Valencia (Spain) 22–25 April*. Rotterdam: Balkema.

Rossman, L. (2000). Computer Models/ Epanet. *Water Distribution Systems Handbook*. USA. McGraw-Hill.

Saegrov, S. (2003). Computer Aided Rehabilitation of Water Networks. *Proc. PEDS 2003, Valencia (Spain) 22–25 April*. Rotterdam: Balkema.

Skarda, B.C. (1996). Diagnosis Methods and Performance Indicators for Rehabilitation Policies. A Swiss Point of View. *Water Supply IWSA. Vol 14.*

Unión Europea (2000). *Directiva 2000/60/CE del Parlamento Europeo y del Consejo de 23 de Octubre de 2000.* Diario Oficial de las Comunidades Europeas, 22/12/2000.

Definition and estimation of resource costs of an aquifer

F. Dehoux
Centre Entreprise Environnement, Université catholique de Louvain, Louvain-la-Neuve, Belgium

ABSTRACT: The European Water Framework Directive stipulates that Member States shall "take account of the principle of the recovery of the costs of water services, including environmental and *resource costs*". The purpose of an internalisation of the external costs is twofold. First, it allows water companies to make the necessary investments (i.e. being financially autonomous) and secondly, it enables a control of the impoverishment of water resources by a proper demand management.

An estimation of resource costs is necessary to a cost-benefits analysis of a water conservation policy, as the benefits will consist of a reduction of resource costs.

After introducing the reasons why estimating an aquifer's resource costs becomes important, we propose a definition of resource costs and compare two common ways for their estimation. We compare the advantages and drawbacks of both methods, talking about internalisation and linking the reasoning to the Polluter Pays Principle.

1 INTRODUCTION

1.1 *Water management and resource costs*

Kliot et al. (2001) point four characteristics that make water resources management very difficult, and which introduce perfectly our study: scarcity, maldistribution, sharing, and finally over-utilisation and misuse.

Water scarcity is becoming widespread in the developing world, mainly due to demographic growth and urbanisation (Kliot et al. 2001). Maldistribution is due to nature and climate (for instance, according to Matricon (2000), China receives 7% of the earth's precipitation, with 21% of its population). The sharing issue (Frey 1993) can be seen on different scales: farmers sharing a small river or an aquifer; countries sharing a river basin or an aquifer – according to Wolf et al. (1999), 45.3% of the land surface of the earth are covered by international river basins. The way to solve the sharing problem will depend on this scale (Yetim 2002). Finally, over-utilisation and misuse are the variables on the basis of which we will develop most of our study, together with scarcity.

Up to now, when a water resource like an aquifer becomes scarce, the authorities often look for a "supply-based solution", like treating that water to improve its quality. But the decision maker becomes more and more aware that a good water resources management should not only be supply-based, but also demand-based. It starts being widely accepted that implementations of supply management projects should be preceded by a careful cost-benefits analysis (CBA, Hanley et al. 2001), a Life Cycle Assessment (UNEP 1996), or any other environmental appraisal method (Tyteca 2002). In case of a CBA, a proper appraisal of the value associated to the resource is needed to value the "benefits" part of the analysis.

The purpose of this work will be to define resource costs (i.e. the costs that the quantitative depletion or the qualitative degeneration of a water resource causes to water uses) and propose a valuation methodology.

1.2 *European legislation and resource costs*

The objective of the European Water Framework Directive (Directive 2000/60/EC, hereafter WFD) is to reach a good water status by 2015.

It is the first European environmental Directive including explicitly some economic aspects. Particularly two articles introduce economics as a tool to reach the general objective. Article 5 indicates that "Each Member State shall ensure that for each river basin district[1] [...] an economic analysis of water use is undertaken [...]". Article 9 states that "Member States shall take account of the principle of the recovery of the costs of water services, including environmental and *resource costs*, having regard to the economic analysis [...], and in accordance in particular with the polluter pays principle."

The cost recovery requested by Article 9 implies several things: a calculation of the financial incomes of the water sector; an analysis of the water demand curve, or at least its price elasticity (to appraise the extent to which a price increase will reduce water consumption); an estimation of costs, including resource and environmental costs.

2 REVIEW OF LITERATURE

While water supply management has always been mainly the field of engineers (Matricon 2000; Cornut 2003), economists since a couple of decades seek to make this supply more efficient and to introduce a water demand management (EEA, 2001), for instance with an appropriate pricing system (Green 2003; Dinar 2000; Davy & Strosser 2002; Rákosi 2000). Water demand management has been poorly studied for such a long time because water is a low unit cost product (Green 2003). But nowadays this demand management and prediction are getting important: several expensive projects have been built to match a predicted growth in demand, but this latter did not often occur (USACE 1995; Acreman et al. 1999; Bowers 1983; Reisner 1993).

As a major health problem in a large amount of countries all over the world, water scarcity has been the subject of significant research. This problem is very often linked to food problems (Yang & Zehnder 2002). But water scarcity has also been studied in Western countries (Davis 2001). They concern either global resources for a region or specific resources, like groundwater (Shiklomanov 1998; EEA 2000). For instance, according to IWMI (2000), in 2025, 78% of the world's population will live in water scarcity. The US Congress (1993) states that in many parts of the world, the level of contained aquifers is falling because of over-abstraction. But on the other hand, it seems that economic analyses of water scarcity have not been often carried out.

We have seen that economic aspects of resource scarcity are sometimes studied, as well as economic aspects of environmental issues; however, the links between resource scarcity, environmental issues and economics often seem to be forgotten; it is however studied in the field of irrigation, i.e. the loss of crops due to the impoverishment of a resource (Terrell et al. 2002).

It seems that resource costs have been more or less forgotten by scientific research (WATECO 2002). This is partly because the demand for such a research has been, up to now, almost non-existent, especially in the water sector. As long as there was little demand management, estimating resource costs made no sense.

Now that demand management starts becoming widely accepted, for instance within the European Union, via the WFD, and pricing and cost-recovery (reflecting, at least partly, the polluter pays principle and the user pays principle) are recognised as instruments for this management, a proper estimation of resource costs has become important, to decide how pricing should be implemented to meet the objective of perpetuating the resource.

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[1]WFD defines it as follows: "'River basin district' means the area of land and sea, made up of one or more neighbouring river basins together with their associated groundwaters and coastal waters, which is identified under Article 3(1) as the main unit for management of river basins." (Article 2 of WFD). The main purpose of those districts is managerial: a management plan must be produced by river basin district.

3 CATEGORISATION OF COSTS

A common way of categorising costs of water services is the one proposed by WATECO (2002), and already suggested by the WFD, which consists of three main categories: financial (or direct) costs, resource costs and environmental costs. Even if this categorisation is not explicitly justified in the WFD (those categories of costs are treated at an equivalent level: they all have to be recovered), we will make it for a practical reason: the methods we will use and the problems we will encounter are noticeably different for each category. We will define them as precisely as possible to ensure we do not make any double-counting of some costs and we do not forget any cost. Let us analyse the three types.

Financial costs. Water uses often require an infrastructure (for abstraction, storage, treatment, distribution, etc.). This category will consist of:

- investment costs realised in order to enable the distribution of water of a satisfying quality, in a satisfying quantity, to all consumers, and to enable its disposal after use in the natural environment;
- operating and maintenance costs of this equipment (for instance, the wage of the workforce);
- administrative costs;
- opportunity costs of capital invested, i.e. the best return the investors could have had by investing in another project.

Resource costs. This category will be defined exhaustively later, but we have seen *supra* that this concept means the costs that the quantitative depletion or the qualitative degeneration of a water resource causes to water uses.

Environmental costs. They are defined by the European Commission (COM (2000) 477 final) as "the costs of damage that water uses impose on the environment and ecosystems and those who use the environment (e.g. a reduction in ecological quality of ecosystems or the salinisation and degradation of productive soils)."

4 EXTERNALITIES AND INTERNALISATION

Resource costs and environmental costs represent externalities: they are a result of water uses, but the ones bearing those costs are mostly not the users who caused them. Those externalities can be either positive or negative. As an example of positive externalities, one could think of return flows: a farmer irrigating his fields with surface water in quantities such that a part of it percolates into an aquifer which is over-exploited induces a positive externality on the users or the aquifer (Briscoe 1997). It will lead to a negative resource cost. This positive externality could be coupled with a negative externality, if that percolating water is polluted. Those externalities could nevertheless be non-existing in case the only user of this aquifer is the farmer.

The purpose of one of the uses of water – sanitation – is to get rid of waste (faecal, industrial etc…). That means that, if the user doesn't clean his water by himself, he automatically creates an externality.

As we saw before, Article 9 promotes the "polluter pays principle". It also requests an "adequate contribution of the different water uses […] to the recovery of the costs of water services" and states that Member States shall ensure "that water-pricing policies provide adequate incentives for users to use water resources efficiently, and thereby contribute to the environmental objectives of this Directive". Those three principles (cost-recovery, polluter-pays and incitative pricing) mean that externalities will have to be internalised (if not for each polluter or water user, at least for the three main sectors which are households, agriculture and industry).

5 DEFINITION

It is very hard to find a definition of resource costs in the scientific literature. We will thus work on the basis of the definition proposed by the European Commission (it is not a legal definition), i.e. "resource costs represent the costs of foregone opportunities which other uses suffer due to the depletion of the resource beyond its natural rate of recharge or recovery (e.g. linked to the over-abstraction of groundwater)."

First of all, we would like to point out that when we write "resource cost", the reader should keep in mind that we talk about the cost of the *impoverishment* of that resource. We start from the principle that all possible uses of a resource must be enabled, and that none of those uses must limit another water use.

The impoverishment can be quantitative (in the case of an over-abstraction of the resource) and/or qualitative (in the case of a pollution reducing the possibilities of some uses). The resource cost must reflect the fact that, beyond a certain natural recovery rate (from a quality point of view) or renewal rate (from a quantity point of view), some opportunities disappear for water users.

Note that the impoverishment of the resource will be valued by the loss of some uses, i.e. we will only take into account direct use values (Turner et al. 1993). These uses can be present or future (for instance if the next generation will not be able to satisfy all its uses).

Another remark is that we have to compare average figures. If, for instance, the renewal rate of an aquifer is such that it takes a month to rainwater to reach it, we do not have to worry about short over-abstraction periods. We will have to compare *mean* abstraction rates to the renewal rate, over a period depending on the renewal rate of the resource. In some parts of the world where there are two seasons per year – one dry and one rainy season – one could for instance imagine that there would be a different resource cost per season. The renewal rate of an aquifer will depend on a lot of factors: precipitation, geology, but also the depth of this aquifer, the presence and the type of vegetation growing on the surface, … (Clothier 2003).

Following the definition of resource costs proposed by the Commission, and the remarks made above, we will define this concept as follows: "Resource costs represent the costs of foregone opportunities which other (present and future) water uses suffer due to the qualitative and/or quantitative impoverishment of the resource beyond its average natural rate of quantitative recharge or qualitative recovery. It is a foregone direct use value."

6 VALUATION

First of all we will outline a general methodology based on the estimation of the demand function. We will talk briefly about the different possibilities for this estimation. We will then sketch the methodologies that are not based on the estimation of the demand curve. We will finally summarise the steps to follow to value resource costs.

We can already notice intuitively that resource costs depend on:

- the value attributed to this resource for the different water uses it can fulfil (represented by the demand curve);
- renewal and recovery rates of the resource;
- the existence of a substitution, i.e. another resource that is able to fulfil the same uses. We will talk about replacement possibilities, which is the same concept.

6.1 *Methods based on the demand curve*

In this section, we will do as if we knew the demand function for each water use. Many textbooks (Hanley et al. 2001) explain how to estimate this function.

We will now illustrate the issue of resource costs with two simplified examples, one illustrating quantity issues, and the other illustrating quality issues.

6.1.1 *Quantity issues*

Imagine a groundwater abstraction. Different categories of users value this water differently: it has an important value to households, but a lower value to farmers who use it for irrigation (the value per m^3 used is usually lower for irrigation than for many other uses; see for instance Rogers et al. 2002). If the piezometric level of this aquifer becomes too low to abstract water from it, resulting in a cut of water supply, the loss per m^3 would be more important for households than for farmers.

This is due to the fact that the best possible use for households (e.g. for cooking food) has for them a higher value than the value attributed to irrigation by farmers. Figure 1 shows an aggregate demand curve, D(p), representing the sum of the individual demands (in our example, the individual demand of farmers being lower than the one of households).

The supply curve represents the prices fixed by the water supplier (according to the neo-classical theory, this should be equal to the long-term marginal costs; however this might not always be the case, as water prices often depend on political decisions or legal constraints). If the renewal rate is greater than the total annual demand (represented by x), there is no resource cost. The equilibrium price will be equal to y. If the renewal rate is lower than x, there is over-abstraction; in this case we will call the equilibrium (x, y) an unsustainable equilibrium.

There are two possible ways of understanding the issue of resource costs. We will explain both and argue why we prefer the second. Anyway, none is perfect and they both have advantages and drawbacks.

The first is the one proposed by WATECO (2002), and consists in calculating the difference between the theoretical price[2] with scarcity and the real price. Let us assume that the renewal rate r, is smaller than the consumption. The difference $(y_r - y)$ is the resource cost per m^3 (see Figure 2).

The total resource costs (RC) will then be equal to the area A on Figure 2. The equation is:

$$RC = (y_r - y) \cdot r \tag{1}$$

This method has one big advantage: according to the neo-classical theory, once internalised, the resource costs will automatically lead us from the unsustainable equilibrium E_1 to the new equilibrium E2, which will be sustainable. As supply equals demand, this solution leads to an efficient allocation from the Pareto point of view – Monasinghe (1992) states that an efficient policy may

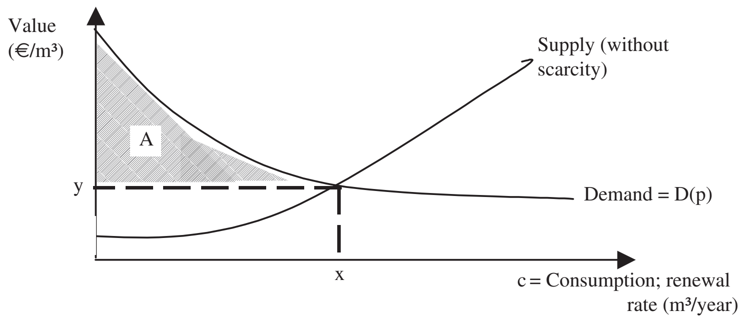


Figure 1. Consumer's surplus.

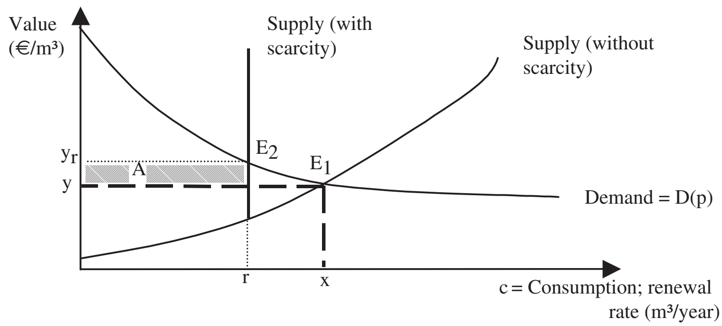


Figure 2. The effect of scarcity.

[2] With theoretical price we mean the price given by the intersection between supply and demand.

be defined as one, which maximises the net benefits accruing to a community from a given course of action, with no consideration paid to the way in which those benefits are distributed. It is also consistent with Article 9 of WFD which is in favour of a water-pricing policy providing "adequate incentives for users to use water resources efficiently". But it does not fit to the definition of resource costs as proposed by the Commission: the result of using this method is not the recovery of the foregone opportunities of water uses, but the shift towards a sustainable economic equilibrium. Another point is that it seems pointless to try to reach an equilibrium given that this point is very hard to estimate, and that water supply and demand depend on a lot of different factors (Briscoe 1996; Bettendroffer et al. 2002). Briscoe (1996) states that "most certainly those […] estimates can never, and should never, be used to make technocratic decisions on allocations and prices (as has sometimes been proposed)." Moreover, an important cut in consumption will not always be possible (Green 2003), as for certain uses, the elasticity is rather small and becomes even smaller when consumption is already low (Bettendroffer et al. 2002). Finally, as we will see later, this method will not be possible to put into practice for valuing resource costs due to a quality degradation, so that there would be inconsistencies with respect to the method implemented for estimating resource costs in the latter case.

This explains why we prefer the following approach, based on an estimation of the surplus. This concept is already old – it has been studied by Dupuit (1844) – and represents the difference between what the consumers are ready to pay and what they are really asked to pay. It is the monetary equivalent of the welfare they get from their consumption at a fixed price. To have an idea of the amount it can represent, we will give the results of a study made on the Alsacian aquifer in France (Willinger & Stenger 1994) on the basis of the contingent valuation method. They estimated the use value to 107 to 125 Euro/household per year (i.e. a total surplus of 57.5 million Euro per year, 80% of the Alsacians being users of this aquifer).

On Figure 1 above, the consumer's surplus is the hatched surface A.

Now, imagine the renewal rate is equal to r, which is smaller than the current consumption rate, as represented on Figure 3. E_1 is an unsustainable equilibrium. To reach long term balance between natural recharge and water consumption (we call it a sustainable equilibrium, like E_2 on Figure 3), the distributor will have to reduce his supply by $(x - r)$ units per year. This will lead to a loss of the consumer's surplus (represented by the wavy area "A–B" on Figure 3), which will be the resource cost of the aquifer. The mathematical formula of the resource costs will then be:

$$\boxed{RC = \left[\int_r^x D(p)dc - (x - r) \cdot y \right]} \qquad (2)$$

If one wishes to recover those costs, one could include them into the water price. This increase would be greater than RC/x (due to the price elasticity, the final consumption will be lower than x).

Nevertheless, this method assumes that the supplier or the authorities will implement some other complementary measures to cut abstractions.

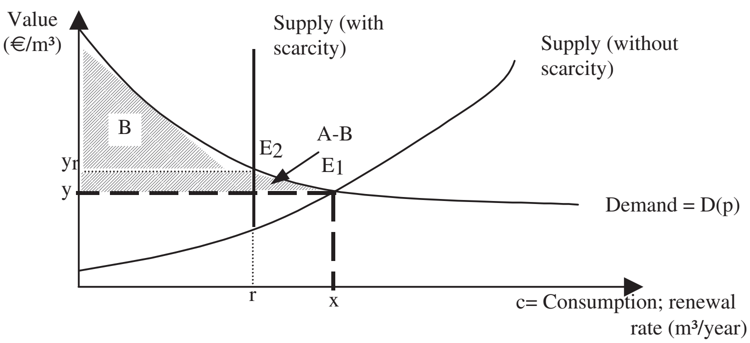


Figure 3. The loss of surplus.

A second important assumption we have made here is that the distributor cuts its supply where it is the less valued (i.e. it is efficient from an allocation point of view); in our example, water supply for irrigation will be reduced. We will see *infra* how it is when this assumption does not hold.

The way we described the calculation of resource costs is directly derived from the definition we have made of those costs. The solution proposed by WATECO (2002) is better than the other if demand management is the only tool to solve the resource problem. But we think this does not fit to the idea of the WFD, which suggests to use economics as a tool to reach the objectives, but not as the only tool. That means that if we combine other approaches, reaching the demand-supply equilibrium only by increasing prices is not an objective in itself.

From a practical point of view, those two methods do not seem possible to implement as such (at least not within the current socio-economic culture), because it would imply a different price for the users of each aquifer. A solution could be to add a "resource fee" depending on the resource from which water is abstracted, but it is not our purpose to discuss this here.

6.1.2 *Quality issues*

In case certain uses cannot be fulfilled because of a degradation of the *quality* of water, the issue is different. The supply curve will not shift, but some individual demand curves will move to the left, or even disappear (i.e. for the same price, users are willing to consume less of the resource). As a consequence, the aggregate demand will move to the left. That is the meaning of the illustration of the two Figures 4 and 5 hereafter. After a shift of the demand curve, the consumer's surplus changes from A to B. Resource costs will equal the difference between the two areas A and B. Note that if the distributor does not adapt its prices to the quantities, i.e. the supply curve is a horizontal straight line ($y = y'$), then the resource cost is equal to the wavy area C.

The resource cost will be:

$$RC = \left[\int_0^x D_1(p)dc - x \cdot y \right] - \left[\int_0^{x'} D_2(p)dc - x' \cdot y' \right] \tag{3}$$

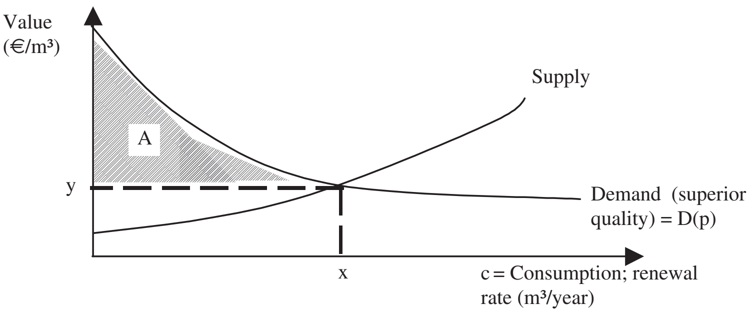


Figure 4. Water quality: initial situation.

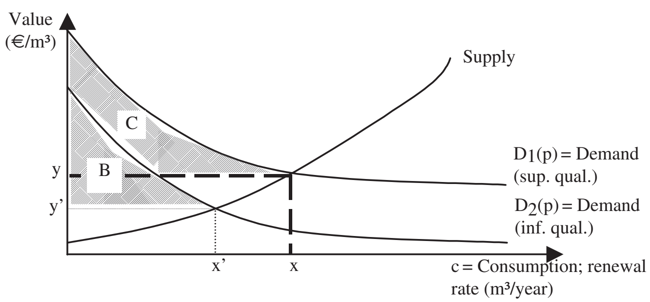


Figure 5. Water quality: situation after degradation.

Note that in the case of a resource cost due to a quality issue, it is not the consumer who should bear those costs via the cost-recovery mechanism, but the polluter (this is in fact the polluter pays principle).

Another important remark is that in fact it is not the quality on itself which influences the demand, but rather the *perception* of this quality by the user. This problem is tackled in Amigues et al. (1995). Amigues et al. (1995) reckon that the phenomenon is twofold: first, the quality of water is relative to the use which is made, and second, the perception of this quality depends on the user. Moreover, the link between water quality and its perception will depend mainly on a few parameters, like odours and transparency.

6.1.3 *Relations between quantity and quality issues*

It can happen that some quantity issues worsen quality issues. This could be the case for instance if the groundwater flows become so slow that bacteria's start developing. But the opposite is also possible: when a pollution is diluted after heavy rains, the qualitative status of water will get better! Green (2003) states for instance that an abstraction from an aquifer can lead to subsidence of the land above.

6.1.4 *Resource cost and allocation decisions*

A misallocation of scarce resources leads to resource costs. We think it would be interesting to the different actors of the water sector to know which part of the resource costs is due to water scarcity or bad quality, and which part is due to misallocation. This would be the case if a government allocates important quantities of water to an inefficient irrigation system, while cities face a lack of water. We will thus separate resource costs into two categories:

- costs of scarcity and bad quality of the resource;
- costs of misallocation of the resource (i.e. opportunity costs of that resource).

Note that a misallocation can only occur if there is a water scarcity or quality issue.

6.1.5 *Substitution possibility and resource cost*

Up to now we made the tacit assumption that there was no substitution to the resource, i.e. it is not possible for a user to find a replacement solution (taking water from another resource, treating it, etc.) at a lower cost than his WTP. We saw that the loss of surplus corresponds to the resource cost. But if a low cost replacement method exits, the estimate of its cost can serve as an upper bound or a lower bound to the resource costs. We will discuss this in the paragraph 6.2.1.

6.2 *Methodologies non based on the demand curve*

Turner et al. (1993) list another set of methods which are not based on the estimation of a demand function. Those are the dose-response method, the replacement costs method, the mitigation behaviour, and the opportunity costs.

We will analyse the replacement costs method, for a simple reason: it is the method that has commonly been used to assess (consciously or not) the resources costs.

6.2.1 *Replacement costs method*

This method has often been used in cost-benefit analyses to estimate the benefits of a project designed to compensate for a resource which is not able to fulfil all uses. When a resource becomes scarce, the method consists of calculating the costs of the cheapest possible measure to get some water of a satisfying quality for all uses from somewhere else, or another way (f.i. abstraction of water from a deeper water body). If resource costs are due to a quality issue, we can add some treatment techniques to those "replacement" solutions. Those methods are already used in countries where drinking water is becoming scarce, like in many Mediterranean countries.

This method has the advantage of being simple and well mastered (cost estimations are in general accurate). However, problems can arise, and the method must be used cautiously.

First of all, instead of being an appraisal of the WTP or WTA of the users of the resource, the result will often depend on public decision making, because the investments of such replacement projects are often huge. It is thus an appraisal of the WTP of the decider.

This method can lead to an underestimate of the resource costs, due to the transfer of part of the resource cost to another resource. Imagine for instance that the abstraction rate of aquifer *A* is higher than its renewal rate. A replacement project consists of importing water from an aquifer *B*. But if this causes the aquifer *B* to be overexploited, the project is not sustainable, and it leads to an underestimate of the resource costs of aquifer *A*. This happens because this replacement method aims at a redistribution of the resources of a region, or country, but if all the resources are structurally overexploited, as it can be in certain Mediterranean countries, this method does not work properly. Such a reasoning is quite simple in this over-simplified example, but will be harder to pursue in reality. This is one of the reasons why Amigues et al. (1995) consider replacement costs as a lower bound to those resource costs.

On the other hand, the method can also lead to an overestimate of the resource costs. If one specific use is restrained by the low renewal rate of the aquifer, but if it has a very small value to the user, it may not be worth realising huge investments in replacement projects. Note that if the decider makes a cost-benefits analysis of the replacement project, this should not happen: if the costs are higher than the benefits, one should not implement the project.

Another possibility, which is very similar to the replacement costs method, is to estimate the costs of protecting the resource (this is very often implemented to protect the quality of water from aquifers; when it is implemented we will consider it as a direct cost). The main difference consists of the moment of the considered solution. A protection method must be implemented before a problem occurs, while a replacement solution will be implemented after.

7 CONCLUSIONS: STEPS TO FOLLOW AT THE BASIN LEVEL

A first important point to emphasise is that resource costs (and environmental costs) must be estimated separately for each case. If there are many cases where a resource cost occurs, it will be hard to have a global idea of resource costs for a whole region or river basin district.

Moreover, the situation can evolve with time. Seasonal evolutions, demand evolution, climatic changes, land use planning are several factors which will influence resource costs.

The steps to be followed to appraise resource costs can be suggested as follows:

– identification of all the overexploited resources and of the polluted resources which are not able to serve for all uses in the long term (this step must be performed with specialists like hydrogeologists, who will assess the renewal rate of the resources);
– for each of these resources, evaluation of the resource costs *via* the methods outlined above, and linkage to the water uses that caused them (in accordance to the polluter pays principle);
– aggregation of all those costs at the river basin district level.

REFERENCES

Acreman, M., Barbier, E., Birley, M. et al. 1999. Managed Flood Releases from Reservoirs – a Review of Current Problems and Future Prospects. *Project R7344, report to DFID*. Wallingford: Institute of Hydrology.

Amigues, J.-P., Bonnieux, F., Le Goffe, P. & Point, P. 1995. *Valorisation des usages de l'eau*. Paris: Economica

Bettendroffer, A., Dehoux, F., Tyteca, D. 2002. PIRENE : Aspects économiques. *Deuxième rapport annuel, Université catholique de Louvain.*

Bouwes, N.W., Schneider, R. 1979. Procedures in Estimating Benefits of Water Quality Change. *American Journal of Agricultural Economics* 61: 535–539.

Bowers, J.K. 1983. Cost-benefit analysis of wetland drainage. *Environment and Planning* A(15): 227–235.

Briscoe, J. 1996. Water as an economic good: the idea and what it means in practice. *The World Bank, paper presented at the World Congress of the International Commission on Irrigation and Drainage.* Cairo.

Briscoe, J. 1997. Managing Water as an Economic Good. *Paper presented at the International Commission on Irrigation and Drainage, Water – An Economic Good?* Oxford, UK.

Clothier, B. 2003. The biggest water filter in the world: The soil-plant ecosystem. *Chaire Tractebel*. Université cathlique de Louvain, Belgium.

Commission of the European Communities 1997. Proposal for a COUNCIL DIRECTIVE establishing a framework for Community action in the field of water policy. *COM(97) 49 final.* Brussels.

Commission of the European Communities 2000. Communication from the Commission to the Council, the European Parliamant and the Economic and Social Committee. Pricing policies for enhancing the sustainability of water resources. *COM(2000) 477 final*. Brussels.

Cornut, P. 2003. *Histoires d'eau. Les enjeux de l'eau potable au XXIe siècle en Europe occidentale*. Eupen: Editions Luc Pire.

Davis, S.K. 2001. The politics of water scarcity in the Western states. *Social Science Journal* 38(4): 527–542.

Davy, T., Strosser, P. 2002. Tarification et gestion durable des ressources en eau. Quelques éléments de réflexion. In Eau et économie. *Congrès de la Société Hydrotechnique de France, 27èmes Journées de l'Hydraulique.* Paris.

Dinar, A. (ed.) 2000. *The Political Economy of Water Pricing Reforms*. Published for the World Bank, Oxford University Press.

Dupuit, J. 1844. De la mesure de l'utilité des travaux publics. *Annales des Ponts et Chaussées.* 2d series. vol. 8.

EEA (European Environment Agency) 2000. Groundwater quality and quantity in Europe. *Environmental assessment report* No 3.

EEA (European Environment Agency) 2001. Sustainable water use in Europe - Part 2: Demand management. *Environmental issue report* No 19.

Frey, F. 1993. The political context of conflict and cooperation over international river basins. *Water International* 18(1): 54–68.

Green, C. 2003. *Handbook of Water Eonomics. Principles and practice*. Chichester: John Wiley & Sons Ltd.

Hanley, N., Shogren, J.F., White, B. 2001. *Introduction to Environmental Economics.* Oxford: Oxford University Press.

IWMI (International Water Management Institute 2000. *World Water Supply and Demand 1995 to 2025.* Colombo.

Kliot, N., Shmueli, D., Shamir, U. 2001. Institutions for management of transboundary water resources: their nature, characteristics and shortcomings. *Water Policy* 3: 229–255.

Matricon, J. 2000. *Vive l'eau*. Evreux: Gallimard Sciences, n° 389.

Monasinghe, M. 1992. *Water supply and environmental management: Developing world applications studies in water policies and management.* Boulder: West View press.

Paré, S. 2003. Les conflits liés à la disponibilité de la ressource en eau. Cas du transfert de l'Ebre en Espagne. *Synthèse technique*. Montpellier: ENGREF.

Rákosi, J. 2000. The role of prices in irrigation in Central and Eastern Europe. In *Economic Instruments and Water Policies in Central and Eastern urope: Issues and Options.* Conference Proceedings. Szentendre.

Reisner, M. 1993. *Cadillac Desert: The American West and its disappearing Water.* New York: Viking.

Rogers, P., de Silva, R., Bhatia, R. 2002. Water is an economic good: How to use prices to promote equity, efficiency and sustainability. *Water Policy* 4: 1–17.

Shiklomanov, I.A. 1998. *World Water Resources: A New Appraisal and Assessment for the 21st Century*. Paris: UNESCO.

Terrell, B.L., Johnson, P.N., Segarra, E. 2002. Ogallala aquifer depletion: economic impact of the Texas high plains. *Water Policy* 4: 33–46.

Turner, R.K., Pearce, D., Bateman, I. 1993. *Environmental economics. An elementary introduction.* Baltimore:The John Hopkins University Press.

Tyteca, D. 2002. Business organisational response to environmental challenges: performance measurement and reporting. *Institut d'Administration et de Gestion, Working paper 50/02.*

UNEP 1996. Life Cycle Assessment: What is it and how to do it. *United Nations Environment Programme – Industry and Environment.*

US Congress, Office of Technology Assessment 1993. *Preparing for an Uncertain Climate*. Vol. 1, OTA-O-567, Washington DC: US Government Printing Office.

USACE (US Army Corps of Engineers) 1995. *National Study of Water Management During Drought: The Report to the US Congress.* IWR report 94-NDS-12. Fort Belvoir: Institute for Water Resources.

WATECO 2002. *Economics and the environment. The implementation challenge of the water framework directive. A guidance document.*

WFD 2000. Directive 2000/60/EC of the European Parliament and of the Council of 23 October 2000, establishing a framework for Community action in the field of water policy.

Willinger, M., Stenger, A. 1994. *Valeur de préservation de la qualité des eaux souterraines en alsace. Une estimation par la méthode d'évaluation contingente.* Strasbourg: Université Louis Pasteur.

Yang, H., Zehnder, A.J.B. 2002. Water scarcity and food import: A case study for southern Mediterranean countries. *World Development* 30(8): 1413–1430

Yetim, M. 2002. Governing International Common Pool Resources: the international watercourses of the Middle East. *Water Policy* 4: 305–321.

Trends in California water metering: a preliminary analysis

L.L. Dale & C.D. Whitehead
Lawrence Berkeley National Laboratory, Berkeley, California, USA

N.A. Williams
Loyola College, Baltimore, Maryland, USA

ABSTRACT: Despite growing water scarcity, many water districts in California sell water to customers on a flat rate basis at a time when many policy makers advise sales on a volumetric basis to conserve water. A transactions cost model clarifies the district choice of water sales practice. The model assumes that districts maximize net consumer benefits when choosing between a flat rate and a volumetric sales option. The model is used to explain the observed pattern of flat rate and volumetric sale practices in the Central Valley of California and to predict the increase in volumetric sales resulting from an increase in the average cost of water. A logistic regression analysis of this data indicates that local water cost explains much of the variation in water sale practices in the Central Valley, as predicted by the model.

1 INTRODUCTION

In California and other western states, increasing pressures are being placed on scarce water resources. Much of the impetus comes from growth in the urban demand for water – between 1990 and 2000, California's population grew 13.6% to 33.87 million (U.S. Bureau of the Census 2001). Although the majority of California's population lives in the coastal areas of Southern California and San Francisco Bay, there are significant urban centers in the Central Valley, including Fresno, Bakersfield and Stockton in the southern portion of the Valley and Sacramento in the northern portion. The urban population in the Central Valley is growing at twice the rate of other urban areas in the State, and there is strong interest to insure efficient urban water use in that region.

A peculiarity of the cities of the Central Valley is that most water services are not metered; that is, most urban water users pay a flat monthly fee for an unlimited supply of water. These residents share the city water supply much like Swiss cantons share grazing areas and their water system is essentially communal.

Communal and incompletely metered water systems in the Central Valley have become the focus of increasing State controversy in recent years as pressures upon limited State water supplies have grown. Those interested in water conservation attack the communal water system as inherently inefficient because it provides users little incentive to conserve water. Indeed, several studies have indicated that per-capita water use in cities with communal water systems is much higher than water use in cities with metered systems. In response to these pressures, the California State legislature has proposed bills to force State municipalities to meter individual service connections.

The adoption of water service meters, by some cities, and the communal water systems, by other cities, provides a unique opportunity to observe and measure the determinants of this type of institutional choice. This paper proposes a model of urban water district behavior, which explains the pattern and extent of incomplete water metering in Central Valley cities. The paper is divided into five sections. Following this introduction, section two briefly summarizes the relevant transactions cost and institutional choice literature. Section three presents a model, which predicts the extent of water metering in a water district and suggests a testable hypothesis of this model. An econometric analysis of data from urban Central Valley water districts is used to illustrate the model in section four. The concluding section five contains a discussion of the policy implications of the analysis.

The model presented in this paper assumes that the district choice for incomplete water metering is an efficient response to water metering transactions costs. To the degree this model is supported by the data on water metering in the Central Valley, caution is advised against regulations that impose water meters to improve water use efficiency.

2 LITERATURE REVIEW

Incomplete water metering may be characterized as non-standard market practice (most goods are sold on a volumetric rather than communal basis). Transactions cost economics has provided a useful perspective for explaining non-standard practices as means to economize on the transactions costs of the market (Coase 1937; Demsetz 1967; Williamson 1985). The existence of the firm, non-standard modes of organization, such as vertical integration, and non-standard sales methods, such as block booking and tie-in sales, have all been explained as measures to economize on transactions costs (Coase 1937; Williamson 1985; Kenny & Klein 1983).

Meters represent part of the cost of measuring the amount of water that is sold. This cost, termed measurement cost, represents one type of market transactions cost. Non-standard practices often evolve in cases where measurement costs are particularly high. For example, the high costs of measuring the individual contributions of members of teams may shape the organization of work and firms (Alchian & Demsetz 1972; Ouchi 1980). Similarly, large effort required to determine the value of complex goods offered for purchase may explain the existence of tie in and block booking. For example, Kenny and Klein have suggested that excessive measurement cost associated with diamond purchases may be avoided by the practice of block booking (Kenny & Klein 1983). In this paper, communal water use is explained as a practice to economize on water use measurement cost.

3 A TRANSACTIONS COST MODEL OF WATER SALES PRACTICES

Our model focuses on an urban water utility facing a decision whether to meter some given segment of its services population; for example, a project has been proposed to meter some specific geographic area or customer group, and the utility has to decide whether to proceed with the project. We model this as a discrete rather than a continuous choice; thus, the question is whether or not to meter rather than how much to meter. This captures the reality that many water utilities face – in practice, there are significant fixed costs to the utility in organizing and executing a metering program, and there are often geographical or logistic constraints that essentially fix the scale of the metering program. The Central Valley contains many residences within many urban water districts. Each district chooses the proportion of residential water service connections within the district to be fitted with meters, termed here the metering coverage (p_i). There are I districts, denoted by i, $i = 1, \ldots I$, and J_i residences within each district, denoted by j, $j = 1, \ldots J_i$.

For simplicity, the following assumptions are made about district water demand and supply. Individuals in all districts have identical demand (benefit) schedules for water (W(q)). The demand schedule is a decreasing monotonic function relating the volumetric price (c_i) and the quantity demanded (q) such that $(\partial W/\partial c_i) < 0$.

A linear demand curve intersects the quantity axis at some finite value, q_2, and the area under the demand curve is finite. The income effect, due to a price change along that curve, is assumed to be negligible. Water supply to each district exhibits constant returns and cost, C per unit for acquisition, delivery and disposal. The cost to the district of installing meters and billing each resident (kj) varies according to residential location. The distribution of meter costs across residences is the same in all districts.

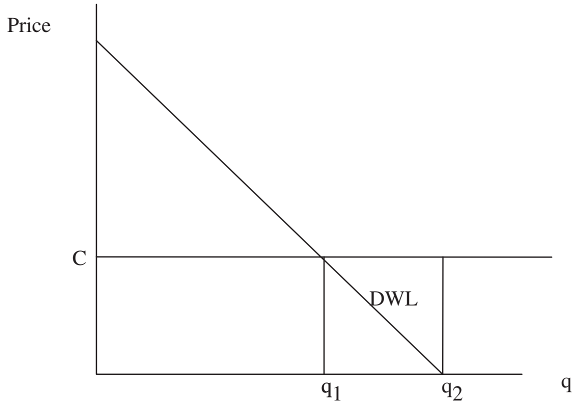


Figure 1. Water demand and dead weight loss under different options for distributing water.

Each district i has two options for selling water to each resident j, a share option (s_{ij}) and a meter option (r_{ij}). Under the share option, a resident is charged a flat fee for water per time period and consumes q_2 units of water. Under the meter option a resident is charged a volumetric fee, based upon quantity consumed, and consumes $q_1(c_i)$ units of water, a variable quantity less than q_2 (see Fig. 1).

Under both options district water revenues equal district costs. Under the share option the district delivers q_2 units of water and charges a flat fee (q_2c_i) to cover the water acquisition and delivery costs. Under the meter option, the district delivers $q_1(c_i)$ units of water, an amount less than q_2. The district charges a volumetric price of c_i and the total water bill is c_iq_1. The district assesses an additional fee (k_j) to cover the cost of meter installation and billing. The model assumes no additional monitoring to limit the water use of metered and non-metered residents.

The district chooses to maximize net benefits summed across all residences within the district. As the problem is defined, the district chooses between s_{ij} and r_{ij} for each resident to maximize net benefits to that resident. The district chooses the communal option, s_{ij}, when $S_{ij} > R_{ij}$, and the meter option, r_{ij}, when $S_{ij} < R_{ij}$, where capital letters denote net benefits under each option. The net benefits to option s_{ij}

$$S_{ij} = \int_0^{q2} W(q)dq - c_i q_2 \tag{1}$$

equal the entire area under the demand curve less the cost of water. Net benefits to option r_{ij}

$$R_{ij} = \int_0^{q2} W(q)dq - c_i q_1 - k_j \tag{2}$$

are restricted to the area under the demand curve to the left of q_2, where marginal benefits exceed the cost of water, less the cost to supply q_2. After differentiating (1) and (2) with respect to c_i and simplifying, it is apparent that

$$\frac{\partial S_{ij}}{\partial c_i} = -q_2$$

and

$$\frac{\partial R_{ij}}{\partial c_i} = -q_1 .$$

Since $-q_2 < -q_1$, and both are negative, $(\partial S_{ij}/\partial c_i) < (\partial R_{ij}/\partial c_i)$. An increase in the cost of water decreases option s_{ij} net benefits more than it decreases option r_{ij} net benefits.

Given free water ($c_i = 0$) and some positive metering cost k^*, equations (1) and (2) indicate that option s_{ij} will always be preferred to option r_{ij}, (assuming $S_{ij} > 0$). As c_i is increased indefinitely, eventually some cost c^* will be reached where

$$\int_0^{q_2} W(q)dq - c_i^* q_2 = \int_0^{q_1} W(q)dq - c_i^* q_1 - k^* \tag{3}$$

and $S_{ij} = R_{ij}$. If some proportion, p^*, of the residences in the district have metering costs below k^*, p^* percent of the district residences in this example will have metered service connections, and $(1 - p^*)$ percent will have share service connections. Similarly, equations (1) and (2) indicate:

$$\int_0^{q_2} W(q)dq - c_i^{**} q_2 = \int_0^{q_1} W(q)dq - c_i^{**} q_1 - k^{**} \text{ for some } k^{**}\text{, where } c^{**} > c^*. \tag{4}$$

From (4) it can be shown that $k^{**} > k^*$, which implies that over p^* of the district residences will be metered given cost c^{**}. Assuming that S_{ij} and $R_{ij} > 0$, it follows in general that $(\partial P_i/\partial c_i) > 0$.

4 EMPIRICAL APPLICATION AND DATA ANALYSIS

4.1 *Data collection*

The model is used to explain the pattern of residential metering shown by water districts in Central Valley urban areas. Empirical analysis required the collection of secondary data as well as primary data. The California Department of Water Resources provided data on the proportion of metered connections in each city in the districts of the Central Valley. The Black & Veatch Corporation provided c_i, the unit water cost charged customers in 1999, q_i, water use per resident and n_i, city/district population. (Recall that average cost is assumed constant so that average and marginal costs are equivalent). Telephone interviews were conducted with representatives of selected water districts to obtain a range for k_i, the average cost of metering service connections, including the costs of meter installation (new and retrofit) and meter reading and billing. These data were used to supplement and update information published in California Department of Water Resources Bulletin 166-4 (August 1994) and covering 70 urban areas in the Central Valley watershed. Water use and metering data were available for 59 urban areas in 1999. These data are analyzed using two techniques. First, the data are used to indicate the minimum water cost needed to justify residential metering and maximize net benefits, assuming average metering costs in a district and efficient water district choice of metering coverage. Second, the data are used in a logistic regression to estimate the actual change in metering associated with the change in water costs across districts in the Central Valley. A comparison between efficient and actual metering permits the analysis of the motivation explaining district water metering choice.

4.2 *Benefit cost criteria for choosing between water sales options*

The switch from share pricing option s to metering option r represents a trade off between the benefits and the costs of metering. When a meter is installed, water use drops but metering costs become positive. Hence, the net benefit of the switch varies according to the expected drop in water use, the value of the drop in water use and the cost of metering. The 59 urban districts which provided water use and metering data may be split into three groups: (1) districts where all residents have water meters; (2) districts where no residents have water meters; and (3) districts where some residents have meters and some residents do not have meters. These three groups contain 27, 10 and 22 districts, respectively, in our sample (see Table 1).

Table 1. Metering practices and water costs in selected cities in the Central Valley (1999).

County	Serving	Percent metered single family (%)	Annual charge per cubic meter (2000$)
Stanislaus	Denair	0.00	390
Fresno	Fresno	0.00	300
Fresno	Kingsburg	0.00	500
San Joaquin	Lodi	0.00	310
Madera	Madera	0.00	240
Stanislaus	Modesto	0.00	540
Sacramento	Sacramento	0.00	350
Kern	Shafter	0.00	560
Sacramento	Sacramento unincorporated	0.00	160
Yolo	Woodland	0.00	180
Fresno	Reedley	0.00	160
Sacramento	Carmichael	0.20	590
Merced	Atwater	0.20	390
Stanislaus	Turlock	0.30	310
Madera	Chowchilla incorporated	1.50	400
Yuba	Roseville	7.70	220
Yuba	Marysville & vicinity	11.80	490
Sacramento	Sacramento	13.40	350
Sacramento	Elk Grove	18.30	150
Merced	Merced	21.60	440
Kern	Bakersfield and vicinity	24.70	350
Tulare	Tulare	26.00	330
Sacramento	Fair Oaks/Orangevale	27.50	540
Fresno	Firebaugh/Las Deltas	27.70	510
Tulare	Visalia & vicinity	31.10	340
Fresno	Selma	35.90	550
Butte/Glenn	Chico & vicinity, Hamilton & vicinity	37.80	360
Kings	Corcoran	38.00	470
Yuba	Yuba City	86.00	250
Fresno	Clovis/Tarpey village	93.90	380
Tulare	Porterville	95.80	390
Yolo	Davis/El Macero	96.70	320
Kern	Arvin	100.0	400
Placer	Auburn/Bowman	100.0	580
Kern	Bakersfield	1.000	360
Fresno	Coalinga	100.0	710
Colusa	Colusa	100.0	280
Kern	Delano	100.0	370
Placer	Brockway/Kings, Beach/Tahoe Vista	100.0	970
Shasta	Burney	100.0	310
Merced	Delhi	100.0	300
Tulare	Dinuba	100.0	360
El Dorado	El Dorado hills	100.0	530
Tulare	Exeter	100.0	200
Merced	Hilmar	100.0	390
Kings	Lemoore	100.0	370
Tulare	Lindsay	100.0	410
Merced	Los Banos	100.0	220
San Joaquin	Manteca	100.0	320
Sacramento	Rancho Murieta	100.0	670
Tehama	Red Bluff	100.0	380
Shasta	Redding	100.0	390
San Joaquin	Stockton	100.0	540
Kern	Taft	100.0	540
Kern	Tehachapi	100.0	380
San Joaquin	Tracy	100.0	930
Placer	Auburn unincorporated	100.0	580
Nevada	Grass valley area unincorporated	100.0	670

Table 2. Survey results of cost of meter installation, meter reading and billing (2000$).

Municipal utility	Meter installation: Retrofit	Meter installation: New	Reading and billing (annual cost)
Chico	$ 225.32	$ 105.15	
Fresno	$ 255.36	$ 82.62	
Redding		$ 97.64	
Sacramento citizens	$ 193.77		
Sacramento county	$ 781.10		$ 7.90
Stockton	$ 225.32	$ 100.00	$ 3.00
Average cost	$ 336.17	$ 96.35	$ 5.45
Annual installation cost*	$ 31.73	$ 9.09	
Total cost**	$ 37.18	$ 14.54	

* Amortized over 20 year life of meter @ 7%.
** Including reading and billing cost.

The average cost of water in 100% metered districts was \$480 per km^3 (km^3 in \$2000) while the average cost of water in 100% non-metered districts was \$350/km^3. The average annual water use per residence in metered and non-metered was 340 km^3 and 420 km^3, respectively, a 25% difference. As a first approximation, these data suggest that a switch from a share to a meter option, to a resident in a district where the cost of water was \$480/km^3, would decrease water use 80 km^3. This amount may be termed excessive water use because it has a marginal value less than its marginal cost. A non-metered residence will always have some excessive water use because each resident acts as though his marginal cost of water were zero.

The value of a drop in water use to a resident equals the avoided cost of the resident's excessive water use. The net value of the drop in water use is termed dead weight loss (DWL). If we assume a linear demand for water, half the cost of excessive water use by a non-metered residence is dead weight loss (Hanke 1982). For example, given a linear demand for water between \$480/km^3 and \$0/km^3, the DWL associated with the 80 km^3 excessive water use is \$20 (See Figure 1). Using these same linear assumptions, the DWL associated with water costing \$320/km^3 is about \$13.64, the DWL associated with \$160/km^3 water is about \$6.82, and the DWL associated with \$80/km^3 water is about \$3.41. Six urban water districts provided cost data based upon recent or on-going metering programs (Table 2). These data indicate that metering costs within a district are quite variable and have a bimodal distribution, reflecting the difference between the costs of retrofitting meters into older residences and installing meters in new residences. Based upon these data, the annual cost of meter installation, reading and billing is estimated to average \$37.18 for a retrofit and \$14.54 for a new residence meter (see Table 2).

A switch from option s to option r is warranted only when avoided DWL is greater than acquired metering costs. The estimates above suggest this to be the case on average for retrofit metering, only when the cost of water is above \$0.9/m^3. Avoided DWL exceeds the cost of new residence metering when the cost of water is over \$0.3/m^3.

4.3 *Logistic regression analysis of water district choices*

Benefit-cost criteria might assist district residents to choose between share and metering options for distributing water. An econometric analysis was done to determine whether similar benefit-criteria affect this choice in practice. Specifically, a logistic recession was run to estimate the change in metering proportion associated with a unit change in water costs across districts in the Central Valley.

Table 3. Regression results[1].

Variable	Coefficients	Standard error	t-stat
Intercept	−1.32020732	2.676082516	−0.49334
Annual charge in \$2000	0.00980136	0.004606344	2.127796
Population	−2.9654E-05	1.11604E-05	−2.65709

[1]Adjusted $R^2 = .16$ with 59 observations.

The logistic regression estimates the metering-water cost relationship assuming a function of the form $y = b_1 + b_2 c + b_3\, pop + e$ where b_1, b_2, and b_3 are coefficients to be estimated, c_i is district average water cost, pop is city population, e is the error term having a zero mean and Weibull distribution, and y represents the logistic transformation of p_i equal to:

$$\boxed{\begin{aligned} y &= \ln\frac{p_i}{1-p_i}, \ \text{if } 0 < p_i < 1 \\ y &= \ln\frac{p_i + \frac{1}{n}}{1 - p_i + \frac{1}{n}}, \ \text{if } p_i = 0 \\ y &= \ln\frac{p_i - \frac{1}{n}}{1 - p_i - \frac{1}{n}}, \ \text{if } p_i = 1 \end{aligned}} \tag{5}$$

This form is used in order to perturb the data from the boundary conditions (Cox 1970). Recall that p_i is the proportion of residences in a district with water service connections that are metered and n is the number of observations in the regression sample. Equation (5) may then be estimated using ordinary least squares (Cox 1970).

The estimated coefficient and associated t-statistic in the regression equation indicates that the water cost variable is positive and significant, as predicted by the model (Table 3). Population is inversely related to metering proportion and is statistically significant. We hypothesize that larger communities in the Central Valley were established long before metering became more commonplace and are thus less likely to be metered. This equation may be used to predict the long run increase in metering caused by changes in the water cost. For example, the equation indicates a 72% metering proportion given \$0.3/m^3 water and indicates a 100% metering proportion, given \$0.9/m^3 water. In this range, a \$690 increase in the cost of water is associated with a 28% increase in metering.

It is noteworthy that district choice of metering options, as summarized in the regression equation, is compatible with the benefit cost criterion for maximizing district net benefits, presented in the example above. Recall that these criteria indicate that retrofit metering is economic, given above \$0.9/m^3 water, and new residence metering is uneconomic given below \$0.3/m^3 water, assuming average costs for retrofit and new residence meters.

In other words, district metering in the Central Valley is probable (100%), when the benefit cost criterion indicates retrofit metering of existing residences is economic, and district metering is less probable (72%), when benefit cost criterion indicates metering new residences is not economic. This comparison between metering practice and metering benefits and costs, suggests that water districts choose metering proportions in large part on efficiency grounds. The data on water costs and metering proportions in different districts illustrate this point most directly. It is generally not correct to accuse water districts of inefficiency merely because district residences are not metered.

5 CONCLUSION

Metering proportions of Central Valley water districts are explained using a model that postulates maximization of district net benefits. Predictions of metering proportions, based upon this model, are compatible with the empirical data. These findings suggest that share water systems in the Central Valley are an efficient response to low water costs.

This analysis only considers the direct cost of water and meters to districts. Broader considerations, such as the volume and quality of return flows and possible under-pricing of water are not dealt with and could modify this conclusion. Nevertheless, these findings also suggest that legislative effort to end share water systems may be misguided. More fundamental inefficiencies in California water use, such as uncertain water rights, hinder water sales between districts and may keep Central Valley water prices artificially low. Legislative effort might be more effective if it were directed to solve these fundamental problems rather than to impose water meters upon Central Valley water districts.

REFERENCES

Alchian, A. & Demsetz, H. 1972. Production, Information Costs and Economic Organization. *American Economic Review* 62:777–95.

Black & Veatch Corporation. 1999. California Water Charge Survey: 1999.

Coase, R. H. 1937. The Nature of the Firm. *Econometrica* 4.

Cox, D. R. 1970. Analysis of Binary Data. London: Methuen.

Demsetz, H. 1967. Toward a Theory of Property Rights. *American Economic Review* 57:347–59.

Department of Water Resources. 1994. Urban Water Use in California. Bulletin 166-4. State of California Department of Water Resources.

Department of Water Resources. 1999. Unpublished PWSS Survey. State of California Department of Water Resources.

Fama, E. & Jensen, M. 1983. Separation of Ownership and Control. *Journal of Law and Economics* 26:301–26.

Hanke, S. H. 1982. On Turvey's Benefit-Cost 'Short-Cut': A Study of Water Meters. *Land Economics* 58(1): 144–46.

Kenny, R. & Klein, B. 1983. The Economics of Block Booking. *Journal of Law and Economics* 26:497–540.

Ouchi, W. G. 1980. Markets, Bureaucracies, and Clans. *Administrative Science Quarterly* 25:120–42.

U.S. Census Bureau: State and County QuickFacts. Online. http://quickfacts.census.gov/qfd/states/00000.html. June 14, 2001.

Vossler, C. A., Espey, J. & Shaw, W. D. 1998. Trick or Treat? An Offer to Obtain Metered Water. *J. Am. Water Resour. Assoc.* 34(5):1213–20.

Walters, L. L. & Young, R. A. 1994. Economic Factors Affecting Residential Water Demand in Colorado. Colorado Water Resources Research Institute, Colorado State University, Fort Collins, CO 80523. 100 pp.

Williamson, O. E. 1985. The Economic Institutions of Capitalism. New York: The Free Press.

The economics of water efficient products in the household

N. Grant
Elemental Solutions, Hereford, UK

D. Howarth
Environment Agency, Worthing, UK

ABSTRACT: Many water efficient household appliances are not considered cost-effective to install from the perspective of the householder or from a water company undertaking a large scale retrofit programme. The current state of the market is reviewed for the following appliances: toilets, washing machines, dishwashers and showers. For each product group an economic analysis has been undertaken that considered water, wastewater, energy costs, the residual life of appliances replaced, life span and implementation period. The results suggest that all the options considered are worthy of further investigation

1 INTRODUCTION

Despite moderately high rainfall in England and Wales (897 mm/year) water availability per capita is low (1334 m^3/annum) and is lower than Spain, Syria and Malawi, due to our high population density (343 /km^2). In its national water resources strategy, *Water resources for the future (2001)*, the Environment Agency depicted a number of catchments, predominantly in the southeast of England, where either there was no additional water available for abstraction or over-abstraction was currently occurring. Against this background most water companies continue to predict growth in demand driven by increasing levels of household per capita use.

Demand management, defined as "the implementation of policies or measures which serve to control or influence the consumption or waste of water" has been on the political and regulatory agenda since the 1988–92 drought affected large parts of southeast England. The government and its regulators, Ofwat and the Environment Agency have published numerous consultation papers and policy papers indicating the need, for economic and environmental reasons for water companies to seriously consider demand management policies in their approaches to water resources management.

2 BACKGROUND/CONTEXT

Despite this regulatory desire and a legislative requirement, water company adoption of demand management measures has been limited. Since 1996 the water companies have had a duty to promote the efficient use of water to their customers and Ofwat measure water company progress by the following inputs 1996–2002 (Ofwat, 2002):

– 419,719 supply pipes repaired
– 73,002 supply pipes replaced
– over 7 m cistern devices distributed
– over 10 m water audit packs distributed to households

– 82,707 household water audits carried out by water companies (or their agents)
– 120,199 water audit packs distributed to institutions and commerce
– 30,532 institutional and commercial water audits carried out by water companies (or their agents)

Water company estimates of the savings resulting from these activities totals 647 Ml/day (4.1% of current distribution input).

The Government's ten-point plan for a better water industry introduced mandatory leakage targets in 1998. As a result leakage has fallen by 35% from its peak in 1995/96. The majority of the estimated savings arising from the water companies' activities to fulfil their water efficiency duty are from the repair and replacement of supply pipes driven by the need to meet leakage targets.

Most of the water companies now claim to have reached, or are very close to, their economic level of leakage. Hence, if future demand reductions are to take place alternative demand management strategies will be required.

Ofwat has stated that it expects a "basic, minimum level of activity from all companies, unless supplies are under pressure where a more active approach is necessary" where a "more active approach" is not defined. Most companies are complying with the requirement to carry out this basic, minimum level of activity and doing little more (with one or two notable exceptions). These activities are low-cost (which should not be confused with cost-effective) delivering low potential water savings, with the exception of supply-pipe repairs.

Water companies are required to submit water resources plans to the Environment Agency every five years. These plans are next due in 2004 when they will also be used as part of the price-setting process. In the 1999 water resource plan submissions and in their water efficiency plans (1996 and 2001) no water company seriously evaluated an appliance exchange programme. It has been suggested that (water) savings from conservation programmes are transient and therefore cannot be guaranteed into the future, and this is likely to be true of the measures that comprise Ofwat's "basic level of activity". However, appliances that work efficiently with a fixed volume (clothes washer, dishwasher, toilet), although with a higher unit cost, should yield predictable and guaranteed savings. To date this is an area that is relatively unexplored.

3 STUDY OBJECTIVES

The objectives of the study were to estimate these wider costs from the perspective of the household customer and from the water company who could be in the position of implementing a wide scale retrofit program. In addition to water, wastewater, energy and detergent costs, CO_2 emissions, life span and reliability of the new appliances, the residual life of appliances replaced and energy label inaccuracies were all considered.

The products reviewed include clothes washers, dishwashers, toilets, showers and direct water heating appliances.

4 PRODUCT REVIEWS

4.1 *Wet white goods*

Data for washing machines and dishwashers are readily available in the form of independent test reports from the Consumer Association. Previous analyses had led to the conclusion that efficient machines tended to cost more but they were also more highly rated against other criteria such as noise, reliability and ergonomics. The concept of "payback period" was therefore complicated as the buyer was getting a better product. For the purchaser or utility only considering water efficiency, the significant over-cost and modest savings meant that the payback period might exceed the typical 10-year life of the machine (Grant, 2002).

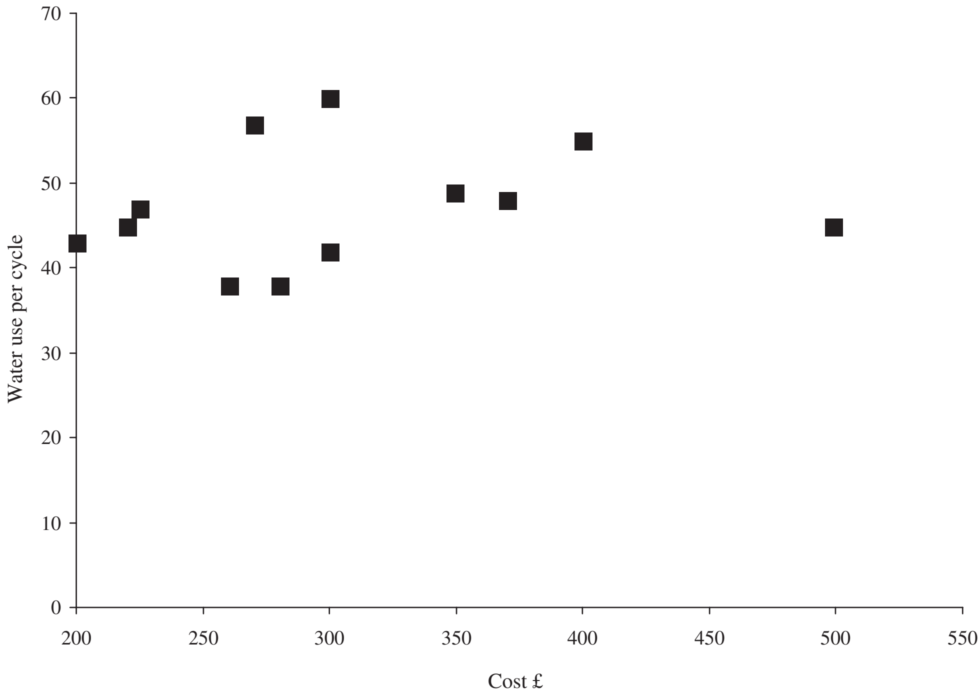


Figure 1. Washing machine water use variability with cost.

The situation has now changed. Figure 1 shows the lack of correlation between water efficiency and machine purchase price based on recent data (Which?, January 2003). The near market-saturation of A rated (energy) machines suggest that the Energy Label has been a success, at least in terms of the measured variables.

Further efficiencies can be achieved by improved control of water (e.g. for part loads) and detergent (to reduce excess rinse requirements) but this would not be reflected on the current Energy Label. A trend towards machines with a larger capacity of 6 kg or even 8 kg could further reduce part load efficiencies. Other uncertainties include the poor accuracy of manufacturer's claims on the Energy Label (EA 2003).

The situation with dishwashers is similar. Again using data from the Which? website we found that all machines tested used less than 20 litres per cycle (12 place settings) and 50% used 16 litres or less.

4.2 *WCs*

Unlike white goods, no independent testing of WC performance and actual water use is available in the UK. Also there is very little published trial data to prove water saving from "low flush" WCs and so analysts typically use assumptions based on nominal flush volumes and, for dual flush, an assumed ratio of 3 or 4 part flushes to one full flush. Neither of these assumptions can be substantiated. Figure 2 summarises the measured average flush volumes for all the "low flush WC" trials that we were able to find. The four domestic 6/3 dual flush trials average 4.6 litres per flush and this figure has been used provisionally in our calculations. Details and references are included in the full report (EA 2003).

In the UK, dual flush typically means 6/4 litre but 6/3 is available as is 4/2 from Scandinavia (not yet UK tested). Manufacturers claim that about 75%–80% of sales are now dual flush (National Water Conservation Group Minutes of meeting 1st October 2002). Whilst dual flush is

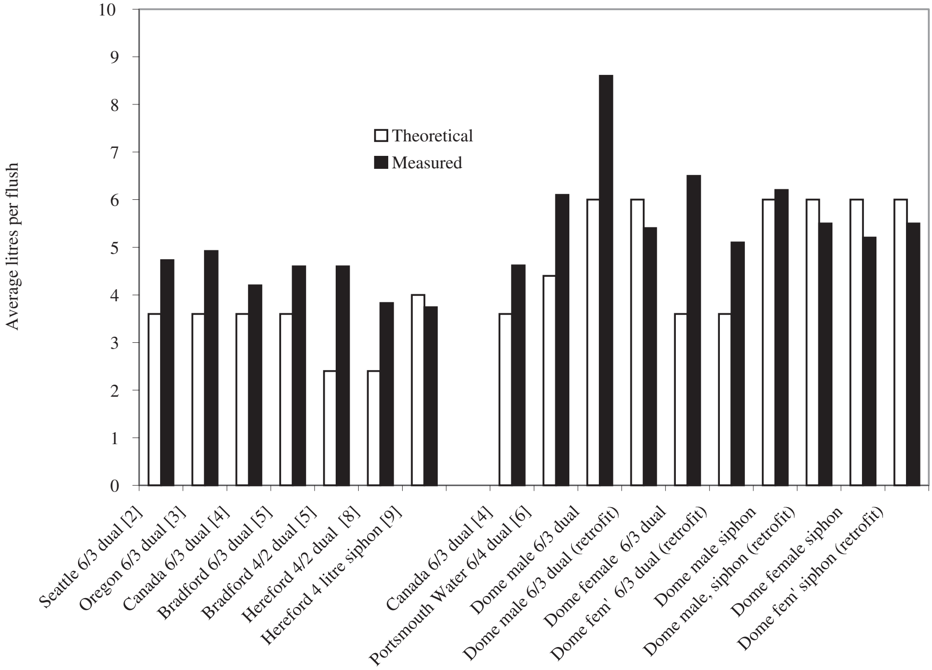


Figure 2. Summary of low flush WC trial data.

often seen as the efficient option, the real world savings are not as high as manufacturers claim and in some trials water use has been higher than with the single flush control group; see graph 2 (EA 2003, Hills et al 2001).

The issue of valves and siphons is also contentious and rational debate has been difficult due to the political and commercial interests involved. It will be many years before we have sufficient UK-specific evidence to show how significant the problem of valve leakage will be. Any problem is likely to be compounded by the low penetration of water metering and a severe skills shortage in the plumbing trade, concerns that have been highlighted by UK manufacturers for many years. Endurance testing of flush valves might eliminate some of the lowest quality products but the testing does not prevent observed failures due to factors such as incorrect installation or jamming of mechanisms, for instance due to calcium deposits in hard water areas.

There is strong anecdotal evidence that many new suites fail to clear the pan effectively at 6 litres (leading to double flushing) and that some plumbers increase the flush volume by raising the water level to reduce customer complaints. Testing to the Water Supply (Water Fittings) Regulations 1999: WC Suite Performance Specifications should address the performance issue.

A number of factors influence actual flush volumes as opposed to quoted nominal volumes. These include manufacturers' tolerances and adjustment of water level at installation. The Regulator's Specification requires the flush volume to be measured with the water supply turned off but in reality extra water enters the cistern during the flush. The volume of wasted water will depend on valve design, water pressure and flush duration.

WC prices tend to reflect design and quality rather than water efficiency. Unfortunately very few WCs currently on the market have independent approval (e.g. WRAS or KIWA) so we cannot make a reliable correlation between price and even nominal flush volumes, as some models may not meet the Regulator's Performance Specification. In the USA it has been shown that flush performance is independent of purchase price for the latest WCs (NAHB 2002).

A promising option is the retrofit of dual flush devices to existing siphon cistern WCs. A trial by Southern Water and the Environment Agency delivered average savings of 2.6 litres per flush (Keating & Lawson, 2000). At the time of writing this option is prohibited by the Water Supply (Water Fittings) Regulations 1999, but a change in the near future is possible.

4.3 *Showers*

As WC flush volumes have fallen and bathing habits have changed, newer households may use more water for baths and showers than for WC flushing.

UK showers can be usefully divided into 3 main types; electric heated, gravity and mains-pressure or "power showers". For electric and most gravity showers, flow rates are modest and there is little scope for water saving. Mains pressure or power showers can deliver very high flow rates but, in practice, users are likely to control the flow rate. Multi-head panels can deliver very high flow rates but have previously been limited to the luxury market. A recent trend is for low-cost models with up to 8 showerheads.

"Water saver" showerheads are available that use the water pressure to produce an atomised spray for enhanced wetting with less flow. Other systems use a venturi to entrain air for a similar effect. Such devices can give the effect of a powerful shower whilst only using 6–10 litres of water per minute but in typical cold UK bathrooms, atomising and aerating heads can lead to a "cold feet" effect. Also these showers can be noisy and the fine spray can aggravate moisture problems in some bathrooms.

In the UK the most likely route to water efficiency for non-enthusiasts would be to regulate flow rates to an acceptable "water sufficient" level. For electric showers, no intervention is required as flow is limited by power input. For pumped and mains pressure showers, dynamic flow regulators can be fitted but to guarantee savings these would need to be integrated into shower heads or mixers to prevent removal. In the US showerheads with a maximum flow rate of 2.5 US Gpm (9.5 litres/minute) at 80 Psi (56.2 metres head) have been a legal requirement since January 1994 and similar regulations are being considered in Australia.

4.4 *Direct hot water systems*

Combination boilers account for over 50% of the domestic boiler market and this is predicted to rise. Combination boilers typically require a warm-up period before hot water is delivered to the tap and this can waste water and energy whilst causing annoyance. Solutions are available such as small thermal stores and keep-warm facilities but the associated energy cost/benefit balance is not yet fully considered in the SEDBUK and SAP rating schemes.

It is unlikely that boiler replacement could be justified on water efficiency grounds alone because of the high cost. Instead water efficiency should be integrated into the SAP and SEDBUK energy ratings and considered in regulations.

5 ECONOMICS

The study focussed on practical product-specific issues rather than the methodology of economic analysis which is covered in depth elsewhere (UKWIR and Environment Agency, 1996). The study has identified a number of practical barriers to appliance exchange or similar programs. For example, for a given product, if savings are uncertain, water-use labelling is inaccurate or installation is complicated by issues of compatibility (style, dimensions, colour), then we are not able to calculate a reliable estimate of the economics or potential uptake of a product replacement or promotion programme.

With the exception of showers the current trend for all the product groups is towards lower water use. Hence there are large potential savings to be achieved by replacing older products with new.

However if the old product is still serviceable then the cost of this option is high. For new purchases the cost difference between efficient and inefficient products, all else being equal, is likely to be minimal but at the same time any savings may be difficult to differentiate in a reliable way.

From the householder's perspective the economics of any option will vary considerably according to compound factors such as household size, water and sewerage charges and number of appliances per household. For example if a family of 5 paying £2.50 /m^3 water and sewerage charges with a single 9 litre WC were to fit a dual flush device, they might achieve savings of; 2.6 litres/flush $\times$ 5 flushes/head/day $\times$ 5 $\times$ £2.50 /m^3 $\times$ 365 days = £59/year. A couple with two WCs paying £1 /m^3 for water and sewerage might save; 2.6 litres/flush $\times$ 5 flushes/head/day $\times$ 2 $\times$ £1 /m^3 $\times$ 365 days = £9.50/year. The cost of converting WCs has been estimated at around £20/unit so the 5 person house achieves a simple payback of £20/£59 = 0.34 years whilst the couple with two WCs achieves £40/£9.5 = 4.2 years, still respectable but 12 times as long. This paper concentrates on the economics from a water company perspective.

Regional average figures can be used to calculate the Average Incremental Cost (AIC) of measures such as dual flush retrofit. Consideration of the household economics suggests targeting larger households with older toilets. However volunteers are likely to be from households with water meters, which tend to be of lower occupancy (where metering is optional) or from homes built since 1989 where the toilet will, at most deliver a 7.5 litre flush.

For a water company to calculate the economic benefit of a retrofit or product replacement programme, the assumed life of the measure is a critical variable. It does not seem reasonable to calculate the benefit over the full life of the new product as after, say 5 years, the existing WC or washing machine may have been due for replacement. Although WCs can have a very long life, it is thought that 50% of domestic WCs are replaced every 16.5 years (Griggs, 2003) but it is not known how many of the same WCs are replaced regularly. Washing machine and dishwasher life is about 10 years.

In predicting savings, for example to determine AIC, we are faced with a number of uncertainties:

- would the consumer have purchased a less efficient machine without a subsidy given the lack of price/efficiency correlation?
- are the manufacturers' claims accurate and proven?
- rebound effects – for example will a washing machine be used for smaller loads if it is perceived to use less water and electricity?
- are our assumptions about usage and savings accurate (WC flushes/day, full:part-flush ratio, washing loads/week etc)?

Values for AIC or Average Incremental Social Cost (AISC) are meaningless without a statement of the assumptions used. For the purposes of this work we have not attempted to include social costs or benefits.

The main variables when calculating the AIC of a proposed measure are:

- Discount rate – different parties can justify different rates but the current 3.5% Treasury rate has been assumed here.
- Opex (operating expenditure) savings – this is the actual cost spared for every cubic metre of water saved, we have used a figure of 6 p/m^3 for electricity, chemicals etc. The value is not critical.
- Loss of revenue – ignored here as Ofwat have stated that they will make allowances for revenue loss at price setting.
- Cost of implementing measure (including administration) – this may be the full price of installing a measure or an incentive such as a voucher or discount.
- Time period of analysis – 30 years, consistent with water resource plans.
- Implementation period – if too long assumptions about technologies will be out of date. Does not alter AIC unless timeframe is exceeded. May be limited by uptake rate.
- Life of option – this has a very significant impact on the calculated AIC. For new installations the expected product life can be used but for replacement and retrofit programs the residual life of the existing product should be used.
- Water saving – where possible trial data should be used and product performance verified.

Table 1. Average incremental cost options (3.5% discount rate over 30 years, 6 p/m^3 variable opex).

Scenario	From	To	Installation period (years)	Life of option (years)	Unit cost (£/household)	Unit saving (unit is household) (literes/day)	Range of AIC (p/m^3)
1	Standard WC	Dual flush retro	5	5–15 (10)	£40 installed	32.5	19–66 (33)
2	Default purchase WC	4.6 litre average flush	10	15–30 (20)	£20–50 (£20) voucher	6.25–12.5 [2] (9.4)	19–178 (34)
3	9 litre WC	4.6 litre average flush	10	5–15 (10)	£400–250 (£350) installed	61	88–699 (176)
4	Existing old washing machine	Efficient new washing machine	3	3–6 (5)	£100–250 (200) voucher to full cost	50	93–466 (229)
5	Standard purchase washing machine	Efficient new washing machine	3	10	£20–50 (£20) voucher	5–10 (5)	58–312 (121)
6	Standard shower	Water saving showerhead	5	5	£40 fitted and tested	5–39 (10)	72–463 (227)

5.1 *Notes on table 1*

The scenarios in the table are intended for discussion, not as reliable benchmarks. For each option the least certain variable(s) are given a range of values. Two calculations are then performed to show the best and worst case. The figures in brackets are our best estimate.

5.1.1 *Scenario 1*

The cost and water savings (2.6 litres/flush) figures are from Southern Water and the Environment Agency. The only value varied was the "life of option" which is the residual life of the converted WCs. Newer WCs might have a longer residual life but should also have a lower flush volume so savings will be less. Other assumptions: 5 flushes/person/day, 2.5 people and 1.7 WCs/household. Considering a 10-year residual life of the converted WC we get an AIC of 33 p/m^3 with a discount rate of 3.5%. Raising this to 6% increases the AIC to 37 p/m^3. If the water saving proves to be only half what is expected (1.3 litres/flush) then at 3.5% discount rate and 10-year life, we get an AIC of 72 p/m^3.

5.1.2 *Scenario 2*

This example assumes that a future label can differentiate between models but that there is minimal over-cost associated with more efficient WCs. It is assumed that savings are between 0.5 and 1 litre per flush when compared with a default purchase. Based on these assumptions the economics look promising but promotion of a given WC type without hard evidence of actual water savings would be unwise. When data and appropriate product labeling are available then a voucher price (after subtracting overheads) can be calculated to achieve any required AIC. Although the detail of the relationship is not known, the higher the voucher price the better the likely uptake.

5.1.3 *Scenario 3*

The installation cost would depend on many factors and it is possible that the customer would contribute. The cost and liability for making good can be a significant barrier. Issues of performance, leakage, and compatibility (WC format, style, colour, dimensions) must be considered as well as the issues in Scenario 2. The assumed 4.6 litre average flush is for 6/3 dual flush or 4.5 litre single flush, both available in the UK with third party certification, see EA 2003.

5.1.4 *Scenario 4*

The water company supplies or subsidises the replacement of old washing machines using an average of 100 litres per wash and with a residual life of 3–6 years. The replacement is assumed to use 50 litres/wash. The economics are not particularly good because of the short residual life of the old machine (it would soon be replaced with a more efficient model anyway). The uncertainties about default purchase and the problems of administration and verification probably rule out this option even before we consider life cycle issues.

5.1.5 *Scenario 5*

If a performance differentiation can be made then an incentive scheme (vouchers, discounts, bulk purchase of products etc) should drive the market towards even more efficient machines. Hence even an apparently uneconomic scenario could "pump-prime" market transformation on a wide scale. The same argument applies to other products but raises the question "who pays?"

5.1.6 *Scenario 6*

Showering presents different problems for economic prediction as the trend is for increased use and higher flows. Insufficient data were available for us to make meaningful AIC calculations so the figures in the table should only be taken as an example. A regulatory approach may be required as in the US and damage limitation may be a more appropriate goal than water saving with respect to current water use.

6 CONCLUSIONS

- With leakage control approaching its economic minimum, future demand management efforts will need to concentrate on water use in the household.
- The price differential between inefficient and efficient appliances has largely disappeared. This is an important finding as the deterrent of paying more for efficiency has been removed.
- WCs and wet white goods use fixed volumes of water so where efficient models are installed, the risk of savings not being made is low.
- Independent testing is needed to verify manufacturers' claims, to allow consumers and water companies (or their agents) to make informed choices – this would be especially important if a water efficiency labelling scheme was to evolve.
- In 1999 Water Resources Plans the following ranges of resource development costs were submitted: new reservoir 23–323 p/m^3, Groundwater scheme 0.2–136 p/m^3, Bulk supply transfer 0.3–181 p/m^3. The options in Table 1 all have ranges of average incremental costs that are at least part within the resource development cost ranges, suggesting that these desktop scenarios are worthy of further investigation in the form of field trials.
- In making progress in this area of encouraging householders to replace their inefficient appliances with more efficient ones, water companies (and their regulators) may need to adopt a "loss leader" approach, to assess what works and what doesn't.

REFERENCES

Environment Agency & UKWIR. 1996. *Economics of demand management; practical guidelines*. Report Ref. No. 96/WR/03/2. UKWIR/Environment Agency.

Environment Agency. March 2001. *Water resources for the future, a strategy for England and Wales*. Bristol: Environment Agency.

Environment Agency. 2003. *The economics of water efficient products in the household*. Worthing: Environment Agency.

Grant, N. 2002. *Water conservation products; a preliminary review*. Watersave Network.

Griggs, J. 2003. *Personal communication*. BRE.

Hills, S., Birks, R. & McKenzie, B. 2001. *The Millennium Dome "WaterCycle" Experiment: to evaluate water efficiency and customer perception at a recycling scheme for 6 million visitors*. Presented to World Water Congress, Berlin, 15–19 October 2001.

Keating, T. & Lawson, R. 2000. *The water efficiency of retrofit dual flush toilets*. Southern Water & Environment Agency.

NAHB (National Association of Home Builders). September 2000. *Water closet performance testing*, Report No.: P01-1660902, prepared for Seattle Public Utilities and East Bay Municipal Utility District by NAHB Research Center Upper Marlboro, MD.

Office of Water Services. 2002. *Security of supply, leakage and the efficient use of water 2001–02*. Ofwat.

The water supply (water fittings) regulations 1999: WC suite performance specifications http://www.defra.gov.uk/environment/water/industry/wsregs99/wcspec/.

Which?. January 2003. (http://www.productpicker.which.net/).

Developing a decision support tool for cost-effective optimum in-house water consumption

F.A. Memon & D. Butler
Department of Civil and Environmental Engineering, Imperial College London, UK

ABSTRACT: Water consumption in large commercial and public buildings can be significantly reduced if water efficient devices are installed in place of conventional devices. A wide range of such devices is currently available in the market. However, before investing in new facilities, an estimate of possible water savings coupled with a cost-benefit analysis is essential. To facilitate this, a decision support tool is currently being developed. This paper explains the rational and implementation of the tool.

The tool has been developed within the Matlab environment by incorporating optimization routines as solvers for linear integer problems. Appliances considered are toilets, urinals and washbasin taps. Their characteristics such as capital, operational and retrofit costs, and water consumption have been used to form constraints for the optimization routines. These routines feed into the cost benefit module using present value algorithms.

Preliminary results indicate that it is possible to achieve water savings for given cost (or flow) constraints, and the payback period can be reduced using alternative scenarios without affecting the desired level of service. The tool is also flexible in its operational scope. In addition to large buildings, it can be applied to assess economic viability of retrofitting programmes for residential blocks of similar water consumption patterns.

1 BACKGROUND

Population growth, particularly migration to urban areas, and climate change are seen as two major factors posing a direct challenge to meeting the growing deficit between the available water resources and increasing demand. Although, apparently, fresh water resources are abundant in holistic terms, they are limited in areas of need. Water consumption over the next three decades is anticipated to increase and put an additional water stress (water consumption/available water) on existing resources (Table 1).

Table 1. Observed and predicted water stress in 1985 and 2025 (Vörösmarty *et al*, 2000).

	Population (million)		Available water (km^3/yr)		Water stress	Change in water stress, relative to 1985, in 2025 (%)		
Area	1985	2025	1985	2025	in 1985	Climate	Population	Combined
Africa	543	1440	4520	4100	0.032	10	73	92
Asia	2930	4800	13700	13300	0.129	2.3	60	66
Australia	22	33	714	692	0.025	2.0	30	44
Europe	667	682	2770	2790	0.154	−1.9	30	31
North America	395	601	5890	5870	0.105	−4.4	23	28
South America	267	454	11700	10400	0.009	12	93	121
Globe	4830	8010	39300	37100	0.078	4.0	50	61

At a global scale, the data shows a rather rosy picture. However, investigations at country level point towards a potential problem of water shortages that will tend to worsen in time unless appropriate measures are taken.

Instead of developing new water resources, which arguably is a less attractive option both financially and environmentally, considerable effort has recently been steered towards the implementation of demand management options. These include design and adoption of water conservation and recycling technologies and strategies.

In urban areas, the major water consumption is typically in households and large commercial/public buildings (schools, offices, hotels, hospitals etc). Domestic water consumption patterns are well established and large water savings are difficult to achieve because considerable investment must be made. However, the case of large buildings is rather different and past studies (e.g. BSRIA (1998)) have shown that considerable water and financial savings are possible if water conservation measures, mainly low water consuming devices, are employed.

2 WATER CONSUMPTION TRENDS AND CONSERVATION POTENTIAL FOR LARGE BUILDINGS

According to a survey carried out in the above-mentioned BSRIA study, the annual water consumption in large buildings, in England and Wales alone, is 966500 Ml, which is approximately 25% of the total public water consumption. About 65% of this is consumed in hotels and motels. The breakdown for the remaining 35% is shown in Figure 1. The typical water consumption pattern in large buildings, as reported in BSRIA (1998), is shown in Figure 2. Maximum water consumption is in toilet flushing followed by urinals and washing through taps. It is estimated that these large buildings annually spend over £491 million on water related services. This excludes sums spent by hotels and motels (£926 million). Annual spending in water services in large buildings is shown in Figure 3. The figure shows considerable scope for financial savings and implementation of demand management options.

Although considerable effort has been expended on raising public awareness and introducing policy drivers (e.g. metering) being a precursor to achieve consumers' behavioral change, the

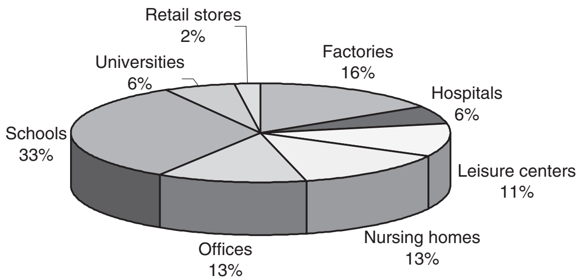


Figure 1. Water consumption in different types of buildings (BSRIA, 1998).

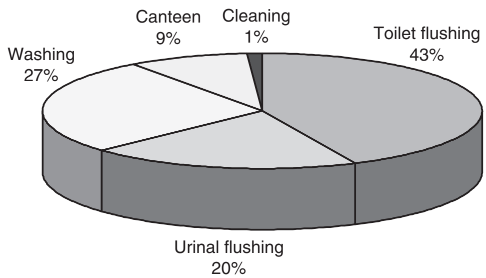


Figure 2. Typical water consumption pattern in offices (BSRIA, 1998).

extent of their success is hard to measure. Therefore, water demand forecasting and future supply provision planning cannot rely solely upon the assumption that consumers will change their habits. Hence introduction of concrete mechanisms to reduce demand at source is essential. This is possible by introducing well-tested low water consuming devices.

Installation of low water consuming devices can reduce water consumption significantly. For example, low/dual flush toilets can reduce water consumption by a factor of 3 (Figure 4). Use of water efficient urinals can reduce urinal-flushing demand by half. The use of spray taps on washbasins and flow restrictors can produce additional savings of up to 80% in flows from conventional devices (EA, 2001). It is estimated that large commercial buildings offer a significant potential for water and associated costs savings. For example, in England and Wales alone about 565 Ml of water could be saved daily. This amounts to annual financial savings of about £189 million (BSRIA, 1998).

Appliance retrofit programmes have yielded considerable savings in the past. An account of financial savings gained through retrofitting projects in the USA is given by Green (2003). For example, in Arizona savings of about US$ 88 million have been reported from a toilet cistern-retrofitting programme implemented by investing US$ 15 million (Diziegieleski and Baumann, 1992). A considerable amount of benefit (£89–396 million) has also been reported by Massachusetts Water Resources Authority; achieved through a modest investment of £22 million in retrofitting projects. In New York City, the cost of replacing 1.5 million high volume toilet cisterns with 6-litre cisterns is estimated as £0.6–0.7 million per Ml/d compared to £1.9–2.8 million per Ml/d for resource expansion (Amy Vickers, 1996). Broadly similar success stories are reported from Australia and some other parts of the world. A guideline methodology for designing retrofitting programmes is given in NMOS (1999 and 2001).

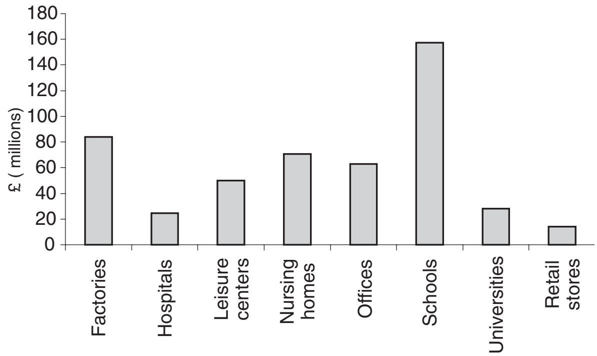


Figure 3. Annual spending on water services (BSRIA, 1998).

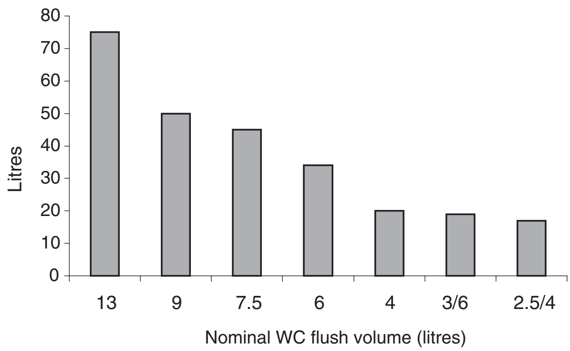


Figure 4. Water use per person per day for toilet flushing (EA, 2001).

The general perception pertaining to demand management options is that these are *not* “fit and forget” measures, and require substantial initial investment and operating costs. In an economic interest driven culture, water conservation measures will be successful only if they are reasonably financially attractive. To address this, a simulation tool has been developed which can assist in the decision-making process and assess the economic viability of demand management strategies. This paper describes the tool structure and demonstrates its implementation.

3 MODEL STRUCTURE

A flow chart for the proposed model is shown in Figure 5. The model consists of three modules:

- Input
- Appliance selection
- Economic assessment

The input module provides the tool user the environment to change appliance characteristics, set cost (or flow) constraints, assign the average number of service points for each appliance, select appliance life and interest rate on savings and incorporate building users’ characteristics (e.g. average use of appliance per day).

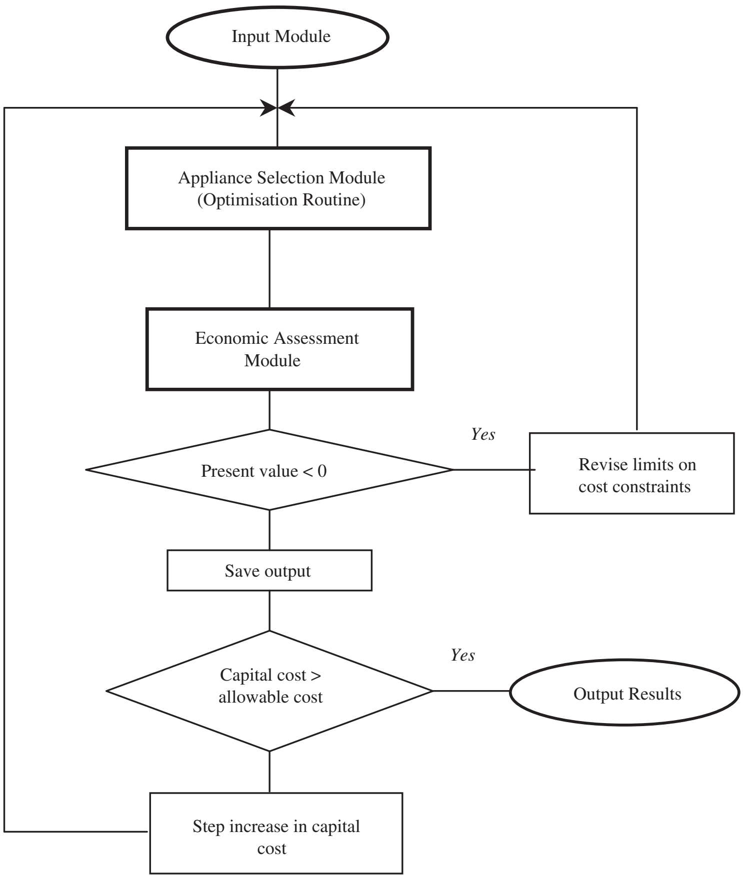


Figure 5. Conceptual flow diagram for the simulation tool.

The appliance selection module, which is the core module, produces an optimized selection of appliance combinations offering minimum water consumption or investment. The module defines the objective function and constraint matrix within the framework of a linear integer programming problem. The objective function *f* for flow minimisation accommodating three appliances (toilets, urinals and washbasin taps), in its generic form, is represented by Equation 1.

$$\boxed{f = \sum_{i=1}^{n} ww_i.x_i + \sum_{j=1}^{n} wu_j.x_j + \sum_{k=1}^{n} wt_k.x_k} \tag{1}$$

where

- n = number of types of appliance considered for each category (WCs, urinals and taps) = 3
- ww = daily water consumption for each type of WC (litre)
- wu = daily water consumption for each type of urinal (litre)
- wt = daily water consumption for each type of tap (litre)
- x_i = number of each type of WC (integer variable)
- x_j = number of each type of urinal (integer variable)
- x_k = number of each type of tap (integer variable)
- i, j, k = counters

The daily water consumption of each appliance is computed as the product of water consumption per use and the average number of uses per appliance per day. The average number of uses per day for each appliance may vary significantly depending on the nature and functions of building in which appliances are to be housed.

Since in this paper three types of each appliance are considered, the total number of variables in the objective function is 9. To solve this form of optimization (minimization) problem, a set of 7 constraint equations is required. These are given as below:

$$\sum_{i=1}^{n} cw_i.x_i + \sum_{j=1}^{n} cu_j.x_j + \sum_{k=1}^{n} ct_k.x_k \leq Cap\_cost \tag{2}$$

$$\sum_{i=1}^{n} mw_i.x_i + \sum_{j=1}^{n} mu_j.x_j + \sum_{k=1}^{n} mt_k.x_k \leq Maint\_cost \tag{3}$$

$$\sum_{i=1}^{n} rw_i.x_i + \sum_{j=1}^{n} ru_j.x_j + \sum_{k=1}^{n} rt_k.x_k \leq Retro\_cost \tag{4}$$

$$\sum_{i=1}^{n} pw_i.x_i = TNW \tag{5}$$

$$\sum_{j=1}^{n} pu_j.x_j = TNU \tag{6}$$

$$\sum_{k=1}^{n} pt_k.x_k = TNT \tag{7}$$

$$\sum_{i=1}^{n} pw_i.x_i + \sum_{j=1}^{n} pu_j.x_j + \sum_{k=1}^{n} pt_k.x_k = TNW + TNU + TNT \tag{8}$$

where

- *cw, cu and ct* = capital cost for each type of WC, urinal and tap; respectively
- *mw, mu and mt* = maintenance cost for each type of WC, urinal and tap; respectively
- *rw, ru and rt* = retrofitting cost for each type of WC, urinal and tap; respectively
- *pw, pu and pt* = number of each type of WC, urinal and tap; respectively
- *TNW* = Total number of WCs to be installed
- *TNU* = Total number of urinals to be installed

TNT = Total number of taps to be installed
Cap_cost = Allowable capital cost
$Maint\_cost$ = Allowable annual maintenance cost
$Retro\_cost$ = Allowable retrofitting cost

In order to convert equations 2–4 into standard form, inequalities need to be removed. To do this, three slack variables (one in each inequality constraint) are introduced (Belegundu and Chandrupatla, 1999). These variables will also appear in the objective function making the total number of variables from 9 to 12 in Equation 1. Since the slack variables will have zero coefficients, they are not shown in the objective function. In order to counterbalance the increased number of variables in the objective function, we need three additional constraints as shown below.

$$\boxed{\sum_{i=1}^{n} pw_i.x_i + \sum_{j=1}^{n} pu_j.x_j = TNW + TNU} \tag{10}$$

$$\boxed{\sum_{i=1}^{n} pw_i.x_i + \sum_{k=1}^{n} pt_k.x_k = TNW + TNT} \tag{11}$$

$$\boxed{\sum_{j=1}^{n} pu_j.x_j + \sum_{k=1}^{n} pt_k.x_k = TNU + TNT} \tag{12}$$

Equations 5–12 also serve as a check to ensure that the level of service points remains unaffected. A commercially available solver based on the branch and bound method has been applied. The optimization problem algorithm has been prepared in Matlab and satisfies the standard format protocol as required by the solver.

The economic assessment module takes the core module's output and calculates annual water consumption for the base case scenario (when no new water saving appliances are fitted) and that for the technology mix obtained after optimization. Annual water savings are then converted into financial savings using per cubic metre water costs. The net financial gain (S) is obtained using Equation 13.

$$S = \textit{Annual financial saving-annual maintenance cost} \tag{13}$$

In case of negative S the optimization procedure is repeated with revised limits on cost related constraints (mainly capital costs (Figure 5)).

The present value for S obtained annually is calculated using the methodology suggested in NWC (1982). According to NWC, the annual net financial gain (S) is assumed invested at an interest rate a for m years (appliance life in this case). The present value of cumulative investment over m years (P_V) can be calculated as shown in Equation 14. Equation 15 gives the future value (F_V) of S at the end of mth year.

$$P_V = S\frac{1-(1+a)^{-m}}{a} \tag{14}$$

$$F_V = S\frac{(1+a)^{-m}-1}{a} \tag{15}$$

A retrofitting programme will be assumed financially viable if P_V exceeds I_C (i. e. sum of initial capital cost for appliances (suggested by the optimization routine) and respective costs incurred on retrofitting).

The tool developed here allows continuous estimation of optimized solutions for step increases in capital costs (within maximum cost limits) set by the tool user. This iterative approach produces a range of solutions offering different levels and water savings, possible appliance type combinations and corresponding $P_V S$. The optimum solution will be the one with the highest value of λ. Where

$$\lambda = \frac{P_V}{I_C} \tag{16}$$

The modelling approach adopted here offers a trade off between water and financial savings. To illustrate this and further clarify the tool implementation, the model has been applied to a scenario as explained in the following example.

4 TOOL IMPLEMENTATION

The financial viability and water savings that can be achieved for a medium sized commercial building have been assessed. Several options for replacing conventional high water consuming devices with water saving devices are considered. The appliances under consideration have characteristics as shown in Table 2, which also shows the levels of service required and average number of uses per appliance.

The objective of Table 2 is to illustrate the input requirements for the model rather than to present precise characteristics of different appliances currently available in the market. Therefore, values assigned to different parameters in the table should be treated as assumptions only.

In the table, *Type 3* appliances are the ones that have the lowest or no water consumption (e.g. vacuum toilets, waterless urinals and infra-red sensor fitted spray taps). Although these offer considerable water savings, they tend to have increased maintenance costs. This very aspect informs and influences the economic assessment module when computing the present value of savings.

Parameters influencing the economic assessment module and respective assumed values are shown in Table 3. The table also shows cost constraints.

The model has been applied for parameter values shown in Tables 2 and 3 and results are discussed as below.

Table 2. Appliances' characteristics used in the tool.

Appliance characteristics	WCs			Urinals			Taps		
	Type 1	Type 2	Type 3	Type 1	Type 2	Type 3	Type 1	Type 2	Type 3
Capital cost per unit (£)	150	180	500	100	120	150	80	120	150
Retrofitting cost per unit (£)	20	25	50	20	30	40	10	15	20
Maintenance cost per unit (£)	15	15	100	10	15	20	1	3	10
Water consumption per use (litre)	9	6	0	2	1	0	5	2	0.5
Total number of units required		15			20			20	
Average number of uses per day		15			50			60	

Table 3. Parameters and assumed values for the economic assessment module.

Parameter	Value
Water cost (£/m^3)	1
Life of appliance replacement programme (years)	14
Business days for the commercial building per year	250
Interest rate (%)	5
Total capital cost (minimum-maximum) (£)	7000–14000
Capital cost increment (£)	250–500
Total allowable annual maintenance cost (£)	2100
Total allowable retrofitting cost (£)	2000

Table 4. Simulation results.

Capital cost (Inc. Retr.) (£) I_C	No. of each type of toilets w_1	w_2	w_3	No. of each type of urinals u_1	u_2	u_3	No. of each type of taps t_1	t_2	t_3	Water saving (m^3/y)	P_v	λ
7470	15	0	0	20	0	0	4	16	0	720	2405	0.3
7780	15	0	0	18	2	0	0	18	2	970	4563	0.58
8065	15	0	0	19	1	0	0	9	11	1160	5870	0.73
8350	15	0	0	20	0	0	0	0	20	1350	7176	0.86
8735	15	0	0	6	14	0	0	1	19	1502	8062	0.92
9080	15	0	0	1	15	4	0	0	20	1637	8884	0.97
9410	15	0	0	2	5	13	0	0	20	1737	9478	1.07
9750	15	0	0	0	0	20	0	0	20	1850	10146	1.04
10030	*7*	*8*	0	0	0	20	0	0	20	1940	11037	1.105
10275	*0*	*15*	*0*	*0*	*0*	*20*	*0*	*0*	*20*	*2081*	*11817*	*1.15*
10585	1	13	1	0	0	20	0	0	20	2030	11086	1.05
10965	0	13	2	0	0	20	0	0	20	2064	10579	0.96
11655	0	11	4	0	0	20	0	0	20	2108	9341	0.8
12000	0	10	5	0	0	20	0	0	20	2131	8723	0.79
12690	0	8	7	0	0	20	0	0	20	2163	7486	0.59
15450	0	0	15	0	0	20	0	0	20	2356	2536	0.16

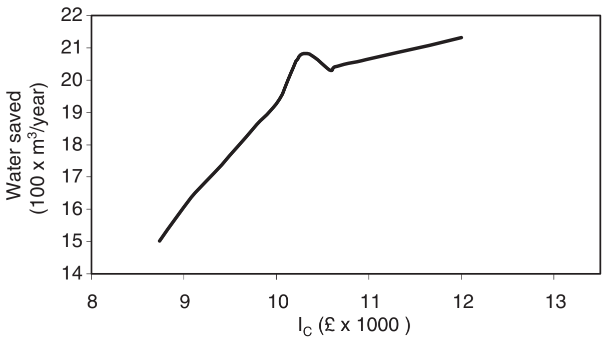


Figure 6. Impact of capital (I_C) investment on water savings.

5 RESULTS AND DISCUSSION

Table 4 is a summary of simulation results showing numbers of each type of appliance offering optimum reduction in water consumption for each cost increment. In all cases, the sum of all types in each category of appliances remains unchanged, meaning the flow reduction is achieved without affecting the required level of service. The table also shows a gradual increase in the volume of water saved annually with increasing cost. However, after a certain point the amount of water saved becomes less attractive in economic terms. This is the point of highest λ value where present value is at a maximum and well above the corresponding initial capital cost. Appliance configuration corresponding to maximum λ (in *italics*) is the optimum (trade off) solution for the given set of cost constraints, appliances' characteristics and water consumption trends. Total annual water savings and associated present values are plotted as a function of initial capital investment and are shown in Figures 6 and 7.

The inherent flexibility embedded in the tool allows investigation of a range of "what if" scenarios including the impact of variation in interest rates, appliance life and water pricing

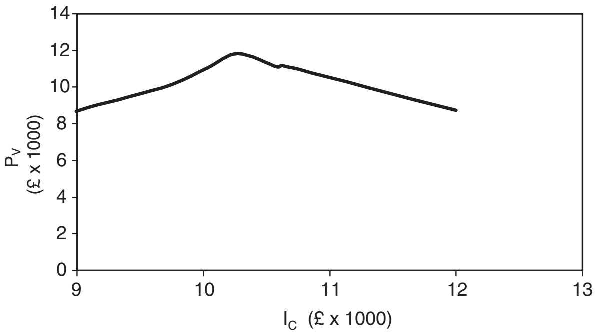


Figure 7. Impact of I_C on present value from annual financial savings.

mechanisms. Although the tool in its present form has been applied to a medium sized single commercial building, it can be used to investigate environmental and financial implications on catchment scale including viability assessment of appliance replacement programmes for residential units of identical consumption patterns.

6 CONCLUSIONS

Past studies have shown considerable water saving potential and economic benefits that can be achieved by implementing demand management measures in large buildings. Among these, probably the most effective is of low water consuming devices as they reduce consumption at source and do not require significant change in users' habits.

To assist implementation of water efficient devices retrofitting programmes, a simulation tool has been developed. The tool helps the user to investigate economic viability of different options and offers a trade off to secure water (and financial) savings within defined constraints.

For robust simulation outputs, it is imperative to have better understanding of water consumption patterns in large buildings. To address this, detailed data collection spread over a reasonable time period is required.

REFERENCES

Amy Vickers and Associates Inc (1996). *Water Conservation Planning: USA Case Studies Project*, Report to the Demand Management Center, Environment Agency, Amherst, MA: Ammy Vickers and Associates.

Belegundu, A D and Chandrupatla, T R (1999), *Optimization Concepts and Applications in Engineering*, Prentice-Hall, Inc, New Jersey.

BSRIA (1998). *Water Consumption and Conservation in Buildings*: *Potential for Water Conservation*, Building Services Research and Information Association Report No. 12586B/3.

Diziegieleski, B. and Baumann, D.D. (1992). The benefits of managing urban water demand, *Environment* 34 (9), 6–11, 35–41.

EA (2001), *Conserving Water in Buildings-Fact Cards*, National Water Demand Management Center, Environment Agency, Worthing, UK

Green, C. (2003) *Handbook of Water Economics Principles and Practice*, John Wiley & Sons Ltd, p. 246

NMOS (1999), *A Water Conservation Guide for Commercial, Institutional and Industrial Users*, New Mexico Office of the State Engineer, Water Use and Conservation Bureau.

NMOS (2001), *A Water Conservation Guide for Public Utilities*, New Mexico Office of the State Engineer, Water Use and Conservation Bureau.

NWC (1985), *An Introduction to Economic and Financial Appraisal, National Water Council (Training Division) Teaching Text 2*, National Water Council, London.

Vörösmarty, C.J., Green, P., Salisbury, J. and Lammers, R.B. (2000). Global water resources: Vulnerability from climate change and population growth, *Science*, 289(7), pp. 284–288.

Paper net present value technologies for asset appraisal of existing water utilities

C.F. Chiaves
C. di C. srl Acqua Energia Tecnologia, Turin, Italy

ABSTRACT: In Italy most structures associated with water utilities are owned by local authorities. The current privatization process implies that all structures must be transferred to new companies by means of a contribution in kind. This contribution requires a preliminary inventory and an accurate evaluation. A specific model is being presented in this paper to determine the correct value of existing water structures. When water assets lose their economic serviceability, they must be put out of duty, and no scrap value remains. Based upon this assumption, two new variables are introduced: the "wearing cost", that measures the progressive fall of serviceability, and the "wearing factor", that measures the growth rate of the wearing cost. NPV analysis leads to two final equations: an equation establishing the relationship between the remaining life of the used asset and the "wearing factor", and an equation relating the value of the asset to its age and to its wearing factor.

1 INTRODUCTION

At the end of 2001 new rules were introduced in Italy (art. 35 Legge Finanziaria 2002) in order to accelerate the process of privatisation in all network utilities. Among other regulations, the mentioned Act stated that the networks and all structures associated to a network utility must be owned and run by a business company. The ownership of the company must be only local government (private shareholders admitted only as a minority).

When the incorporation of this type of business refers to an existing utility, the local authority transfers all public properties associated to the existing service to the new company by means of a contribution in kind. Consequently the prerequisite for the incorporation is a detailed inventory and a correct appraisal of the value of all the set of infrastructures the facility consists of.

The present paper offers a mathematical model that relates usage to maintenance and gives a method to make a correct evaluation of water assets for an existing water utility.

2 OBJECTIVES

As we know the water service consists of three service components: the potable service, the wastewater collection and the wastewater treatment. All these components have their own structures such as source supply works, potable water treatment plants, pumping stations, pipelines, consumer's connections, metering system, sewerage network, wastewater treatment plants etc. We can easily calculate how much is the cost of the replacement of an existing structure. But what is its value today, assumed it is an existing, and correctly operating, but not new, structure? Can we theoretically determine when a structure is not correctly operating and when is the right time to replace it? An answer is that this "time" comes when the capital investment for replacement is minor than the savings in maintenance and in production costs of the old asset.

3 TYPICAL FEATURES OF WATER ASSETS

All typical assets used in water systems or wastewater systems have a value that is related to their function, to their serviceability. If hypothetically for any reason one of them is not capable of producing any service its value vanishes even if it is still on site. An evidence of this comes out if we observe underground pipes. When there is the need to put one of them out of duty, the old pipe is always left abandoned in the ground and does not have any scrap and realisable value: actually the cost of removing it from the soil is commonly higher then its scrap value.

4 DEFINITIONS AND ASSUMPTIONS

We will define some concepts and quantities we are going to deal with in the course of this paper. Some of them are just a recall of well-known notions in mathematics, in economics and in management science. Others are original definitions we are quoting to introduce new concepts.

In order to make the mathematical development easier we will try to treat all quantities as if they were physical quantities. Hence for each definition the dimensions of units are given.

4.1 *Time: periods of accounting n and time t*

For the variable "time", the symbol t is used: in physics the unit of time is typically the second. In this paper we can use any unit we like, with the only obligation of using always the same unit for all the quantities that are function of time. The variable p is used for the period of time during which the interest i is accrued and it is also the normal duration of an accounting period. We will use n to indicate a number of periods p from the beginning to the end of the considered interval of time t, so that it is $n = t/p$. The period p, as well as the variable t, are of dimensions 1 in time (year, month, day … second), whilst n is an integer number, and has no units.

4.2 *Interest rate i, compound interest and continuous compound interest j*

The rate i is the rate paid for the use of borrowed funds over a particular period of time p. With a compound interest capitalisation, at the end of n periods the capital becomes:

$$A_n = A_o\,(1+i)^n \tag{1}$$

According to the definition of n, we can write equation (1) as follows:

$$A_n = A_o\,(1+i)^{t/p} \tag{2}$$

When the interest is compounded continually inside each period of time p, the capital A is given by the following exponential equation:

$$A = A_o\,\mathrm{e}^{jt} \tag{3}$$

$$\text{where } j = [\ln(1+i)]/p \tag{4}$$

We have given j the name of "rate of continuous compound interest". Whilst i is a rational number and has no units, j is of dimensions -1 in time.

4.3 *Present value*

The compound capitalisation gives a solution to the problem of determining how much a given sum will be worth at the end of n periods since now. The "present value" (or better, the discounted present value) gives a solution to the opposite problem of determining how much a sum A_0 is worth "now", if it will be available (received or spent) in the future, i.e. n periods hence. From (1) we have easily the present value A_n:

$$A_n = A_0\,(1+i)^{-n} \tag{5}$$

or, according to the relationship between t and n

$$A_n = A_0\,(1+i)^{-t/p} \tag{6}$$

If expression (3) is used equation (6) becomes

$$A(t) = A_0\,(1+i)^{-t/p} = A_0\,e^{-jt} \tag{7}$$

4.4 *Asset value vs. age u: initial value A_0, actual value A(t) and value index a(t)*

When the given asset was new, the sum A_0 was spent to purchase it. The value A_0 is defined as the initial value of the asset. We make the assumption that any time we want to purchase a new asset we can find a new one with the same feature and the same price A_0. The value A_0 is of dimensions 1 in currency (Euros, pounds etc.).

The value of an asset changes with time. It starts from the beginning value A_0 and during its life decreases because of wear and tear. The actual value is a function of time t and we use the expression $A(t)$. The actual value A is of dimensions 1 in currency (Euros, pounds etc.).

We introduce also the "value index" (or simply "value") defined as the ratio between $A(t)$ and its initial value A_0. For this ratio the small letter is used: $a(t) = A(t)/A_0$.

At the beginning of its life ($t = 0$), an assets has a value index $a = 1$. At time t, *w*hen the given asset is aged u, its value index decreases as a function of age $a = a(u)$.The value index has no units, as it is a rational number.

4.5 *Assets' useful life T and number of periods N*

When assets are purchased, they are useful to a water company for a limited period of time during which its services are received. The time T from the purchase of the new asset up to its final wearing out and replacement is called "useful life". If we refer to the number n of period of time p, instead the time t, we give the name of "number of periods of life" to the ratio T/p and we use for this the symbol N. The life T is of dimensions 1 in time (years, months etc.). N is an integer number.

4.6 *Total costs C, and "Wearing-cost" W for a given asset*

Let us sum all the "costs" related to the management of a given asset at the nth period of time: operational workers, maintenance, repairs, chemicals, energy, etc. We call this sum "total cost" and indicate it with the symbol C_{pn}.

At the initial period, when the asset is new, "total cost" has a magnitude that can be indicated with the symbol C_{po}. In the following periods, as effect of the process of wear and tear, the magnitude of "total cost" increases.

The process of wear and tear also affects the contribution to the for-sale services theoretically associated to the given asset and causes a drop in production: losses of pumps capacities, reduced water flows due to scale built in pipes. These losses must be converted into equivalent currency values and then treated as costs and added to the operational and maintenance costs to form the "total cost".

For our purposes, we are interested to the increasing process of the expenditure, i.e. the difference between the "total cost" C_{pn} affecting the asset at the nth period and its initial magnitude C_{po}. We give this difference the definition of "wearing cost" and we use the symbol W_{pn}:

$$W_{pn} = C_{pn} - C_{po} \quad \text{or} \quad C_{pn} = C_{po} + W_{pn} \tag{8}$$

We will consider the "wearing cost" a continuous function of time t and use the term $W(t)$ instead of the term W_{pn}. Also the total cost C_{pn} will be considered a function of time t and equation (8) becomes:

$$C(t) = C_o + W(t) \tag{9}$$

Quantity C_{pn} and quantity W_{pn} are of dimensions 1 in currency (Euro, Pound etc.), whilst $C(t)$, $W(t)$ are of dimensions 1 in currency and –1 in time (e.g.: Euros per day, Euros per year etc.).

4.7 *Relative wearing cost w(t), cumulative and present values for the wearing*

As already done for the value of the asset $a(t)$, the wearing out W can be expressed as a ratio between the wearing currency value $W(t)$ and the initial value A_0 of the asset itself. For this quantity we will use the small letter: $w(t) = W(t)/A_0$. The variable $w(t)$ has the favourable feature of being of dimensions –1 in time, the same as the rate of continuous compound interest j. We will call w simply "wearing", and the adjective "relative" is omitted, when not necessary.

Our analysis needs as well the definitions of the cumulative value of the wearing x and also the "present" value for the wearing y and for its cumulative value z. The definitions are:

cumulative (relative) wearing $x_{pn} = \Sigma_{on}\, w_{pn}$
present (relative) wearing $y_{pn} = (1 + i)^{-n}\, w_{pn}$
cumulative present (relative) wearing $z_{pn} = \Sigma_{on}\, (1 + i)^{-n}\, w_{pn}$

If the summation symbols are substituted by an integral sign the above quantities become:

cumulative (relative) wearing $$x(t) = \int_0^t w(t)\mathrm{d}t \tag{10}$$

present (relative) wearing $$y(t) = w(t)\, \mathrm{e}^{-jt} \tag{11}$$

cumulative present(relative) wearing $$z(t) = \int_0^t w(t)\, \mathrm{e}^{-jt}\, \mathrm{d}t \tag{12}$$

5 NET PRESENT VALUE METHODOLOGIES

If hypothetically a water structure is not affected by any wearing, it has always the same value as a new one: to match these limit features an asset must keep unchanged performances, production capability, operations and maintenance costs.

As a matter of fact "wear and tear" occurs over time: the asset deteriorates, it falls off in efficiency and requires increased expenditures on repairs and maintenance. For all this, we have used the term "wearing cost", represented by the symbol W_{pn}. Our target is to determine correctly the lifetime T of a given asset and the variation of its value in the extent of its life. Therefore we can forget the fraction of operations and maintenance costs that does not change over time and focus our attention on the cost component W_{pn}.

In order to provide a relationship between W_{pn} and the lifetime of the asset, we will adapt the Net Present Value methodologies. This method leads to a comparative analysis between the investment expenditure A_0 for replacement and the present value of costs (or sacrifices), when all costs are discounted back to the present time. This is done assuming a rate of interest i.

Suppose we have decided replacements occur at the end of all life cycles of duration T the condition that makes the replacement worth is that the (discounted) difference between the two cumulative wearing costs over the next cycle T be equal to the investment expenditure A_0. The equations, which fulfill this condition are:

$$\boxed{\int_0^T W(\tau)e^{-jt}\mathrm{d}t - \int_0^T W(t)e^{-jt}\mathrm{d}t - A_0 = 0} \tag{13}$$

$$\boxed{\int_0^T w(\tau)e^{-jt}\mathrm{d}t - \int_0^T w(t)e^{-jt}\mathrm{d}t - 1 = 0} \tag{14}$$

where $\tau = T + t$.

Equation (13) or (14) states the condition under which the value of a given asset vanishes and its useful life has reached its end. The particular value T at which t solves equation (13) or (14) is the "useful life time" of the given asset. If we know the time rate of change of the wearing-cost, i.e. the function $w = w(t)$ that relates w to t, equations (13) or (14) can be solved and will provide the solution T.

6 DEFINING THE TIME RATE OF CHANGE OF THE WEARING-COST

Starting from their construction and going on over all their life, corrosion and water or soils action affect pipes. Rust and lime scale build-up progressively reduces the pipe cross section and affects the management cost. All these factors are depending on use and can be classified in what we have called "wearing-cost". Similar discussion can be done for cast iron pipes, pumps, electrical equipment, meters, pressure filters etc. The use of all these items in water industry is approximately uniform over time, consequently the wearing w can be assumed proportional to time and the relationship comes in the simple form:

$$w = k\,t \tag{15}$$

The suggestion for this extreme simplification comes from our experience on water system management. There is no reason to use involved expressions for the function $W = W(t)$ or $w = w(t)$. We call equation (15) "simple linear wearing".

For all water assets equation (15) can be assumed as correct enough. We have called k "wearing factor". It should be determined by a direct investigation according to the operational conditions and actual use. The factor k is of dimensions -2 in time and w is of dimensions -1 in time.

In table 1 an example of factor k determination is carried out for a steel 4-inch water main, 600 meters long, 25 years old. The cost of construction with conventional method has been evaluated Euros 110,000.00 inclusive of 40 connections, digging and pavement reconstruction. Two annual

Table 1. Determining the magnitude of wearing factor k for a 25 years old 4" steel main.

Item	Symbol	Unit	No.	Unit cost Euros	Results
Cost of replacement	A_0	Euros			110,000.00
Number of periods (age)	N		25		
Duration of each period	p	year	1		
Time	$t = Np$	year	25		
Wearing cost (initial period)	W_0	Euros/year			
Wearing cost (current period) = repairs	W_t		2	950.00	1900.00
Relative wearing cost (current period)	$w_t = W_t/A_0$	year^{-1}			1.73%
Growth factor of wearing cost	$k = w_t/t$	year^{-2}			6.9×10^{-4}

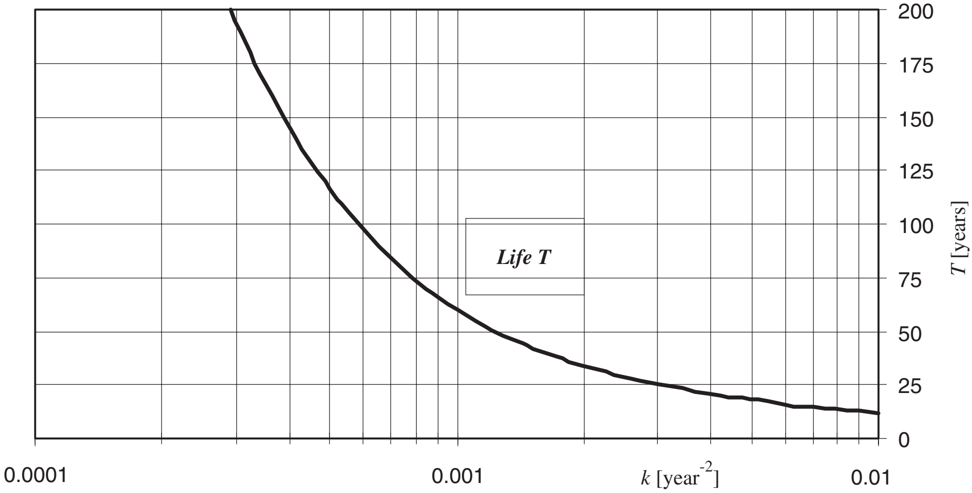


Figure 1. Asset useful life T vs. wearing growth factor k ($i_{year} = 6\%$; $j = 0.0583$ year^{-1}).

leaks repairs have been supposed (that means a heavy condition of deterioration). The results is $k = 0.00069$ years^{-2}.

7 CALCULATING THE USEFUL LIFETIME

The equations that give the rate of wearing with time allow us to determine the useful lifetime of an asset by a simple deduction. If we substitute (15) in equation (14), we obtain:

$$\int_0^T k \cdot (T+t)\mathrm{e}^{-jt}\mathrm{d}t - \int_0^T k \cdot t\, e^{-jt}\mathrm{d}t - 1 = 0 \qquad \text{or} \qquad kT\int_0^T e^{-jt}\mathrm{d}t - 1 = 0$$

which can be solved with respect to k, and indirectly yields the lifetime T:

$$k = \frac{j}{T(1-\mathrm{e}^{-jT})} \tag{16}$$

In the graph of figure 1, T has been plotted vs. the growth factor k. The graph gives the duration that makes the replacement "economically necessary".

For the example of 4" steel main table 1 has given a wearing factor $k = 6.9 \times 10^{-4}$ year^{-2}. The graph of figure 1 gives $T = 85$ years. This means that the assumed heavy conditions for the 25 years aged pipe of the example are non sufficient to make the replacement worth.

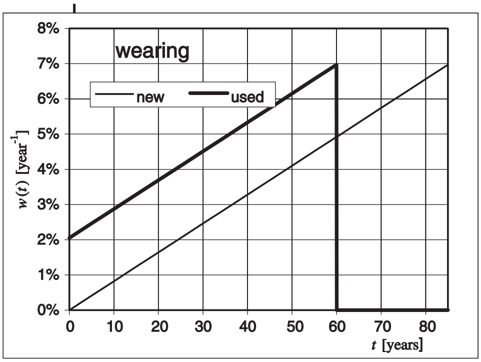
If analyses are carried out, like the one outlined in table 1 for a steel main, k factors can be found for other typical water assets and the corresponding lifetime T can be determined by means of figure 1 graph. Table 2 shows that factor k and time T varies in a wide range.

8 USED ASSET EVALUATION

The asset value, that initially was A_0, vanishes when getting at that final time T. Now we want to find the actual value of the asset at "any" time t between the initial time 0 and the final time T.

Table 2. Examples of wearing growth factors k and lifetime T as resulting from equation (16) ($i = 6\%$).

Type of water asset	Wearing factor k Year^{-2}	Wearing factor k Month^{-2}	Useful life T years
Steel pipes	6.9×10^{-4}	5.8×10^{-5}	85
Cast iron pipes	5.6×10^{-4}	4.6×10^{-5}	105
Pump stations	2.9×10^{-3}	2.4×10^{-4}	26
Polyethylene pipes	5.1×10^{-4}	4.3×10^{-5}	114
Meters	5.0×10^{-3}	4.15×10^{-4}	18
Pressure filters	1.8×10^{-3}	1.54×10^{-4}	36
Electrical transformers	3.0×10^{-4}	2.53×10^{-5}	25



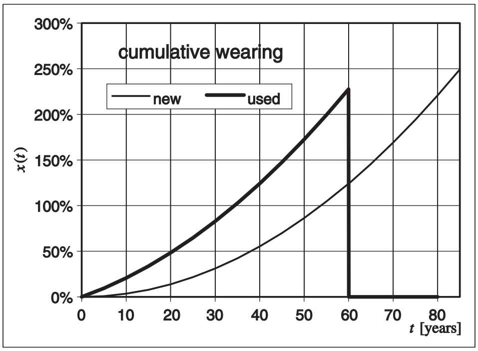


Figure 2. Steel pipe new vs. 25 years old. Wearing w (left) and cumulative wearing x (right).

8.1 *Approximated methods*

A rough method of calculating the value of a used asset could be a simple proportion to the "remaining time of life" i.e.:

$$\boxed{A(t) = \frac{T-u}{T} A_0 \qquad \text{or} \qquad a(t) = \frac{T-u}{T}} \tag{17}$$

where A = value; a = value index; u = age; T = useful life.

If the "remaining time ratio" r is introduced, equation (17) assumes this elementary form:

$$a = r \tag{17b}$$

where r =ratio between the residual life of a used asset and the full life of a new one.

For the example of table 1 Equation (17) gives: $35/85 \times 110{,}000.00 = 77{,}647.00$ Euros.

The simple equations (17) and (17b) have the appreciable feature of being very simple, but they are a first approximation. They do not take into account that the operations and maintenance costs are not constant over the asset lifetime: consequently the drop of the asset value may not be considered evenly distributed over the lifetime. The graphs of figure 2 show (left) the difference between the wearing w for a new steel pipe and for a 25-years-old-steel pipe.

Figure 2 (right) shows the cumulative wearing by integration of equation (10):

$$\boxed{\text{new asset} \quad x(t) = \int_0^t k \cdot t \, \mathrm{d}t \qquad x(t) = \tfrac{1}{2} k\, t^2} \tag{18}$$

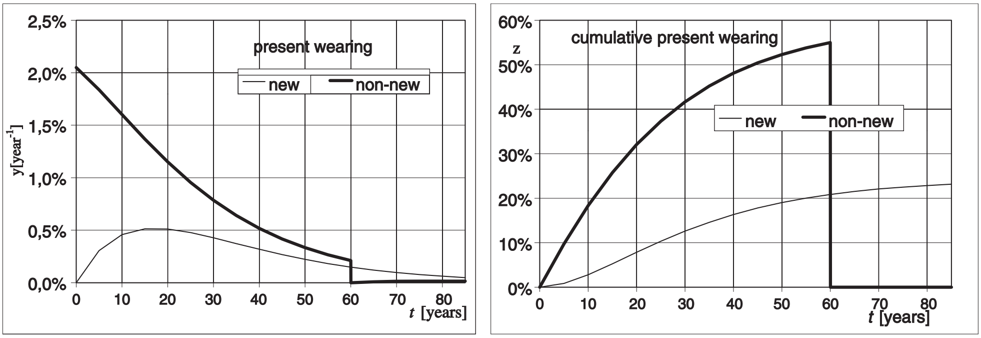


Figure 3. Steel pipe: new vs. 25 years old. Present wearing (left) and cumulative present wearing (right).

$$\text{used asset} \quad x(t) = \int_0^t k \cdot (u+t)\,\mathrm{d}t \qquad x(t) = \tfrac{1}{2}\,k\,t^2 + k\,u\,t \tag{19}$$

8.2 *Present value analysis*

Calculations done with the above summation of the wearing costs does not take into account the time the singles expenditures occur. Amounts spent at different points in time must be reduced to equivalent terms by means of the present-value-procedures presented in section 4.3. After reduction, we can sum them up to give comparable total sums. The equation (15) of simple linear wearing combined with equation (11) gives the following expressions for y:

$$\text{new asset} \quad y(t) = \mathrm{e}^{-jt}\,k\,t \tag{20}$$

$$\text{used asset} \quad y(t) = \mathrm{e}^{-jt}\,k(t+u) \tag{21}$$

where u = age of the asset.

For the examples of the steel pipe presented in the previous sections, the graphs of $y(t)$ have been plotted in figure 3 (left) according to equations (20) and (21).

The right side of figure 3 shows the graphs of cumulative present wearing for the same example of a new pipe vs. a 25 years old pipe. The graphs have been plotted by means of the integral equation (12) in which $w(t)$ has been substituted by the simple linear wearing as given by equation (15). For an asset that has started its depreciation process when its initial value was the purchase value A_0 (new asset), the cumulative present wearing is:

$$z_0(t) = \int_0^t k t \mathrm{e}^{-jt}\,\mathrm{d}t \tag{22}$$

The integration gives the solution:

$$z_0(t) = k/j^2\,[1 - \mathrm{e}^{-jt}\,(j\,t + 1)]. \tag{23}$$

Changes are necessary to equation (22) for a used asset:

$$z_u(t) = \int^t k \cdot (u+t)\mathrm{e}^{-jt}\,\mathrm{d}t \tag{24}$$

Table 3. List of capital and management expenditures for new and used asset (net present values).

	New asset	Used asset
Purchase of the asset at time $t = 0$	A_0	$a(u)A_0$
Cumulative wearing over time $0\ldots T$	$z_0(\mathrm{T})\,\mathrm{A}_0$	
Cumulative wearing over time $0..T-u$		$z_u\,A_0$
Replacement at time $T-u$		$e^{-j(T-u)}A_0$
Cumulative wearing over time $T-u\ldots T$		$z_r\,A_0$
Realize of used asset at time T		$a(u)\,e^{-jT}A_0$
Total present costs (in currency terms)	$A_0 + z_0(T)\,A_0$	$\mathrm{a}(u)A_0 + z_u\,A_0 + e^{-j(T-u)}\,A_0 + z_r\,A_0 + a(u)\,e^{-jT}\mathrm{A}_0$
Total present costs (in relative terms)	$1 + z_0(T)$	$a(u) + z_u + e^{-j(T-u)} + z_r + a(u)\,e^{-jT}$

where u = asset age at the initial time $t = 0$. The integration of (24) gives:

$$\boxed{z_u(t) = z_0(t) + \frac{ku}{j}(1 - e^{-jt})} \tag{25}$$

8.3 *Calculating the value of a used asset: correct method*

Let us consider the sequence of events that affect management and capital expenditures during the whole cycles of life T in the two conditions: new asset and used asset aged u. Table 3 lists all expenditures discounted back to the present time.

The unknown index $a(u)$ is the one that fulfils the condition that for the new asset cycle the total present costs be equal to the one for the used asset cycle (see table 3), i.e.:

$$1 + z_0(T) = a(u) + z_u + e^{-j(T-u)} + z_r - a(u)\,e^{-j(T-u)} \tag{26}$$

where $z_0(T) = k/j^2\,[1 - e^{-jT}\,(j\,T+1)]$ (see equation 23), whilst z_u, z_r are:

$$\boxed{z_u = \int_0^t k\cdot(u+t)\mathrm{e}^{-jt}\mathrm{d}t = z_{T-u} - \frac{ku}{j}\mathrm{e}^{-jT}} \tag{27}$$

$$z_r = \int_{T-u}^{T} k\cdot(t-T+u)\mathrm{e}^{-jt}\mathrm{d}t = z_0 - z_{T-u} - \frac{k}{j}(\mathrm{e}^{-jT} - \mathrm{e}^{-jT(T-u)})(T-u) \tag{28}$$

Substitution of (27), (28) in equation (26) gives:

$$\boxed{a(u) = \frac{1 + \frac{ku}{j}(\mathrm{e}^{-jT} - 1) + \frac{kT}{j}\mathrm{e}^{-jT}(\mathrm{e}^{-jT} - 1) - \mathrm{e}^{-jT}}{1 - \mathrm{e}^{-jT}}} \tag{29}$$

which eventually can be written in this simpler form:

$$\boxed{a(u) = \frac{(\frac{k}{j}T - 1)}{1 - \mathrm{e}^{-jT}}(\mathrm{e}^{-jT} - 1) + \frac{k}{j}Tr} \tag{30}$$

where r = remaining lifetime ratio $(T-u)/T$.

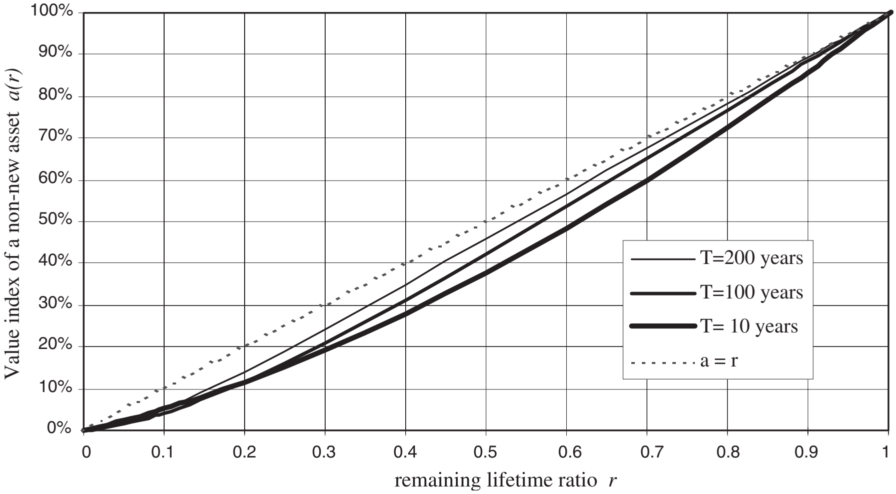


Figure 4. Value index of used assets as function of remaining life ratio r ford different duration of life T (interest rate $i_{\text{year}} = 6\%$).

Equation (30) has been plotted in figure 4: the value index of the asset a is expressed as a function of remaining life ratio r for typical values of the full life of the asset T.

It will be noted that the influence of the full life T is not great and the graphs do not vary appreciably even for reasonable variations of the life T. For $T \rightarrow \infty$ the curves approximate to the expression $a = r$, represented in the figure with the dotted line.

9 CONCLUSIONS

The proposed model for the evaluation of used water assets consists of three steps:

1. Determining the "wearing cost" W, and the "wearing factor" k i.e. the rate of increasing of annual costs due to wear and tear; as it comes from management and accounting records.
2. Calculating the duration of the useful life of a new asset T, by means of equation (16) which relates T to k.
3. Calculating the relative value of the given used asset a, by means of equation (30) or equation (29), as a function of the variables T, u and k.

The first step concerning the "wearing cost" W must be carried out for the various types of assets by a direct investigation on the operational conditions, serviceability and actual use of the given asset.

As far as the second step is concerned, it has been shown that the process of deterioration, which occurs time by time in water networks is very low for most of the items. As a consequence the number of years a water asset can continue to work profitably before replacement is longer then usually believed. The useful lifetime strongly influences the mathematical model and must be calculated by means of a correct method. The proposed method seems to be accurate enough as a first approach to this problem.

For the third step, the proposed mathematical formulation gives the value of a used asset as a function of both the effective asset age and the rate of management costs increase.

In all this paper the interest rate i has been considered constant, and the value of 6% has been assumed as a fear possible value for the present days. The choice of this term is of fundamental importance because it seriously affects all results. It is obvious that if the cost of currency decreases the feasibility of an investment will increase.

As a conclusion, when existing structures and facilities are to be transferred to a new owner, they must be evaluated by an accurate scientific method, which implies collection of data records, serviceability analysis, and a correct estimate of its remaining useful life. Any methods by which the value of a new asset is roughly reduced to determine the value of the used asset, must be rejected because this often results in arbitrary values where the assets are underestimated.

REFERENCES

Jacquet, J.L. & Miller, W.C. 1992. *The accounting cycle: a practical guide to accounting basics*, Crisp Publications, Inc.

Meigs, W.B. & Johnson C.E. 1977. *Accounting: the basis for business decisions*. McGraw Hill Book Company.

Thelma T. Increasing privatization: a global phenomenon. In *Water & Wastewater International*, February 2000.

Patrick R. & Russel L.A. 1963. A manager's guide to operational research, J. Wiley.

Comitato Vigilanza Servizi Idrici. Relazione annuale al Parlamento sullo stato dei servizi idrici anno 2001, Rome July, 2002.

LIST OF SYMBOLS

Symbol	Definition	Unit
t	Time as a physical concept	time
p	Accounting time period	time
i	Rate of interest	number
j	Continuous rate of interest	time^{-1}
n	Number of periods	number
A_0	Initial value: sum invested or to be invested, or purchase cost of an asset	currency
$A(t)$	Actual value (appraisal value) of used asset as a function of time t	currency
$a(t)$	Value index of used asset as ratio between value and its purchase cost	number
N	Useful life of an asset in terms of number of periods	number
T	Useful lifetime of an asset in terms of real time (months, years etc.)	time
C_{pn}	Total operations and maintenance costs at the nth period p	currency
$C(t)$	Total operations and maintenance costs as a function of time t	currency time^{-1}
W_{pn}	"Wearing cost" of a given asset recorded during the nth period p	currency
$W(t)$	"Wearing cost" of a given asset occurring at time t	currency time^{-1}
$w(t)$	(relative) wearing $w = W/A_0$ occurring at time t	time^{-1}
$x(t)$	(relative) wearing present value occurring at time t	time^{-1}
$y(t)$	Summation over all (relative) wearing-values since time 0 to time t	number
$z(t)$	Summation over all (relative) wearing-present-values since time 0 to time t	number
k	Wearing factor, time rate of growth for W (or w)	time^{-2}
u	Age of a used asset	time
r	Remaining life time ratio $r = (T - u)/T$	number
ln	Natural logarithm	

Key words: appraisal, asset value, net present value, replacement, used assets, useful life, water assets, wear and tear, wearing cost, whole life costing.

Cost analysis of alternative scenarios for drinking water supply in remote Quebec communities

A. Mailhot & J.-P. Villeneuve
INRS-Eau, Terre et Environnement, Sainte-Foy, Québec, Canada

ABSTRACT: Many isolated communities in northeastern Quebec, commonly referred to as the Côte-Nord (North Shore) region, face serious drinking water supply problems. Although water is abundant in this region, it is of such poor quality that filtration facilities will be required to ensure potable water comply with the recently adopted provincial regulation. The cost of building and operating these units is a major hurdle, considering the limited financial resources of the local economy. This study examines alternative methods for supplying drinking water to the region, such as transporting water from neighbouring treatment sites. A cost analysis comparison of transportation versus technological solutions (local treatment facilities) is presented in this article. The analysis involves seven communities, with populations ranging from 22 to 479. Other important issues related to operation, staff availability, maintenance and affordability of these solutions are also discussed.

1 INTRODUCTION

The government of Quebec has recently adopted new Drinking Water Quality Regulations (DWQR) (Government of Quebec 2001). Municipalities have until June 2005 (populations of 50,000 or less) or June 2007 (populations over 50,000) to comply with these new regulations. The DWQR includes provisions for a mandatory filtration treatment system for surface water that does not meet specific requirements many of which related to turbidity and Total Organic Carbon (TOC) concentrations (Coulibaly & Rodriguez, 2003). This obligation poses a real challenge to small isolated municipalities in Quebec. This is the case for the Québec's North Shore region where the vast quantity of surface water available has high TOC concentrations. Other regional characteristics further complicate the region's drinking water supply and distribution situation: extremely cold winters; thin layer of ground soil (water pipes are not buried deeply and therefore freeze in winter); non-existent road link-up between certain villages; and scarcity of financial resources to fund municipal infrastructures.

This article presents the findings of a study examining alternative solutions to drinking water supply and distribution. The solutions consider the possibility of transporting drinking water from neighbouring production or supply sites. In this context, "beverage-quality" drinking water (delivered to points in homes where it can be ingested) is supplied by transportation and stored in homes, whereas non "beverage-quality" water (delivered to other points for other uses) is distributed via a water network (Cotruvo & Cotruvo Jr. 2003; Cotruvo 2003). These solutions are compared to more "traditional" approaches, i.e. construction of surface water treatment facilities, from where water is distributed via a distribution network. An implementation/operation cost analysis of these solutions was also conducted. The study involved seven villages.

2 WATER SOURCES AND SUPPLY IN THE NORTH SHORE REGION

Quebec's North Shore region is located in the eastern part of the province, extending from Baie Comeau to Labrador on the north shore of the St. Lawrence estuary. Figure 1 shows a location map

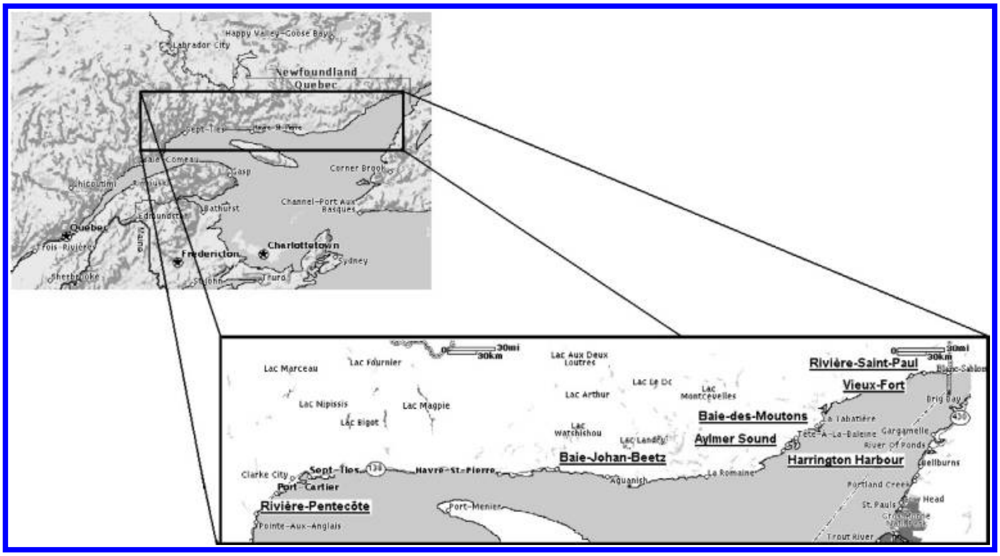


Figure 1. The North shore region showing the location of seven villages examined in this article (underlined names).

of the seven villages examined in the study. From west to east, they are: Rivière-Pentecôte, Baie-Johan-Beetz, Harrington Harbour, Aylmer Sound, Baie-des-Moutons, Vieux Fort and Rivière-Saint-Paul. A distance of 750 km separates Rivière-Pentecôte and Rivière-Saint-Paul. As the road ends a short distance past Baie-Johan-Beetz there is no ground access to villages located east of Baie-Johan-Beetz. These villages receive supplies by boat, except during winter months. Roads within villages and those connecting communities are snow-covered during winter months (usually from November to April). Snowmobiles are the only means of transportation during the cold season.

Table 1 details the principal characteristics of the current drinking water supply and distribution systems in these villages. However, a few explanatory comments are necessary. All utilities identified in Table 1 are supplied by surface water, except Baie-des-Moutons where a well was dug close to the village. Water sources include inland ponds and rivers. However, none of these villages draws water from the St. Lawrence gulf because of its high salt content (salted water). A number of attempts have also been made to locate groundwater. The presence of unfractured rock over large areas prevents water from penetrating the surface, thereby explaining the lack of success in finding groundwater.

As Table 1 shows, many of these distribution networks have common features. Most of the networks are poorly-looped or unlooped, which for a small population means water sometimes remains in pipes for long periods, thereby compromising water quality. Many of these villages, in fact, have experienced recurrent microbiological contamination problems in past years.

Another feature is the very thin loose layer of top soil in these regions and the fact that the ground surface is comprised primarily of rock. For this reason, water pipes are difficult to bury and some are even at ground level. During winter's very cold temperatures water pipes inevitably freeze. Moreover, in summer, water temperatures in pipes rise to unacceptable levels, which can compromise water quality. To offset these problems, some villages have installed systems that maintain continuous water flow at network end points. Baie-Johan-Beetz has even installed two tanks that heat water to approximately 3°C before it enters the network.

Among these utilities, Harrington Harbour merits particular attention due to certain unusual characteristics. The village is located on an island (that is almost entirely rock) in the St. Lawrence gulf a few kilometres off-shore. Two natural basins in the upper part of the island have been modified to serve as a drinking water reservoir. The surface area of the watersheds discharging into these basins limits the overall capacity. The distribution system is comprised of two water mains. Only a few homes are

Table 1. The primary characteristics of the current drinking water supply and distribution systems in the seven villages examined in this study.

Village	Population supplied by public network[1]	Distribution network	Current treatment
Rivière-Pentecôte	96	– serves 16% of the population – low pressure problem – unlooped – water pipes not buried deeply	– chlorination
Baie-Johan-Beetz	87	– water pipes not buried deeply – water-heating system averts winter freezing	– chlorination
Harrington Harbour	320	– water pipes not buried deeply – most homes unconnected to network	– low pressure filter – UV – chlorination
Aylmer Sound	22	– low pressure problem	– UV
Baie-des-Moutons	– a few houses and the school.	– no network (the school and a few homes are supplied by a "private" network). – a well dug at the entrance of the village; inhabitants have access to very high quality underground water	– no treatment (sur face water)
Vieux-Fort	323	– weakly looped – low pressure problem – water pipes not buried deeply	– chlorination
Rivière-Saint-Paul	479	– weakly looped – low pressure problem – water pipes not buried deeply	– chlorination

[1]Number of inhabitants currently connected to the network. The remaining population has private wells or, as in the case of Baie-des-Moutons, rain collection systems.

hooked up to the distribution network. Other homes have basement storage facilities. Many of them use swimming pools. Pumps circulate water in the home. Water is pumped in to these residential storage facilities on a bi-weekly basis through hoses connected to the water main valves.

3 ALTERNATIVE SCENARIOS FOR DRINKING WATER SUPPLY

Under the new Quebec DWQR, TOC and turbidity levels in surface drinking water sources in all villages are too high to forego installing filtration facilities. One technological approach currently being considered is nanofiltration (NF), which is well-suited to small-scale production units. However, it is evident that beyond compliance with DWQR treatment requirements, major improvements will be necessary in order to solve the reported problems. This will especially involve upgrading and completing water pipe networks. This analysis only addresses the issue of treatment.

Important issues must be considered when seeking a solution to drinking water supply in remote areas. The first pertains to the ability of local economies to support such infrastructures. Although building costs are primarily assumed by the central government, maintenance and operating costs (much higher in isolated areas) are charged to the local community. The second is availability of qualified labour to operate treatment facilities, which may be problematic in the case of a technological solution such as nanofiltration.

Alternative approaches that are less costly than conventional approaches may be sought for providing safe drinking water. The alternative scenarios considered in this study are based on the idea that domestic water volumes can be separated into two components according to use (Cotruvo &

Table 2. Description of the four scenarios examined in the study.

Scenario	Description
A	Tank truck delivery to homes from a neighbouring production or supply site; each home has a drinking water storage tank.
B	Truck delivery of bottled water to homes. Water bottled at a neighbouring production or supply site; each home has a drinking fountain.
C	Tank truck delivery to homes from a small local production plant; each home has a drinking water storage tank.
D	Truck delivery of bottled water to homes. Water bottled at a local production site; each home has a drinking fountain.

Cotruvo Jr. 2003; Cotruvo 2003). First, tap water which is meant for drinking and cooking purposes. Second, water used for non-drinking purposes (showers, toilet, washing machines, etc.). This distinction between drinking and non-drinking use of domestic water is not considered in Quebec's DWQR which requires that all distributed water must be of "beverage-quality". Having made this distinction, alternative scenarios have been developed involving drinking water transported from production or supply sites to homes. Non-drinking water is delivered through the water distribution network, assuming it has an acceptable microbiological quality, but no filtration treatment is required due to its use. Table 2 describes the four scenarios examined in this study.

These scenarios entail two types of in-house storage (tank and bottles) and two types of water supply (neighbouring production or supply sites and local small treatment plant). The capacity of the local treatment plant is sufficient to meet drinking water needs only.

Scenarios in which all domestic water is transported and delivered to homes have also been considered for Rivière-Pentecôte and Baie-Johan-Beetz. This type of distribution system is currently used in the 14 Kativik Regional Administration villages, located in northern Quebec. Tank trucks (6,800–13,600-litre capacity) are filled with water at neighbouring lakes or rivers and water is delivered to each home. Each house has tanks (1,136-litre capacity) to store water, the exact number depending on how many individuals live in the building. A light on the front of each house indicates to the driver if a water delivery is necessary. Details of this scenario's mechanics in Rivière-Pentecôte and Baie-Johan-Beetz can be found in Villeneuve et al. 2003.

The feasibility of scenarios A and B depends on availability of good quality water at a neighbouring site. If it is unavailable, then scenario C or D should be considered. The results of a preliminary screening based on these considerations appear in Table 3. The Table also details the daily required drinking water volumes for each village estimated on the basis of 10 litres/day/person. The total volume estimates were increased 10% to take institutional and commercial drinking water demand into account.

Scenarios A and B were considered for Rivière-Pentecôte and Baie-Johan-Beetz since the production capacity of nearby towns is sufficient to meet their drinking water requirements. The town of Port-Cartier, located 35 km east of Rivière-Pentecôte, was identified as a potential supply site for Rivière-Pentecôte. A treatment plant with a 2700 m^3/day capacity was built in this community in 2001. The municipality of Havre-Saint-Pierre, located 65 km west of Baie-Johan-Beetz, could supply drinking water to this village. Five wells supply Havre-Saint-Pierre with water. A volume of approximately 2,045 m^3/day is currently distributed.

3.1 *Hypotheses and parameters of each scenario*

To tabulate the costs of implementing and maintaining systems identified in each of these scenarios, a number of parameters and hypotheses must be defined, namely those pertaining to: i) supply and delivery of drinking water to homes; ii) vehicles used to supply and distribute drinking water; iii) in-home water storage facilities; iv) municipal storage and vehicle maintenance facilities; v) small-scale treatment facilities (scenarios C and D); vi) necessary staff; and vii) water purchase

Table 3. Estimated daily volume of drinking water and feasibility of scenarios.

Village	Estimated daily volume (m^3)	Scenario A	Scenario B	Scenario C	Scenario D
Rivière-Pentecôte	1.0	Yes	Yes	Yes	Yes
Baie-Johan-Beetz	1.0	Yes	Yes	Yes	Yes
Harrington Harbour	3.3	No (limited transport facilities)	No (limited transport facilities)	Yes	Yes
Aylmer Sound	0.24	Yes	Yes	Yes	Yes
Baie-des-Moutons	1.8	Yes	Yes	Unnecessary (well meets drinking water demand)	Unnecessary (well meets drinking water demand)
Vieux-Fort	3.6	No (no nearby supply sites, limited ability to transport in winter)	No (no nearby supply sites, limited ability to transport in winter)	Yes	Yes
Rivière-Saint-Paul	5.3	No (no nearby supply sites, limited ability to transport in winter)	No (no nearby supply sites, limited ability to transport in winter)	Yes	Yes

(scenarios A and B). The sections below provide details on the hypotheses and parameters used to conduct the cost analysis.

3.1.1 *Supply and delivery of drinking water to homes*

Vehicles would deliver drinking water Monday through Friday. Water would also be transported from the supply site to the village (scenarios A and B) Monday to Friday. We assume that each home in the village requires water every two days. An identification system, such as placing a notice in a window, would indicate when a home requires water delivery. In the case of Aylmer Sound, given that the village is completely isolated and can only be reached by boat in summer or snowmobile in winter, storage facilities have been planned to ensure water supply in the event of poor weather.

3.1.2 *Drinking water supply and distribution vehicles*

As indicated above, various vehicles must be considered. Table 4 details the vehicles required to supply water to the villages studied. Non-traditional vehicles must be considered since some villages (Aylmer Sound, Harrington Harbour) have wooden sidewalks where only All Terrain Vehicles (ATV) can travel. Snow remains on roads and streets throughout the entire winter in all villages, except in Rivière-Pentecôte and Baie-Johan-Beetz, so the only method of travel is snowmobile.

The reasoning used to estimate the number of vehicles was that five houses could be visited in one hour, tank filling would take 30 minutes on average and, where necessary, the distance between the supply site and the village would be taken into account. Table 5 details the estimated costs to purchase equipment. Maintenance and operating costs are also presented in Table 5. All costs are given in Canadian dollars. These values are based on actual retail prices and estimates provided by retailers and equipment owners.

3.1.3 *In-house water storage facilities*

Scenarios A and C assume a storage facility will be installed in each home and that there are no space restrictions. The facility cannot be installed outdoors due to cold winter weather. The holding

Table 4. Type of vehicles available in each village to supply and distribute drinking water.

Village	Vehicles used to supply and distribute water
Rivière-Pentecôte	Minivans equipped with cooling tank
Baie-Johan-Beetz	Minivans equipped with cooling tank
Harrington Harbour	ATV with tank trailer and snowmobiles with tank trailer (winter)
Aylmer Sound	ATV with tank trailer and snowmobiles with tank trailer (winter)
Baie-des-Moutons	Minivans equipped with cooling tank and snowmobiles with tank trailer (winter)
Vieux-Fort	Minivans equipped with cooling tank and snowmobiles with tank trailer (winter)
Rivière-Saint-Paul	Minivans equipped with cooling tank and snowmobiles with tank trailer (winter)

Table 5. Costs to purchase, maintain and operate vehicles.

Vehicles	Purchase cost	Annual maintenance cost	Operating cost
Minivans equipped with cooling tank	\$75,000	\$4,000	\$0.25/km
ATV equipped with tank trailer	\$20,000	\$2,000	\$0.13/km
Snowmobile equipped with tank trailer	\$20,000	\$2,000	\$0.10/km

capacity of each storage facility must be sufficient to supply the residents of each house for five days. Storage units with a 100-litre capacity were considered. The total number of units was established on the basis of the above hypothesis.

The estimated average cost for purchase and installation of each storage unit in scenarios A and C is \$2,500. This price was quoted by a company currently providing this type of service, plus an additional 25% to take into account the remote location of the communities.

Under scenarios B and D, bottled water is delivered. A water fountain must therefore be installed in each home. Each unit sells for \$240 in the Quebec City region (retail price). An additional 25% was added to the price to take into account transportation costs to the North Shore region. This brings the purchase cost to \$300/unit, while installation cost was assumed to be negligible.

3.1.4 *Municipal building to store and maintain vehicles*

We assume no space is currently available to accommodate the additional vehicles required to supply and distribute water. Existing buildings must be expanded or new structures built. The additional surface area was estimated on the basis of the type and number of vehicles required to supply and distribute drinking water. The estimated unit construction cost for these buildings is \$1.57 /m^2. Annual maintenance and operating costs were established as 5% of the construction cost.

3.1.5 *Small-scale treatment facilities (scenarios C and D)*

These small-scale units would be installed in the village to treat surface water for drinking water purposes. Their capacity should be adapted to the estimated drinking water demand. Port-Cartier (35 km east of Rivière-Pentecôte) was used as a test site for this scenario. In 1995, a membrane filtration unit was installed in that town, with a system capacity of 23 m^3/day. A distribution point was installed where Port-Cartier residents can fill their bottles. The maintenance and operating costs in this study are based on estimates provided by the Port-Cartier municipal water service. The company that sold the system currently used in Port-Cartier was contacted to obtain the current cost of purchasing one of these systems.

Two system capacities were examined: a 1 m^3/day capacity unit for Rivière-Pentecôte, Baie-Johan-Beetz and Aylmer Sound; and a 6 m^3/day capacity for Harrington Harbour, Vieux-Fort and Rivière-Saint-Paul. The purchase costs of these units are \$30,400 (1 m^3/day unit) and \$42,200

(6 m^3/day unit). Construction and other equipment costs (pumps, electrical installation, etc.) to complete the installation must be added. The total cost to install these units was an estimated \$99,300 (1 m^3/day unit) and \$109,300 (6 m^3/day unit). Maintenance costs, based on estimates from the company supplying these systems, totaled \$6,000/year for the 1 m^3/day unit and \$10,000/year for the 6 m^3/day unit.

3.1.6 *Staff requirements*

The municipal staff required to operate the drinking water supply and distribution service was determined on the basis of parameters indicated above. An annual salary of \$26,000 was determined. The total time to operate the treatment unit is based on estimates provided by the company supplying these systems.

3.1.7 *Purchase of water (scenarios A and B)*

On the basis of scenarios A and B, drinking water would be supplied by neighbouring municipalities or villages. A purchase price was therefore included in the cost analysis. This price is based on costs in the Quebec City region, i.e. \$0.352/m^3. This is based on an "urban" situation and could be higher if smaller production units are considered and depending on how it is estimated. Considering all other costs involved in scenarios A and B, this cost essentially means we assume that the cost associated with the purchase of water is negligible.

4 COST ANALYSIS

Cost analyses for scenarios A to D are based on the hypotheses and parameters defined above. The installation, maintenance and operating costs of each scenario have been estimated for each village. Total equipment costs (buildings, treatment plant, vehicles, storage facilities) have been reported annually on the basis of an annual interest rate of 5% and a 20-year service life for buildings, treatment units and storage facilities, as well as a 10-year service life for vehicles and drinking fountains. Annual maintenance and operating costs were added to equipment costs to obtain the annual total.

Solutions involving membrane technology units for filtering raw surface water were compared. As indicated above, these units should have an adequate water production capacity to meet total domestic demand. The estimated capacity of these units was determined based on the current condition of distribution networks and the extensive water loss caused by continuous flow that is necessary to avert frozen water pipes in winter and high water temperatures in summer. The estimated capacities for each village are reported in Table 6.

Two configurations for membrane processes were considered: nanofiltration (spiral elements), and the so-called integrated membrane treatment (Bouchard et al. 2000; Lozier et al. 1997). The latter configuration was examined because of high TOC concentrations in raw water at some sites. The installation and operating costs of these units were estimated using data obtained from companies supplying these technologies (Bouchard et al. 2000) related to treatment units in Canada and Norway. Table 6 details the installation, maintenance and operating costs of these treatment units as they apply to each village.

Table 7 details the total annual cost per resident of setting up a treatment plant and the alternative scenarios. It shows that setting up a treatment plant would invariably lower the total cost when compared with alternative scenarios. Even in Aylmer Sound, which has the smallest population of the municipalities studied, operating a membrane process treatment plant would be the least expensive option. In fact, the cost of vehicle-distributed drinking water would be higher than the potential savings generated in the context of no treatment plant (scenarios A and B) or building a smaller-capacity treatment plant (scenarios C and D).

Another striking observation revealed by Table 7 is the relatively high cost associated with the lowest cost estimate. It is important to bear in mind the very limited financial capacity of each local

Table 6. Costs associated with implementing a membrane treatment system.

Village	Treatment capacity (m^3/day)	Nanofiltration Installation cost	Nanofiltration Annual maintenance and operating costs	Integrated treatment Installation cost	Integrated treatment Annual maintenance and operating costs
Rivière-Pentecôte	80	$757,500	$18,840	$862,500	$19,140
Baie-Johan-Beetz	80	$757,500	$18,840	$862,500	$19,140
Harrington Harbour*	270	$921,000	$24,240	$1,762,500	$30,240
Aylmer Sound	20	$682,500	$16,740	$750,000	$17,040
Baie-des-Moutons	150	$825,000	$21,240	$937,500	$21,840
Vieux-Fort	300	$952,500	$26,040	$1,237,500	$27,840
Rivière-Saint-Paul	450	$1,027,500	$27,840	$1,537,500	$32,040

* Since the available surface water volume in Harrington Harbour is limited (see section 3), reverse osmosis technology was considered. Integrated treatment, in this case, consists of an ultrafiltration/reverse osmosis system.

Table 7. Total annual cost per resident (Canadian dollars).

Village	Alternative scenarios A	B	C	D	Membrane processes treatment NF	Integrated
Rivière-Pentecôte	$1,208	$1,044	$1,352	$1,177	$821	$911
Baie-Johan-Beetz	$1,290	$1,126	$1,415	$1,240	$876	$972
Harrington Harbour*			$594	$462	$309	$539
Aylmer Sound	$4,162	$3,564	$3,800	$3,563	$3,218	$3,474
Baie-des-Moutons	$877	$709			$541	$601
Vieux-Fort			$721	$591	$314	$390
Rivière-Saint-Paul			$511	$402	$237	$321

* See Table 6 footnote.

economy and the negative impact for many citizens of raising municipal taxes. Even if we consider only maintenance and operating costs (assuming the central government would defray building and equipment costs), residents would still have difficulty supporting related service charges.

5 CONCLUSION

Alternative scenarios for water supply and distribution were developed and analysed for seven villages in Québec's North Shore region. Within these scenarios, drinking water supply and distribution (i.e. water intended exclusively for drinking or cooking) are separated from the supply and distribution of water intended for other purposes (washing, toilet facilities, etc.). Drinking water is distributed via vehicles such as specially-adapted minivans, ATVs and snowmobiles. Non-drinking water is distributed by a piping network. The four scenarios considered were developed according to a combination of different water supply options (transporting water from a neighbouring supply site or installation of a small treatment unit for drinking water) and storage options (in-house storage tank or storage of drinking water in bottles). The suitability of a particular scenario to each village was then examined on the basis of the potential of nearby towns or sites to supply drinking water.

A cost analysis was conducted for each scenario, based on the hypotheses and parameters defining the requirements for services, equipment, building and staff.

These scenarios were compared with scenarios where treatment plants are installed. Each plant's capacity was estimated on the basis of the total current demand. Two configurations of membrane process technologies were considered, namely nanofiltration and integrated technologies. Estimated costs (installation, maintenance and operating) were compiled according to data obtained from companies supplying these technologies as well as actual cost estimates from operating plants.

The results of this cost analysis clearly show that solutions involving a membrane process treatment plant and a water pipe network for distribution are less costly than solutions in which drinking water is distributed via a transportation system. In fact, the cost of transporting water exceeds possible gains generated by building and operating smaller-scale treatment plants or even where there is no local treatment plant.

In addition to cost concerns, other important problems associated with water supply in remote communities should be examined, such as the local economy's ability to financially support and operate new equipment and infrastructures. As indicated above, this is a critical point because even if a central government covers the cost of equipment and infrastructures, local communities must be able to fund operations. A second important issue is the difficulty of finding qualified employees to operate specialized water treatment equipment. Improperly operated equipment may pose a threat to public health or reduce the normal service life of equipment.

Some components of the analysis reported in this article should be further developed. Issues pertaining to distribution network upgrading or construction should be incorporated into the analysis. An integrated water supply and distribution solution for these small communities should also include wastewater facilities and sewer network. Indeed, as indicated above, few homes are directly hooked up to the water distribution network at this time in some villages. All homes and buildings could eventually be connected to the water pipe network. The impact of an upgrade of the distribution network on the overall water demand must be considered in order to make sure that villages currently have adequate sewer network or wastewater treatment facility. In situation where each home has a septic tank and if the water distribution network is extended, water consumption demand may exceed forecasts and lead to sanitary problem related to wastewater disposal and treatment.

REFERENCES

Bouchard, C., Kouadio, P., Ellis, D., Rahni, M. & Lebrun, R. 2000. Eau potable, Les procédés à membrane et leurs applications en production d'eau potable. *Vecteur environnement*, July 2000. 33(4): 28–38.

Bouchard, C., Ellis, D., Sérodes, J., Rodriguez, M. & Lebrun, R. 2000. Les filtrations sur membranes: mythes et réalités. Presented at the *12ième Atelier sur l'eau potable*, November 28–29, Laval, Québec, Canada.

Cotruvo, J.A. & Cotruvo Jr., J.A. 2003a. Nontraditional approaches for providing potable water in small systems: Part 1, *Jour. AWWA*, 95(3): 69–76.

Cotruvo, J.A. & Cotruvo Jr., J.A. 2003b. Nontraditional approaches for providing potable water in small systems: Part 1, *Jour. AWWA*, 95(3): 69–76.

Coulibaly, H.D. & Rodriguez, M.J. 2003. Portrait of drinking water quality in small Quebec municipal utilities, *Water Qual. Res. J. Canada*, 38(1): 49–76.

Gouvernement du Québec 2001. *Règlement sur la qualité de l'eau potable.* Ministère de l'environnement, Québec.

Lozier, J.C., Jones, G. & Bellamy, W. 1997. Integrated membrane treatment in Alaska. *Jour. AWWA*, 89(10): 77–89.

Villeneuve, J.-P., Mailhot, A. & Nepton, M. 2003. *Étude de la problématique de l'approvisionnement en eau potable de sept localités de la Côte-Nord*, INRS-Eau, Terre et Environnement, Research report N R633-b, Sainte-Foy, Québec, Canada.

The sustainability of irrigation in England and the impact of water pricing and regulation policy options

J. Morris, K. Vasileiou, E.K. Weatherhead, J.W. Knox & F. Leiva-Baron
Institute of Water and Environment, Cranfield University, Silsoe, Bedford

ABSTRACT: Growing pressure on water resources has lead to limits on abstraction licenses, and consideration of the use of the use of economic instruments, such as increased abstraction charges or tradeable licenses to encourage wise use of water. This paper examines the use and value of water on indicative case studies of irrigation in England, reviewing irrigation performance against a set of indicators of sustainability. The costs of irrigation are estimated at about £0.34/m^3 of water applied, rising to between £0.45 and £0.60/m^3 if winter reservoirs are required. Where irrigation can assure high quality, high value produce, financial benefits, economic value added net of subsidies, and social benefits in terms of employment are still substantial compared to the rainfed alternative. There are however some potential negative environmental impacts. An analysis of alternative policy instruments which might be used to reduce water consumption by irrigators showed that, for the cases explored, the response to and impact of pricing and regulatory instruments can be very different according to value of water (£/m^3) at the margin of use. For example, although increased water charges can reduce demand in low value applications, very large increases in charges are needed to reduce water demand where the financial returns to water are high, resulting in large decreases in farm incomes (and transfer of income to the regulating agency) before consumption significantly changes. In such circumstances, regulation through licensing may be more effective and potentially equitable. Where conditions allow, however, there could be scope for trading of abstraction permits but this would be within overall regulated abstraction quantities.

1 INTRODUCTION

Although irrigation typically accounts for less than 2% of total water abstraction in England and Wales, during peak periods it can account for up to 70% of total abstraction in intensively irrigated areas of eastern England (Knox et al., 1996). It is particularly in these catchments that water resources are over-committed, and additional licences for irrigation abstraction are unobtainable (EA, 2001a). Yet recent research predicts a continued and significant rise in the demand for irrigation water (Downing et al., 2003).

In temperate climates such as in the UK, supplemental irrigation can provide significant yield and more importantly crop quality benefits over and above rain-fed production. For vegetables and potatoes in particular (Table 1), irrigation may be essential to meet market demands for a reliable supply of high-quality produce (Weatherhead et al., 1997). Indeed, quality criteria are increasingly specified as a condition of contracts to supply supermarkets. Failure to meet these quality standards often leads to large price reduction and possibly rejection. In general, the extra financial benefit per m^3 of irrigation water applied is highest for soft fruit (e.g. strawberries), vegetables, potatoes and orchard fruit, and lowest for grass, cereals and sugar beet.

In some catchments, irrigation demand is now outstripping supply. Constraints on abstraction are becoming increasingly necessary, particularly where priority uses include environment protection

Table 1. Allocation of irrigated land and water volume to crops in 2001. (National Irrigation Survey 2001)

	Volume		Area	
Crop	1000 m^3	%	ha	%
Early potatoes	5,872	4	7,628	5
Maincrop potatoes	70,057	53	70,006	47
Sugar beet	4,633	3	9,755	7
Orchard fruit	896	1	1,578	1
Small fruit	3,312	2	3,774	3
Vegetables	34,114	26	39,164	26
Grass	2,470	2	4,104	3
Cereals	1,471	1	4,615	3
Other crops	8,841	7	7,272	5
Total of outdoor crops	131,755	100	147,895	100

measures. Recent European legislation (EU Water Framework Directive 2000/60/EC) and pending national legislation (UK Water Bill) will seek to ensure better abstraction control and greater environmental protection.

In this context, and as part of a wider study of European irrigation (Twite et al., 2001), this paper reports on an assessment of the impact of water regulation versus pricing as mechanisms to reduce pressure on water resources associated with abstraction for irrigation. It also considers the impact of these methods on farm incomes and the performance of farming systems when judged against a set of economic, social and environmental criteria.

2 DATA AND METHODOLOGY

The impacts of changes in the licensed quantities of water and of water prices on agricultural irrigation demand, farm income and a range of performance indicators have been simulated using a linear programming model. In this paper, two indicative case study farms are used to illustrate the impacts of these policy interventions which may be adopted to achieve water rationing in situations of declining resources.

The main characteristics of irrigation in England and Wales have been derived from a national survey of irrigation (Weatherhead and Danert, 2002), regional farm business survey data (Lang, 2002), a dedicated survey of 25 irrigated farms (Weatherhead et al., 2002), and previous studies of irrigation (Weatherhead et al., 1997, Morris et al., 1997a).

The main irrigation farming systems are:

- field-scale vegetables, dominated by potatoes in rotation with other irrigated field-scale vegetable crops such as sugar beet, parsnips, carrots, vining peas and brassicas, as well as non-irrigated crops such as cereals;
- horticultural crops including high value, high quality salad crops such as lettuce, celery, and spring onions, and herbs such as parsley;
- fruit, including soft fruit such as strawberries and raspberries, and top fruit such as apples and pears. This is a relatively small but locally important activity, mainly grown in the south east of England where trickle irrigation is widely used.

A cluster analysis of 1400 irrigating farms from a national survey, showed that the top 30% of farms, ranked by volume of water applied, accounted for approaching 85% of both total water applications and total irrigated area. 96% of the irrigated area uses mobile rain guns or booms. The dominance of potatoes and vegetables in these larger scale irrigation enterprises is apparent. Potatoes alone account for about 52% of the total irrigated area and 58% of the total volume of water

Table 2. Characteristics of the case study farming systems.

Characteristic	Potato/Veg system	Potato/Sugar beet system
Location	Cambridgeshire, (Mepal), average annual rainfall 600 mm; Max ET 3.5 mm/day; soil type sandy loam	Midlands, (Gleadthorpe), average annual rainfall 622 mm; Max Et 3.6 mm/day; soils sandy loams
Farm area	300 ha	300 ha
Command area	300 ha	300 ha
Irrigated area (annual)	150 ha	150 ha
Cropping pattern: *options*	*Rain-fed*: wheat, barley, oil seed rape, beans, peas, sugar beet, set-aside *Irrigated*: wheat, potatoes, sugar beet, onions, carrots, Brussels sprouts, vining peas	*Rainfed*: wheat, barley, oil seed rape, beans, peas, sugar beet, set-aside *Irrigated*: wheat, potatoes, sugar beet, vining peas
Cropping constraints	Rotation for cereal/non-cereal break crops. Vegetables: minimum yearly interval between crops: potatoes and carrots 6, others 4	Rotation for cereal/non cereal break crops. Vegetables: minimum yearly interval between crops: potatoes, others 4
Water supply	Groundwater (borehole) and surface water. Summer abstraction (55%), winter abstraction (45%) of total licensed quantity	Borehole (40%) and surface (60%) sources. Summer 60% Winter 40% of total licensed quantities
Irrigation system	Clay lined reservoir for winter storage, mobile hose-reel system. 25 mm on 7 day interval	Unlined reservoir for winter storage, mobile hose-reel system. 25 mm capacity on 7 day interval
Typical irrigation application depths	125–220 mm on potatoes depending on needs	125–220 mm on potatoes depending on needs
Labour and machinery	Regular labour plus casual for peak periods, fully mechanised system.	Regular labour plus casual for peak periods, fully mechanised system.

applied. In the dominant potato growers group, two distinguishable clusters are discernible: potatoes with field scale vegetables such as onions and carrots vegetables, and potatoes with sugar beet and other crops such as vining peas.

Farm surveys and cluster analysis confirm that there is much variation amongst irrigating farms, such that no one farm conforms with the average. For this reason, indicative farm models were created to represent the dominant types of irrigation. For the current paper, two main systems are examined, namely potatoes with field scale vegetables, and potatoes with sugar beet. The cases are named accordingly. The former is characteristic of eastern England and the latter of the English Midlands. Table 2 summarises the characteristics of the two farms.

Although technically the cropping option for the two farms are much the same, the choice of cropping is influenced by marketing and institutional factors, notably access to markets for potatoes, contracts with nearby factories for sugar beet and peas, as well as husbandry skills and existing farm infrastructure. For the purpose of analysis, farm size is kept the same for both cases, as is the proportion that is irrigable. In practice, evidence suggests that there is a large degree of variation in the details of these variables amongst farms.

Crop yields and prices, crop area-payments, labour and machinery costs were drawn from multiple sources, namely Lang (2002), Nix (2000; 2002) and dedicated farm surveys of irrigation (Weatherhead et al., 2002). Potato and vegetables prices and yields were derived from trends over the preceding 5 year period using published sources (DEFRA, British Potato Council), assuming premium quality for irrigated crops.

Irrigation water requirements were estimated using the Irrigation Water Requirements Model (Hess 1994, 1996) to determine optimum water use for agriculture (Weatherhead et al., 2002), confirmed by farmer reported application depths, for the dominant regional climatic and soil conditions. Using regional weather data, water needs were estimated for the average rainfall year (10th driest year in 20) and the commonly used design year (5th driest year in 20).

In England and Wales, water resources are managed by the Environment Agency (EA). All withdrawals from surface and groundwater for spray irrigation require an abstraction license under the Water Resources Act (1991). Abstraction is subject to licensed quota restrictions, specified by source (surface or groundwater), season, (summer or winter), and abstraction rates (per season, per day and sometimes per hour). Charges for irrigation water are set by the Environment Agency to recover the costs of administering the licensing system. In 2001/2, prices were £0.028 p /m^3 for summer water and £0.0028 p/m^3 for winter water in the eastern Anglian Region, and marginally lower for the Midlands region. Irrigators often have multiple ground and surface water sources, and often a mix of summer and winter licenses. Restrictions on summer abstraction have encouraged investment in storage of winter abstracted water.

Table 3 summarises irrigation costs by the major variables of water source and season. Typical costs for relatively large areas irrigated (50 ha and over) are about £0.33/m^3 applied in the field, rising to about £0.45/m^3 with clay lined storage reservoirs, and as over £0.60/m^3 with artificially lined reservoirs. Water costs are less than 7% of total costs. Thus, at current abstraction charges, summer direct abstraction is always cheaper per m^3 than winter stored water. Summer charges would need to rise to about £0.15p/m^3 or so for winter stored water to be a cheaper option. But in many situations additional summer water is not available.

A set of indicators are used to assess the impact of variations in water availability and water charges on the economic, social and environmental performance of the farms, and thereby measure the degree of achievement against sustainability criteria (Berbel et al., 2002). It is assumed here that farmers will manage their overall rainfed and irrigated farming systems in order to maximise profitability subject to the constraints imposed by market conditions and fixed resources. In practice, actual performance is determined by many factors whose value and influence are difficult if not impossible to predict with accuracy. For this reason, farmers adopt risk reducing and coping strategies, of which irrigation is one.

Table 3. Estimated irrigation costs* (£/m^3, 2001 prices) by water source, storage and infield system.

Source storage in field	Surface none hosereel	Surface reservoir lined hosereel	Surface reservoir unlined hosereel	g/water none hosereel	g/water reservoir lined hosereel	g/water reservoir unlined hosereel
Costs by component £/m^3						
supply and distribution						
fc	0.07	0.08	0.08	0.11	0.10	0.10
vc	0.10	0.06	0.06	0.12	0.08	0.07
Total	*0.17*	*0.14*	*0.14*	*0.22*	*0.18*	*0.17*
Reservoir						
fc	0.00	0.25	0.11	0.00	0.25	0.11
vc	0.00	0.09	0.04	0.00	0.09	0.04
Total	*0.00*	*0.34*	*0.15*	*0.00*	*0.34*	*0.15*
Infield						
fc	0.09	0.09	0.09	0.09	0.09	0.09
vc	0.05	0.05	0.05	0.05	0.05	0.05
Total	*0.14*	*0.14*	*0.14*	*0.14*	*0.14*	*0.14*
Total fc	0.16	0.42	0.28	0.20	0.44	0.30
Total vc	0.14	0.19	0.15	0.16	0.21	0.16
Total	*0.30*	*0.61*	*0.42*	*0.36*	*0.66*	*0.46*

The economic performance of irrigation is expressed in terms of farm income, government support and value added after support. This reflects the type and areas of rainfed and irrigated crop production. Water management is assessed in terms of irrigation practices, water use, and the financial contribution of water at the margin of use. Employment and seasonality of work are used as broad social indicators of participation in farming activities.

Environmental performance, in addition to water use efficiency, is captured through a mix of indicators. A soil cover indicator to assess erosion risk is calculated using a weighted index reflecting plant cover during the farming year, although for UK conditions this indicator, in the absence of local topographic and environmental information, does not in itself provide a good estimate of soil erosion risk.

Estimates of nitrogen and phosphorus emissions (kg/ha) to surface water were obtained using an export coefficient model in accordance with the application rates and local soil and climatic. Pesticide leaching to surface water was estimated using the SWAT model (Hollis and Brown, 1994) using data on application type and rate from Chadwick (1999), and local soil and climatic data type. Pesticide risk, measured as a concentration in μg/l, was expressed as an index using that associated with wheat (=100) as reference point. Estimates of energy inputs, including fertilisers and pesticides, seeds, labour, electricity and fuel use, and machinery replacement and maintenance were obtained from multiple sources, namely ABC (2000), Chadwick (1999), Audsley (1997), Nix (2000), Hülsbergen (2001), Leiva and Morris (1997) and Wells (2001).

3 ANALYSIS AND RESULTS

Linear programming was used to model irrigation on the indicative case study farms. It was first assumed that licensed quantities of water were adequate to meet crop water requirements. The irrigated area, set at 50% of the command area, was assumed to be constrained by system capacity and crop rotation requirements.

The financial performance of irrigation is very sensitive to the prices obtained for irrigated crops. As previously mentioned, most irrigation in UK is focused on quality assurance. A review of potato, vegetable, fruit and salad crop prices showed that prices for first quality produce are typically 40% to 50% above prices for average or second grade produce. It is assumed here that, in areas of irrigation need, irrigated crops receive an additional premium price above prevailing average prices.

The optimum allocation of water amongst irrigated crops for each farm was determined together with the estimated net revenue and other indicator values as shown in Table 4. They were then subject to incremental restrictions on water availability and to increases in water abstraction charges. This process was carried out for the average (10th driest year in 20) irrigation year and for the 5th driest year in 20. The results presented here are for the average year.

In the unrestricted water supply situation, on both farms, all of the potential irrigated land is irrigated, with rainfed cropping given to wheat and set-aside. Incremental reductions in water available impose a switch to rainfed cropping, resulting in successive reductions in the areas of irrigated crops in accordance with their relative returns to water. It is assumed that these crops could not be grown reliably to the quality standard required by their markets in the absence of irrigation. Net margins per ha fall by about 80% and 60% respectively for the potato/vegetable and potato/sugar beet systems as a result of a switch to a cereal-based rainfed regime. Value added, net of subsidy payments also fall significantly. Total labour employment reduces by at least 50% in both cases, with greater variation in the seasonality of work.

The risk of nitrate leaching increases under the rainfed regimes compared to the irrigated ones, due to the increased presence of cereals. There are, however, significant reductions in phosphate and pesticide leaching, and total energy input falls by about 10%.

Figure 1 shows the marginal value product (mvp) of water, that is the contribution to total net revenue (£) per unit of water at the margin of use. This "shadow" price ranges from £0.03/m^3 (the price paid for summer water) when water is not limiting to about £2/m^3 when no irrigation water is available at all. For this case, the average mvp for water is about £1.58/m^3 for the potato/vegetable system

Table 4. Cropping pattern and irrigation system performance indicators under alternative water supply conditions (% of unconstrained demand) for the average rainfall year.

	POT/VEG			POT/S BEET		
Water availability	100%	50%	0%	100%	50%	0%
Cropping pattern per 100 ha						
Rainfed						
Winter wheat (first crop)	45.47	50.00	34.23	45.47	50.00	50.00
2nd winter wheat (2nd crop)	0.00	0.00	31.57	0.00	0.00	0.00
Spring wheat	0.00	0.00	0.00	0.00	0.00	0.00
Winter barley	0.00	0.00	0.00	0.00	0.00	0.00
Winter field beans	0.00	0.00	0.00	0.00	0.00	0.00
Combined peas	0.00	0.00	0.00	0.00	0.00	0.00
Winter oil-seed rape	0.00	13.90	20.00	0.00	2.00	18.17
Sugar beet	0.00	7.47	5.63	0.00	25.00	25.00
Set aside	4.53	6.40	8.57	4.53	5.20	6.83
Irrigated						
Winter wheat	0.00	0.00	0.00	0.00	0.00	0.00
Potatoes	16.67	5.57	0.00	16.67	16.67	0.00
Sugar beet	0.00	0.00	0.00	25.00	0.00	0.00
Vining peas	0.00	0.00	0.00	8.33	1.13	0.00
Brussel sprouts	0.00	0.00	0.00			
Bulb onions	16.67	0.00	0.00			
Carrots	16.67	16.67	0.00			
Economic						
Net margins (£/ha)	1345	881	263	819	812	341
(Value added) (£/ha)	1225	706	28	699	674	153
Public support (£/ha)	120	175	235	120	138	188
Social						
Farm employment (hrs/ha)	29.3	18.5	9.4	20.3	19.6	11.4
Labour seasonality index	99.7	110.1	115.8	32.1	39.8	67.2
Water use and values						
Water use (m^3/irrigable ha)	1366.7	683.3	0	1058.3	529.2	0
Water marginal value price (£/m^3) delivered	0.265	1.924	2.205	0.289	0.395	2.397
Water marginal value price (£/m^3) abstracted	0.017	1.676	1.957	0.011	0.117	2.119
Environment and energy						
Crop diversity	5	6	4	5	6	4
Soil cover index	5.3	4.9	4.8	4.7	4.8	4.7
Nitrogen leaching (Kg N/ha)	30.2	30.3	32.8	26.2	27.8	30.1
Pesticide risk index	115	102.5	87.8	105.3	110.9	97.7
Energy balance (MJ/ha)	88969	87006	82359	91597	95440	84672
Energy consumed (MJ/ha)	20149	18907	18539	20362	20460	18274

and about £1/m^3 for the potato/sugar beet system. This confirms the high returns to water where irrigation supports the production of high value, quality produce.

Figure 1 also shows the water price: quantity relationship for irrigation water, showing the benefit derived from water use, and implicitly farmer willingness to pay for water at different quantities supplied. The two systems show different marginal values for water over the range of water use, and hence different elasticities of response to price changes. On the potato/vegetable farm, demand for water is very price inelastic at existing levels of water use. That is, demand is not very responsive to price changes. A given % increase in water price results in a much lower % change

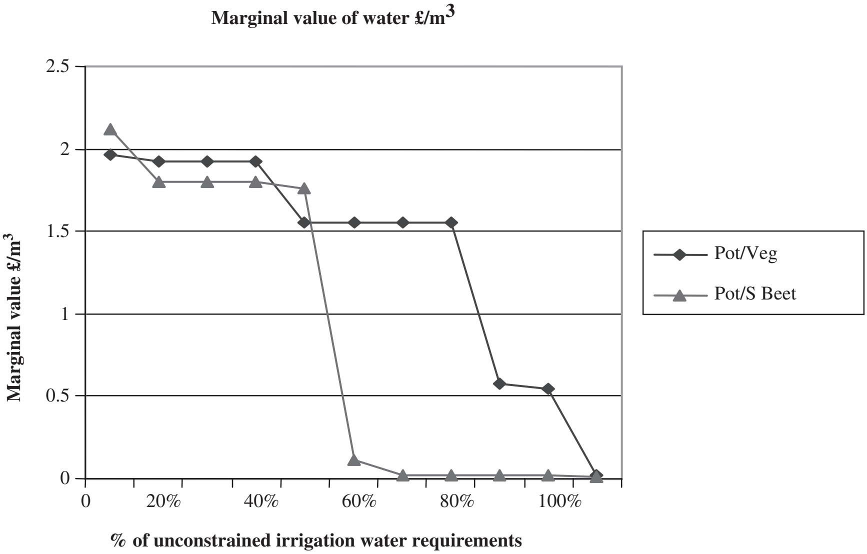


Figure 1. Marginal value of irrigation water (£/m^3) for two irrigation farming systems.

in water use: water prices would need to rise from their current levels of about £0.03/m^3 to about £0.60/m^3 to provide a reduction in demand of about 15% for the assumptions made. For the potato/beet farm, however, the demand for water is relatively sensitive to price changes at existing levels of use. A relatively small % increase in price results in a relatively large % reduction in water use. A rise in the price of water to about £0.12/m^3 would reduce demand by about 40% on this farm as a consequence of rendering the irrigation of sugar beet (and other possible but marginal crops) infeasible. Admittedly this is a significant increase from existing levels, but one that is likely in the event of tradeable abstraction permits (Morris et al., 1997b).

These differences in price elasticities of demand for water between the farming systems, reflecting the differences in benefits of irrigation on different crops, have important implications for the choice of policy instrument for a water resource management agency wishing to ration water use and/or increase efficiency in use. It means, for example, that the effectiveness of water pricing as an economic instrument to achieve reductions in demand varies according to type of irrigated farming.

The impact on farm incomes associated with actions to achieve given reductions in water use by farmers is given in Figure 2. For example, for the potato/vegetable farm, achieving a 10% reduction in water use using the water price mechanism would result in a 40% reduction in income to farmers. Farmers would, in theory, be inclined to absorb the price increases before they significantly changed their water consumption. However, a 10% reduction in water use imposed through an equivalent reduction in licensed quantities available would result in a 6% fall in net margin to farmers.

In the case of the potato/sugar beet farm, the effect of water pricing and regulation as means of water rationing are similar in their impact on behaviour and income, as shown in Figure 2.

Of course, using pricing to ration water will result in significant income transfers to the water management agency over that part of the water demand curve which is price inelastic. Beyond abstraction charges of £1/m^3, extra revenues to the agency would fall as farmers become much more price responsive and their demand for water declines.

The estimates of water values and responses to water pricing were derived assuming full average costs, inclusive of depreciation for regular labour, mechanisation and water supply. Estimates were also derived for variable cost assumptions, excluding regular labour and amortisation of capital items. These latter assumptions did not make significant differences to the overall outcomes for this intensive cropping regime, although irrigation appears more profitable (and the marginal value product of

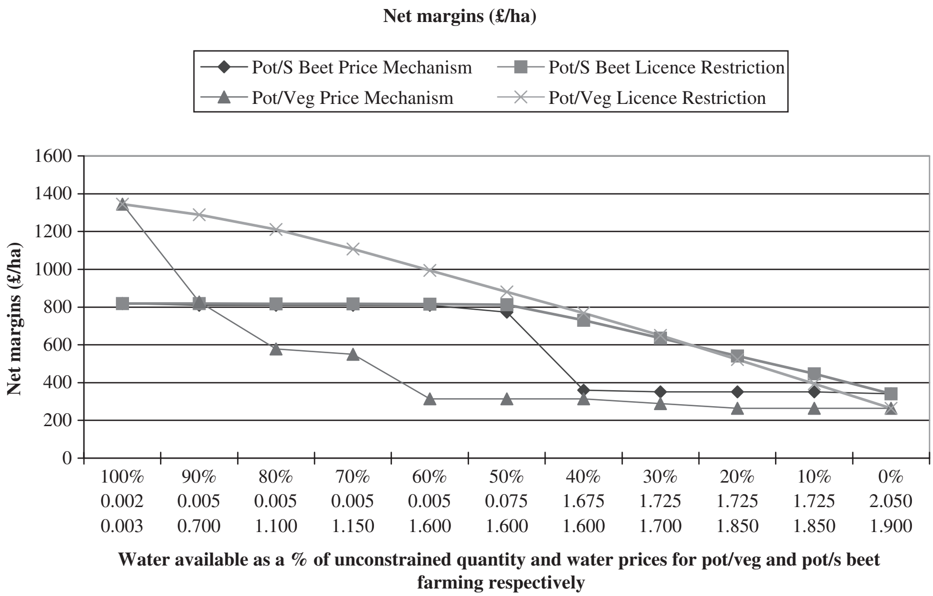


Figure 2. Net margins (£/ha of total cropping) for two case studies in response to changes in water prices and licensed quantities available.

water is higher) when farmers are seeking to recover only the variable costs of irrigation and related cropping activities. It may be that irrigation of less profitable and less water responsive crops such as sugar beet and wheat may be feasible if in the short term it is only necessary to cover variable costs.

While caution must be expressed about the absolute values given here, the underlying message is important. The effectiveness and efficiency of alternative policy instruments such as restrictions on abstraction licenses and water pricing vary according to the potential benefit of irrigation at the margin of water use.

For high value, water responsive crops, the use of water pricing to ration water is likely to transfer income from farmers to the regulator before it will change consumption. Indeed, irrigators might be willing to pay higher prices for increased security of supply. They have demonstrated this in practice for example by incurring extra winter storage costs of between £0.15/m^3 and £0.30/m^3, and in some cases use mains water at £0.60/m^3. Water storage in on-farm irrigation reservoirs nearly doubled between 1984 and 1995 (Weatherhead et al., 1997). However, in some parts of the country, unacceptable flow regimes or over-licensed abstractions are reducing the availability of even winter water for storage (EA, 2001b).

For relatively low value, less water responsive crops, the use of water pricing can result in significant reductions in use at individual farm level. Taken in aggregate, however, as shown by Table 1 earlier, by far the greatest commitment of irrigation water is on potatoes and vegetables. This suggests that there may be limited scope for very significant total savings in water use through the use of water pricing within the bounds of acceptable price levels, especially in water deficit catchments in England and Wales.

4 DISCUSSION: IRRIGATION PERFORMANCE AND POLICY CHANGE

The case study examples draws attention to a number of important issues regarding the performance of dominant irrigation farming systems in E & W, the value of water and the best way of promoting efficient use of an increasingly scarce resource.

Compared to the rainfed alternative, irrigation can provide significantly higher financial net revenues per ha and lower absolute dependency on agricultural support payments, with the notable exception of sugar beet for which prices are guaranteed by quota. Loss of irrigated cropping would most likely be substituted by imports, before it would relocate to less drought-prone areas. Irrigation also generates much greater employment of regular and casual labour, with less variance in employment during the year. From an environmental point of view, although nitrate risk is lower, there is increased risk of phosphate and pesticide risk associated with higher input use in vegetable cropping. The potential effect of irrigation on soil erosion risk varies according to site conditions, especially soils and topography, and the management of in-field irrigation.

Water can generate very high financial returns where supplementary irrigation assures first class quality in high value crops. In order to facilitate mechanisation of field operations, vegetable and root crop production has tended to move to drought prone lighter soils. In the absence of irrigation, land use would revert to extensive cereal cropping. The profitability of irrigation depends considerably on the price differentials offered for quality produce in the market.

In situations of crop water deficit, and returns per m^3 of water are high, as they are in the case of vegetables, rationing water through increased water prices could have a major impact on farm incomes before it substantially changes water use behaviour. In such situations restrictions on licenses are likely to be a more effective and equitable mechanism to achieve beneficial change, although some increase in abstraction charges could help fund water resource management initiatives by the responsible agency.

In the absence of summer licenses, farmers have shown a willingness to switch to winter water incurring an additional cost for storage of at least £0.15/m^3 and in some cases as much as £0.30/m^3 to secure water for irrigation. This could possibly be achieved through increases in summer abstraction charges but, from the water agency's viewpoint the outcomes would be uncertain, farmers may absorb price rises until they eventually make the switch, and such increases in abstraction charges would no doubt meet political opposition. It might be far better to regulate for a more certain outcome.

This last point emphasises the need to be clear about "policy objectives" when choosing water "policy instruments". Regulation in the form of quotas, as the case study above suggests, is probably the most effective way of overall control and rationing of water use where there is pressure on total supply. By comparison, abstraction charges may not achieve the desired reductions in water consumption without unacceptable impacts on user incomes. They do, however, generate additional revenues for the water agency which can be used to fund other support mechanisms, such as local area or on-farm storage, or the development and promotion of water saving technologies. There may also be scope to create a market in tradeable abstraction licenses within an overall quota ceiling. This would in theory facilitate the movement of water to its most beneficial use.

REFERENCES

ABC (2000). *The Agricultural Budgeting & Costing Book*, November 2000. Agro Business Consultants.

Audsley E. (1997). *Systematic Procedures for Calculating Agricultural Performance Data for Computing Systems*. Working Paper, Silsoe Research Institute, Silsoe Bedford.

Berbel J., Morris J., Boymanns D., Bazzani G., Gallerani V., Viaggi D., Gomez M., Twite C. and Weatherhead K. (2002). WADI Project: *Proposal for Joint Agricultural and Water Policy Scenarios*. The Sustainability of European Irrigated Agriculture under the Water Directive and Agenda 2000: EVK1-CT-2000-00057.

Chadwick L. (1999). *Farm Management Handbook 1999/2000.* 20th Edition. Edinburgh: Scottish Agricultural College, 1999.

Environment Agency (2001a). *Water Resources for the Future – A Strategy for England and Wales.* Bristol: The Environment Agency.

Environment Agency (2001b). *Water Resources for the Future A Strategy for Anglian Region.* Bristol: The Environment Agency.

Hess T.M. (1994). IWR. An irrigation water requirements programme (IWR). User manual. Silsoe College, Bedford (unpublished).

Hess T.M. (1996). A microcomputer scheduling program for supplementary irrigation. *Computers and Electronics in Agriculture*, 15: 233–43.

Hollis J.M. and Brown, C.D. (1994). SWAT- A simple Model to Calculate and Extrapolate Concentrations of Pesticides Entering Surface Waters from Agricultural Land, SSLRC, Cranfield University, Silsoe, Bedford

Hülsbergen K.-J., Feil B., Biermann S., Rathke G.-W., Kalk W.-D. and Diepenbrock W. (2001). A method of energy balancing in crop production and its application in a long-term fertilizer trial. *Agriculture, Ecosystems and Environment*, 86: 303–321.

Knox, J.W., Morris, J., Weatherhead E.K. and Turner A.P. (2000). Mapping the financial benefits of sprinkler irrigation and potential financial impact of restrictions on abstraction: A case-study in Anglian Region. *Journal of Environmental Management*, 58: 45–59.

Knox J.W., Weatherhead E.K. and Bradley R.I. (1996). Mapping the spatial distribution of volumetric irrigation water requirements for maincrop potatoes in England and Wales. *Agricultural Water Management*, 31: 1–15.

Lang B. (2002). *Report on Farming in the Eastern Counties of England 2000/2001*. Cambridge University, Dept. of Land Economy, 2002.

Leiva F. and Morris J. (1997). Mechanisation and Sustainability in Arable Farming in England. *J of Agricultural Engineering Research*, 79, 1, 81–90, online www.idealibrary.com.

Morris J., Weatherhead E.K., Knox J.W., Gowing D.J.G., Dunderdale J.A.L., Green C. and Tunstall S. (1997). Practical implications of Introducing Tradeable Permits for Water Abstractions. Report to the Royal Society for the Protection of Birds, by Cranfield University, School of Agriculture, Food and Environment and Middlesex University, Flood Hazard Research Centre. Cranfield University, Silsoe, Bedford.

Morris J., Weatherhead E.K., Mills J., Dunderdale J.A.L., Hess T., Gowing D.J.G, Sanders C. and Knox J.W. (1997a). *Spray Irrigation Cost Benefit Study*. Final Report to Environment Agency, Cranfield University, Silsoe.

Nix J. (2000). *Farm Management Pocketbook,* 30th edition, Wye College, London University, Wye, Ashford, Kent.

Nix J. (2002). *Farm Management Pocketbook,* 32th edition, Wye College, London University, Wye, Ashford, Kent.

Twite C., Morris J., Knox J. and Weatherhead K. (2001). *Inventory of Irrigated Agriculture:United Kingdom.* WADI Project 2001, Cranfield University, Silsoe, Bedford.

Weatherhead E.K. and Danert K. (2002). *Survey of Irrigation of Outdoor Crops in 2001 – England, Cranfield University*, Silsoe, Bedford.

Weatherhead E.K., Knox J.W., Twite C.L. and Morris J. (2002) *Optimum Use of Water for Industry and Agriculture: Part B Agricultural Component*. Environment Agency Water Resources R&D Project W6-056/TR.

Weatherhead E.K., Knox J.W., Morris J., Hess. Bradley R.I. and Sanders C.L. (1997) *Irrigation Demand and On-farm Water Conservation in England and Wales.* Final Report to Ministry of Agriculture, Fisheries and Food. Ref. MAFF Project OC9219. Cranfield University, Bedford.

Wells, Colin (2001). *Total Energy Indicators of Agricultural Sustainability: Dairy Farming Case Study*. Technical Paper 2001/3. Ministry of Agriculture and Forestry, Wellington, New Zealand.

A DSS for water conservation policies impact assessment in agriculture

G.M. Bazzani
Institute for BioMeteorology (IBIMET), National Research Council, Bologna, Italy

ABSTRACT: The paper presents DSIRR "Decision Support System for Irrigation" which permits to conduct an economic environmental assessment of agricultural activity focused on irrigation. The program simulates farmers' decision process, which is economically driven, permitting an accurate description of the production and irrigation processes with endogenous investment choices. Mathematical programming techniques in a multicriteria framework are adopted to solve farm models which are aggregated at catchment's scale. The final user can construct and run different simulation thanks to a Graphical User Interface. An application to an Italian case study that analyses the joint impact of the Water Framework Directive and of the Common Agricultural Policy Mid Term Review and considers irrigation technology is presented. The results confirm the utility of the proposed approach to forecast the demand for water in agriculture and to estimate at low cost social, economical and environmental impacts, generating valuable information to support policy analysis.

1 INTRODUCTION

In the Mediterranean region, as in many others, irrigated agriculture is important in terms of land use, value of production and employment. Growing population, higher living standard and environmental concern rise strong pressures on water, perceived more scarce both in qualitative and quantitative terms. A severe reform in water allocation mechanism is urgently requested at international forum. Many experts suggest to reduce water use in agriculture from 60–80% of current total consumption via economic instruments. In this direction moves also the EU Water Framework Directive 60/2000 (WFD) defining criteria for water management, regulation and pricing. Economic instruments are in fact recommended to put into action the cost recovery and polluter pays principles adopted by the WFD: water price (WP) should provide incentives to reduce water use and pollution. The WFD implementation at regional and catchments level and the definition of river basin plans require long term forecasts of supply and demand for water.

The impact of water policy on agriculture is complex (Tsur et al., 2002). Many critics suggest that irrigation is not efficient neither in technical terms nor in economical terms, others have shown that irrigated agriculture has important social impact favouring employment in less favoured area and representing a key element in the landscape creation. The need for more studies exploring the links existing among agriculture, environment and society is urgent since past research showed that a strong local dependency from contingent situations does not permit easy generalization of policies and interventions (Johansson et al., 2002). This calls for proper tools.

This paper aims to present DSIRR "Decision Support System for Irrigation" a program which permits to conduct an economic environmental assessment of agricultural activity focused on irrigation, reducing cost, time and effort to conduct sound studies which can support policy analysis and definition in a plurality of contexts. The DSS integrates driving economic models with agronomic and engineering information to tackle the water resource allocation and management problem in agriculture at catchment's scale.

The remainder of this paper is organized as follows: after a review of modeling irrigation in agriculture, the program will be illustrated in a non technical way. A case study related to annual crops in the Po Basin, Italy, will be illustrated. Finally considerations and conclusion.

2 MODELLING IRRIGATION CHOICES

An excellent review on modelling water resources management at basin level is offered by McKinney D. C. et al. (1999). DSIRR restricts its field of analysis to the agricultural sector but tries to describe accurately the farmers' joint choice of cropping pattern, water application levels, irrigation technologies, employment, conditioned on farm characteristics, labour and capital endowment, market, institutional and policy conditions, recognizing different production systems operating simultaneously in the catchment.

The farmers' decision process and the policy implication seem well represented and analyzed by many economic models. Economic theory shows that *demand curves*, functions describing the price/quantity relation, permit to generate relevant information, i.e. water consumption, farmers' income and water agency revenue at different water rates, and to see their variation in response to price changes (Howitt, 1980). Since real data for water demand are at the best available only for the range close to the existing consumption level, the curve estimate can be obtained via model simulation using Mathematical Programming Techniques (MPT). A body of economic literature exists on models based on MPT focusing on irrigation among which: Amir et al. (1999), Doppler et al. (2002), Schaible (1997), Varela-Ortega et al. (1998).

Recent literature shows that farmers' behaviour can be better captured and described via the Multi Criteria (MC) paradigm than by the simple profit maximizing one adopted by the standard economic theory (Romero et al., 1989; Berbel et al., 1998; Gomez-Limon et al., 2000; Gomez- Limon et al., 2002).

Most of the economic models present however a severe drawback on agronomic and environmental aspects. The integration of the two faces of the problem, socioeconomic and environmental, seems therefore one of the real issues at hand. In this direction goes Mimouni et al. (2000) who analysed the tradeoffs between farm income and the reduction of erosion and nitrate pollution in a MC framework adopting a multiobjective programming model. At territorial level Garrido (2000) presented a model to evaluate water markets within agricultural sector which included the crop yield response to water and nitrates at farm level.

Nearly all the economic models have a very reduced applicability for two main reasons: firstly, they have been created for specific purposes and therefore are not easily accessible for further application; secondary they cannot be used directly by the final users, if not expert in modelling which is not the common situation, for the absence of a Graphical User Interface (GUI). DSIRR represents a step to fill in the previous gap.

An aspect which deserves specific attention in water modelling is the scale definition; in fact while the catchment represents the right level to deal with the environment and to define policy, farms identify the level where production is carried out and decisions on water use, land allocation, technology adoption are taken. In fact, at farm level all the actions, which in their interaction determine the state of the environment, are taken by independent actors moving in a common framework which is generally market driven. In the past catchment's modelling focused mainly on the aggregate. Application showed the main limits of this approach: a very simple household representation, with inadequate behavioural assumption, the production process poorly described, technology and innovation seldom considered. As a consequence the outcome of the analysis is not reliable since the agricultural activity is ill represented. The approach adopted by DSIRR permits the definition of distinct farm models, necessary to reach a good representation of the reality, and consider the constraints and opportunities deriving from the catchment, level of aggregation of the previous models.

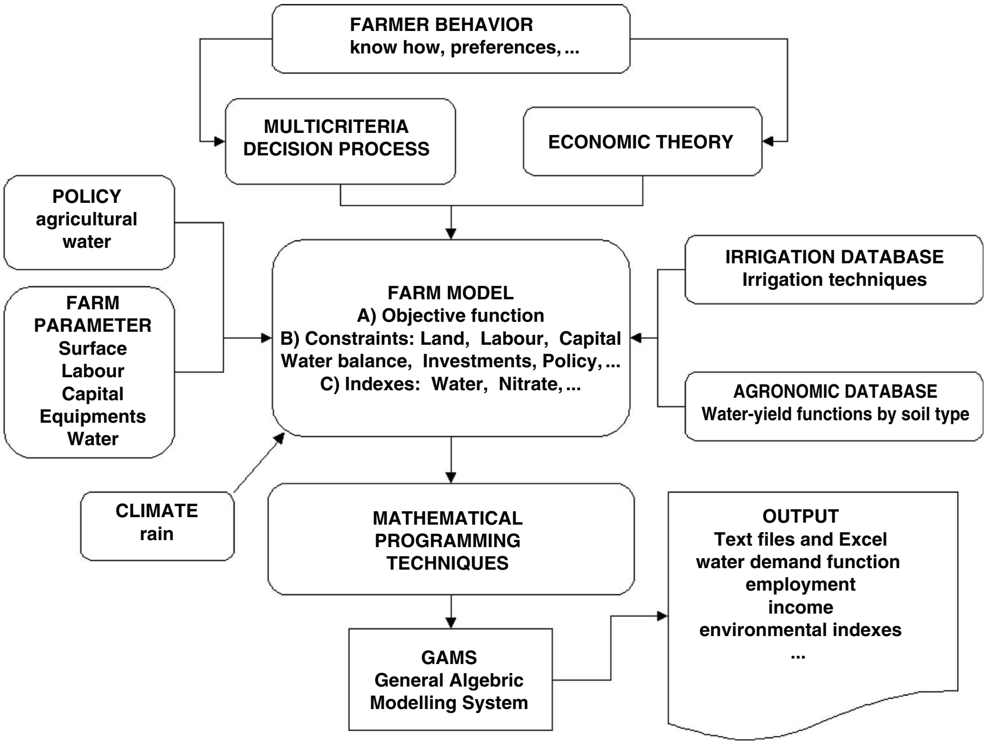


Figure 1. DSIRR farm model representation.

3 DSIRR

DSIRR can be seen as a scenario manager for agroeconomic models implemented in GAMS (General Algebraic Modelling System) (Brooke, 1992). The present beta non commercial version operates as a 32 bit Windows application, on a PC with at least 32 MB of RAM, more is recommended to improve speed and problem dimension. The code is written in Visual Basic and C++. The program requires GAMS package installed on the PC.

The DSS basically reproduces choices taken by actors (farmers) and estimates impacts in the social, economical and environmental dimensions. The approach followed is derived by the economic theory, enriched by the multicriteria paradigm, integrated with institutional, agronomic and environmental aspects, which represent input, output and condition of the production process (Fig. 1).

All the relevant aspects of the irrigation problem are considered: farming conditions, productive potential, crop diversification, technological packages, water availability, as well as labour and capital requirement and environmental dimensions. Different types of farms can be modelled ranging from small scale family farms to large commercial ones, from intensive fruit and horticulture productions to extensive cereals and industrial crops.

The user can construct the simulation to run defining the types of farm to consider and their specific characteristics without any specific knowledge of MPT and modelling techniques, but only of the agricultural and irrigation problem at hand, thanks to:

- a Graphical User Interface (GUI);
- modifiable implemented database to introduce coefficients;
- predefined equations which adapt to the context;
- internal logical rules which verify the coherence of the users' choices.

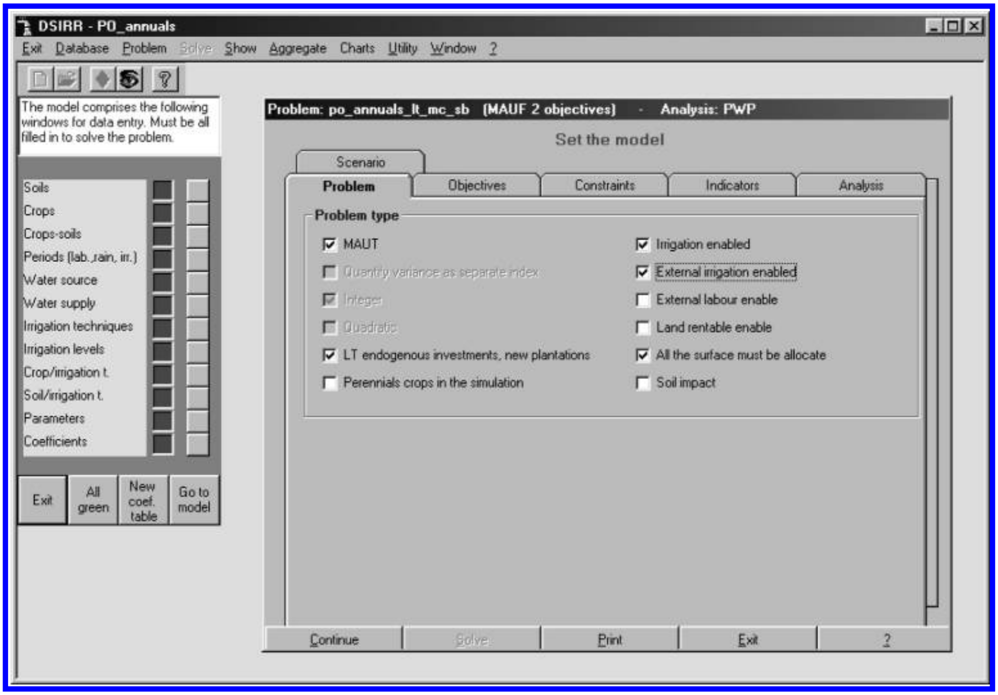


Figure 2. DSIRR user interface.

The necessary steps, i.e. set definition, parameter implementation, equation writing, model resolution, output generation and display are controlled by the main menu, the toolbar and the dialog windows of the GUI. Different utilities permit to access and modify databases, view reports and tables, create charts, adapt the working environment to specific needs (Fig. 2).

DSIRR can export the results to Excel in table and graphical form and maintain them in text form permitting an easy interfacing with other models and programs. Standard output includes: crop mix, balance sheet, irrigation level by crop and technology, labour, water requirements, environmental indices.

4 THE FARM MODELS

DSIRR analyses as an optimisation problem the conjoint choice of crops mix, irrigation level, technology and employment at farm scale. In general the farmer's problem is cast as a constraint maximization and in the simpler case can be formalized as:

$$\max_{\{X,W\}} \mathrm{INC}=\sum_{c}\sum_{i}\sum_{s}\left\{\mathrm{X}_{c,i,s}\left[\mathrm{p}_{c,i}\,\mathrm{q}_{c,i,s}\left(\mathrm{wr}_{c,i,s}\right)+\mathrm{su}_{c}-\mathrm{vc}_{c,i,s}\right]\right\}-\sum_{k}\sum_{l}\sum_{p}\mathrm{W}_{k,l,p}\,\mathrm{wp}_{k,l,p} \qquad (1)$$

subject to:

…

$$\sum_{s}\sum_{c}\sum_{i}\mathrm{X}_{c,i,s}\,\mathrm{ir}_{c,i,s} \le \sum_{l}\mathrm{W}_{k,l,p} \qquad \forall k,p \qquad (2)$$

where the indices represent: *c* crop, *i* irrigation level, *s* type of soil, *k* water source, *l* water provision level, *p* period. To distinguish between variables (endogenously determined) and parameters (exogenously fixed) the former are written in capital letters: *INC* income (€), $X_{c,i,s}$ activities (ha), $p_{c,i}$ crop market price (€/t), $q_{c,i,s}(wr_{c,i,s})$ crop production as function of water (t), $wr_{c,i,s}$ crop water requirements (m^3), su_c subsidies (€), $vc_{c,i,s}$ variable costs (€), $W_{k,l,p}$ water consumption (m^3), $wp_{k,l,p}$ water price (€/m^3), $ir_{c,i,s}$ crop irrigation requirements (m^3).

In equation 1, representing a farmer's objective function, production q is expressed as a function of water and irrigation costs are kept apart. This approach permits the derivation of *water demand function* (3) via parametrization of price or quantity.

$$W = f\left(wp; Q\right) \tag{3}$$

Function 3 determines the quantity of water W demanded by a farmer in a given district in a certain period as an inverse relation of its price wp, given the farm production possibilities and characteristics Q:

The objective function, which represents at farm level the farmer's behaviour, is the operating rule which the model follows. It can be specified as mono or multi criteria, distinctly by farm. In the latter case the Multi Attribute Utility Theory (MAUT) paradigm with a linear utility specification is adopted (Ballestrero et al., 1998), which Hwang (1981) showed approximate more complicated non linear form remaining far easier to use and understand.

In a MAUT framework the selection of the relevant objectives and the estimate of the related weights, representing their relative importance, can be conducted in different ways. A weighted goal programming minimizing the distance of model results from observed farmers' choices can be adopted (Sumpsi et al., 1996). Alternatively weights can be derived via interactive procedure with the decision maker.

Income can be defined as gross margin (GM), net income (NI) or profit (PR). GM represents the cash flow difference i.e. the operating income, NI considers also fixed costs, PR is the residual after farmer's own factors remuneration. Risk can be incorporated via the quadratic variance/covariance minimization or linearly as MOTAD, maximum and total semivariance (Romero, 2000). Among other criteria to minimize labour and a difficulty management index measuring farmer's concern of technical and organizational complexity.

From an economic point of view specific differences exist between short term (ST) and long term (LT) analysis as pointed out by Ward (2002). DSIRR can run both, differing in terms of decision variables, objectives and constraints. The ST represents the preseason moment in which the farmer must decide the crop mix, the agronomic aspects and the irrigation scheduling but is constrained by the existing investment in land, orchards and equipments. No new plantations are possible, neither farm size variation. Seasonal labour is another decisional variable. In the ST income is generally defined as GM. LT represents the planning period in which the farmer can modify completely the farm structure and make new investments. The size of the farm can be changed and new orchards can be planted. As far as irrigation is concerned new irrigation techniques can be adopted. This choice is endogenously determined. NI or PR identifies the income objective. For this latter opportunity costs for farmer' own factors, labour and assets, as well as investments depreciation must be considered. Inadequate perceived remuneration can push farmer to leave agriculture and to migrate elsewhere if conditions exist.

In all models a water balance cheques that water consumption, quantified on the basis of crop irrigation requirements, does not exceed farm water allotment which can be exogenously fixed. The user can specify various water delivery systems, i.e. private wells and basins, cooperative water distribution systems diversified according to the network (open canals, pipes, …). Water availability is defined at farm gate distinctly for periods. Water prices can be differentiated among them and for level of water consumption, simulating a block tariffs policy. If seasonality is considered more equations are required and separate demand curves derived. In this case a total per year water availability constraint can be introduced, water allocation among periods is endogenously determined following the marginal productivity rule.

Irrigation techniques can be fixed (like furrow and drip) or moveable (like sprinkler and guns), each characterized by proper coefficients defining: irrigation capacity (measured in litres for second of water distributed), irrigation efficiency at farm level, energy and labour requirements for distribution, equipment cost as annual depreciation, conservation and interest.

Water yield functions describing crops response to water can be constructed starting from experimental data or pseudo number generated by other models. Given the nature of MPM the consideration of the response curve, which is continuous in nature, is limited to few distinct points defined irrigation levels, this permit to keep the model linear.

Labour requirements for irrigation, diversified among technologies enter the model respecting the seasonality in the labour balance equation block. Labour is diversified in family and external. If requested different skill levels can be added.

Separate equations permit to consider financial aspects; this block controls that the operating cost, plus the investment quota in the LT, does not exceed the available financial capital plus borrowing which, if activated, determines another voice of cost.

Great attention has been given to agronomic aspects which often determine links among crops. Rotational constraints of different types can be easily included.

Environmental conditions, i.e. rain, water tableau, crop's water requirement, are considered. Ad hoc indexes can be estimated to assess nitrate, chemicals and energy use and soil covering.

Policy equations block reproduces the existing CAP regulation, the option to completely decouple subsidy suggested by the Mid Term Review is present.

Different analyses can be performed. Water demand curve can be derived via parametric analysis of water price or water allotment; in the latter the marginal of the constraint quantifies the shadow price of the resource. Income/risk efficient frontier can be estimated. Scenario analysis can be carried out to explore different policy options (variation in input and output prices, coupled and decoupled subsidies, environmental taxes, quota), environmental condition (rain distribution, water tableau level, water/yield function), technologies innovation.

5 A CASE STUDY

Italy is characterised by very differentiated climate and environmental conditions even if it is generally described as a Mediterranean country with cold winters and hot summers. Given this great variability local specificities must be properly considered and differentiated studies conducted, WFD explicitly requires analysis at catchment scale. The case study here presented considers the Po Basin, the larger irrigated plain area in Italy. Here some homogeneity can be easily identified, in fact statistical data available at national and regional level show that distinct production systems characterized by different crop mix coexist according to local specificity and vocationality. Among them: dairy and other animal breeding, rice and annual crops in the west side, fruits and vegetables in the east side. Each system represents in fact a highly specialized production district, this permits to describe the complex reality referring to representative farms in homogeneous areas.

The paper focuses on the annual crop system for the relevance in terms of surface covered and water demand, furthermore cereals which represent the main crop in this system, are highly subsidized under the current CAP.

Data collection integrates official statistical data (ISTAT), with private sources represented by the water board archives, farmer and producer association records and *ad hoc* on field inquiries conducted in the previous years via direct interviews to farmers to elicit their preferences and attitude to invest in new technologies or processes.

In the district different types of farm are present but family is the prevailing form. In the reference farm maize and sugar beet cover nearly 75% of the arable land. Industrial tomato even if limited to 11% represents an important crop for the higher return offered. Soya bean is marginally present, less than 5%. Wheat represents a feasible alternative in dry farming. Set aside complies with the existing CAP (10%). All the previous crops entered the simulation, plus alfa-alfa which at the moment is concentrated in the dairy production system but could be an interesting option under the Mid Term CAP Review.

The analysis explicitly considered seasonality. Five periods have been introduced, covering the dry period June–August with appropriate rain and water tableau apport specified on a monthly base. These data were later used to calculate the irrigation requirements and permit to analyse climate

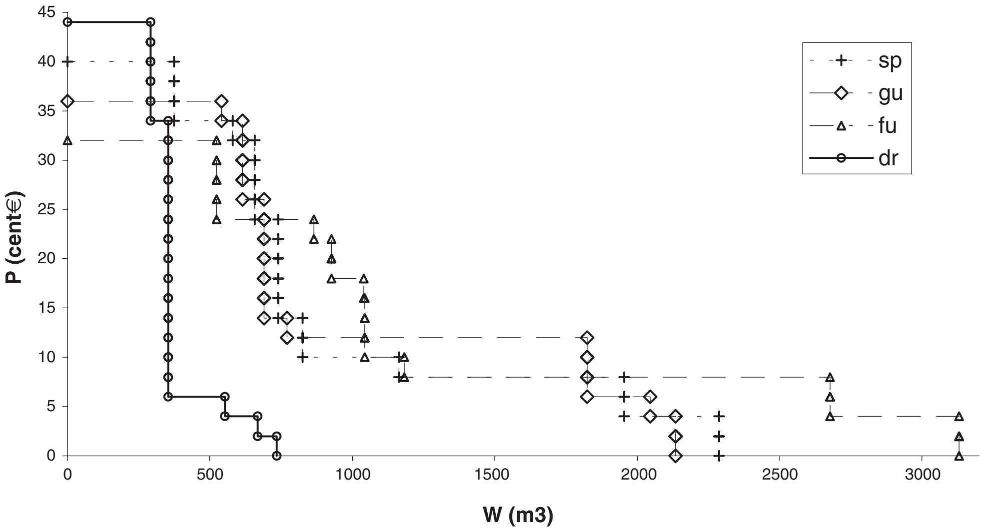


Figure 3. Water demand curves by irrigation technologies.

variability. Five irrigation levels have been defined to reflect water-yield relations, including the no irrigation option. The data have been derived from the experimental research conducted in recent years by the Reclamation Board for Emilia Romagna Channel (CER 1986). Data respect the decreasing marginal productivity of water. About 2300 m^3 are requested for maize full irrigated, production rises to 105 t/ha from 75 t/ha of rain feed on average. Higher irrigation intensity is requested for the tomato which cannot be produced under rain feed. Sugar beet full irrigated requests about 1600 m^3/ha, since irrigation reduces the sugar concentration the crop suffers a reduction in the market price. Soya bean is the less demanding crop in terms of water 900 m^3/ha. The irrigation levels specified into the model are characterized by proper requirement of the other inputs and variation in cost due to services are also considered. Prices and quantities refer to an average of the period 2000–2002. Flat rate are currently used.

Four irrigation technologies have been modelled, they are all potentially present in the region: gravity via furrow (fu), sprinklers (sp), self moving gun (gu), drip irrigation (dr). An increasing efficiency at farm level is gained via a progressive substitution of labour with capital. *Fu* has a negative impact on production, opposite effect has *dr* due to a positive interaction with the plant/soil system.

Figures 3 shows the WD curves estimated. The four curves present similar decreasing pattern but different shapes and intercepts. At zero water price the water consumption presents an impressive variation moving from 3130 m^3/ha (*fu*) to 734 m^3/ha (*dr*); the associated crop mix is: tomato, maize and sugar beet full irrigated, only with *dr* maize is not present. The incidence of irrigated surface drops with the most efficient technology *dr* (90%) from 100% to 45% excluding rain fed setaside. The curve, to the left, presents a strict vertical pattern starting at a price of 6 cents/m^3, level at which irrigated sugar beet leaves the field. Only tomato is irrigated up to a WP of 45 cents/m^3. To the right, opposite pattern shows gravity irrigation (*fu*) with an on farm efficiency of only 50%. Intermediate position are occupied by *sp* (70% eff.) and *gu* (80% eff.). From left to right a progressive substitution of labour with capital, which increases irrigation efficiency at farm level but also energy requirements and investment cost can be observed.

The four curves, respecting engineering and economical expectation, were used in the second part of the research to assess the impact of a water pricing policy and of CAP reform. The methodology adopted follows Gomez-Limon et al. (2002).

In the observed district the prevailing irrigation technology is *gu* and not *fu*. The analysis shows that at zero water price the latter would be the most profitable farm choice for a pure income maximizer

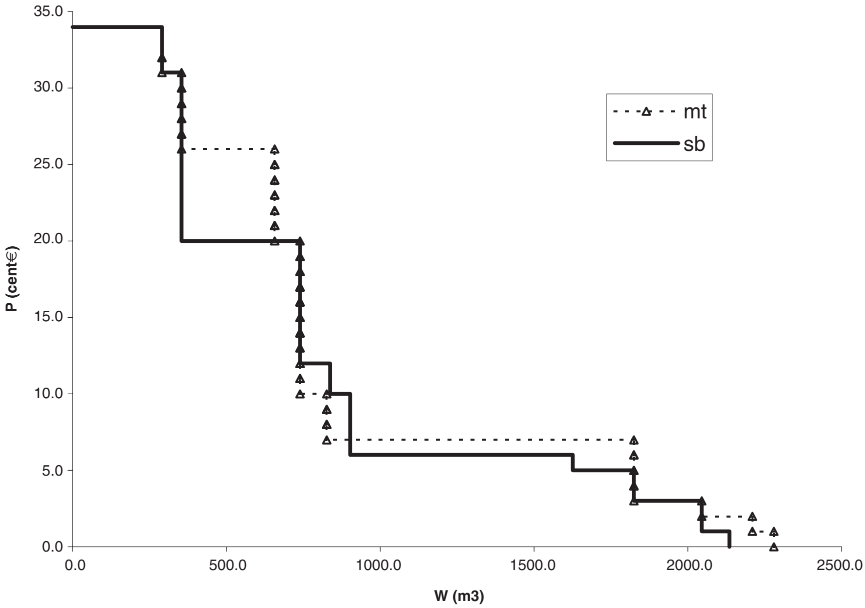


Figure 4. Water demand curves by policy scenario.

(MO) giving a NI of 670 €/ha requiring 34 h/ha, so the MC paradigm was tested. MAUT model was in fact able to reproduce the observed crop mix with higher precision having a bicriterial objective function composed by farm income to maximize (weight 0.68) and family labour to minimize (weight 0.32). Farm net income is over 600 €/ha about 91% of the maximum, but labour drops to 21.5 h/ha 64.5% of the previous figure. It should be noted that assuming a 7.5 €/h remuneration for family labour in MO residual PR is negative, but it turns slightly positive in MC for the reduction of worked hours.

Having calibrated the model at the current zero price, these weights were used to estimate water demand functions under two scenario, current CAP (SB) with a fixed subsidy of 420 €/ha for cereal and protoleaginous (COP) and Mid Term Review (MT) with a decoupled support fixed at 5000 €/farm, nearly 93% of the previous level and a moderate +3% increase of COP market price, and a reduction of −10% of alfa-alfa, set aside still apply.

Figure 4 shows water demand function in the two policy scenario analysed. Under SB the water consumption has a maximum at the null price of 2.135 m^3/ha and becomes null at a price of 34 cent €/m^3. At zero cost of water the optimum crop mix is given by maize and sugar beet full irrigated in a ratio 2/1, due to rotation, tomato at the maximum let by the market constraint, plus the set-aside requirement. Irrigation technology is *gu*.

The rising of the WP determines three interlink adaptations regarding: irrigation levels, technology, crop mix, which are all endogenous to the models. In fact the curves can be divided in three regions adopting different irrigation technology: the first (range 0–5 cent €/m^3) *gu*, the second (range 6–19 cent €/m^3) *dr*, the third (range 20–32 cent €/m^3) *dr*.

The cropping pattern are quite differentiated among them. Maize characterizes the first, this crop can be irrigated only below a WP of 5 cent €/m^3 under SB and of 6 cent €/ m^3 under MT. This crop is the first to leave the field to rain-fed wheat and irrigated soya bean, this determine a sharp drop in the demand. The smaller jumps inside regions are due to the progressive decrease of the crop irrigation levels. In the second range sugar beet and soya bean are partially irrigated. Irrigation on soya is abandoned at a WP of 11 cent €/m^3. Over 19 cent €/m^3 only processing tomato is irrigated. At a WP higher than 32 cent €/m^3 irrigation is not active and the crop mix is: wheat, soya bean and set aside.

Table 1. Indicators.

		SB						MT		
WP cent €/m³	WQ m³/ha	NI €/ha	LAF h/ha	SU €/ha	WAR €/ha	WQ cent €/m³	NI m³/ha	LAF €/ha	SU h/ha	WAR €/ha
0	2135	614	22	249	0	2280	622	23	231	0
5	1626	499	19	278	81	1824	508	21	231	91
10	838	439	18	278	84	739	432	19	231	74
15	739	404	19	249	111	739	396	19	231	111
20	354	328	14	362	71	657	356	18	231	131

The dotted line in figure 4 represents MT scenario: at null WP demand slightly increases +6.8% to 2280 m^3/ha. This change is due to the substitution of some maize and sugar beet with alfa-alfa, which becomes profitable under the new regime and covers about 20% of the surface. Both the curves present very similar decreasing pattern, but MT shifts to right in the WP range 0–6 cents € since alfa-alfa requires more water. In the range 7–12 MT exhibits lower values: soya bean is no more convenient and left the base. The shift from the second to the third region rises from 20 to 26 cent €/m^3. At higher WP the two curves coincide.

The indicators in table 1 show some impacts of water pricing and CAP reform. A WP of 5 cent €/m^3 permits to save 23% of the resource (WQ), while NI decreases of −19%, family employment of −10% (LAF), subsidies (SU) increase +12%. In the MT scenario the negative impact on NI presents the same magnitude, the reduction is only in part due to a transfer to Water Authorities (WAR). Extensivation is captured by the LAF index showing a decreasing trend.

6 CONCLUSIONS

DSIRR is an innovative simple DSS which permits to support demand management in agriculture, estimating water consumption patterns via MPT integrating micro analysis at farm level with macro analysis at catchment's scale. The program permits long and short term demand forecasts, paying high attention to the influence of socio-economic structure and conditions. Sensitivity analysis can be carried out to verify the influence of key parameters linked to climate, technologies, markets. Among other advantages of the DSS can be mentioned:

– a Graphical User Interface which permits a direct control of the simulation by the user;
– integration of economic, agronomic and engineering components;
– flexibility to adapt to different data availability in distinct production contexts;
– personalized database which can be adapted and reused;
– a rich set of models that apply well founded MPT;
– high realism in the farmer's behaviour representation due to a multicriterial approach;
– simple derivation of water demand function;
– richness of information produced covering socio, economic and environmental aspects;
– sensible reduction of cost, time and effort to conduct sound studies.

DSIRR is constructed in a modular approach. This permits a continuous development of the program which can be easily linked with other models and software.

The DSS is currently used in the context of the EU WADI project to study the impact of WFD on irrigated agriculture in Italy (Bazzani et al., 2002). Among the main conclusions derived from the ongoing research emerge clearly that the impact of water pricing is highly diversified among the production systems. Irrigation will continue to be adopted, even at higher WP, at least in the most professional and highly profitable ones such as orchards, vegetables, flowers, dairy and other animal breeding, in the more vocated agricultural areas. Structural changes due to the concurrent action of the CAP reform and WFD will probably determine reduction of incomes in agriculture and a major expulsion of labour from the sector. This process can be accelerated or contrasted by macro-economic situation and the definition of specific interventions.

The annual crop system of the north west Po Basin is quite sensible to a water price increase. The implementation of such policy which could have here strong effect in terms of water saving must anyhow consider the technical feasibility and the transaction cost involved. At present great part of the existing network is represented by open channels, in this situation metering it is difficult to implement. Rationating the resource could in fact represent a feasible alternative but this could increase water tableau extraction with negative environmental impacts, opposite effect can have the use of surface water which can favour their recharge.

The application showed the relevance of the adoption of more efficient irrigation technologies to save water, i.e. shifting on annual crop system from furrow, to drip, via sprinklers and guns; similar results appear in other production systems, moving for instance to micro-irrigation in orchards and vegetables. This shift is anyhow costly for the farmers, therefore the adoption of proper interventions, which can be analysed *ex ante* with the DSS, can play a determinant role to move the system toward more sustainable patterns.

The support coming from DSIRR to define water conservation policies seems valuable.

REFERENCES

Amir, I., Fisher, F.M., 1999. Analyzing agricultural demand for water with an optimizing model. *Agricultural System* 61, 45–56.

Ballestero, E., Romero, C., 1998. *Multiple criteria decision making and its applications to economic problems*. Kluwer Academic Publishers, Amsterdam.

Bazzani, G., 2003. A DSS For An Integrated Analysis Of Irrigation At Farm And Catchment Level: DSIRR, paper accepted for presentation at *Modsim 2003 International Congress on Modelling and Simulation*, 14–17 July 2003, Townsville, Queensland, Australia.

Bazzani, G., Di Pasquale, S., Gallerani, V., Viaggi, D., 2002. Irrigated agriculture in Italy and water regulation under the EU Water Framework Directive. *Irrigation water policies: Micro and macro considerations* Proceedings of International Conference, Agadir, Morocco, 15–17 June 2002.

Berbel, J., Rodriguez, A.A., 1998. MCDM approach to production analysis: an application to irrigated farms in Southern Spain. *European Journal of Operational Research* 107, 108–118.

Brooke, A., Kendrick, D., Meeraus, A., 1992. GAMS *A user's guide*. The Scientific Press.

Consorzio di Bonifica di secondo grado per il Canale Emiliano Romagnolo, 1986, *Schede irrigue*, Edagricole, Bologna.

Dinar, A., Subramanian, A., 1997. Water Pricing Experiences. An International Perspective. *World Bank Technical Paper* No. 386. Washington, DC.

Doppler, W., Salman, A.Z., Al-Karablieh, E.K., Wolff, H., 2002. The impact of water price strategies on the allocation of irrigation water: the case of the Jordan Valley. *Agricultural Water Management* 55, 171–182.

European Council Directive 2000/60 Establishing a Framework for Community Action in the Field of Water Policy.

Garrido, A., 2000. A mathematical programming model applied to the study of water markets within the Spanish agricultural sector. *Annual of Operational Research* 94, 105–123.

Global Water Partnership, 2002. *Policy Guidance and Operational Tools, Toolbox*, Integrated Water Resources Management.

Gómez-Limón, J.A., Arriaza, M., 2000. Socio-Economic and Environmental Impact of Agenda 2000 and Alternative Policy Choices for Market Liberalisation on an Irrigated Area In North-Western Spain. *Agricultural Economics Review* 1, 18–30.

Gómez-Limón, J.A., Arriaza, M., Berbel, J., 2002. Conflicting Implementation of Agricultural and Water Policy in Irrigated Areas in the EU. *Journal of Agricultural Economics* 53, 259–281.

Howitt, R.E., Watson, W.D., Adams, R.M., 1980. A Re-evaluation of Price Elasticities for Irrigation Water. *Water Resources Research* 16, 623–28.

Hwang, C.L., Yoon, K., 1981. *Multiattribute decision making*. Springer Verlag: New York.

ISTAT (various years): *Statistiche agricole*, Roma.

ISTAT (various years): *Statistiche ambientali*, Roma.

Joahansson, R.C., Tsur, Y., Roe, T.L., Doukkali, R., Dinar, A., 2002. Pricing irrigation water: a review of theory and practice. *Water Policy*. 4, 174–199.

Mckinney, D.C., Ximing Cai, Rosengrant, M.W., Ringler, C., Scott, C.A. 1999. Modeling Water Resources Management at the Basin Level: Review and Future Directions, *SWIM Paper n. 6*, System-Wide-Initiative on Water Management, Colombo, Sri Lanka.

Mimoumi, M., Zekri, S., Flichman, G., 2000. Modelling the trade-offs between farm income and the reduction of erosion and nitrate pollution. *Annals of Operations Research* 94, 91–103.

Romero, C., 2000. Risk Programming for Agriculture Resource Allocation: A Multidimensional Risk Approach. *Annals of Operations Research* 94, 57–68.

Romero, C., Rehman, T., 1989. *Multiple Criteria Analysis for Agricultural Decision*. Elsevier: Amsterdam.

Schaible, G.D., 1997. Water Conservation Policy Analysis: An Interregional, Multi-Output, Primal-Dual Optimization Approach. American *Journal of Agricultural Economics* 79, 163–177.

Sumpsi, J.M., Amador, F., Romero, C., 1997. On farmers' objectives: A multi-criteria approach. *European Journal of Operational Research* 96, 64–71.

Tsur, Y., Roe, T.L., Doukkali, R., Dinar, A.,D., 2002. Efficiency and equity implications of irrigation water pricing. *Irrigation water policies: Micro and macro considerations.* Proceedings of International Conference, Agadir, Morocco, 15–17 June 2002.

Varela-Ortega, C., Sumpsi, J.M., Garrido, A., Blanco, M., Iglesias, E., 1998. Water Pricing Policies, Public Decision Making and Farmers response: Implications for Water Policy. *Agricultural Economics* 19, 193–202.

Ward, F.A., Michelsen, A., 2002. The economic value of water in agriculture: concepts and policy applications. *Water policy* 4, 423–446.

Chapter 9 – Water consumption trends and demand forecasts

Lifestyle and its effects on domestic water use

David Burnell
Management Science Department, Lancaster University

ABSTRACT: This paper sets out a fresh approach to a core water industry challenge, to explain how water is used by its wide spectrum of domestic customers. Most raw data on domestic water use comes from monitoring panels, scaled up as if water use variation were random. Analysis of panel data in this study shows most households have a recurring pattern of daily water use. However, there are a wide variety of patterns in the average diurnal usage for these sites – a diversity of water-use lifestyles.

Principal Component Analysis helps use-patterns to be classified. Key differences arise on night and middle-day use. Differences in diurnal profile over time, e.g. getting-up water-use patterns on weekdays and at weekends, offer extra insights on water-use lifestyle. Changes in diurnal patterns can also be seen in metered District data during half-term holidays and at the start of Ramadan. Combining these insights with ward-level data from the UK 2001 Census should enhance understanding of water-use diversity.

1 INTRODUCTION

A solid understanding of the makeup of domestic water use is essential in day-to-day network management (e.g. leakage control, demand prediction) and to longer-term water resource planning. This paper, which is part of long-term research on fully exploiting District Meter data, outlines some ways to enhance this understanding.

Conventional wisdom is to scale up panel and cul-de-sac study data as if water use per occupant is broadly homogeneous for each region and housing type. This paper demonstrates a diversity in patterns of usage between households within the day, and proposes ways of classifying and exploiting these.

Raw data for the study is logged 15 minutes usage data taken from a water company water-use monitoring panel. Since the main purpose of this paper is to present ideas, only a subset of this data is used in the analysis. The subset comprises 311 urban terraced houses in autumn 2001. This period excludes summer weather effects, which warrant separate analysis.

2 A LIFESTYLE MODEL OF DOMESTIC WATER USE

2.1 *Raw data on domestic water use in the UK*

The UK does not have universal revenue metering on domestic water users. Knowledge of water use is drawn from special panels of customers and from cul-de-sac studies put together by the water companies (Herrington 1996, Scottish Office 1993). Each method encounters uncertainties on how the sample should be scaled up to represent the whole population, and on measurement issues (e.g. distinguishing network leakage from persistent in-house wastage).

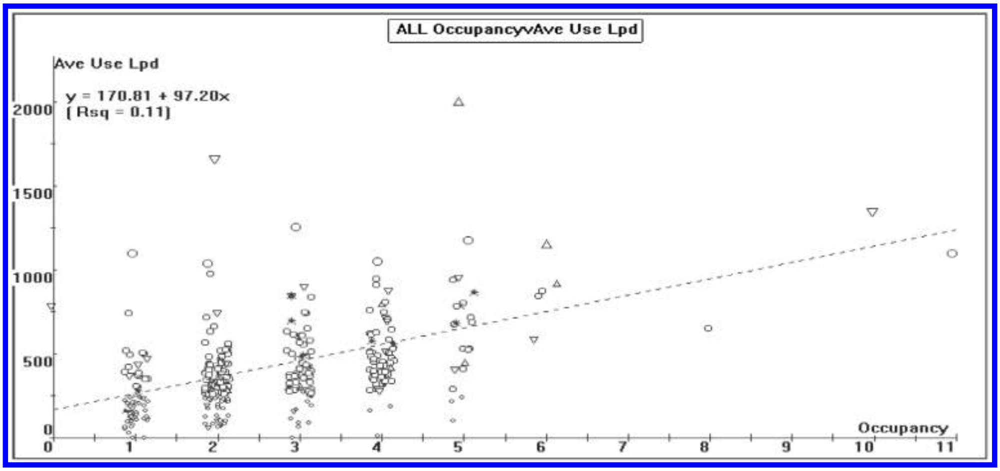


Figure 1. Occupancy vs average daily use for sites in sample.

The underlying assumption on scale-up is that domestic water users are broadly interchangeable, once household size and housing have been taken into account, so differences will "cancel out" once a large number of such customers are aggregated. This is fine for large-scale, long-term planning. However, if one wants to predict and make sense of usage in local communities – such as a metered District – then one needs to quantify and categorise differences which may arise in these communities.

The broad consensus between the various sources on average domestic water use (e.g. the Scottish study (1993) claims consistency with results from Ofwat) masks a great deal of variation within the sites studied. This variation is illustrated in Fig. 1, which plots average daily usage for the sample sites against occupancy. Each occupancy has a huge spread of usage values.

This variation may be purely random, in which case it will even out as the sample is aggregated; or it could be systematic and varying from one type of household to another, in which case this variation needs to be classified and explained.

2.2 *Water-use lifestyles*

This study seeks ways to assemble and classify such differences. The starting point is to recognise that households

- have a variety of occupants, at various stages in life;
- work to different timetables and deadlines;
- apply different attitudes e.g. to personal cleanliness and appearance;
- behave differently at different times of the day and week;
- vary their behaviour over the year (e.g. term-time vs holidays).

In short, households have a range of lifestyles which will be reflected in their style of water use.

Water-use panels provide a long history of diurnal usage patterns (typically 15 minutes frequency) for a sample of a few thousand cooperative households. This gives data not only on overall usage and the mix of large- and small-usage events, but also information on when and how often these events occur. The following "water-use lifestyles" will lead to a diversity of usage patterns within the day and over time:

a) whether someone in the household has to get up for a tight work-deadline;
b) whether there is anyone regularly at home during the day;

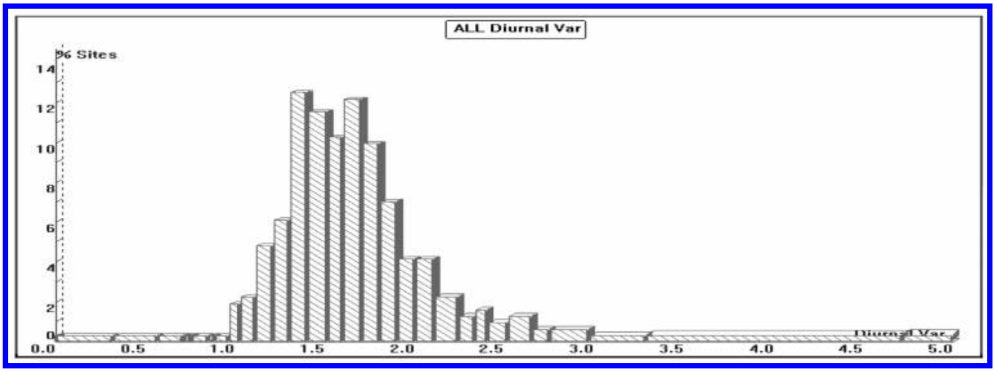


Figure 2. Distribution of diurnal variability for sites in sample.

c) the frequency of washing machine use, which may depend on the type of job and the importance of smartness in the workplace;
d) whether there are babies who are provoking night activity;
e) school-age children, who will rise and go to bed later during school holidays;
f) if and how often the household goes away on holiday together;
g) bathing and showering regime for each occupant (e.g. morning vs evening, daily vs weekly);
h) the amount of water normally used in each of these bathing activities;
i) the amount of social activity in the evening;
j) the spread of times at which the household normally goes to bed;
k) whether the weekend's activity, in terms of water use, is different from weekdays;
l) whether religious festivals (e.g. Ramadan) will change the household routine;
m) the "cohesion" of the household, e.g. is it operating as a family or as a collection of individuals, such as a household of students.

Lifestyle patterns will reflect habits formed in upbringing and so be persistent over time.

2.3 *Diurnal consistency within individual households*

Inspecting sites within the panel shows individual households generally have a persistent water-use pattern. One way to quantify this is to compute, for each site, its variability in usage for each distinct quarter-hour of the week over the study period. This is then summarised for each site by comparing the ratio of the average standard-deviations over the week to the average mean. Across the sample sites this "diurnal variability" has an average value of 1.62, as shown in Fig. 2.

To see if this is good enough for pattern-recognition purposes, the sites are sorted in ascending order of diurnal variability so one can inspect sites along this range. For a site with low diurnal variability, and superposing usage over the 24 hours on each Tuesday and Wednesday of the study period gives a strongly repeated pattern, as shown in Fig. 3.

For a mid-range site, there is still a good level of consistency, as shown in Fig. 4

Even for a site with high diurnal variability, 75% of the way along the variability distribution, superposing successive Tuesdays and Wednesdays shows a consistent pattern, see Fig. 5.

Whilst these three sites are each self-consistent, they are very different from each other; each site is using its water at different times during the day.

2.4 *Diurnal variation between sites*

So individual sites are self-consistent in their water-use pattern – the household's habits give rise to a repeated "fingerprint". But how do these fingerprints differ from site to site? To quantify this each day is divided into standard "time-slices" to capture key aspects of water use:

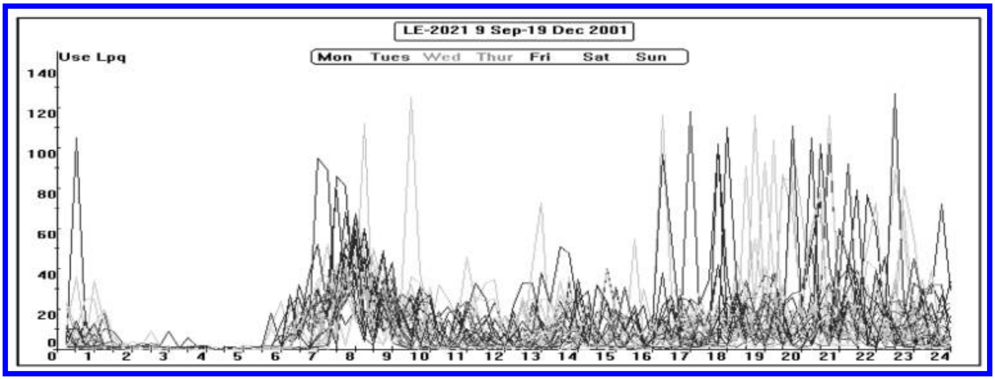


Figure 3. Superposed diurnal usage, Sept–Dec 2001, for a low-variability site.

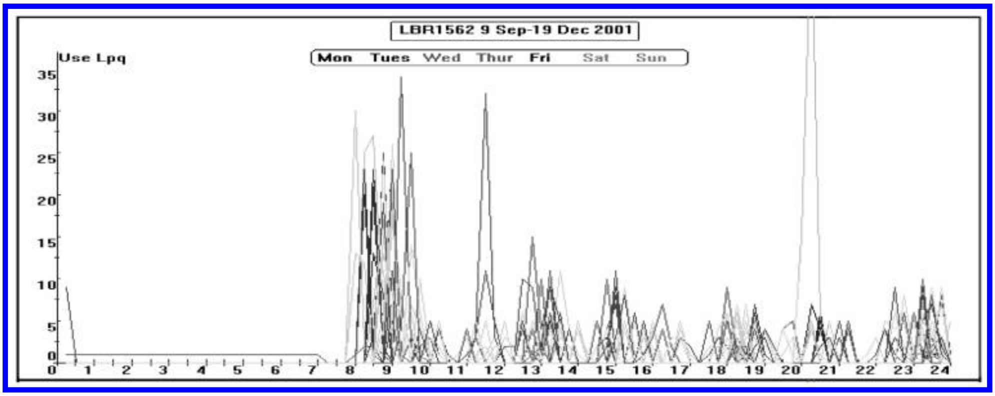


Figure 4. Superposed diurnal usage, Sept–Dec 2001, for a medium-variability site.

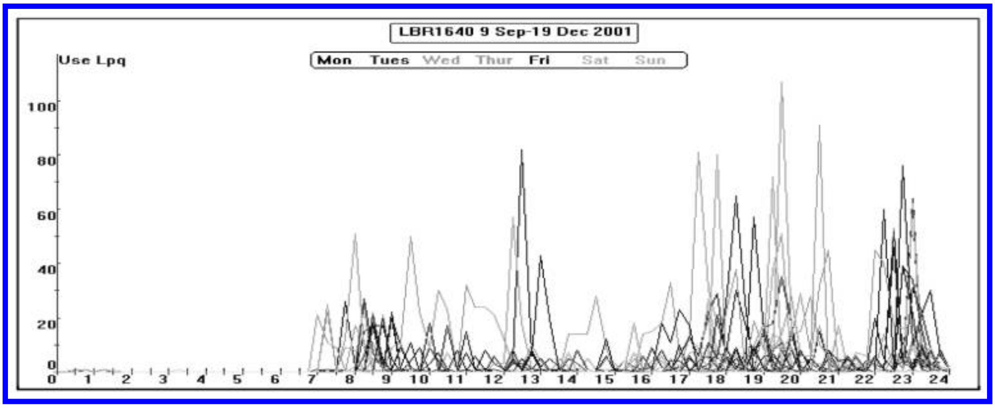


Figure 5. Superposed diurnal usage, Sept–Dec 2001, for a high-variability site.

- Night-time midnight–3 am
- Dead-of-night 3 am–6 am
- Rising 6 am–9.30 am
- Morning 9.30 am–12 noon
- Afternoon 12 Noon–3 pm
- Kids home 3 pm–5 pm
- Teatime 5 pm–7 pm
- Evening 7 pm–midnight

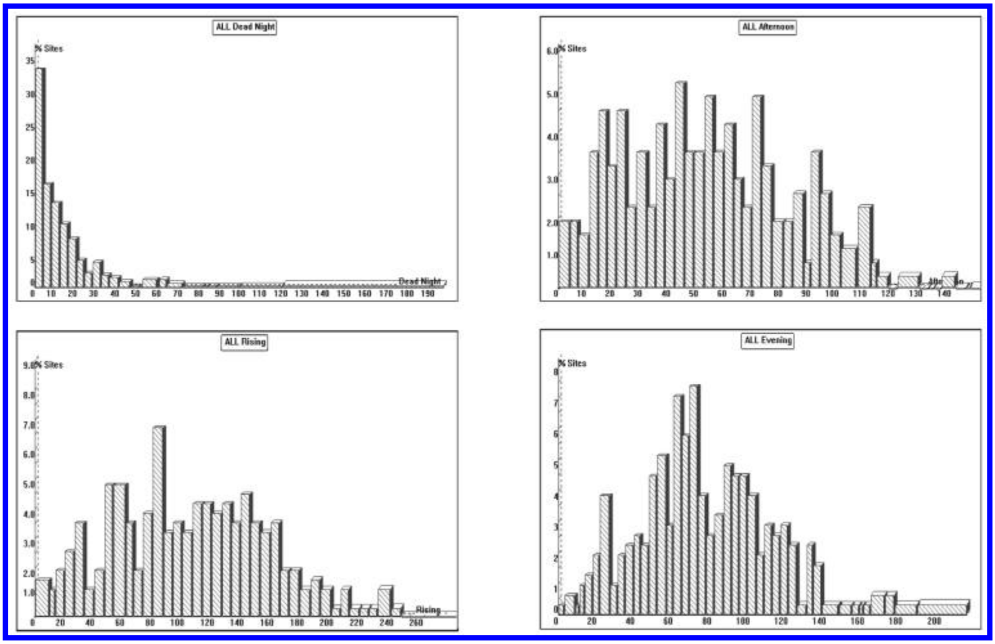


Figure 6. Distribution of normalised flowrates for the dead-of-night, rising, afternoon and evening time-slices.

Site usage is computed by time-slice. The average usage-rate during each time-slice is then normalised by comparing it with the flow-rate for that site over the day.

Fig. 6 shows a wide range of values across the sites in all but the dead-of-night time-slice, and no consensus on the proportion of a site's usage which should occur. Even in the "rising" time-slice – where one might expect all households to have a lot in common – there is substantial variation from 0 to 240% of average use. Thus the "standard diurnal domestic profile" frequently cited (e.g. Race & Burnell, 2001) turns out to be made up of a wide variety of diurnal shapes.

2.5 *Classifying time-slice-use variation: principal components*

One way to classify time-slice variation is to apply Principal Component Analysis to pick out the most significant differences across all sites. This method, well-described in Krzanowski, (1988), looks for the most distinctive linear transformations which can be applied to the data. The explained-variance and details of the first six Principal Components are given in Table 1.
Examining the largest absolute values and their sign in the first three columns:

- The first column, with all values similar, captures the size of site usage
- The second segregates on night and dead-of-night usage
- The third distinguishes rising use and high daytime usage

The first and second components of time-slice usage by site is shown in Fig. 7. Similar points will be clustered together. The size of third and fourth components of each site are indicated by the horizontal and vertical lines at each point.

Fig. 7 provides a starting point for grouping sites into common-lifestyle classes. For future analysis it is encouraging that variation in day-time use emerges as a key distinguishing feature, since this may be linked to economic factors such as employment. The 2001 UK Census captured employment and a wealth of other data (e.g. ethnicity) at Ward level. Using boundary polygons and data on property locations, this is transferable to Metered Districts. Further work could then refine water-lifestyle classification schemes for different types of community.

Table 1. Principal components of time-slice flowrates for sample sites.

Variance	6.039	0.723	0.493	0.266	0.237	0.118
	75.5%	9.0%	6.2%	3.3%	3.0%	1.5%
Night	0.33	−0.52	−0.26	0.35	0.46	0.45
Dead-of-night	0.32	−0.63	−0.11	−0.26	−0.50	−0.35
Rising	0.32	0.38	−0.60	0.34	−0.47	0.11
Morning	0.36	0.21	0.45	0.51	−0.06	−0.24
Afternoon	0.39	0.01	0.38	0.11	0.02	0.01
Kids home	0.39	−0.03	0.30	−0.21	−0.04	0.11
Tea-time	0.36	0.30	0.03	−0.56	−0.03	0.49
Evening	0.36	0.21	−0.34	−0.23	0.55	−0.59

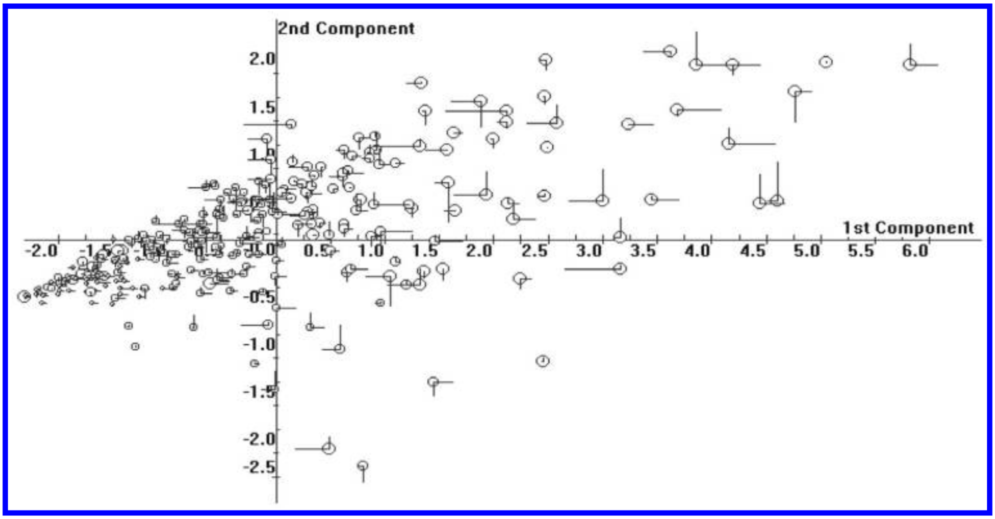


Figure 7. First two principal components of variation in time-slice usage for sites in sample.

3 PATTERN DIFFERENCES OVER TIME

Further insights on water-use lifestyle come from the way patterns for individual households and for communities vary over time in response to common events. These include holidays, religious festivals and hot-weather. The effects can be seen both on site and District usage data.

3.1 *Comparisons of weekday and weekend rising patterns*

In general:

- Households which are economically active or have school-age children will have water-use patterns at weekends which differ from weekdays. There will be fewer early-morning deadlines and the pattern of water use then may be significantly different.
- Households which are economically inactive and without children are much more likely to have a similar rising pattern – whatever it is – at weekends as on weekdays.

The following calculation for each site allows this to be quantified:

- compare the cumulative usage for average weekday, weekend profiles from 5 am to 10 am;

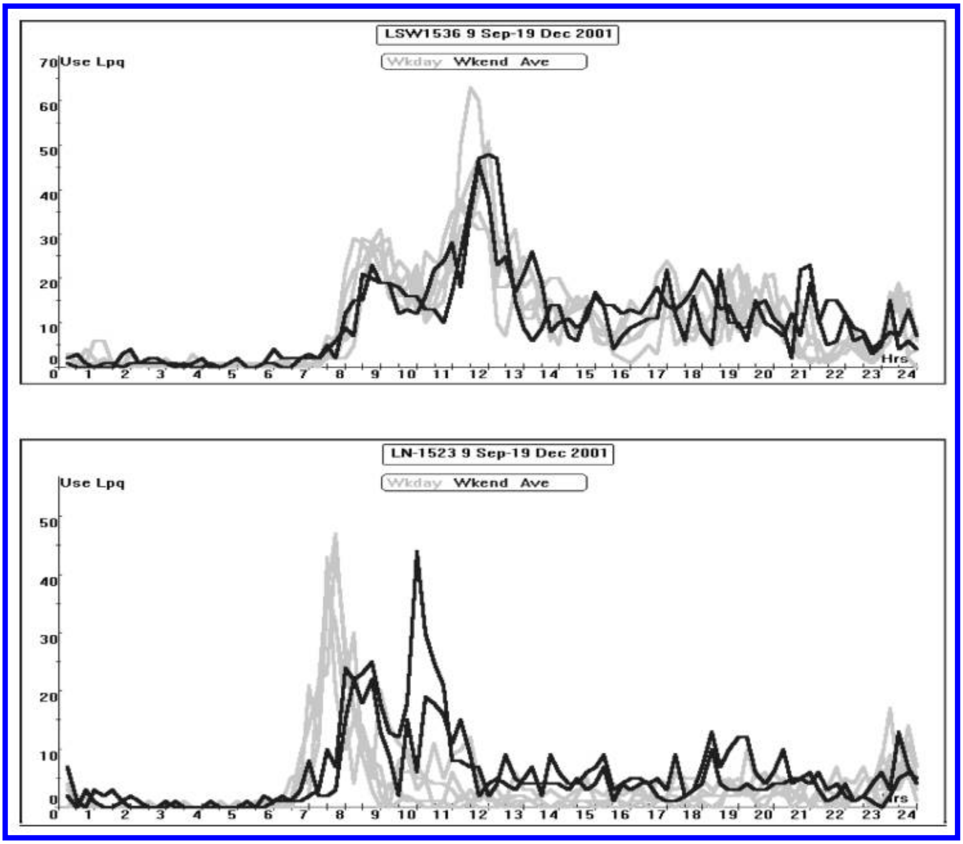


Figure 8. Average weekday and weekend usage profiles for similar and less-similar sites.

- find the average difference between these cumulative values;
- express the difference as a ratio of average use to give a "weekend-difference factor".

If the sites are then arranged in ascending order of this factor, inspection gives the threshold at which significant differences emerge.

The 50th site out of 311 (16%) has average Monday–Friday profiles and Saturday–Sunday profiles as shown in Fig. 8. There is virtually no difference in "rising lifestyle". In contrast, by the 130th site, 40% of the way through the list, a significant contrast emerges.

Fig. 9 plots the number of children at each site against the weekend-difference measure, most of the low-difference sites are ones with no children. Many of these may be retired households.

3.2 *Pattern variation in metered District usage: Ramadan*

Another "common event" which may affect local communities is religious festivals. Since the ethnicity of the sample water monitoring panellists is unknown, this is best studied by comparing diurnal patterns in metered Districts before and after the key dates. One event which can be studied in this way is the start of the Muslim fast of Ramadan, as shown in Fig. 10.

If the weekday diurnal profiles pre and post Ramadan are superposed, a clear pattern emerges, as shown in Fig. 11. Observers of the Fast rise at 4–5 am so they can breakfast before dawn; there is correspondingly lower usage in the normal 8 am morning peak.

If leakage is being estimated from the District's Minimum Night Line, care is needed to avoid confusing extra legitimate night use at, say, 4 am, with a rise in leakage.

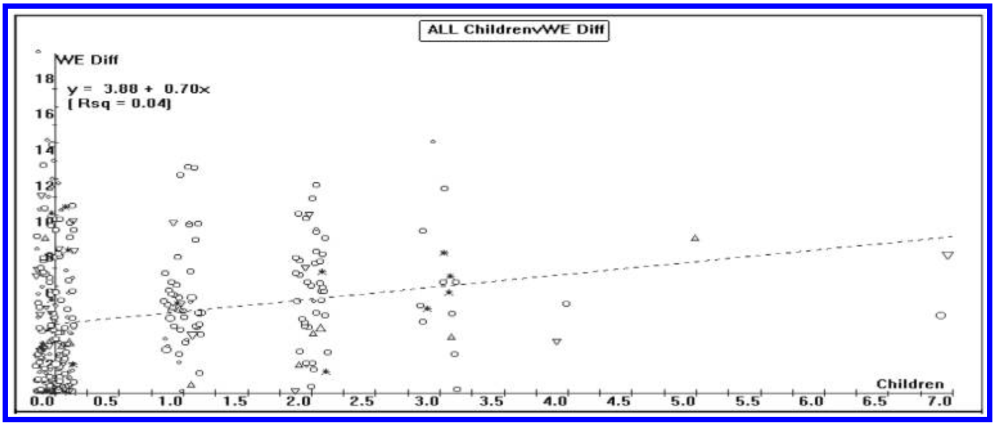


Figure 9. How weekend variation increases with number of children in the household.

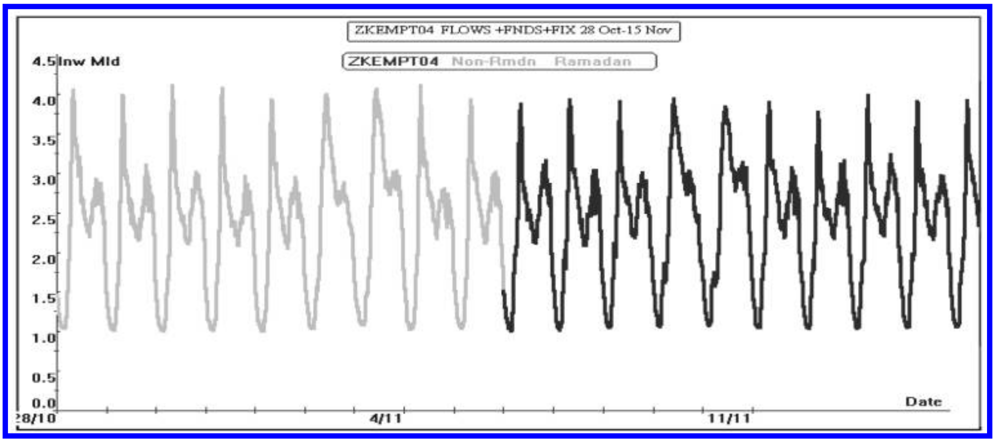


Figure 10. Metered usage in a Muslim District over the fortnight covering the start of Ramadan.

3.3 *Pattern variation in metered District usage: half-term holidays*

The effect of half-term holidays on usage can be similarly studied. Weekday diurnal usage patterns in term time and at half-term for the District are shown in Fig. 12. There is a pronounced "slump", presumably as many school-children rise later.

This change in pattern could provide a cross-check on the panellists with children. These diurnal pattern changes need to be taken into account when interpreting metered District data, as discussed elsewhere (Burnell 2003).

4 CONCLUSIONS AND FURTHER POSSIBILITIES

A single-product undertaking such as a water company requires the best possible understanding of how is main product is used by its wide range of customers, both to operate its existing assets effectively and for longer-term water-resource planning.

For most UK companies the prime source of understanding is domestic water use monitoring panels. These give data not only on total use per site but also on when this usage occurs over the day.

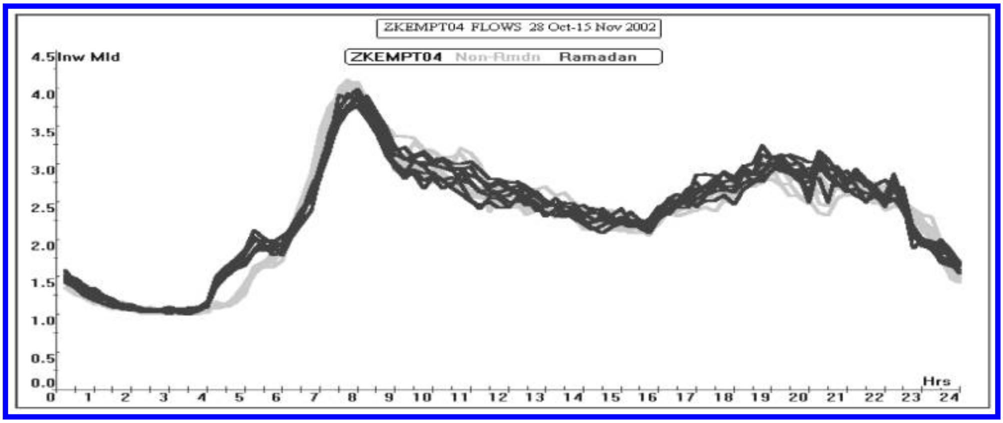


Figure 11. Metered District diurnal usage before and during Ramadan.

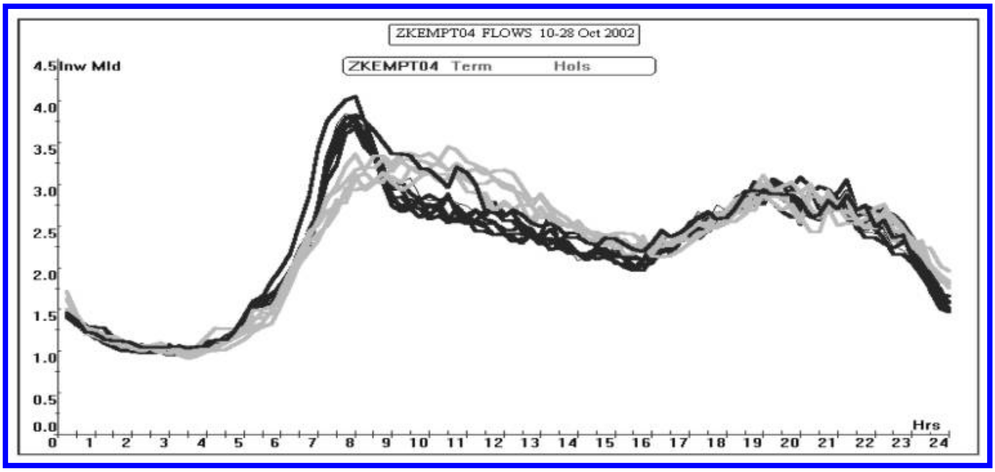


Figure 12. Metered District diurnal usage pattern in term-time and in half-term holidays.

However, there is substantial variation in diurnal usage patterns between sites, reflecting a diversity of water-use lifestyles. This paper has explored how these usage patterns might be classified and linked to the variety of ways (and times-of-day) that water is used within the home.

Principal Component Analysis can be applied to the pattern of use in each part of the day. The fraction of use in the middle of the day is picked out as one significant factor. These ideas could be developed into a formal water-lifestyle classification scheme.

Differences in diurnal profile for the same sites or communities over time are also significant. Ways of discriminating between sites using their weekday and weekend rising patterns are presented, with childless sites shown to have very similar rising patterns. Clear differences are also shown at the start of Ramadan in a mainly-Muslim District and during half-term.

Metered districts offer an alternative way of measuring domestic usage – providing the district is hydraulically "tight" and its demographic makeup is known. The 2001 Census data contains a great deal of information which will soon be available at the Ward level. This should allow metropolitan districts to be characterised not only in household size and housing but also by employment, wealth and ethnicity.

REFERENCES

Herrington P, 1996. *Climate Change and the Demand for Water*, Department of Environment.
Scottish Office, 1993. *Domestic Water Consumption in Scotland in 1991*, Scottish Office.
Race J.J. & Burnell D.A., 2000. *Analysis of Domestic Demand Patterns*, American Society of Civil Engineers, Minneapolis.
Krzanowski W.J., 1988. *Principles of Multivariate Analysis.*
Burnell D.A., 2003. *Auto-validation of District Meter*, Advances in water supply management.

Pattern of real water demand and its influence on water meters recordings

Wojciech Koral
Institute of Water and Wastewater Technology, Silesian University of Technology, Gliwice, Poland

ABSTRACT: Before 1989 real water usage in Poland was estimated as the water supplied to the water network system and real water losses was unknown. After economical changes the sale of water became the main source of profit for Water and Sewage Utilities, so measurement of water usage is now the most significant issue. At the same time unitary daily water consumption dropped drastically, i.e. in chosen objects (described in paper) more than two times . It causes very serious problem with correct water meter sizing (for new connections and during modernisation) so that the range of measurement covers all recorded flows (both maximal and minimal – leakage in water supply system).

The paper shows the results of research of real water demands (recorded every second during minimum ten days) for a few eleven-storey buildings with different number of flats (from 80 to 330 flats) and influence of demand patterns on correctness of water meter indications. Additionally the results of comparison research of water meters with the same diameter but from different producers was run to show the fact that usage of "C" class water meters (but still oversized) has little influence on water balance for chosen buildings in comparison with water meter of smaller diameter but chosen in accordance with real water demand pattern.

1 INTRODUCTION

Since 1992 a decrease in water consumption has been recorded in Poland. Is was caused by common usage of water meters, application of new, leakproof and water saving sanitary equipment. Probably recession and increase in price of water were additional reason.

At the same time standard used to choose a water meter diameter haven't changed since 1992 (this standard is based on German DIN 1988). Equation used:

$$q_o = 0.682*(\Sigma q_n)^{0.45} - 0.14 \quad (1)$$

$$q_{nw} = 2*q_o \quad (2)$$

where: q_o = design flow [dm^3/s], Σq_n = sum of nominal flow of sanitary equipment [dm^3/s], q_{nw} = nominal flow, used to choice a diameter of water meter [dm^3/s] reflects the state of sanitary and house equipment used 10 years ago. It makes it necessary to find a new method of choosing water meter diameter.

Research which was done in cooperation with Water and Sewage Utility in Gliwice, Poland, shows that usage of oversized water meters (chosen in accordance with standards) causes serious measuring error of water meters indications and increases apparent water losses, revealed by water utilities.

The paper shows the results of research of real water demands (recorded every second during minimum ten days) for a few eleven-storey buildings with different number of flats (from 80 to 330 flats) and influence of demand patterns on correctness of water meter indications. Additionally the results of comparison research of water meters with the same diameter but from different producers was run to show the fact that usage of "C" class water meters (but still oversized) has little influence on water balance for chosen buildings in comparison with water meter of smaller diameter but chosen in accordance with real water demand pattern.

2 METHODOLOGY

Research was done in three chosen objects: two separate buildings and a group of four buildings with different number of flats. All buildings are eleven-storey and they are supplied locally with warm water.

All objects are supplied by a local pumping station equipped with frequency converter controlled pumps. There are 80 flats in first building ("I"), 132 flats in second one ("II") and totally 330 flats in the third object ("III").

Research of demand pattern in the first and second buildings was done by means of an electromagnetic flow meter, installed in series with water meter. In the third object readouts of flows were done by means of water meter equipped with optoelectronic transmitter. Water meter was installed in local pumping station supplying object "III". Studies were done in summer months 2001, 2002 and at the turn of 2003. Flows were recorded every second. In the third object flows have been recorded since September 2002.

Comparative studies of "B" class, "C" class and compound water meters were done between May and November 2001 in object "I". All water meters were installed in series with compound one. Distance of five diameters before the meters and three diameters after the meters was kept. Additionally water supply system was equipped with filter to prevent water meters from mechanical damage.

3 WATER CONSUMPTION (1993–2002)

Since 1992 in Gliwice drop in water consumption has been recorded. It refers to both industry and housing. That tendency can be also observed in the objects under research, which is showed in Figure 1.

Simultaneously in this objects diameters of water meters didn't change (calculated according to standards). To compare design and average flow, calculation was done for simultaneous water usage for toilet, sink and shower in all the flats in chosen building (Table 1).

Results of calculations (especially ratio Q_{max}/Q_{ave}) show that water meters are oversized. It can be a reason for problems with balance of sold water (measured by main water meter in object) and water

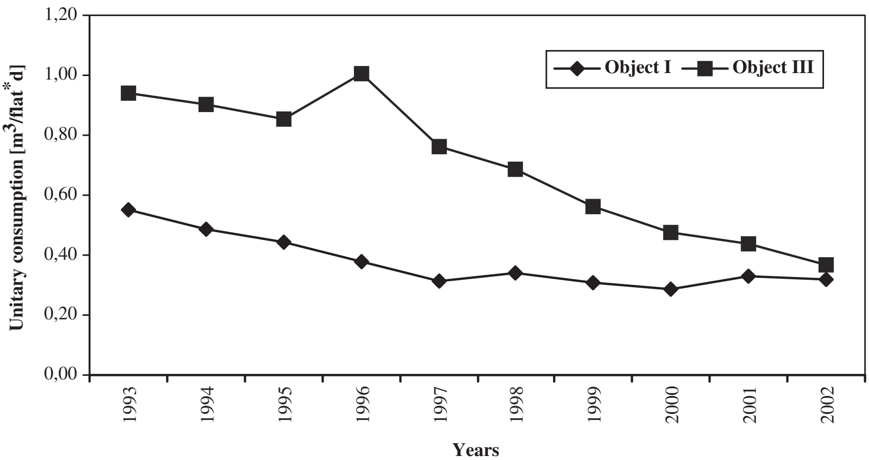


Figure 1. Changes in water consumption for objects under research between 1992 and 2002.

Table 1. Calculation of maximal and average flow for chosen objects.

No of object	Number of flats	Maximal flow of cold and warm water (according to PN-92/B-01706) Q_{max} [dm^3/s]	Q_{max} [m^3/h]	Average flows (2002) Q_{ave} [m^3/h]
I	80	3.39	12.20	1.05
II	132	3.85	13.86	2.10
II	330	4.81	17.32	5.04

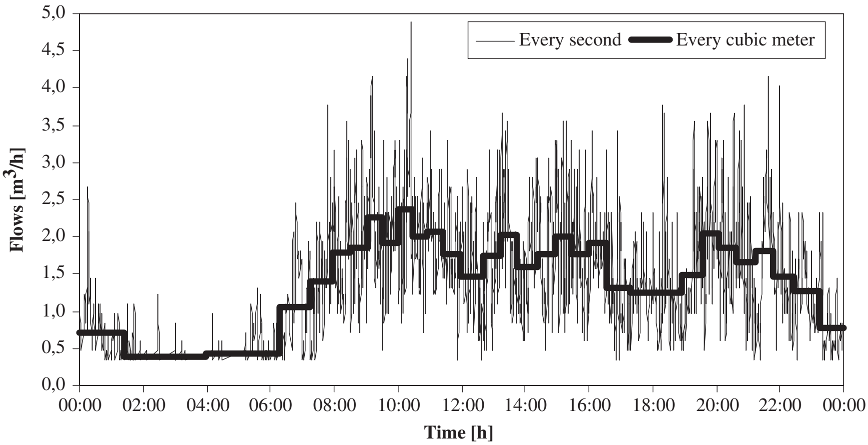


Figure 2. Comparison of pattern of water demands for different frequency of flow recordings (object "I").

bought by residents (a sum of water measured by individual water meters, fitted in every flat). This problem is confirmed by experience of housing association which manages objects II and III.

Results of calculation showed above, forced the author to study pattern of real water demand and its influence on water meter recordings.

4 PATTERN OF REAL WATER DEMAND

4.1 *Frequency of measurement*

Prior to the study the problem of frequency of measurement was considered. Preliminary research was done to find the best frequency:

- every 2 minutes (frequency of measurement used in local pumping station in Gliwice);
- every 1 m^3 (by using water meter with Reed contact);
- every second (by using electromagnetic flow meter).

Preliminary studies (done in object "I"; flows were recorded on Saturday – week day of maximal water usage, Figure 2) showed that recorded flows can change very dynamically.

Figure 2 shows significant differences between pattern of water demands recorded with distinct frequency: maximal flows measured every second are twice higher than measured every cubic meter. In this case one second frequency was chosen for research.

Additionally, Figure 3 shows two very important issues: flows smaller than 0.5 m^3/h were recorded during over 20% of total measuring time (those flows are identified as leakage of water supply system in object) [Koral, 2003, Buchberger at all, 2003]. But simultaneously those small flows are only a few percent of total volume recorded by water meter. That fact will be described in detail below.

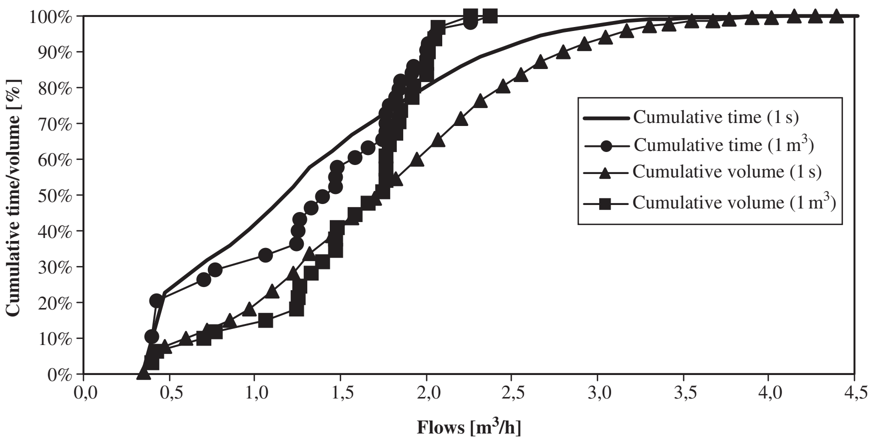


Figure 3. Comparison of cumulative time/volume curve for different frequency of flow recordings (object "I").

4.2 *Error curve of water meter (according to ISO 4064/1)*

Error curve of water meter has five characteristic points:

– starting flow rate, for which water meter starts to work; measuring error for this flow is unknown;
– minimal flow rate; below this flow measuring error is higher than −5%;
– transitional flow rate; above this flow measuring error is smaller than 2%;
– maximal working flow rate; for this flow water meter can work continuously about 1–2 hours without any mechanical damage;
– maximal flow rate; water meter working with flow higher than that can be damaged.

While considering the diameter of water meter it is important to choose water meter which will work most of the time between transitional and maximal work flow rate with maximal accuracy. Application of oversized water meters causes shift of measured flows towards lower flows and higher measuring error.

4.3 *Studies of real demand pattern for object "II"*

Research done in object "II" in February 2003 showed that during about 20% of total operating time water meter recorded only leakage in water supply system in this object. Simultaneously during 25% of operating time measured flows were smaller than minimal flow for water meter ($q_{min} = 0.70\,m^3/h$) and about 60% of operating time – smaller than transitional flow ($q_p = 2.0\,m^3/h$). Additionally maximal recorded flow was $6.0\,m^3/h$; it means that it was twice smaller than design flows calculated according to Polish standards (Table 1). Results are shown in Figure 4.

Additionally results in Figure 5 showed that more than 40% of total volume were recorded with measuring error higher than 2% and 6% – with error higher than −5%.

According to the results of the research water meter with nominal flow $6.0\,m^3/h$ should be used for this object (instead of water meter with $q_n = 15.0\,m^3/h$).

4.4 *Studies of real demand pattern for Christmas time (object "III")*

Water meter diameter, calculated according to Polish standard, is designed for maximal flow, assuming that nominal flow of water meter is twice smaller than calculated one. That is why additional studies for Christmas time (maximal yearly water consumption) was done in object "III".

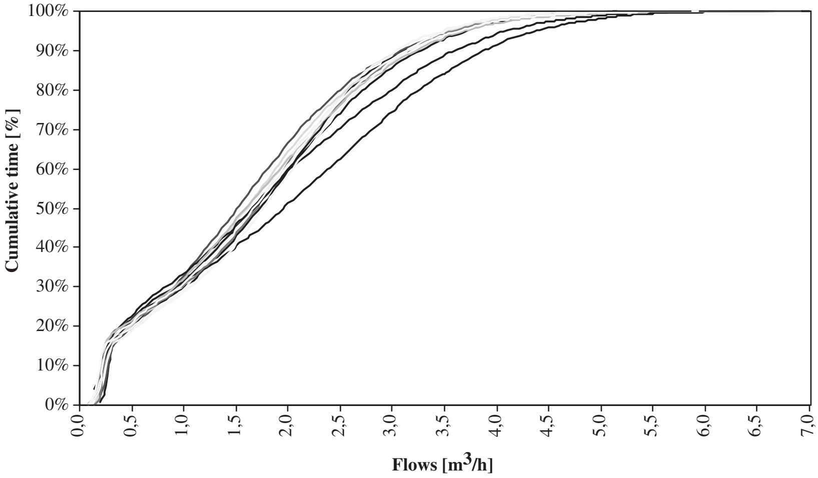


Figure 4. Curve of cumulative time (object "II").

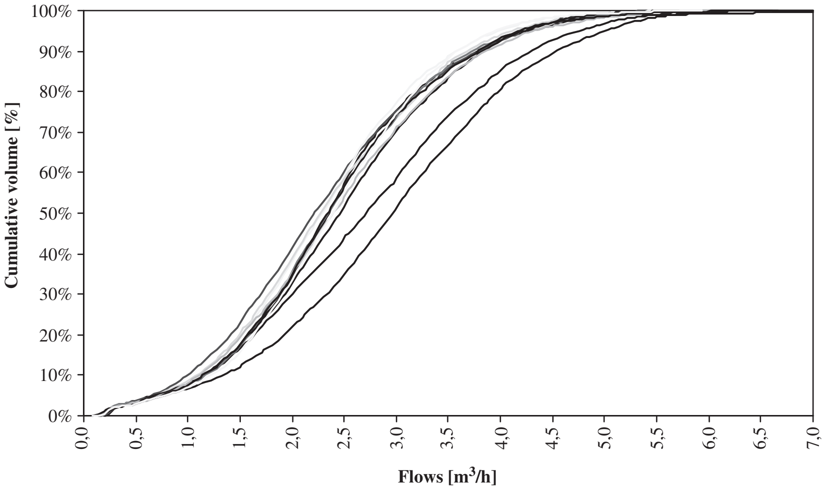


Figure 5. Curve of cumulative volume (object "II").

Flows were recorded every second between 20 December 2002 and 3 January 2003. Real demand pattern shown in Figure 6 was plotted for three characteristic days: two days of maximal flows (21 and 24 December) and one day of minimal water consumption (3 January). Those minimal flows were caused by failure of frequency converter which controls pumps supplying object "III".

Results (Figure 6) showed that maximal recorded flow is smaller than 15.0 m^3/h (nominal flow of water meters in object "III"). Additionally only a few times flow was higher than 12.0 m^3/h, which flow is about 30% smaller than design flow calculated according to Polish standards. (Table 1) and total sum of time for this flow is about 10 minutes during the whole day.

Density curve of flows for 2 January (Figure 7) shows very important information. As the result of failure of frequency converter controlling pumps night pressure was 0.2 MPa higher than usual and day pressure was 0.1 MPa lower than usual. That situation cause that night flows increased from 0.50 to 0.68 m^3/h and density curve of day flows shifted towards lower flows. It emphasises strong relationship between pressure in water supply system and water demand.

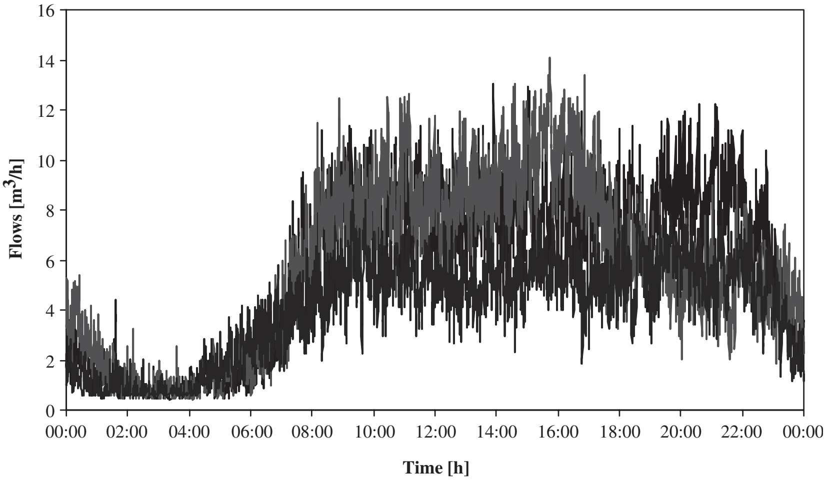


Figure 6. Real demand pattern for object "III" (Christmas 2002).

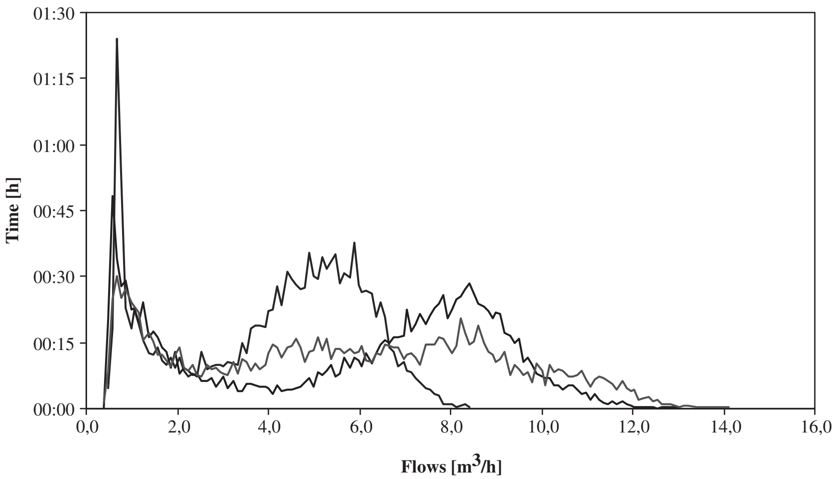


Figure 7. Density curve of flows (object "III", Christmas 2002).

On the basis of the results described above water meter with nominal flow equal to $10.0\,m^3/h$ could be used for this object (instead of water meter with $q_n = 15.0\,m^3/h$).

5 COMPARATIVE STUDIES OF "B" CLASS AND "C" CLASS WATER METERS

Comparative studies were done in object I in during the year 2001. Four water meters from different producers were chosen in order to find the best one. Compound water meter was used as a standard. All water meters had nominal flow equal to $15\,m^3/h$ according to calculations showed in Table 1. Metrology of each water meter was shown in Table 2.

Water meter "IV" has a very interesting metrology i.e. very good parameters especially for low flows. Those parameters are comparable with compound water meter, which is emphasized by the producer.

Table 2. Metrology of chosen water meters.

Water meter (class)	Kind of water meter	Starting flow rate [m^3/h]	Minimal flow rate [m^3/h]	Transitional flow rate [m^3/h]	Nominal flow rate [m^3/h]	Maximal flow rate [m^3/h]
–	Compound	0.015	0.050	0.200/3.0*	15	35
"I"(B)	Propeller	0.060	0.150	1.0	15	30
"II"(C)	Multi-jet	–	0.100	1.0	15	30
"III"(C)	Single-jet	0.015	0.090	0.225	15	30
"IV"(C)	Single-jet	0.032	0.060	0.080	15	30

* bigger value for water meter DN50.

Table 3. Results of comparative research of water meters.

Water meter	Average divergence of total measured volume of water in comparison with compound water meter		
	m^3/d	m^3/year	%
I	−3.92	−1431	−14.0
II	−3.48	−1270	−12.7
III	−3.95	−1442	−13.0
IV	−5.51	−2011	−20.3

Every day during one week research readouts of each water meter were done. Results were shown in Table 3.

Results of research reveal that total volume of measured water is comparable for water meters "I", "II", and "III". This fact shows that it is not necessary to use more expensive "C" class water meter instead of "B" class one for this building. But results of studies for water meter "IV" are very surprising – the best metrology given by the producer turned out to be the worst for real dynamic water consumption.

At the same time all water meters recordings are more than 12% smaller than readouts of the standard, compound water meter. During the whole year, it is about 1.5 of monthly water usage for this object.

6 SUMMARY

Studies of pattern of water demand showed that maximal water usage is smaller than maximal flow calculated according to Polish standards. Following standards led to usage of oversized water meter and problem with balancing of supplied and sold water. It made Utilities show overstated water losses (in fact – apparent losses) and waste of time and money spent on identifying non existing failure and leakage. Adjustment of water meters diameter to real water usage allowed for precise balancing of supplied and sold water for all housing estates supplied by local pumping stations.

Additionally research revealed that perfect metrology of water meters, presented by producers differed significantly from results of measurement of water usage done in real operating conditions. Indications of water meters under research were over 12% lower than those of compound water meter. Also, indications of "B" class are comparable with "C" class water meter, which is more expensive.

Results of research showed that apparent losses caused by oversized water meters are a real problem of Polish Water and Sewage Utilities.

REFERENCES

PN-92/B-01706 Installations for water supply design (in Polish)

DIN 1988 Technische Regeln für Trinkwasser-Installationen. Dezember 1988

ISO 4064/1 (www.iso.ch)

W. Koral: "Are apparent losses caused by water meters a real problem? Practical experiences" II International Conference on Efficient Use and Management of Water for Urban Supply, Tenerife April 2003, Spain

S. G. Buchberger & G. Nadimpalli: "New method to estimate leakage rates in drinking water distribution systems", Pumps, Electromechanical Devices And Systems, Valencia April 2003, Spain

Brochures of water meters producers: PoWoGaz (www.powogaz.com.pl), Mirometr (www.mirometr.pl), ABB (www.abb.com), Schlumberger (www.schlumberger.pl)

Stochastic prediction of the minimum night flow demand in a District Metered Area

Vicente García
PhD Student. Institute for Water Technology. Universidad Politécnica de Valencia, Spain

Enrique Cabrera, Jorge García-Serra & Francisco Arregui
Institute for Water Technology. Universidad Politécnica de Valencia, Spain

Javier Almandoz
Department of Nuclear and Fluid Mechanics Engineering. Universidad del País Vasco, Spain

ABSTRACT: One of the best well known and widest used method to evaluate water losses in a water distribution network is by means of the minimum night flow in a well defined District Metered Area (DMA). The measured flow is compared with the background minimum night flow, which is the sum of the customer night use and the background night flow losses. These components have to be assessed from "typical" values, so the error of the method depends to a large extent on the adequate estimation of these values. The authors have developed a stochastic model to simulate residential water demand, which can be used to evaluate residential night use. The model has been checked, providing reasonable results, with measurements performed in Spain and in the United States.

1 INTRODUCTION

Water distribution systems consist of hundreds of kilometers of pipes that are often buried and forgotten, but water losses through leakage occur continuously. Leakage of water takes place from mains and service connections at joints and fittings, from breaks due to excessive loads and holes arising from corrosion. In the last years an increased awareness of leakage has been taken into account by water companies, due to several factors: a greater sensitivity to conserve the environment, as a way to diminish the risk of pathogenic intrusions (Funk et al., 1999), and as a tool to manage a drought (American Water Works Association, 1992).

According to Lambert et al. (1998), an active leakage control (ALC) is one of the four complementary methods of effective leakage management. An ALC policy is intended to locate and repair detectable but unreported leaks. Although the most appropriate method is specially determined by local conditions, continuous or intermittent night flow measurements into DMAs of moderate size are an effective way by which leakage control teams can evaluate unreported bursts, in order to prioritise activities to locate and repair them. Those measurements comprise the flow rate supplied to the whole district during night hours. It is then necessary to know the customer night use, to deduct from the measured night flow. Because it is not feasible to measure night flow consumed by every customer, customer night use is usually evaluated by means of typical values. An alternative to the use of typical values consists of using a stochastic model to determine water demand.

The stochastic model presented in this paper is built from just some few measurements of the water demand in individual customers. The model is based on a rectangular pulse point process, in which every single consumption is described in terms of a rectangular pulse of a given duration and intensity. Although it has been applied to simulate residential demands, it could be extended to other uses, provided enough measurements are available.

It is obvious that in order to establish adequate patterns of consumption, it is necessary to measure a wide sample of different types of consumers. With the model proposed, it is possible to obtain a great information from these measurements, because the model can generate different consumption scenarios, according to an observed pattern. Water demand is essentially a stochastic process, and hence it must be studied taking into account its random nature.

Besides its usefulness, the model can be constructed in a straightforward way. It includes a total of 9 free parameters that define 5 different statistical functions. These parameters can be easily calculated once individual consumption pulses have been identified. In order to identify pulses, water demand must be continually measured in single properties at short time steps along a period (usually several weeks) that allows to extract the daily pattern.

2 NIGHT FLOW METHODOLOGY

As has been mentioned, this methodology is basically a water balance during the night period, when the water consumption is minimum and leaks are maximum, and hence there are few error possibilities. The details of whatever is exposed here are in reports E and F (UK Water Industry, 1994a; UK Water Industry, 1994b).

Night flow measurement is a method extensively used in the United Kingdom to estimate water losses. In the UK only about 10 percent of the domestic customers are metered (Walski et al., 2001). Instead of metering individual customers, distribution systems are divided into smaller zones, named District Metered Areas (DMAs), which can be isolated by valves and are fed by a small number of meters.

Minimum night flow measurements in well defined DMAs of moderate size is an effective method to detect unreported bursts. The reason to measure minimum night flows (usually the minimum daily flow is produced between 1:00 and 5:00 hr) is that during this period, flows have the highest leaks proportion, so a significant increase in the minimum night flow can be a reliable indicator of some system anomaly, most often produced by a burst in the network (Figure 1).

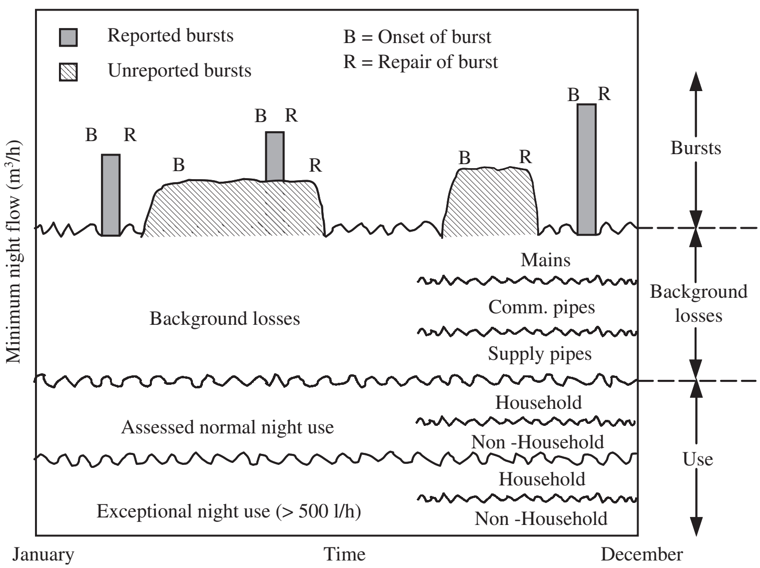


Figure 1. Bursts detection by means of night flow measurements.

In large DMAs, individual bursts (especially on service pipes) will have a small relative impact, and will be difficult to identify from the night flow measurements.

As can be seen in Figure 1, total losses consist of reported bursts, unreported bursts and background losses on different parts of the distribution system and supply pipes. Bursts are defined as individual significant incidents with definite durations, where the loss rate exceeds a threshold value of 500 l/hr at 50 m pressure. Smaller flow rates are considered to compose the "background" losses. Background losses on mains and services depend on the general condition of the infrastructure and pressure. For infrastructure in "average" condition and 50 m, the values proposed are: 40 l/Km/hr for distribution mains and 4 l/property/hr for services.

The estimated customer night use is the part of the night flow that is legitimately used by three types of customers: exceptional uses (>500 l/hr), household uses and non-household uses. The sum of these components and the estimated background losses must be compared with the measured minimum night flow. Any difference may be due to unreported bursts.

While exceptional uses must be individually determined, household and non-household uses can be estimated from typical values. In order to establish these values, several researches were undertaken in the UK between 1991 and 1993. The average household night usage from the tests is 1.7 l/property/hr; this does not include exceptional use for hosepipes or purposes equivalent to a fully open tap. The data obtained indicate that a small percentage of high-use active households (e.g. washing machines, dishwashers) can have a high influence on the average household night use. It is typical of the components of the night flows that a small proportion of the sample can significantly affect the average and standard deviation.

Assuming that on any night the average night flow is generated by a proportion "p" out of "N" households, the statistical distribution of the nightly number of active properties can be calculated by the binomial distribution. If the number of active households can all be considered to have the same use of z l/h, the mean and standard deviation of the night use can be calculated by using the formulae shown in Table 1. As can be observed, the spread of average night use values decreases quickly as the size of district increases. Because not all the active households have the same average use, a better approximation can be achieved by assuming a multinomial distribution.

The average use from different types of non-households shows a wide variation. The active non-households are classified into five groups (A to E) with similar average use characteristics (0.9, 6.2, 12.6, 20.5 and 60 l/property/hr). Non-household night use can be substantial and very variable within individual DMAs. Two methods of assessing non-household night use are proposed. In small DMAs the more detailed method (based on groups A to E) may well be justified, together with consideration of sampling variability. A more simplified method consists of evaluating the average flow as 7 or 8 l/non-household/hr. It is observed, as in the case of household night use, that a tiny proportion of high night flow users has a great influence on the average night flow use. Standard deviation of assessed non-household night use can also be calculated by assuming a binomial distribution.

3 STOCHASTIC MODEL FORMULATION

The model presented herein is more extensively developed in (García, 2003).

Water consumption process in a residence along the day can be represented as a sequence of n rectangular pulses (Wells, 1994; Buchberger and Wells, 1996), with starting times $\tau_1, \tau_2, \tau_3, \ldots, \tau_n$, all of them referred to the beginning of the day (0:00 hours) and being $\tau_1 < \tau_2 < \tau_3 \cdots < \tau_i < \cdots \tau_n$, with $\tau_i \in [0, 86{,}400]$ s, $\forall_i$.

Table 1. Household night use statistics.

	l/hr	l/prop./hr
Mean	Npz	pz
Standard deviation	$z\sqrt{Np(1-p)}$	$z\sqrt{\frac{p(1-p)}{N}}$

Each pulse has an associated duration T_i (s) and intensity I_i (l/s), so water consumption process can be derived from superimposing the pulse sequence along the day. Therefore, total daily consumption can be obtained as:

$$\forall_d = \sum_{i=1}^{n} T_i(\tau_i) \cdot I_i(\tau_i) \tag{1}$$

Variables $\forall_d$, T_i and I_i are described in terms of appropriate statistical functions, while the time occurrence of pulses (τ_i) is conceptualized as an independent stochastic point process. It is also assumed that durations and intensities are independent variables. The formulation of each component is detailed hereafter.

3.1 *Daily water consumption ($\forall_d$)*

This is the variable that can be determined more easily, and more tangible from a hydraulic point of view, as it represents the volume of water needed to satisfy the daily demand. According to several previous researches (Bowen et al., 1993; Buchberger and Wells, 1996; Billings and Jones, 1996), the log-normal distribution with mean μ and standard deviation σ is proposed, and indeed yields to the convincing fitting to empirical data.

3.2 *Pulse durations (T_i)*

It is assumed that this variable is exponentially distributed, with parameter α. The choice of this distribution is due to the observed fact that there is a large number of short pulses combined with fewer ones of larger durations. Besides, there is a mathematical convenience because of the simplicity of exponential function, that contributes to model parsimony.

Corresponding density and cumulative distributions are given by (2) and (3):

$$f(T) = \alpha \cdot e^{-\alpha \cdot T} \tag{2}$$

$$F(T) = 1 - e^{-\alpha \cdot T} \tag{3}$$

with average pulse duration equal to α^{-1} (s).

3.3 *Pulse intensities (I_i)*

This variable is described by means of a Weibull distribution, with scale parameter λ (l/s) and shape parameter β. The selection of this function is mainly motivated by empirical criteria, taking into account the observed bias of the samples, and having verified the good agreement of the Weibull distribution to the sample distributions. The corresponding density and cumulative distributions are given by (4) and (5):

$$\boxed{f(I) = \frac{\beta}{\lambda} \cdot \left(\frac{I}{\lambda}\right)^{\beta-1} \cdot e^{-\left(\frac{I}{\lambda}\right)^{\beta}} \qquad I \geq 0} \tag{4}$$

$$F(I) = 1 - e^{-\left(\frac{I}{\lambda}\right)^{\beta}} \qquad I \geq 0 \tag{5}$$

with a mean pulse intensity given by (6):

$$E(I) = \lambda \cdot \Gamma\left(1 + \frac{1}{\beta}\right) \tag{6}$$

where

$$\Gamma(\xi) = \int_0^\infty z^{\xi-1} \cdot e^{-z} \cdot dz \tag{7}$$

3.4 *Pulse occurrence process in time ($\tau_1, \tau_2, \tau_3, \ldots, \tau_n$)*

The series of time occurrences along the day that defines the starting times of pulses can be conveniently described by a non-homogeneous Poisson process (Parzen, 1967). The corresponding intensity function $v(t)$ describes the expected frequency of occurrences along the day, and is formulated as follows:

$$v(t) = C \cdot g(t) + \varepsilon(t) \tag{8}$$

where C is a constant for a given day and represents the expected value of the number of pulses during the day, $g(t)$ is a continuous function that defines the daily temporal pattern, and can be viewed as a normalized arrival rate, while $\varepsilon(t)$ is a random component with zero mean and standard deviation σ_r.

Function $g(t)$ could be defined in terms of a Fourier analysis, though using simple functions with three or four parameters seems to be more adequate, in order to describe the essence of daily variation in probabilistic terms. In this application, a cubic polynomial has been selected:

$$g(t) = \frac{1}{10000} \cdot \left[A_3 t^3 + A_2 t^2 + A_1 t + C_0\right] \qquad t \in [0,24] \ \ hours \tag{9}$$

where the only free parameters are A_1, A_2, and A_3, while C_0 is obtained through (10):

$$\int_0^{24} g(t) \cdot dt = 1 \tag{10}$$

3.5 *Model closure*

Once the variables have been analyzed, there must be found a link between the different parameters. This link is obtained by taking into account that daily consumption is the sum of the water uses $\forall_i$ corresponding to each one of the rectangular pulses occurred along the day:

$$\forall_d = \sum_{i=1}^{C} \forall_i \tag{11}$$

According to the proposed formulation, the expected value of daily consumption can be obtained through (12):

$$\boxed{E[\forall_d] = C \cdot E[T] \cdot E[I] = C \cdot \left(\frac{1}{\alpha}\right) \cdot \lambda \cdot \Gamma\left(1 + \frac{1}{\beta}\right) = \mu} \tag{12}$$

Aggregated daily variables must be preserved by the model, in particular total daily consumption. For this reason, it is advisable to introduce an additional degree of freedom in the model, by allowing variation of C between different days, in which case C can be considered as a random variable. Equation (12) could then be rewritten as:

$$\boxed{E[C] = \frac{\mu \cdot \alpha}{\lambda \cdot \Gamma\left(1 + \frac{1}{\beta}\right)} = \mu_C} \tag{13}$$

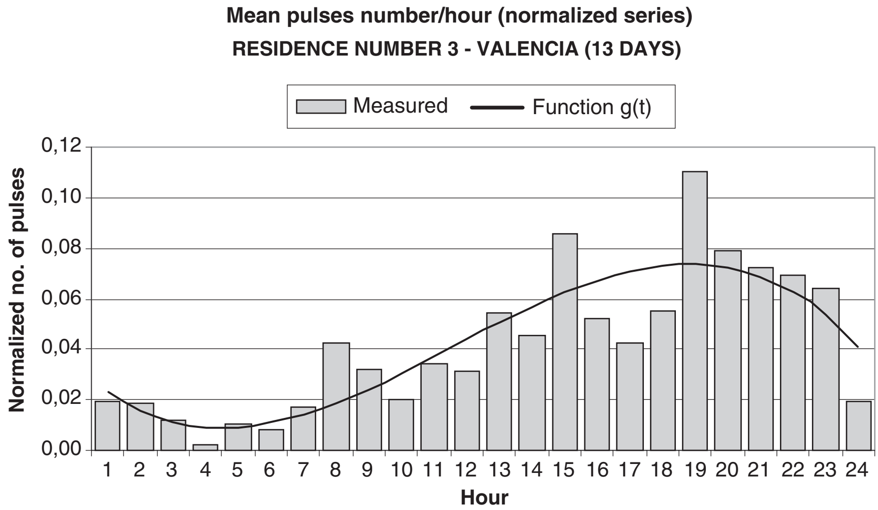


Figure 2. Mean number of pulses per hour.

Equation (13) represents the closure of the model, because a link between the two blocks of parameters (parameters of consumption and parameters of occurrence) is derived.

4 MODEL APPLICATION TO NIGHT FLOW METHODOLOGY

As has been previously explained, night flow measurements in DMAs of moderate size can be a useful method to identify unreported bursts. If DMA size is large, method effectiveness is lower, because individual bursts will have a small influence on the measured night flows. On the other hand, in small or medium DMAs the spread of night use can be very large, and average night use is greatly affected by a small proportion of high-use active properties. This means that in these cases, night use is very variable, so it will not be enough to estimate its average value by means of typical values.

The stochastic model proposed can be used for this purpose, because it can simulate different water demands based on a measured pattern. For its construction, detailed measurements have to be made in individual properties. In this paper two case studies are reported, in which data are available for a set of residences in Valencia (Spain) and Milford (USA).

With regard to Valencia data, the measurements were taken in 15 residences during the month of September 1998 (15 days). Detailed description of the sample can be found in (Arregui, 1998). Milford data were obtained in 20 residences the year before (31 days), and the details are in (Buchberger and Lee, 1999). From the initial set of residences, only 7 of them were selected for analysis in the case of Valencia, and 14 in the case of Milford. Some residences were discarded because they did not have water consumption for one or more days.

Parameters of the model were estimated for the ensemble of residences in Valencia and in Milford, and synthetic series were simulated for comparison with historical series.

One of the features of the model is that it tends to smooth peak and valley demands, as a consequence of the polynomial trend chosen for function $g(t)$ (Figure 2).

Because of that, several simulations have been made in this example:

1. First, model parameters have been estimated for all the pulses registered along the day, and night demand from 00:00 to 06:00 has been determined, from a 24 hour simulation.
2. Second, only pulses produced between 00:00 and 06:00 have been considered, and a simulation has been made for the period between these hours.

Table 2. Ensemble of Valencia residences.

Simulation period	Average night consumption (l) Synthetic series	Historical series	Relative error (%)
T = [0,24] hr	308.59	227.70	35.53
T = [0,6] hr	252.50	227.70	10.89
T = [3,6] hr	90.47	86.36	4.75

Table 3. Ensemble of Milford residences.

Simulation period	Average night consumption (l) Synthetic series	Historical series	Relative error (%)
T = [0,24] hr	1090.55	793.23	37.48
T = [0,6] hr	809.30	793.23	2.03
T = [3,6] hr	587.37	595.42	1.35

3. Finally, the period elapsed between 03:00 and 06:00 has been chosen, and the same process as in step 2 has been performed.

The results obtained are shown in Tables 2 and 3. It is observed that in both cases, the shorter the simulation period is, the better the agreement between synthetic and historical series is. In general, synthetic series overestimates night demand, because for night hours, function $g(t)$ is greater than the normalized number of pulses registered. But when only night pulses are chosen to estimate model parameters, and synthetic series are obtained for night hours, the results can be considered quite accurate.

If night use is expressed in l/property/hr, it can be observed that the corresponding values in Valencia and Milford are much greater than those in UK. For example, in the case of Valencia night use is 5.42 l/property/hr for T = [0, 6] hr, while in the case of Milford is 9.44 l/property/hr for the same period. The main reason for this is that in the case of Valencia and Milford, all the residences were active during all the days of the historical series. Another reason is that consumption patterns are quite variable between different cities and countries, so it is necessary to make extensive measurements to characterize consumption patterns. The stochastic model can help to make the most of this information.

5 CONCLUSIONS

Leakage is an indicator of efficiency in water distribution systems, so leak detection is an important task in the maintenance of these systems. Measuring minimum night flows has become a method extensively used for this purpose.

Customer night use is one of the components of minimum night flow that shows a great variability. Traditionally it has been estimated from typical values obtained by measuring different types of customers. In this paper a stochastic model has been proposed as an alternative to make accurate calculations. This model can simulate each of the individual consumption pulses that takes place in a single customer along the day, so it can be used to obtain water consumption in any period. Model result is not deterministic, so by simulating different days an estimation of use variability can be obtained, according to a measured consumption pattern.

In the two cases studied, a good agreement between synthetic and historical series has been observed. Because the function used to reproduce the temporal variation of pulse frequency is smoother than that measured, the best result is obtained when only night pulses are considered to construct the model.

The model has been applied to household use, but it could be extrapolated to other uses with minor changes. In any case, enough data from extensive measurements should be previously obtained.

ACKNOWLEDGEMENTS

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REFERENCES

American Water Works Association (1992). *Drought Management Planning.* Denver. Colorado. USA. AWWA.

Arregui, F. (1998). *Propuesta de una metodología para el análisis y gestión del parque de contadores de agua en un abastecimiento.* PhD. Thesis, Universidad Politécnica de Valencia. (In Spanish).

Billings, R. B. and C. V. Jones (1996). *Forecasting Urban Water Demand*. Denver, CO. American Water Works Association.

Bowen, P. T., J. F. Harp, J. W. Baxter, and R. D. Shull (1993). *Residential Water Use Patterns.* Denver, CO. AWWA Research Foundation.

Buchberger, S. G. and Y. Lee (1999). *Water Industry Systems: Modelling and Optimization Applications,* Chapter Evidence Supporting the Poisson Pulse Hypothesis for Residential Water Demands, pp. 215–227. Research Studies Press Ltd.

Buchberger, S. G. and G. J. Wells (1996). Intensity, duration and frequency of residential water demands. *Journal of Water Resources Planning and Management 122, No. 1,* 11–19.

Funk, J., S. Goes, D. Wood, and M. LeChevallier (1999, July). Pathogen intrusion into water distribution due to systems transients. San Francisco. USA. ASME & JSME Joint Fluids Engineering International Conference.

García, V. (2003). *Modelación de la demanda urbana de agua.* PhD. Thesis, Universidad Politécnica de Valencia. (In Spanish).

Lambert, A., S. Myers, and S. Trow (1998). *Managing Water Leakage. Economic and technical issues.* London. Financial Times Energy.

Parzen, E. (1967). *Stochastic Processes.* San Francisco, California. Holden-Day.

UK Water Industry (1994a). Report e: Interpreting measured night flows. Technical report, UK Water Industry. Engineering and Operations Committee.

UK Water Industry (1994b). Report f: Using night flow data. Technical report, UK Water Industry. Engineering and Operations Committee.

Walski, T. M., D. V. Chase, and D. A. Savic (2001). *Water Distribution Modeling.* USA. Haestad Methods, Inc.

Wells, G. J. (1994). Statistical characteristics of residential water demands. Master's thesis, University of Cincinnati.

Measuring microcomponents for demand forecasting

Andrew Ball & Magdalena Styles
Southern Water

Ken Stimson & Mark Kowalski
WRc plc

ABSTRACT: Forecasting of future domestic demand – the driver behind investment into water resources schemes – relies on good knowledge of current domestic water use. The understanding of water use behaviour in the home – the microcomponents of domestic demand – is particularly important. Previous microcomponents studies relied on extensive invasive sub-metering of domestic properties. Identiflow®, a new method, is a non-invasive and more flexible approach. The technique measures flow at a single entry point to the property and determines microcomponents by classifying flow events using specialised software. The method has been tested on over 600 properties in the UK. Southern Water has used the Identiflow® analysis method since 2000 to improve its knowledge of the microcomponents of domestic demand and understand any differences amongst customer groups. The data have been used to assess normal and peak domestic use and improve domestic demand forecasting. In 2003, Identiflow® will be used to understand the impact on demand patterns from optant metering. The paper discusses the technology used, specific examples of the approach as used by Southern Water, and concludes that microcomponent analysis is vital for demand forecasting.

1 INTRODUCTION

Domestic water consumption consists of several microcomponents. Typically these are the uses of devices such as toilets, showers, bath, washing machines, dishwashers, internal and external taps. Water use for a particular purpose can be achieved using one or more devices. For example toilet flushing uses a single device, whereas personal washing can involve a bath, shower or tap use.

UK Water service providers are required to use microcomponent data in demand forecasts for planning and operation (UKEA 2003). Initial work in this area was undertaken in a study of the effect of climate change on water use (Herrington 1995). In the Herrington model, device use is the product of device ownership, frequency of use and volume per use. In turn it has been recognised that these factors can be influenced by economic prosperity, type of housing, occupancy, climate and technical developments in water-using devices. Until recently this approach relied on limited data.

Recent advances have been based upon improved monitoring technology, allied to a proven method for recognising and quantifying microcomponent use. A national study in the UK (Kowalski & Stimson 2001) of 250 properties proved the method, derived key national statistics, and provided the basis for Water Companies to undertake further studies. The national study did not cover seasonal demand in any detail.

However, for many water companies in the UK, including Southern Water, summer, or "peak" demand is a major driver of investment in both new water resource developments and demand management. Southern Water therefore undertook two investigations in the years 2001 (Glennie et al. 2002) and 2002 (Patel et al. 2003), both using Identiflow, aimed at understanding and quantifying the drivers of peak demand. This work was funded by Southern Water, Mid Kent Water and the Environment Agency (Southern Region).

The information gleaned was initially used to improve targeting of water efficiency programmes. The results of the studies were then used with other data on household appliance ownership to develop a microcomponent-based demand forecast.

2 MEASURING MICROCOMPONENTS

2.1 *The WRc Identiflow system*

The WRc system (Fig. 1) for measuring microcomponent use comprises a flow meter and logging system supported by Identiflow analysis software.

The flow meter and logger system can be installed in an external meter boundary box and records water consumption at a relatively high resolution. The system is non-intrusive and is therefore designed to minimise any effect on customer behaviour.

The Identiflow software has automatic facilities which identify water use "*events*" and classify them as a particular microcomponent. The software also provides interactive facilities which permit the experienced user to review and refine the analysis.

The analysis provides the characteristics of each water-use event, calculating duration, volume, average flow and maximum flow, and identifying the microcomponent. The statistical analysis provides a range of reports including device characteristics, frequency of use, volume per use, and the contribution of each device to overall demand.

2.2 *Customer information and contact*

In order to support and corroborate the microcomponent analysis, information is sought from the customer. The customer is asked information about the number of occupants, the water-using devices owned, the type of plumbing (direct or indirect), and any water saving measures in place. Depending upon the nature of the investigation, customers may be asked about factors affecting usage e.g. whether the customer has a large garden or is a keen gardener.

Interest to date has focussed upon unmeasured UK customers i.e. those who pay for water based on a rating system, rather than by direct metering. The use of the Identiflow system therefore

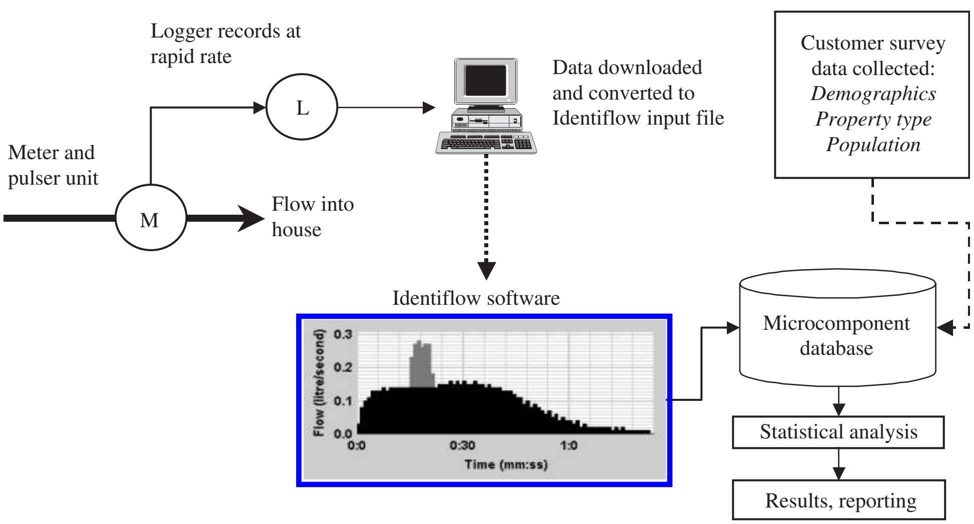

Figure 1. The Identiflow® system.

involves the temporary installation and removal of a meter and a few minutes interruption to supply on each occasion.

Households for microcomponent studies are therefore recruited voluntarily in advance, in order to maintain goodwill and cooperation.

3 MICROCOMPONENT STUDIES

3.1 *Introduction*

The UK national study and Southern Water studies are described below. Key conclusions and results from the national study helped to guide the direction of later studies. The main objective of the national study was to investigate the applications for producing a microcomponent monitor for unmeasured customers. Such a monitor, at national, regional or local level consists of a representative set of households. The microcomponent use behaviour of these properties can be used to represent water use of the respective unmeasured population.

Water use data for a microcomponent monitor is best monitored at times of average (i.e. non-peak) demand, typically October to March. This approach does not capture seasonal effects and supplementary information is required to quantify this use as described in the two studies for Southern Water.

3.2 *The national study*

A project was carried out in 2001 by WRc to investigate the use of the Identiflow method as a means of developing a national microcomponents monitor (Kowalski & Stimson 2001). The trial involved monitoring around 250 properties from ten water companies during a period of "normal" (i.e. non-peak) demand and assessing domestic water use at the microcomponent level.

The average *per household consumption* (phc) for all households analysed was 367.4 litres/property/day. The volume per use results were in broad agreement with common knowledge. Figure 2 compares microcomponent data from the WRc study with values commonly quoted by the UK Environment Agency which are believed to be the best estimates available in 2001. The figures show broad agreement, and any differences are probably accounted for by differences in demography and sample size.

The study proved the methodology. It was found that a well-chosen sample of about 100 properties would be sufficient to provide water use results (e.g. frequency of shower use or average volume per toilet flush) with an error of no more than 10%. There was little variation in use from one

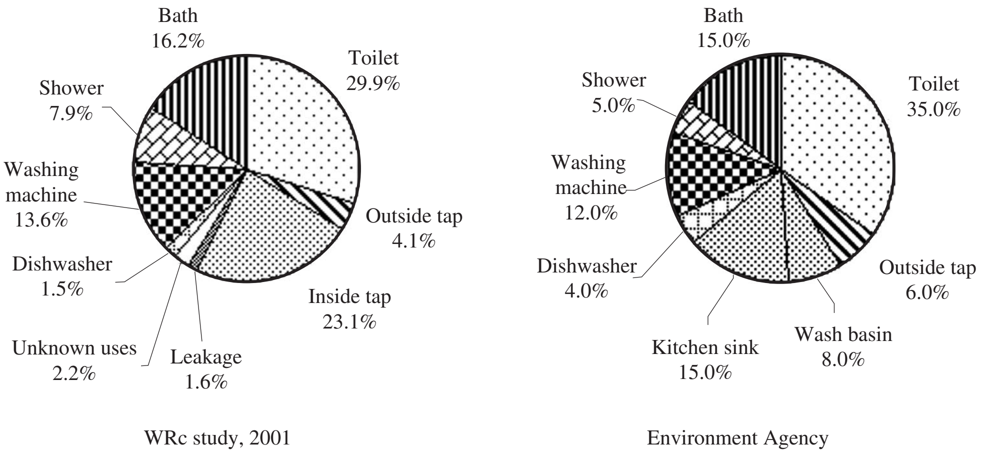

Figure 2. Total consumption by microcomponent.

week to the next and so the methodology was refined to monitor households for two weeks in order to select and analyse a typical week. It was concluded that the sample selected for a micro-component monitor should be representative of the area in terms of house type and occupancy.

3.3 *Seasonal studies at houses served by Southern Water*

3.3.1 *The monitoring regimes*

There were two studies in 2001 and 2002 respectively (Glennie et al. 2002, Patel et al. 2003). In 2001, six properties in Hastings and seven properties in Horsham were monitored twice, initially in April/May and then in July/August. The two periods were chosen to capture average and peak behaviour respectively. In 2002, a total of 28 further properties in Horsham, Hastings and Balcombe were successfully monitored once in July/August at a time of anticipated high demand.

The Hastings and Horsham areas were specifically selected because the balance between water available for supply and forecast demand means that further investment in water efficiency and new water resources is required in both areas.

The 2001 sample was chosen primarily to reflect a range of occupancies and was targeted mainly at households where there were keen gardeners, although a small number of those not keen on gardening were included. A range of house types and garden sizes was also achieved. In 2002 Southern Water wrote to 250 suitable households requesting volunteers. The 100 positive replies were grouped into geographic clusters and those with small gardens and without a keen gardener were excluded in order to identify houses to be monitored.

3.3.2 *Zonal demand and weather data*

For both studies the houses were clustered in a small number of district meter areas (DMAs). Demand in these areas is routinely monitored at 15-minute intervals. Relevant DMA data were made available for April to August 2001 and for the whole summer period of 2002. In addition operational information (e.g. re-valving) affecting the DMAs was gathered.

Daily meteorological data comprising maximum temperature, sunshine hours and rainfall at the nearest weather stations were collected for the same time periods.

3.3.3 *Analysis of summer use 2001*

The 2001 exercise had five key steps, viz.

- Comparison of daily consumption between households in a DMA with reference to weather influence
- Analysis of diurnal water-use profiles within each DMA
- Compare of increased water use and weather
- Compare of differences in appliance use for each household between the two monitoring periods
- Identify of "peak" and "non-peak" days and identify which appliances were likely contributors to peak use and at what time of day.

Owing to operational activities in Horsham, the useful DMA flow data were limited and therefore several definitive results apply to Hastings only. In each area there was a significant increase in water consumption for the two groups of properties between April/May and July/August. In each area a single property contributed to most of this increase. In Hastings, the DMA consumption was strongly influence by all three weather parameters and the chosen properties followed the same trends as the DMA. Figure 3 shows weather parameters and demand in Hastings.

Peak water use days for the Hastings DMA mainly coincided with peak water use days at the individual properties. The additional water use on peak days was mainly through outside taps. There was some evidence of increased shower use. Diurnal profiles for peak days were similar to those of non-peak days but with a marked evening peak. There was a consistent difference between weekdays and weekends. Personal washing and toilet use were higher at the weekends. On the basis of this evidence, a microcomponent monitor based on measurements at times of average

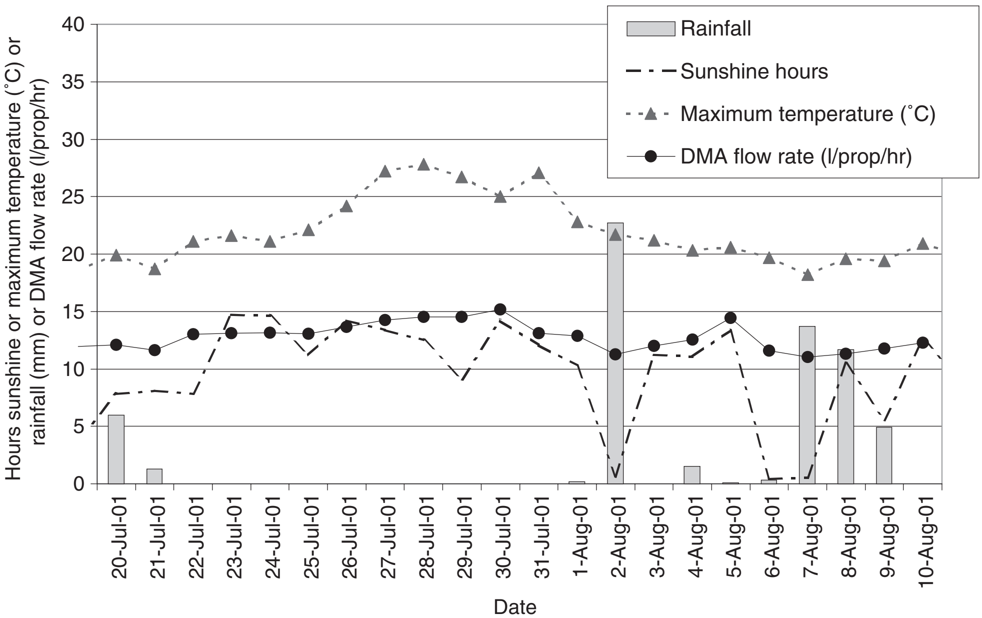

Figure 3. The influence of weather on demand, Hastings.

demand is generally representative of annual behaviour for all microcomponents with the notable exception of external use.

3.3.4 *Analysis of summer use 2002*

In 2002 the monitoring was confined to two sessions of two weeks within a four-week period, selected in anticipation of peak demand behaviour. Peak and non-peak days were identified for each property and for each relevant DMA. The microcomponent uses contributing to increased demand on peak days were investigated for monitored properties. As an additional check summer use at these properties was compared with winter use at a sample of eighteen different properties monitored in the year 2000.

DMA peak demand days fell within the monitoring period for all properties except for thirteen properties in Horsham. The DMA peak demand days generally coincided with peak demand days at the remaining fifteen properties. There was considerable variation amongst properties: Figure 4 indicates significant variation in external use across properties and peak and non-peak days. Very large peak volumes were attributable to a few houses.

All microcomponents contributed to peak demand. External use was the major contributor and personal washing was the second largest contributor. The summer/winter comparison confirmed increased daily consumption and significant increase in external use.

The second study therefore reinforced the earlier conclusions. A small number of houses again made a major contribution to increased demand. If such houses could be easily identified, they could be candidates for demand management measures e.g. the use of water butts or more efficient external devices. This, in turn, could be reflected in a demand forecast.

4 USING THE MICROCOMPONENT DATA

The results of the microcomponent analysis have been used in two ways. Firstly to design appropriate water efficiency campaigns and secondly, when combined with other data, to produce a microcomponent based demand forecast. This section focuses on the latter of these two.

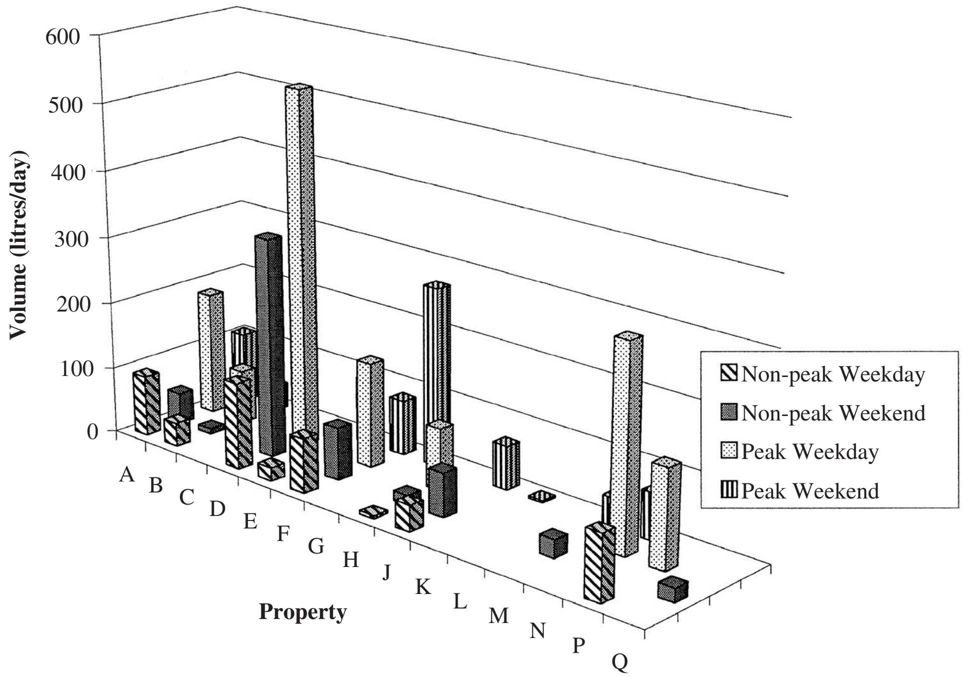

Figure 4. Variation in external use amongst properties on peak, non-peak, week and weekend days.

4.1 *Development of the microcomponent-based demand forecast*

The data from the Identiflow studies provides important information on household consumption during peak and non-peak periods. In particular if provides information on volume and frequency of use of different appliances. By taking this data and information on appliance ownership (both present and future) it was possible to develop a microcomponent-based demand forecast.

4.1.1 *Survey details*

There are several sources of information on appliance ownership (e.g. UK Office of National Statistics). However, for the purposes of producing a demand forecast, Southern Water wanted to produce a forecast for different classes of customers (e.g. metered and unmetered customers) so appliance ownership by customer type was required. In addition, Southern Water's distribution area is divided into several zones which have different socio-economic characteristics. Therefore in order to obtain such detailed information, a questionnaire was sent to 60,000 customers in November 2002. Approximately 25,000 responses were obtained from this survey.

From the results of the survey it was observed that there were differences in appliance ownership by area, property type and account type (metered or unmetered). Examples of the results are given in the figures below which show dishwasher ownership (Fig. 5) and hose pipe ownership (Fig. 6) by account type for 8 water resources zones (Isle of Wight, Hampshire North, Hampshire South, Sussex North, Sussex Coast, Kent Medway and Kent Thanet) and the company as a whole (SWS).

4.1.2 *Forecasting of appliance ownership*

The information from the survey combined with the results of the Identiflow studies provides ownership, frequency and volume information for the present. However, as the results of the demand forecasting exercise are sensitive to assumptions used, to produce a forecast, information

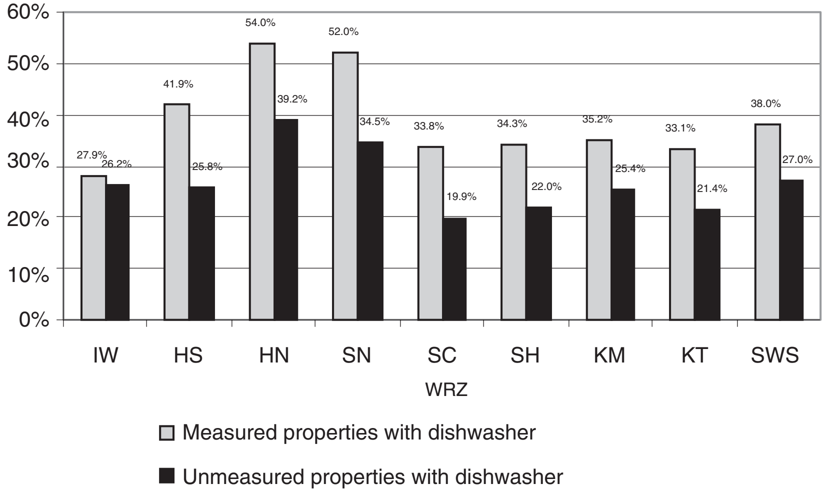

Figure 5. Ownership of dishwashers by account type and resource zone.

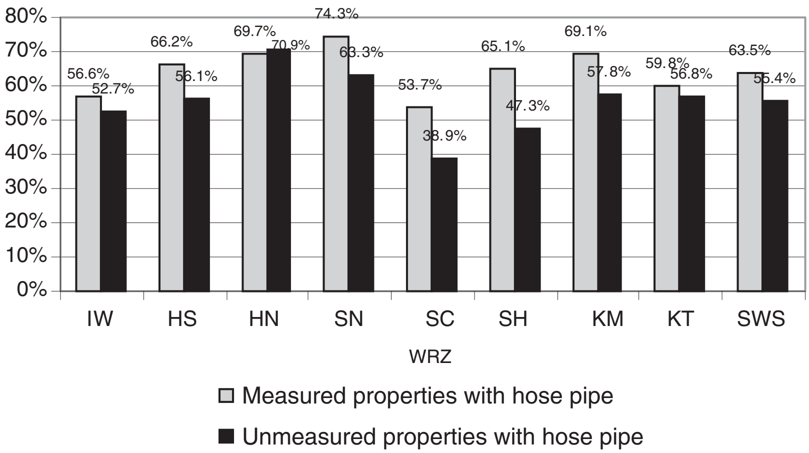

Figure 6. Ownership of hose pipes by account type and resource zone.

on future appliance ownership and volumes is required. For instance the current stock of toilets has an average flush volume of 9.5 litres whereas modern European cisterns are 4.5 to 6 litres. Given that around 30% of household water use is used for toilet flushing the rate of replacement of toilet cisterns has a material impact on microcomponent use. Information on appliance replacement rates was obtained from Experian Business Strategies.

4.1.3 *Results of the forecasting methodology*

By combining information on the use of appliances from Identiflow, with the results of the current and forecast appliance ownership surveys it is possible to develop forecasts of consumption for each microcomponent. Figure 7 shows the forecast of water used for toilet flushing as a percentage of current volumes. Similar analysis has been completed for other appliances to derive a per capita consumption (pcc) forecast.

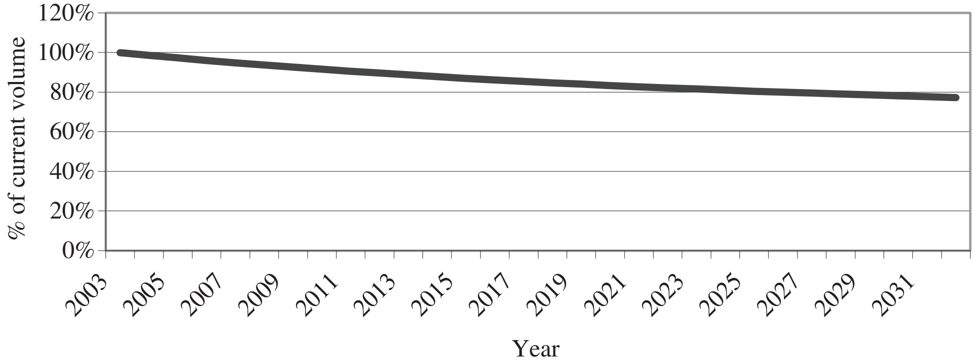

Figure 7. Forecast changes in toilet flush volume as percentage of current volume.

5 CONCLUSIONS

5.1 *Microcomponent information is essential for demand forecasting*

The three key aspects in microcomponent forecasting are ownership, volume and frequency of use. Two of these should be directly measured. Frequency of use represents current customer behaviour and is essential. Indirect methods e.g. keeping a diary of use have been shown to be very inaccurate.

Volume per use needs to be measured for appliances in actual use. Modern washing machines have many programmes and so the range of volumes used should be determined. The size of a bath only determines the maximum volume used.

In contrast, ownership predictions are best determined by market studies. Trends in occupancy, house type, economic prosperity, lifestyle and attitudes to water use need to be assessed by demographic, economic and other studies.

5.2 *Microcomponent analysis quantifies the impact of revenue metering on consumption*

Domestic demand forecasting consists of both unmeasured and metered customers. Most attention to date has been on unmeasured customers. However the same forecasting principles apply to metered customers and microcomponent studies of this increasing customer group will be needed.

Furthermore, the take-up of metering is likely to be over a protracted period and therefore the anticipated behaviour of meter optants will be significant. Southern Water are currently undertaking a microcomponent study of meter optants who will be monitored before meter installation, just after meter installation and about six months later in order to determine short term and longer-term changes in water use.

REFERENCES

Herrington, P. 1995. Climate change and the demand for water. *Final Report to the Institute of Hydrology and the Department of the Environment*.

Glennie, E.B.G. et al. 2002. Microcomponent analysis and peak demands. *WRc Report UC3992*. Swindon: WRc.

Kowalski, M & Stimson, K. 2001. Towards a national microcomponent monitor. *WRc Report UC3798*. Swindon: WRc.

Patel, A. et al. 2003. Microcomponent analysis and peak demands 2002. *WRc Report UC6117*. Swindon: WRc.

UK Environment Agency. 2003. Water Resources Planning Guideline, Version 3.0.

Chapter 10 – Demand management – country experiences

Water demand: a UK perspective

A. McDonald
Professor of Environmental Management, Earth and Environment Faculty, University of Leeds, UK

S. Bellfield
Water Scientist, Arup Water, Admiral House, Rose Wharf, Leeds, UK

M. Fletcher
Associate Director, Arup Water, Admiral House, Rose Wharf, Leeds, UK

ABSTRACT: Demand dictates water supply requirements thus water demand is an integral part of the UK water industry. The development of accurate estimates and forecasts of water demand and best practice measures to manage demand will have far reaching benefits to the water industry for operational management, strategic objectives and financial planning. Further, the ability to accurately estimate, forecast and manage demands will aid in the promotion of a sustainable water resource management strategy and the achievement of some of the objectives of the Water Framework Directive. This is particularly important since water resources in the UK are considered to be closer to sustainability limits than ever before (Mitchell, 1999; DEFRA, 2002a).

From a day-to-day operational viewpoint, the water utilities would be able to identify with certainty how much water is required to meet demand and how much water is unaccounted for, due to factors such as leakage and illegal use, resulting in a more cost effective and efficient service. Furthermore, it will aid in the mitigation of detrimental environmental impacts that result from unnecessary over-abstraction of water resources. From a financial and strategic perspective, accurate calculations of demand will ensure that water utilities determine their correct revenues and levels of investment to ensure that water supplies are secure both now and in the future. Appropriate targets can, therefore, be set for resource development, leakage reduction and demand management. These targets can, undoubtedly, aid in the promotion of a sustainable water resource management strategy. It may, for example, help to inform an investment strategy to ensure sustainable development of water resources, which consider social, environmental and economic impacts.

The development of effective techniques to estimate, forecast and manage demand depends significantly on the current knowledge and understanding of demand and the factors that influence it. In the UK, knowledge of demand is high by international standards due to the implementation of water consumption monitors. There remains, however, a paucity of water demand knowledge driven largely by our lack of 'good' quality metered data. This paper, therefore, aims to provide an in-depth, detailed and holistic 'state of the art' review of water demand in the UK. Definitions will be clarified and 'best practice' measures for estimating, forecasting, controlling and conserving demands will be detailed. The overall aim of this paper is to go some way to promote a blueprint for a sustainable water resource management strategy.

1 INTRODUCTION

Water is arguably the world's most precious resource. In developed countries such as the UK, society expects, and legislates for, a clean and reliable supply of potable water. Scientific investigation of

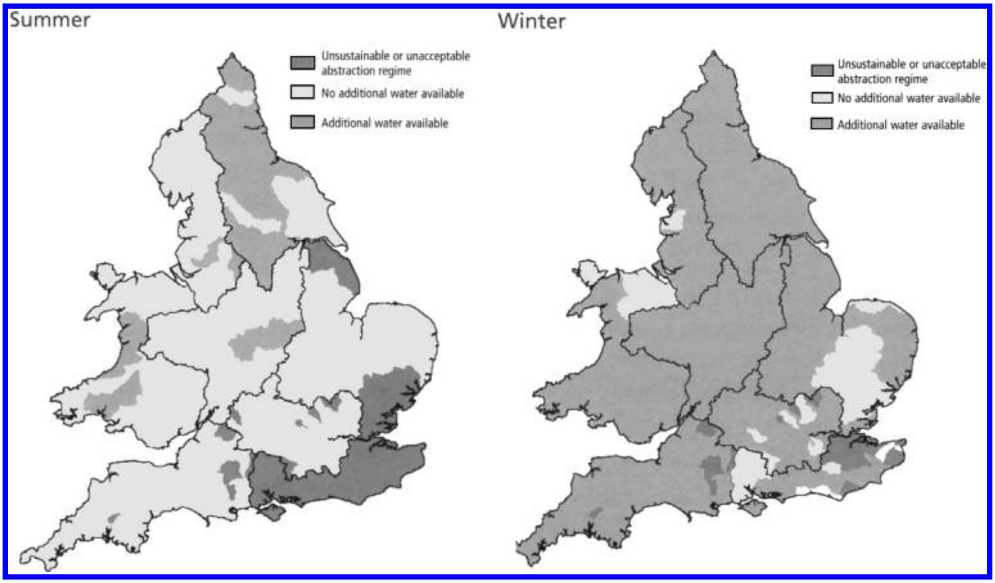

Figure 1. Summary of current water availability in England & Wales.

potable water quality in the UK started in the 18th Century and is now claimed to be set at a standard far higher than necessary to safeguard health (Department of the Environment, 1994). Despite this, immediately following the privatization in 1989 of the water industry in England and Wales, the focus of water industry investment was on potable water quality. However, since the water resource problems during the droughts of 1988–1992 and the well-publicized problems of 1995–1996, the supply of potable water has received increased attention. This attention has been further intensified by reports that current increases in demand (attributed to an absolute growth in both household numbers and household water consumption) along with climate change-induced uncertainty over supply are forcing UK water resources closer to sustainability limits than ever before (Mitchell, 1999). On present UK water demand trends, it is thought unclear that there will be sufficient water resource available beyond 2025 to meet demand (DEFRA, 2002a). Figure 1 indicates that surface water abstractions are already at their limit in the summer for much of England and also in the winter for parts of the south and east.

Demand management is viewed increasingly as a key element of a sustainable water resource management strategy. This led to a number of initiatives, established by the UK government, to promote efficient and sustainable water use. Since 1998, the Director General ensures that every water company gives high priority to reducing leakage from its distribution system, that water companies promote the efficient use of water by customers and in there own operations (DEFRA, 2002b). Effective demand management requires accurate water demand forecasts. These forecasts should predict all components of water use (e.g. domestic, industrial, agricultural) on different planning horizons (e.g. the short-, medium- and long-term) for different spatial scales. The development of accurate detailed water demand forecasts that are fit for purpose depend on our understanding of demand and the factors that (i) promote or constrain demand and (ii) confound the analysis. The paucity of water demand knowledge, resulting from confused analysis, poor data and inadequate definition, results in crude and unrealistic forecasts of future demands. Furthermore, the impacts of demand control (e.g. pricing) and demand conservation (e.g. leakage control, water saving technology, wastewater reuse and recycling, water bylaws and education and awareness of residents) are far from clear. This paper aims to provide an overall perspective of water demand in the UK, clarifying definitions and detailing measures for estimating, forecasting, controlling and conserving demands. This paper aims to raise awareness of 'demand' in the UK, with the overriding aim of going some way to promote a blueprint for a sustainable water resource management strategy.

2 WHAT IS DEMAND?

Much confusion surrounds the definition of 'demand'. The conventional definition used in the industry refers to the quantity of water demanded or just water use, rather than the stricter definition of the relationship between price and quantity. The conventional definition can be interpreted in several ways; (i) demand on resources (ii) demand on works or (iii) customer's true demand. These definitions are strongly interrelated. They depend significantly on the individual's perception of both supply and demand. Of course, the different interpretations of demand are not incorrect, however, this paper refers to demand as the customer's true demand, that is the actual amount of water used by the customer. Where another form of demand is discussed this will be made clear. These differences in definition result in large differences in apparent demand for example from unaccounted for water usage, some of which are legal (e.g. fire fighting, mains flushing), some of which are illegal (e.g. unregistered private supplies), and some of which are unknown (e.g. leakage).

3 SUPPLY/DEMAND BALANCE

To reduce the disparities between figures derived for different interpretations of demand and, hence, to control the amount of water 'lost' in supply, the water utilities have a statutory duty to report the disparity in the amount of water supplied and that demanded by the customer. This procedure called the supply/demand balance. Each year, OFWAT assesses the water utilities supply/demand balance outputs (OFWAT, 2002a) to ensure that targets set at the last price review, such as leakage performance, are being met. Every five years, the water utilities are also required to produce a long-term strategy for maintaining the balance between supply and demand. This strategy must account for future changes in demand as well as incorporating the ability to connect new premises to their networks (OFWAT, 2002b). Accurate calculation of the supply/demand balance is, therefore, vital for the water utility to determine their appropriate level of investment to ensure security of supply and levels of service. It is also vital for OFWAT who use the supply/demand balance to ensure that water utilities have selected the correct targets for leakage reduction, resource development and demand management. OFWAT also use the supply/demand balance as the basis for setting price limits as part of the Periodic Review. A key element in the supply/demand balance is, undoubtedly, the water utilities' ability to estimate and forecast the customers true demand for water.

4 COMPONENTS OF DEMAND

Water in England and Wales is abstracted for four main purposes (i) public water supply (ii) power generation (iii) agriculture and (iv) other industry. The public water supply component can be subdivided further into measured and unmeasured households and non-households and water not delivered. The water industry is required to meet the customer's demand for water for each of these components. However, many of these components of demand are unmeasured. In the UK, for example, only about 14% of domestic supplies in England and Wales were metered in 1999 (Twort *et al.*, 2000). To derive components of water demand, the water industry must subtract known demands from the total amount of water supplied and often uses the residual to estimate other components of demand.

5 WHY MEASURE DEMAND?

In the UK, water utilities have a requirement to estimate and forecast water demand: by OFWAT (as part of the annual June Returns and the Periodic Review) and to provide a cost effective and efficient service. The water utilities are unable to state with certainty the amount of water that is used, the factors that affect its use and how much water will be required in the future. Gaining this understanding and being able to develop accurate demand forecasts will have several benefits that will aid a sustainable water resource management strategy. These benefits will include the mitigation of detrimental over-abstraction. These forecasts will help to avoid the premature development of water resources. This knowledge can also be applied to resource needs forecasting for example in relation to new housing. With the proposal of 3.81 million houses to be built between 1996 and 2025 (Office of the Deputy Prime Minister, 2003), 22% of which will be built in the southeast of England where there is currently an unsustainable or unacceptable abstraction regime (DEFRA, 2002a), resource needs forecasting would be an invaluable tool.

6 MEASURING DEMAND

In the UK, water utilities are able to provide reasonably accurate estimates of non-household demand as most of these customers are on a measured tariff. Similarly, the water utilities are able to state with confidence the water demanded by measured households. However, for the largest component of demand, unmeasured households, water utilities are unable to state with certainty the amount of water consumed. To improve estimates of unmeasured household water use most water companies in the UK have conducted or are planning their own domestic consumption studies. The scale, design and purpose of these studies vary significantly from company to company.

Two main approaches are used in domestic water use estimation (OFWAT, 2002b). The first is zonal metering, where the flow of water to a group of households is measured. These groups of households are typically in a *cul de sac*, a small water supply or a district meter area. The number of households included in these studies, therefore, varies depending on the chosen location of the zonal meter. In general, *cul de sacs* include about 30 households, small water supply areas include less than 100 households, and district meter areas can include up to 2000 households. The second approach is through individual household metering. Individual meters can be placed at the boundary of the property or they can be installed inside the property at the stop tap position.

Components of domestic water use have also been studied to measure demand (e.g. Thackray *et al.*, 1978; Hall *et al.*, 1988; DeOreo *et al.*, 1996). The strength of using the component approach is that each main item of domestic water consumption, such as personal washing, toilet flushing, clothes washing and garden watering is identified and treated separately. Earlier work in this field generally relied upon the co-operation of participating households to keep daily records of water use (Thackray *et al.*, 1978; Hall *et al.*, 1988). However, a more advanced technique, developed in the United States, used flow traces that were so precise, signatures associated with all major water use categories could be identified (DeOreo *et al.*, 1996).

Several advantages and disadvantages are associated with each metering technique. The major advantage of individual household and component metering relates to the direct measurement of water consumption (with internal meters). This means that unlike zonal meters, leakage in the distribution system and communication pipe work does not affect the measured water consumption. Supply pipe leakage, however, remains a factor. When using individual household or component metering, the true figure for household water consumption is obtained if the sample is representative of the population. Several biases affect these measures (i) selection of households and self-selection (ii) the Hawthorne Effect (iii) financial advantage bias and (iv) sample decay and monitor maintenance. These are discussed in the sections that follow.

6.1 *Selection of households and self-selection*

The greatest bias occurs if the sample households are not representative of the wider population. However, no work exists which evaluates whether an area or zonal meter is representative although intuitively *cul de sac* sites, chosen as a relatively simple situation unreliant on metered area differences, are unlikely to be typical. Per capita consumption can range enormously for properties in the same area or street (Russac *et al.*, 1991). This makes it extremely difficult to select typical households. It is estimated, for example, that probably less than 10% of households regularly use a hose-pipe or a garden sprinkler. A very large number of households would, therefore, be required to participate in the study to ensure that the sample was representative with respect to such use. If a classification such as ACORN is being used to profile water use, it is possible that than an individual household will not be typical of the ACORN class for the enumeration district in which it is situated. Water companies use different measures to stratify their samples – property type, ownership type, head of household's occupation etc. The same concern, the representation of the sample in relation to the population, exists for all measures.

Further bias may also be introduced as installation of individual household and component monitors requires the agreement of the householder. Therefore, a degree of 'self-selection' is inevitable; for example, customers who use excessive amounts of water (or who ignore restrictions) will be unlikely to agree to take part in such a study. This will tend to bias the sample in favour of low water users. Also, there is a strong possibility that customers agreeing to participate in the study will be in the same socio-economic grouping, probably well educated middle-class citizens who realise the objectives of such studies. Self-selection may, therefore, result in a cross-section of the population who are low water users. In comparison, zonal metering produces more accurate and representative estimates of the population (i) because they do not require the agreement of the householder and (ii) as a group of households are monitored, the individualistic behaviour of single households is smoothed out.

6.2 *The Hawthorne Effect*

Bias may arise because occupants are aware that they are being monitored. Demand may be suppressed resulting in the underestimation of overall per capita consumption. Even if meter installation has no impact on the householder's water bill, the presence of a meter may encourage householders to deal more quickly with dripping taps for example. Also, householders are more likely to comply with regulations set by the water company such as hose-pipe bans. This is known as the 'Hawthorne Effect'. The Hawthorne Effect is named after a study performed in the 1930's in Hawthorne, Illinois in which it was discovered that the act of merely studying individual behaviour could impact upon that behaviour (Wickstrom and Bendix, 2000). Thackray *et al.* (1978) suggested that this effect is temporary; however, during meter readings, customers are reminded that they are being monitored, particularly when using internal household meters, which require the permission of the householder to be read. This effect may be exacerbated further during component studies where the occupants record their own water use and hence, are reminded constantly that they are being monitored. In a zonal meter, however, customers are unaware that their water consumption is being monitored and, therefore, there can be no question of the presence of a meter directly influencing the results.

6.3 *Financial advantage bias*

A specialised form of self-selection is where there is a preferential uptake of domestic consumption monitor opportunity because of perceived financial advantage. Financial advantage bias is inevitably also the prime reason for customers leaving domestic water use studies. Some customers realize that it is more financially viable to opt for a meter, rather than paying for water according to the rateable value of properties. Often these customers only agree to participate in water use studies for confirmation that they are low water users. They perhaps also wish to adopt the preferential meter installation rates frequently available through water use studies. Certainly, the opt out rate in Domestic Consumption Monitor populations in favour of a metered tariff has been up to three times that experienced in the general population. In turn, there may be a disproportionate number of high water users remaining.

6.4 *Sample decay and monitor maintenance*

The perception among domestic consumption monitor administrators is that it is becoming increasing difficult to recruit new volunteers for household studies (Ridgewell, 1999, pers. comm.). There will be further losses from the study due to factors such as death or relocation. The average residence time in a property in the UK is about 10 years; therefore, about 10% of the domestic consumption monitor households will change each year based solely on relocation. Average residence time is based on the gross migra-production rate (GMR) calculated using the 1981 census of population, which states that over a lifetime, a person in the UK moves 6.5 times. If the average life expectancy of seventy years is used, the average time in one place is 10.77 years (Stillwell, 2000, pers. comm.). In early studies the characteristics of the households in studies were surveyed only at the start of the monitoring programme. Thus factors such as family growth, deaths, unemployment or new employment, divorce etc., will all reduce the validity of the original data. In recognition of these problems, several companies have introduced household re-surveys. However, this both complicates the handling of the domestic water consumption data and drives Hawthorne Effects.

Regular monitoring, maintenance and replacement of water meters and loggers is essential to the success of any domestic consumption study whether it be a zonal study, an individual household study, or a component study. Without regular maintenance, the potential for errors in the water consumption will undoubtedly increase. In any event meter under-registration is likely to occur driven by age or, a more recent view, by gross throughput irrespective of age. A figure of 2–3% is generally accepted as under registration.

7 FORECASTING DEMAND

The water industry employs a range of time horizons. For strategic purposes, such as bids for licence renewal, the industry is required by OFWAT to make estimates of demand and resource availability on a 25 year time horizon. Intermediate time horizons, for example in the planning, commissioning and implementation of new major works, is in the order of 10–15 years. Management time horizons are in the order of 3–5 years but tactical management ranges from a few months to a year. Operational management is in the order of weeks. Figure 2 illustrates the range of time

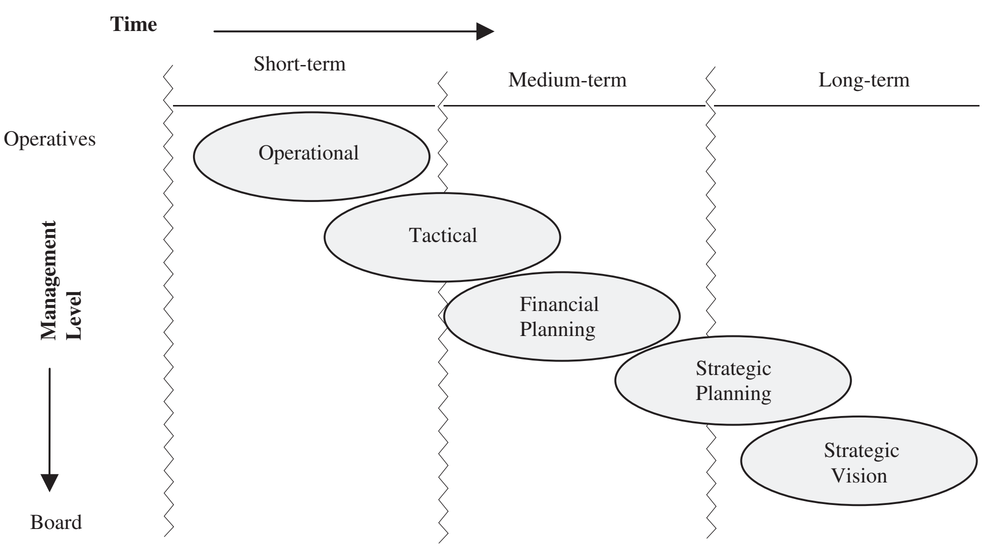

Figure 2. Time horizons employed in the UK water industry.

horizons employed and, since there is no consistent nomenclature on which to draw, attempts a logical characterisation of time horizons.

The water utilities are required to forecast each component of demand in the different time-horizons. Non-household demand is driven primarily by the state of the local economy, an index of moisture deficit and price structures (Mitchell, McDonald and Williamson, 2000). Accurate forecasts of non-household demand are dependent, therefore, on accurate forecasts of these three components. Forecasting household demand is, however, more problematic. Household demand is dependent on a number of variables. These include population growth, household size, household type, socio-economic grouping, migration and immigration (both legal and illegal), appliance ownership, garden size, meter uptake rates and the behavioural characteristics of the individual to name but a few. These factors, combined with uncertainties relating to climate change, the uptake of demand management measures and the mechanism for charging for water makes domestic demand forecasting extremely problematic.

8 DEMAND MANAGEMENT

Demand management is viewed increasingly by water utilities and governing bodies as a potential means of aiding the security of future water supplies. There are two ways in which demand can be managed; (i) by controlling demand and/or (ii) by conserving or displacing demand.

8.1 *Controlling demand*

The UK is almost unique in Europe in that households are not generally charged for the amount of water used. The marginal cost of water is in effect zero, which promotes little incentive to economise on water use. Some believe that if customers are charged for water on a measured tariff, bill increases might result in the customer being even less willing to take measures to reduce demand (even in times of shortages such as 1995) (Environment Agency, 1997). Despite this, the overall consensus is that water metering with appropriate tariffs has significant potential to manage demand (DEFRA, 2002b). Evidence reported by Herrington (1998) suggests that domestic metering may have a significant effect upon reducing peak demands. Figures vary but trials have shown reductions of between 10–20% for average demands and 15–30% for peaks primarily due to immediate behavioural changes (Environment Agency, 1997). However, retrofitting of meters may not be economic in the short-term and invites unfavourable comment on access to this important resource.

8.2 *Conserving demand*

At present, the uptake of water conservation measures in the UK is relatively small and is generally only apparent where there is obvious economic gain. As part of the Periodic Review 2004, however, OFWAT have proposed a way of promoting efficient water use. If the cost of saving water by adopting a demand management measure is less than the cost of delivering additional water, then the company has a statutory duty to promote demand management measures. OFWAT have, therefore, stated that there is an economic level of demand management activity (OFWAT, 2002b).

Current water conservation measures include leakage control, water saving technology (e.g. pressure reduction valve and cistern displacement devices), wastewater reuse and recycling, water bylaws and education and awareness of residents. There are numerous studies that assess the impact on consumption of demand management measures. Whitcomb (1991), for example, estimated that per-capita in-house water use would decline by 6.4% due to the installation of water efficient showerheads. Similarly, dual flush devices were found to create savings of up to 64% in water used for toilet flushing (Jones, 2002). On a larger scale, several major waste minimization projects, including Project Catalyst in the Mersey Basin, have shown that a significant reduction in water use occurs through the adoption of cleaner technologies. The fourteen companies that participated in Project Catalyst have collectively reduced water consumption by 5.3 Ml/d with measures that have paid for themselves in less than one year (Environment Agency, 1997).

Despite (i) the proven capabilities of water conservation measures and (ii) the UK government's proactive approach in promoting demand management measures (primarily through the Environment Agency's Demand Management Centre), there is currently limited interest in saving water in the UK. Some water conservation measures, such as direct wastewater or rainwater reuse, has been dismissed in the UK largely because of the possibility of cross contamination between the potable and non-potable systems and the consequent risk of harm to public health. For this reason, it has been assumed that direct reuse would not be acceptable to the public. However, recent research, albeit on a very small sample (Coventry University, 1996, pers. comm.), suggests that this may not be the case. Of 118 people who responded to a postal survey, 93% were in favor of rainwater reuse in domestic properties. However, the most likely reason why there is limited interest in demand management measures in the UK is because there is little financial incentive to economize on water usage. Demand management schemes would be best implemented with customer metering and an appropriate tariff structure, which encourages demand reductions by introducing an economic incentive to reduce wastage, leakage and the uneconomic use of water (UKWIR, 2002). A water conservation and demand management strategy should, therefore, be a coherent whole and not a number of isolated, uncoordinated options.

9 CONCLUSION

A greater understanding of demand management is essential for optimum use of water at a local, regional and national level. A fundamental appreciation of what 'demand', 'supply/demand balance', 'demand forecasting', 'demand management' mean can help inform the wider debate of how we should improve management of our water resources.

Greater emphasis on control and conservation of demand has arisen from 'raised awareness of the fundamental role water plays' in the public domain. The trend for improved understanding and informed demand management within the regulatory framework ultimately improves the service provided by the water industry to the customer.

REFERENCES

DeOreo, W. B., Heaney, J. P. and Mayer, P. W. (1996) 'Flow trace analysis to assess water use', *Journal of the American Water Works Association,* **88**(1), 79–90.

Department for Environment, Food and Rural Affairs (2002a) *Directing the Flow – Priorities for Future Water Policy*, London.

Department for Environment, Food and Rural Affairs (2002b) *Second Consultation Paper on the Implementation of the EC Water Framework Directive (2000/60/EC),* Welsh Assembly Government, October 2002.

Department of the Environment (1994) *Sustainable Development – The UK Strategy,* HMSO, London.

Environment Agency (1997) Saving Water: Taking Action – The Responses to the Consultation Report on Water Conservation and Demand Management, National Water Demand Management Centre, October 1997.

Hall, M. J., Hooper, B. D. and Postle, S. M. (1988) 'Domestic per capita water consumption in South West England', *Journal of the Chartered Institution of Water and Environmental Management,* **2**, 626–631.

Herrington, P. R. (1998) 'Analysing and Forecasting Peak Demands on the Public Water Supply', *Journal of the Chartered Institution of Water and Environmental Management,* **12**, 139–143.

Jones, I. (2002) *Dual–Flush Toilets Save Water, Study Shows*, (online), Environment Agency, available from www.environment-agency.gov.uk/news/341762, accessed 01-05-03.

Mitchell, G., McDonald, A. and Williamson, P. (2000) 'A SIC Coded Strategic Planning Model of Non-Household Water Demand for UK Regions', *Journal Chartered Institute of Water and Environmental Management* **14**(3), 226–233.

Mitchell, G. (1999) 'Demand forecasting as a tool for sustainable water resource management', *International Journal of Sustainable Development and World Ecology*, **6**, 231–241.

Office of the Deputy Prime Minister (2003) *Housing Statistics – Projections of Households in England 2021*, www.housing.odpm.gov.uk/statistics/publicat/household/project/08.htm

Office of Water Services (2002a) *OFWAT Forward Programme 2002–03 to 2004–04,* OFWAT, March 2002.

Office of Water Services (2002b) Security of Supply, Leakage and the Efficient Use of Water 2001–02 Report, OFWAT, October 2002.

Ridgewell, C. (1999) *Domestic Consumption Study Administrator*, Essex & Suffolk Water, personal communication.

Russac, D. A. V., Rushton, K. R. and Simpson, R. J. (1991) 'Insight into domestic demand from a metering trial', *Journal of the Institution of Water and Environmental Management,* **5**(3), 342–351.

Stillwell, J. (2000) School of Geography, University of Leeds, personal communication.

Thackray, J. E., Cocker, V. and Archibald, G. G. (1978) 'The Malvern and Mansfield studies of domestic water usage', *Proceedings of the Institution of Civil Engineers*, **64**(1), 37–61.

Twort, A. C., Ratnayaka, D. D. and Brandt, M. J. (2000) *Water Supply,* 5th Edition*,* London, Arnold, 676pp.

UK Water Industry Research Limited (2002) *The Economics of Balancing Supply & Demand (EBSD) Main Report*, Report Ref. No. 02/WR/27/3.

Whitcomb, J. B. (1991) 'Water reductions from residential audits', *Water Resources Bulletin*, **27**(5), 761–767.

Wickstrom, G. and Bendix, T. (2000) 'The Hawthorne Effect – what did the original Hawthorne studies actually show?' *Scandinavian Journal of Work, Environment and Health,* **26**(4), 363–367.

Water demand management in Syria: technical, legal and institutional issues

M. Salman
IPTRID/AGL, FAO, Rome, Italy

W. Mualla
Faculty of Civil Engineering, Damascus University, Damascus, Syria

ABSTRACT: Syria is classified amongst the semi arid countries of the Middle East. It has a population of 18 million in the year 2002, and its Total Renewable Water Resources (TRWR) is estimated around 16 BCM per year. In other words, the per capita TRWR is now less than the water scarcity index (1000 m^3/person/year). Although, this would currently rank Syria amongst countries with moderate water stress, it will be soon classified as a country with severe water stress if its population continues to grow at its current rate (about 3%) and water use efficiency is not increased effectively.

In Syria and until fairly recently, emphasis has been put on the supply side of water management. Demand management and improvement of patterns of water use has received less attention. Water managers and planners have given high priority to locating; developing and managing new water resources. The aim was always to augment the national water budget with new water. The most popular way of achieving this aim was to control surface flows by building new dams and creating multi-purpose reservoirs. (There are now around 160 dams in Syria with a total capacity of 14 BCM). Irrigation schemes were also built and agricultural activities were expanded greatly to achieve self-sufficiency in essential food products and food security.

Over the years, however, the most attractive alternatives for the development of water resources infrastructure have already been implemented and, it is hard to think of feasible alternatives for a further increase in supply. In addition, the cost of developing less accessible water is high and time consuming. Therefore, the emphasis has now been shifted to demand management. And since irrigation is the main user of water in Syria, special attention is now given to address its management deficiencies.

This paper describes some of the measures that have been taken recently on the technical, institutional and legal sides of demand management in the water sector of Syria. Early signs indicate that these measures have already played a significant role in alleviating some of the mounting pressure on water resources especially in periods of draught.

1 WATER SUPPLY AND USE IN SYRIA

In Syria, the total estimated water use volume is about 15 billion m^3. The Euphrates and Orontes basins account for about 50% and 20% of the water use respectively. Table 1 shows water availability and use in the various basins of Syria. As shown in this table, water balance in most basins has been in deficit (except in the coastal basin and the Euphrates basin). This will be exacerbated further especially in those basins encompassing large urban areas such as Damascus and Aleppo.

Agriculture is the largest water-consuming sector in Syria accounting for about 87% of water use. The domestic and industrial water use stand at about 9% and 4% respectively. While the urban water demands is rapidly increasing due to strong population growth rate (about 3% per annum) and industrial growth, new water sources are becoming scarce and extremely expensive to develop.

Table 1. Water availability and use (World Bank, 2001).

Basin	Irrigation ($m.m^3$)	Domestic ($m.m^3$)	Industrial ($m.m^3$)	Total use ($m.m^3$)	Renewable water resources ($m.m^3$)	Deficit ($m.m^3$)
Yarmouk	360	70	10	440	500	60
Aleppo	780	280	90	1150	500	−650
Orontes	2230	320	270	2730	3900	1170
Barada/Awaj	920	390	40	1350	900	−450
Coastal	960	120	40	1120	3000	1180
Steppe	340	40	10	390	700	310
Euphrates	7160	250	110	7520	N.A.	N.A.
Total	12750	1390	570	14700	–	–
% Share	87%	9%	4%	100%	–	–

Water deficits are expected to worsen placing additional stress on all uses. Since drinking water needs are given top priority in the government's policy, water availability for agriculture use could face severe constraints.

2 PRESSURE ON WATER RESOURCES

Pressures on water resources of the country come from all sectors of the economy with highest demand from agricultural sector.

Agriculture dominates the Syrian economy. It contributes about 32% to the GDP, and employs nearly 31% of the workforce, with another 50% of the manufacturing force dependent on it for employment.

In 2000, the cultivated land area in Syria was estimated at 5.5 million ha, which accounted about 30% of the total country area. 20% of the cultivated land area (1.2 million ha) was irrigated.

The total irrigated area increased from 650,000 ha in 1985 to 1.3 million ha in 2002. This remarkable expansion of irrigation is mainly attributed to the rapid increase in groundwater irrigation (Table 2). 60% percent of all irrigated area in Syria is currently irrigated by groundwater, which are all privately developed and operated.

This great expansion of groundwater-irrigated agriculture has resulted in groundwater being overexploited in most basins of the country. Continuous decline in groundwater tables have been accounted affecting some surface sources such as spring flows and causing seawater intrusion in land areas adjacent to the sea.

A substantial portion of the increase in groundwater use is related to increases in irrigation for wheat, cotton, citrus, and sugar beet. Area increases have been substantial in the last decade in sugar beet (32%), cotton (75%), irrigated wheat (40%), and citrus (40%). Much of the expansion in wheat has been driven by rapid expansions of its price while water cost has remained low. Farmers from public irrigation schemes obtain water at an extremely subsidized rate, and groundwater costs do not reflect their real value because the energy required for pumping is also subsidized.

Government policies have contributed to the tremendous increase in groundwater irrigation. Wheat supported prices which have been higher than the world prices for several years, coupled with subsidized energy costs have proved to be strong incentives for farmers to take up groundwater irrigation in many areas.

Legally, licenses are required to drill and use wells. Licenses specify the extent of water use and require renewal every ten years. However, poor enforcement has resulted in a large increase in the number of illegal wells in recent years (almost 50% of the total number of wells) that has contributed to the groundwater table declines in many areas, especially in Damascus countryside (Table 3).

Table 2. Irrigated area by source of irrigation (Somi et al, 2001, 2002).

Year	Surface irrigated (1000 ha)	Groundwater irrigated (1000 ha)	Total irrigated area (1000 ha)
1985	334 (51%)	318 (49%)	652
1990	351 (51%)	342 (49%)	693
1995	388 (36%)	694 (64%)	1082
2000	512 (42%)	698 (58%)	1210
2002	583 (43%)	764 (57%)	1347

Table 3. Licensed and non licensed wells by region, May 2002 (Ministry of Agriculture).

	Number of wells		
Governorate	Legal	Illegal	Total
Damascus Countryside	5833	32688	38521
Al-Sweida	604	172	776
Qonaitra	244	134	378
Daraa	2957	1300	4257
Homs	11053	9041	20094
Hama	5551	6734	12285
Al-Ghab	1859	2865	4724
Idlib	9298	1079	10377
Aleppo	5647	19285	24932
Tartous	8384	3846	12230
Latakia	1041	4724	5765
Al-Raqa	3297	496	3793
Al-Hassaka	18747	10351	29098
Deir Al-Zour	2262.0	3195.0	5457.0
Total	76777	95910	172687

Conveyance efficiencies of surface irrigation canals in general do not exceed 50–60%. On farm water efficiency is low (about 40–60%) due to over irrigation by farmers, the use of traditional irrigation techniques, and the inadequacy of land leveling.

Urban water demand has also increased rapidly during the last decade due to strong population growth (around 3%) and industrial growth. The first objective of the national water policy has always been the provision of safe drinking water. 95% of the population in urban areas and 80% of the population in rural areas have access to safe potable water. Urban and rural water supply and sanitation facilities have been enlarged and upgraded regularly to accommodate the expanding population. However, unaccounting for water has exceeded 50% of the produced water in some urban water distribution systems. These are mainly attributed to nonphysical losses such as illegal connections (25%) and physical losses such as leakage (25%). These figures suggest that significant potential for reducing domestic and municipal water use does exist.

Barada/Awaj basin, where Damascus is located has no significant water sources, both surface and groundwater, other than the Barada and Figeh Springs which supply drinking water to the inhabitants of Damascus. As most of water resources of the basin are being dedicated continuously to support Damascus increasing demand for drinking water, internal conflict over water has risen. Farmers, in Damascus countryside, who have been using groundwater for irrigating their lands for years, have protested the drying up of their wells caused by the unsustained groundwater extraction.

Damascus Water Supply and Sewerage Authority (DAWSSA) is planning a huge inter-basin water transfer project from the coastal region, where there still excess fresh water, to Damascus city to meet its rapidly growing domestic water demand. However, the project is still in its feasibility design phase and may not materialize before a decade which makes it only a long term solution.

3 MAJOR WATER SECTOR CHALLENGES

As new water sources have become increasingly inaccessible and the cost of augmenting water supply has become very high, the emphasis has now been shifted to demand management. And since irrigation is the main user of water in Syria (85%), special attention has been given to address its management deficiencies.

Syria's strategy to adopt demand management and water conservation involves measures taken on the technical, legal, and institutional sides.

On the technical side the goal was to increase the efficiency of water use in all sectors of the economy, domestic, agricultural and industrial, and to protect the environment. On the legal side, it has been realized that adequate legislations and regulations must be introduced to protect water resources, to encourage people to adopt water conservation measures, and to promote the efficient use of water in all sectors of the economy. On the institutional side, the aim was to build management capacities in the various institutions of water sector in order to promote to the role of water manager rather than water supplier, which has been the traditional role.

What follows is a summary of some of the measures that have been taken recently and fall under the category of demand management:

3.1 *Technical*

The Ministry of Irrigation has started an ambitious plan investing about 32 billion Syrian Pounds (600 million US$) for the next 4 years on the rehabilitation and modernization of old irrigation projects to improve conveyance efficiency and minimize distribution losses through converting open irrigation canal systems to pressurized pipe systems.

As to protect the environment and provide additional sources of water, the Ministry of Housing and Utilities has started a programme for the reuse of treated and industrial wastewater. Most major cities in Syria have wastewater treatment plants either in operation (Damascus, Aleppo, Hamma and Homs) or in the phase of construction (Table 4). The government policy is to ultimately utilize within each basin all treated wastewater. This should provide about 475 MCM of treated wastewater available on an annual basis.

The Ministry of Agriculture is encouraging farmers to use advanced on-farm irrigation techniques like drip and sprinkler irrigation to improve on-farm irrigation efficiencies and conserve water. The ministry has provided generous loans to meet the capital costs of sprinkler and drip irrigation systems. The coverage of the credit has been expanded from 85 to 100% of the total capital costs in response to recent draughts. However, the present level of adoption of these techniques is still fairly low, with sprinkler irrigation covering about 125,000 ha and drip irrigation covering about 42,000 ha (Table 5).

The Ministry of Agriculture is planning annual crop intensity, crop rotation, and crop patterns according to the available renewable water resources in each year.

3.2 *Legal*

The Ministry of Irrigation has banned the drilling of new wells in all basins except in the coastal basin. According to the new regulations, all drilling rigs must be placed under a specific depositary controlled by the government. Drilling contractors are required to get permits for moving the rigs for any new job. All existing wells in the country are to be equipped with discharge meters. Maximum extraction level will be specified for each well.

The Ministry of Irrigation is in the process of regularizing all illegal wells. Farmers and other citizens must register with the proper authorities any illegal well within their properties, and apply for a license. A committee is set up in each basin to study each application and decide whether to grant the license or to close the well. In case a license is granted, a discharge meter is installed

Table 4. Domestic wastewater treatment plants (Ministry of Housing and Utilities).

Location	Status	Population served	Daily discharge (m^3/day)
Damascus	Operating	2,200,000	485,000
Alleppo	Operating	2,000,000	255,000
Hama	Operating	500,000	70,000
Homs	Operating	655,000	133,900
Selemiyah	Operating	45,000	5,850
Dar'aa	Under Construction (2003)	124,000	21,800
Idleb	Under Construction (2003)	182,537	30,000
Latakia	Under Construction (2004)	506,600	100,830
Tartous	Under Construction (2004)	154,370	33,437
Al-Sweida	Planned (2004)	138,200	18,750
Hasakeh	Planned (2005)	207,300	37,314
Al-Raqqa	Planned (2005)	330,000	61,000
Deir Al-Zour	Planned (2005)	250,000	45,000
Total	–	7,293,007	1,297,881

Table 5. Irrigated area with modern irrigation techniques, May, 2002 (Ministry of Agriculture).

	Areas irrigated using modern on-farm irrigation technologies (ha)			
Governorate	Sprinkler	Drip	Improved surface	Total
Damascus Countryside	1828	7997	39	9864
Al-Sweida	0	1578	0	1578
Qonaitra	65	1620	0	1685
Daraa	3813	9699	0	13512
Homs	6802	7194	377	14373
Hama	28715	1454	1083	31252
Al-Ghab	9681	94	0	9775
Idlib	28773	4100	0	32873
Tartous	240	3765	0	4005
Latakia	124	1201	0	1325
Aleppo	14119	1447	462	16028
Al-Raqa	3412	427	110	3949
Al-Hassaka	27416	1332	394	29142
Deir Al-Zour	33	41	450	524
Total	125021	41949	2915	169885

and a maximum extraction is specified depending on well location, irrigated land area and other factors (Table 6).

In some areas, the government has proposed well consolidation as an alternative to well closures. This involves the closure of private wells and the provision of water to farmers through a much more limited number of collective wells. This reduces well interference problem and allow well being carefully located where resources are sufficient. In addition it establishes clear points where control could be exerted over extraction levels and water use efficiency could be encouraged. In Aljezira region, Northeast Syria, several irrigation projects and dams were constructed relying on the flow of Al-Khabour River. This river receives its main discharge from Ras-Al-Ayn aquifer and a group of springs surrounding it. The vast number of wells drilled in the Ras-Al-Ayn area and the overexploitation of groundwater by farmers has led to a severe decline in groundwater table and the discharges of Ras-Al-Ayn aquifer and the surrounding springs threatening the regular constant flow of Al-Khabour River and of course the projects relying on its water. The government has taken an immediate action to recover the situation flow by drilling 68 wells with an average discharge of 200 l/sec and extracting

Table 6. Regulated wells since August 2000 (Ministry of Agriculture).

Governorate	Number of regulated wells
Damascus Countryside	2367
Al-Sweida	354
Qonaitra	83
Daraa	830
Homs	1193
Hama	1596
Al-Ghab	206
Idlib	1337
Aleppo	4577
Tartous	300
Latakia	30
Al-Raqa	551
Al-Hassaka	2680
Deir Al-Zour	321
Total	16425

water from these wells to support the flow of Al-Khabour River. It has also taken a long term plan to consolidate illegal wells already exist in Ras-Al-Ayn area by establishing an irrigation project based on the concept of "cooperative farming" by drilling a number of wells with high discharges to replace farmers illegal wells. This project is set to irrigate an area of 20,000 ha. Its feasibility study has completed and it will be soon implemented.

Due to the fragmented nature of current laws and the absence of a comprehensive and unified water law that matches the development of irrigation and land reclamation projects, the Ministry of Irrigation has drafted a new bill to supersede and replace existing water laws. The new law is currently being considered by Parliament and is expected to be passed soon. The new law confirms established rights on public water but gives the government the authority to nullify them in return for adequate compensation. It specifies that a license must be obtained for digging wells or installing pumping equipment. Each license specifies the extent of water use. The law requires that a meter to be installed to monitor extraction level. The minister of irrigation has the right to nullify any license if allowed extraction levels are exceeded. Licenses for wells are renewed every year, while licenses for the installation of pumps are valid for ten years. A fee is prescribed for issuing or renewing every license while irrigation tariffs are based on irrigated land area not on metered water consumption. The new law also includes sections about water pollution control and legal actions in case of violation. It prohibits the disposal of wastes that may cause pollution from any source into any public waterway.

3.3 *Institutional*

One of the most significant institutional strengthening steps in water resources management that has been taken recently is the establishment of the Water Resources Information Center (WRIC) within the framework of the Ministry of Irrigation and in collaboration with Japan International Cooperation Agency (JICA). The long-term objective of this center is to achieve integrated and sustainable surface and groundwater management (quantity and quality) in the whole basins of Syria. A water resources information system which comprises of hydrological and meteorological observation stations, and computer system and computer network is established at the Main Center in Damascus and at two Basin Centers (Barada/Awaj and the Coastal Basins). Other Basin Centers will be established at a later stage. The project also involves the preparation of a monitoring programme of meteorological, hydrological, groundwater, water quality, and unconventional water data in Barada/Awaj Basin and the Coastal Basin, in the first stage, and in the remaining basins at a later stage.

There are currently 143 industrial facilities within the private sector in Syria that produce on-farm irrigation equipments. However, there is a need for standardization of the equipment in order to ensure

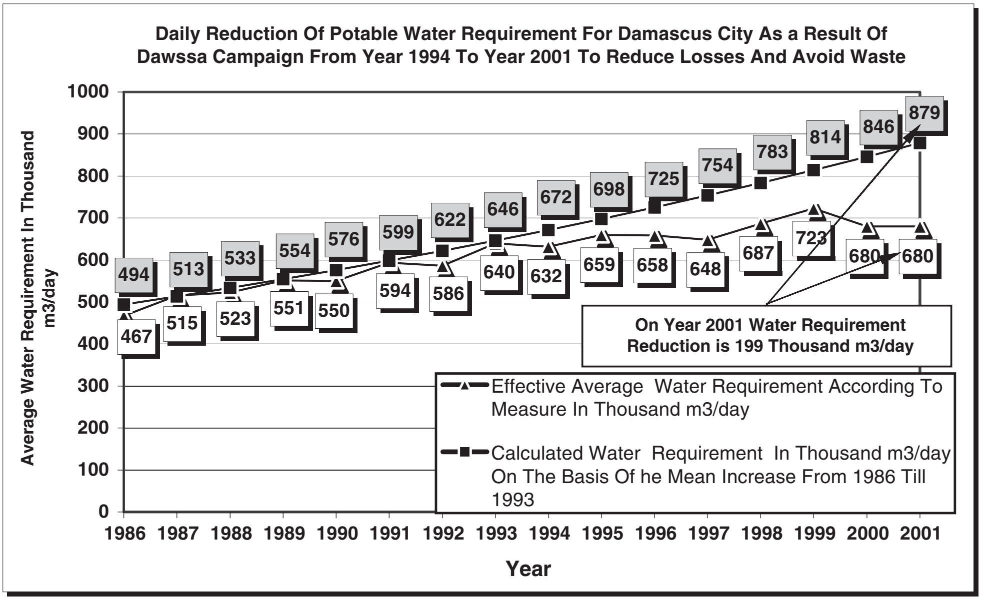

Figure 1. Daily reduction of portable water requirements for Damascus City.

quality control. The Ministry of Agriculture is currently preparing (with the help of FAO & UNDP) the Syrian Standard Specification for the on-farm irrigation equipment. Compliance with this Standard shall be prerequisite for the qualification of any bank loans.

The Ministry of Higher Education is currently establishing water resources management center. This will provide training programmes on wide array of social, economic and technical issues necessary for water management. Research programmes pertaining to water resources management will also constitute essential part of the center's activities.

4 CASE STUDY 1: DAMASCUS WATER SUPPLY AND SEWERAGE AUTHORITY

Faced with increasing demand on drinking water as a result of population growth and socio-economic development, and the limited available of water resources in Damascus region, DAWSSA has launched in the past few years an integrated programme of water conservation. The programme involves working in different directions simultaneously.

The first direction aimed at reducing physical losses from the distribution system. DAWSSA has modernized its capacities in locating and repairing pipe breaks and leaks by establishing a modern leak detection unit and providing adequate training for its staff. Furthermore, a programme for replacing older pipes of the network was also initiated with the help of JICA. In total, more than 200 kilometers of pipes of various diameters have been replaced in the last three years (The total length of pipes scheduled for replacement is about 340 kilometers). This has led to a significant reduction in the daily water consumption. For example, demand for water has remained almost steady (680,000 m^3/day) from 2000 to 2001 despite the fact that the number of subscribers has increased by 21,000 (see Figure 1). The second direction aimed at reducing non-physical losses from the network. Almost, 42,000 illegal connections have been removed in the past few years, and replaced with metered service connections. The third direction was to improve DAWSSA's capabilities in managing the distribution system in an optimal way. A database management system for the entire production and distribution system has been designed, and a Supervisory Control and Data Acquisition System (SCADA) have been installed. This has helped very much in designing efficient O&M strategies for the distribution system which ultimately contributed to the water conservation campaign.

It is evident from Figure 1 that since DAWSSA started its water conservation campaingn in 1994, substantial reduction in the daily water consumption (almost 200,000 m^3/day) has been achieved.

5 CASE STUDY 2: MODERNIZATION OF OLD ALYARMOOK IRRIGATION PROJECT

Old Alyarmook project is part of Alyarmook basin located in southern Syria at some 10 km northwest of Daraa city with a total designed irrigated area of 7175 ha. The main water sources of the project are Almzerib lake, Zeizoun lake, and a group of springs within the project area.

In view of the rapid decrease in the discharges of the project main sources and due to the government policy to give drinking water needs top priority, the project has reached a level of shortage that urged the government to look for an immediate action to sustain the project agricultural water demand. For example, the discharge of Almzerib Lake has decreased from 1202 l/sec in 1992 to 760 in 2002 and from its current discharge 620 l/sec is allocated to supply Daraa city and Swaida city leaving irrigation from this source only with 140 l/sec.

The government has implemented three measures to recover such a shortage: further increase in water supply, improvement in conveyance efficiency, and improvement in field efficiency.

The government has started the exploitation of two projects to extract an average discharge of 740 l/sec from nearest water resources, Alhrair and Altabariaat. These two supply projects are still under construction. Since 1997 and with a total budget of 823 million Syrian Pounds (16.5 million US$) the government has started a project of converting all open canals to pressurized submersed pipes in order to save water and enhance equity rights through improving conveyance efficiency and minimize distribution losses (an average of 20–30%), and by preventing farmers from abstracting water illegally from open canals. The first phase of the conversion was completed while the second is due next year. The government has encouraged farmers to use advanced on-farm irrigation techniques like drip and sprinkler systems to improve field efficiencies and conserve water. The Ministry of Agriculture has provided loans to farmers to meet the capital cost of modern systems (interest free for the first five years and 4.5% after). The coverage of the credit has been expanded to cover almost 100% of the total capital cost in response to the recent draughts and severe water shortages (average of 55,000 Syrian Pounds/ha, according to the type of the system). The Ministry of Agriculture has been providing any farmer who is interested in converting his irrigation technique to a modern one with detailed feasibility study and cost estimation. An area of 3200 ha of the total project area has already converted to drip irrigation mainly cropped with olive, vine, and summer vegetables.

6 ISSUES STILL NEED TO BE TACKLED

Many important issues still need to be tackled. Syria has not addressed seriously yet the most important factor that is putting the highest pressure on the country's resources in general and water resources in particular, namely the population growth rate. Averaging around 3% per annum, this rate is considered amongst the highest in the world and should be brought into control.

Many contradictory policy issues need to be settled. For example, government policy of encouraging farmers to invest in modern on-farm irrigation technologies is at odd with the government irrigation tariff policies which do not provide incentive to farmers to conserve water since the operation and maintenance charge for the public surface water irrigation schemes is a flat fee based on field size and unrelated to actual water consumption.

Although there has been a remarkable expansion on the use of modern irrigation techniques, still farmers' perception on the practice of these new techniques is poor. The government should implement intensive training programmes to enhance farmers' knowledge.

Communication and information systems are essential to bring the message of Water Demand Management to the end users. Long term investment programmes to transfer knowledge on actual crop water needs and the development and adaptation to higher value and less water intensive cropping patterns from research centers to farmers need to be implemented.

The institutional aspects of treated wastewater reuse still needs attention through better organization and regulation on the basis of well defined standards and specifications. The environmental and health implications of treated wastewater reuse need to be disseminated to farmers who also need more training on measures to minimize adverse effects.

REFERENCES

Damascus Water Supply and Sewerage Authority (2002) "Water Production Report for the City of Damascus in the year 2001", (Arabic).

JICA (2002) "Establishment of Water Resources Information Center in the Ministry of Irrigation, Syria", project document.

Ministry of Agriculture and Agrarian Reforms (2002) "Tables of Actual Transformation to Modern Irrigation Methods", Syria.

Ministry of Agriculture and Agrarian Reforms, (2002) "Tables of Legal, Illegal and Regularized Wells", Syria.

Somi, G., Zein, A., Shayeb, R, and Dawood, M. (2001) "Participatory Management of Water Resources for Agricultural Purposes in the Syrian Arab Republic", internal paper, Ministry of Agriculture and Agrarian Reforms, Syria, (Arabic).

Somi, G., Zein, A., Dawood, M. and Sayyed-Hassan, A (2002) "Progress Report on the Transformation to Modern Irrigation Methods till the end of 2001", Internal Report, Ministry of Agriculture and Agrarian Reforms, Syria, (Arabic).

Somi, G. and Zein, A (2002) "The Syrian National Plan for the Transformation to Modern Irrigation Methods", Paper presented at the International Water Day, March, 2002, (Arabic).

World Bank (2001) "Syrian Arab Republic Irrigation Sector Report", Rural Development, Water and Environment Group, Middle East and North Africa Region, Report No. 22602-SYR.

World Bank/UNDP, "National Environmental Action Plan for the Syrian Arab Republic", January 1999.

World Bank/UNDP, "State of the Environment in Syria", April 1998.

Study on stratagem of developing water resources for Northwest China

L. Jianlin, H. Lingmei & S. Bing
Xian University of Technology, Xian, China

M. Takezawa
Nihon University, Tokyo, Japan

ABSTRACT: The Shanxi, Gansu, Qinghai, Ningxia and Xizang region of Northwest China is characterized by an arid climate with rare rainfall and "poor" environment. Water is the key factor in the sustainable development of Northwest China with an area of $3.1 \times 10^6 \,\mathrm{km}^2$ (32% of the total area of China) and a population of 88.08 million people (7% of the total population of China). Although Northwest China has a strong potential for developing its economy, the water shortages and the weak ecological environment need to be overcome. The current situations, utilizations, countermeasures and a stratagem of water resources in Northwest China are described in this paper.

1 INTRODUCTION

The water shortage of Northwest China is undeniable. The poor condition of the land, the population explosion, poor sanitation, overuse of water, and environmental deterioration, and other conditions of the region sharpen the water shortage in Northwest China. In order to realize the sustainable development of the society and economy in Northwest China, it is essential to thoroughly solve the problem of water resources and the water environment at once. The climate is arid and rainfall is rare in Northwest China; the annual average rainfall is only 263 mm. This is the only place where the rainfall is lower than the water demand of the crops and the natural vegetation in China. The irrigation agriculture system with the typical desert and oasis characteristics has become the norm, and the view of "the green with water and the desert without water" is taken on.

2 SITUATION

The average volume of annual water resources in the basins of the Yellow River, the inland river, the Yangtze River and the Lancangjiang River is $2.3 \times 10^{11}\,\mathrm{m}^3$, which is only 6.5% of the total volume in China, and the distribution of water resources within the region is uneven. The total volume of water designated as being hard to use or not fully utilized is as high as $1.0 \times 10^{11}\,\mathrm{m}^3$, most of which is supplied by the upper reaches of the Yangtze River, the Langcangjiang River and Xinjiang Uygur Autonomous Region. Table 1 shows the water resources of Northwest China in 2001, where (1) Shanxi (2) Gansu (3) Qinghai (4) Ningxia (5) Xinjiang (6) Northwest China (7) China.

Characteristics of water resources in Northwest China are as follows:

1) The distribution of rainfall is not even across space. Water shortage occurs in the central belt of the developed economy and in the agricultural areas within the oasis. The exploitation of water

Table 1. Water resources of Northwest China in 2001.

Area name	Rainfall (mm)	Surface water ($10^8 m^3$)	Ground-water ($10^8 m^3$)	Total water resource ($10^8 m^3$)	Runoff coefficient	Water resource (m^3/person)	(m^3/mu*)
(1)	625.1	370.3	147.3	403.4	0.28	1121.8	523.1
(2)	261.7	195.2	125.8	206.9	0.18	821.4	274.5
(3)	283.3	615.7	264.9	621.6	0.30	12357.9	6023.3
(4)	214.0	9.2	30.9	11.2	0.07	208.2	58.9
(5)	166.6	914.3	662.6	978.7	0.29	5602.2	1637.2
(6)	263.0	2104.7	1231.5	2221.8	0.26	2495.6	919.6
(7)	704.0	32729.0	9400.0	34017.0	0.48	2231.8	1428.0

* 1 mu = 0.0667 ha.

Table 2. Water supply of Northwest China in 2000. ($\times 10^8 m^3$).

Area name	Surface water	Groundwater	Other	Total
(1)	42.9	34.99	0.77	78.66
(2)	93.93	28.80	0.36	123.09
(3)	23.43	4.45	0.00	27.88
(4)	80.79	6.40	0.60	87.79
(5)	425.51	54.20	0.28	479.99
(6)	666.56	128.84	2.01	797.41
(7)	4440.4	1069.2	21.14	5530.7

resources is very severe in local areas, especially when the retentive capacity of the water resources is very limited.

2) The seasonal distribution of water resources does not match water demand of the economic activities and the natural vegetation.
3) The transformation between surface water and groundwater is relatively severe. Because rainfall is rare in many areas, all groundwater is recharged from surface water. The natural water cycle is easily influenced by the climate change and human activities, which affect the natural ecological system.

3 UTILIZATION

The water supply and the available discharge of Northwest China are shown in Tables 2 and 3. Table 4 shows water consumption patterns in Northwest China in 2000, where is (1) Shanxi, (2) Gansu, (3) Qinghai, (4) Ningxia,(5) Xinjiang,(6) Northwest China, (7) China.

As shown in Tables 2, 3 and 4, the total water supply is $7.9741 \times 10^{10} m^3$, the total available discharge is $7.9644 \times 10^{10} m^3$ and the total water consumption is $5.0819 \times 10^{10} m^3$ in Northwest China. The water supply is slightly higher than the available discharge in Northwest China, so that the water supply and the available discharge are basically balanced. Surface water accounts for 83.6% of the water supply in Northwest China, while groundwater accounts for only 16.2%. Of the available discharge, agricultural irrigation accounts for 76.4% of this quantity.

If the discharge from forestry, livestock and fishery is added to the discharge from crop irrigation, the available discharge is as high as 89.0%. The rural population accounts for 70% of the population in Northwest China, but only 2.6% of the water use. The largest share of water used goes for irrigation; due to the arid climate and high water consumption by the crops, in addition to poor irrigation

Table 3. Available discharge of Northwest China in 2000. ($\times 10^8 m^3$).

Area name	Irrigated crops	Forestry, livestock, fishery	Industry	Urban domestic use	Rural domestic use	Total
(1)	50.61	5.19	12.66	5.19	5.01	78.66
(2)	91.05	6.37	17.71	3.43	4.17	122.73
(3)	20.01	1.22	3.81	1.30	1.53	27.87
(4)	71.63	9.12	4.78	1.09	0.61	87.23
(5)	374.88	78.33	10.90	6.45	9.39	479.95
(6)	608.18	100.23	49.86	17.46	20.71	796.44
(7)	3466.9	316.6	1139.1	283.9	291.0	5497.6

Table 4. Water consumption of Northwest China in 2000. ($\times 10^8 m^3$).

Area name	Irrigated crops	Forestry, livestock, fishery	Industry	Urban domestic use	Rural domestic use	Total	Rate of water consumption
(1)	36.40	3.71	6.29	2.07	5.01	53.48	68.0
(2)	55.27	4.22	8.35	1.66	4.17	73.67	60.0
(3)	13.89	0.82	0.39	0.27	1.53	16.90	60.6
(4)	32.43	3.66	1.18	0.28	0.61	38.56	44.2
(5)	260.25	54.84	4.16	1.62	4.71	325.58	67.8
(6)	398.64	67.25	20.37	5.90	16.03	508.19	63.8
(7)	2188.85	209.18	291.79	73.56	249.02	3012.40	54.8

techniques, the efficiency of water use is less than 40%. As a result of withdrawing a large amount of groundwater, the ground in this region has been sinking by a few inches per year. By contrast, the degree of development and utilization of surface water resources in Northwest China is relatively high in the inland river basin, and the efficiency of the surface water use is as high as 70%.

4 MAIN PROBLEMS IN WATER SUPPLY

The development of the society and economy is determined by geographical location, climate, and water supply. Water resources are also important for protecting the ecological environment. The residents of Northwest China have built water projects and diverted water for irrigation in the course of developing the economy and the society of Northwest China. It is well known that the degree of water conservation is closely related to the level of the regional development. Water conservation in Northwest China has developed considerably, but the natural water cycle has been affected and a series of problems have occurred due to exploitation of the water resource in order to develop the economy.

4.1 *Lack of infrastructure and water conservation*

In the water supply budget, water diversions accounts for 64% of the water used; water quantity and reliability are very low. Water resources distribution is uneven in time and space, and rational regulation is impossible. Shortages in water supply due to shortcomings in control and regulation of projects in part of the region, also severely affected economic development and the environment.

4.2 *The degree of exploitation in part of the region exceeds the regenerative capacity of water resources*

The distribution of water resources is uneven; water resources are wasted in some area, the environment is damaged because due to high population growth in some areas, and economic development and industrial activity places a stress on water resources in other areas. For example, the degree of development and utilization of water resources has exceeded the regenerative capacity leading to a competition for the available discharge from industries sharpening the contrast between the upper and lower reaches of the Shiyang River and the Heihe River in Gansu, the Tarim River in Xinjiang, the Guanzhong Plain in Shanxi, and other river systems. Thus the distribution of water resource severely affects the development of the society and the economy.

4.3 *The utilization coefficient and the production efficiency are very low*

Incomplete infrastructure, insufficient management, water wastage, and low consciousness for saving water has elevated the per capita water consumption rates and the available discharge in industry and agriculture over the mean level for China, and has caused salinization in the low reaches of the area.

4.4 *Water use severely impacts the environment*

The area of the artificial oases is growing with the increasing scope and scale of human activities. The availability of discharge in the natural environment decreases while the use of discharge for production and domestic use increases sharply. Therefore, reduction of the natural oases and lakes, water shortage in rivers, and desertifications are prevalent, and environmental deterioration is further sharpened.

4.5 *Soil and water loss is very severe*

The upper and middle reaches of the Yellow River cross the Loess Plateau, where very severe soil and water losses due to the natural conditions are aggravated by human activities. Levels of soil and water loss due to human activities have not been effectively kept limited.

4.6 *Water availability and poverty*

Now there are 9.5 million people living in areas of poverty or mountainous areas; the poor condition of the water works makes it hard for the people and the livestock in Northwest China to acquire adequate water. Water is a factor which potentially separates to the rich from the poor.

4.7 *Water management is very extensive*

Extensive management practices of the water infrastructure causes the inefficient distribution of water resources and the non-harmonious relationship between people and water.

5 COUNTERMEASURES

Water is the most important factor for living, economic development and ecological function, and in the case of Northwest China, the development of sustainable water resources is essential for the smooth implementation of western-style development in China and the realization of economic prosperity. It is necessary to translate understanding about water resources into practical action in order to realize the full exploitation and utilization of sustainable water resources. The following reform measures were determined to be key to this process: realizing unified management for water resources, managing water resources using market mechanisms, reforming the pricing mechanism for water, and modernizing the water works by introducing new technology. These ideas for the sustainable development of water resources were applied to the case of Northwest China.

5.1 *Ecological agricultural construction with saving water function*

Northwest China is rich in natural resources, including land, sunlight, and minerals, which are the key to solving the future food shortage in China. Of the entire land area in China, farmland and irrigated areas account for 65% and 59%, respectively. Water is available at 919 m^3 per mu. The amount of uncultivated arable land in Northwest China is 1.333×10^8 ha. The shortage of water resources is the bottleneck that restricts soil reclamation; traditional agriculture has been faced with the serious challenge. In the irrigated areas of Northwest China, the water resources infrastructure has aged and hasn't adequately maintained, reducing the portion of usable discharge. As a result, the groundwater level is increasing and the non-effective evaporation rate is relatively high, resulting in higher salt level in soil and unstable and lower crop yields. The key to transforming and modernizing traditional water conservation method is reforming the large-scale irrigation areas and developing agricultural practices that use less water while achieving high yields. Since 1999, reform of the nine large-scale irrigation areas in Guangzhong was made using loans from the World Bank. Ecological agriculture coordinates regional climatic conditions with well-suited crops; water-saving agricultural methods matching the regional water resource promotes development of the regional society and economy. The sustainable development of agriculture in Northwest China will reflect the sustainable development of water resources.

5.2 *Distribution and unified management of regional water resources*

The management of water resources in Northwest China illustrates the sharp contrast among the water resources, economic development and the ecological balance. In the upper reaches of the rivers, a large amount of water is diverted and stored by the water works, making local water cycling severe. On the other hand, the small amount of surface water from the upper reaches is diverted fully in the lower reaches, but it is not enough to sustain production and human populations. Therefore, rivers have zero-flow and lakes dry up or are drawn down during times of drought. For example, in the Tarim River of Xinjiang, of which the largest tributary is the Yerqiang River, the surface water is fully diverted into the irrigation areas or reservoirs. Now no water flows into the Tarim River. The main stream of the Tarim River flowed as far as 1321 km 50 years ago and ended at Luobupo Lake. By 1952, the flow of Tarim River ended at Taitema Lake because of a dam built at the mouth of the Layin River. The Daxihaizi reservoir was built in 1972, and became the end of the Tarim River. The flow of the Tarim River has been zero for the 266-km lower reaches. Only during a flood period 2 years since, there is a small amount of water flowing into Daxihaizi reservoir. The rational distribution and unified management are evaluated for water resources of Northwest China. The area of Northwest China is very large, but the distribution of water resources depends on seasonal variation. For the rational distribution of water resources, there are three basic tasks: to control water demand, to supply water effectively, and to protect the water quality. Historically, the construction of water resources followed the form of "five weighty and five light". They are favor construction over management, large-scale projects over small-scale ones, and backbone projects over conveyance system. Evaluation of the newly-built projects, distribution systems, management, and protection of water resources should be based on the projects and on the practical benefits. Under the four-no-management style, responsibility for water fields is not responsible for water supply, the responsibility for water supply is not responsible for water drainage, the responsibility for water drainage is not responsible for sewage treatment and the responsibility for sewage treatment is not responsible for reclaimed water when the unified management of water resources is carried out. The distribution of water resources is strengthened and both measures of engineering and non-engineering are the largest benefit of economy, society and environment from the macroscopic height with the sustainable development. For the sustainable development and utilization of water resources, it is necessary to consider the feasibility of the individual project and its compatibility with the entire basin. It is also necessary to consider the size and standard of the individual project, the balance of individual projects in the context of the water resources system, and the demands of social development. Only the coordination of the water distribution system and the science of construction and management of water resources can ensure that water resources projects better serve the development of the society and economy. Nowadays, the Chinese government has implemented unified management and the planning of the available discharge as a water resource within some basins, irrespective of the administrative management boundaries. The management bureau of water resources has been founded, and it is urgently needed to reform the administrative measures regarding water resource management. The following accomplishments have been made on the Tarim, Heihe, and the Yellow rivers: the river sections were extended for the Tarim and Heihe rivers, the flow in the Tarim River now stands at $3 \times 10^8 m^3$ from the Daxihaizi Reservoir to the lower reaches, and even under non-high water conditions, the flow of the Yellow River, does not reach zero-flow.

5.3 *Ecological engineering*

The speed of the environmental deterioration is increasing because of water shortage and human activities in Northwest China. Northwest China consists of various land formations: plateau, mountainous gully, wind-sand hilly areas, desert, oasis, the Gobi Desert, and areas of uncultivated land. Therefore, contrasts among populations, water resources, and environment are sharp and the treatment and management of water resources is most difficult. "The mountain and river beautiful project" is a main impetus for western development in China, and serves as the focal point for realizing the goal of sustainable utilization of water resources in this region. The main objective is to return the sloped reclaimed farmland, which is not suitable for planting crops to forest (or grassland) while considering the local conditions of the Loess Plateau. According to field investigations, land with a steep slope (an angle of more than 25º) accounts for 77.6% of the total farmland on the Loess Plateau. The yield of crops decreases with increasing slope: based data from an experimental area in Dingxi, the yield per $667 m^2$ (y, $kg/667 m^2$) of wheat was negatively related to the slope of the farmland (x) by the regression equation $y = 242.59 - 3.33x$ ($r^2 = -0.9820$). Soil and water losses are very severe and the ecological environment is deteriorating due to the large amount of the reclaimed farmland. In this area of Northwest China, the key to improving the planting conditions is to improve the ecological environment of agriculture, to raise the crop yield, to make the mountains and rivers beautiful, to return the reclaimed farmland to forest (or grassland), and to conserve water through the development of forest and grassland. Carrying out "the mountain and river beautiful project" will focus the ecological environmental develop in directions compatible with human existence and development. At the same time, it can improve the runoff-production condition, increase the groundwater level in the region and relax the contradiction of drinking water and water for daily use in rural areas. For example, the Huijiagou, in Tongchuan, Shanxi province, is a mountainous area located on the Weibei plateau typified by "three ridges, four hills and one gully". By their own initiative, the local people returned the reclaimed farmland by planting trees and grasses in 1993. The degree of cover of the vegetation in the gully and on the ridges increased gradually. By the end of 1995, water began flowing in the gully. A soil dam was built on the gully to store water. Aside from supporting agricultural production, the drinking water issue has also been solved. Similarly, "the mountain and river beautiful project" can be used to accomplish control over soil and water loss and to increase the available of water resources in some areas of Northwest China.

5.4 *Measures for developing new water fields and economizing on expense simultaneously*

Northwest China has a shortage of water resources. Exploitation of available surface water in some areas of Northwest China has been exceeding the local water resource capacity. However, in order to meet the water resource needs of development of society and economy in Northwest China, the demand shows for only a gradually increasing trend. Therefore, the smooth implementation of western development in China must require the availability of water resources. To realize the sustainable utilization of water resources, it is necessary to simultaneously develop new water resources and economize on expense, to regulate the surface water and groundwater, to combine the inter-basin water transfer and the water generation within local regions, and to suitably use the cloud and water resources in the atmosphere. Currently, the utilization of groundwater only accounts for 13.4% of the total in Northwest China. It has potential for exploitation, especially in areas using large-scale irrigation. The exploitation of groundwater can not only increase the water fields, but also reduce soil salinity and improve growing conditions for crops, and allow potential to increase crop yields. There are several projects for inter-basin water transfer within Northwest China that have released the region from water shortages. The largest project for inter-basin water transfer is a western line project, the south to north water transfer project, is being planned and designed. After the project is completed, it will increase the water supply for Northwest China by $1.3 \times 10^{10}\,m^3$. In addition, the upper reaches of the Yellow River in Qinghai province of China are rich in cloud and water resources, and artificial rainfall is one of many measures that can solve the zero flow problem of the Yellow River; according to the People's Daily, $6 \times 10^9\,m^3$ of artificial rainfall was produced in the upper reaches of the Yellow River during the five years from 1997 to 2001, and $1.36 \times 10^9\,m^3$ of water was introduced to the Yellow River. The rate of the cost to benefit was 1:19. Artificial rainfall was also produced in spring 1999, in Hotan, Xinjiang; the volume equaled the storage capacity of 4 Wuluwati Reservoirs and water shortage was relieved because of the spring arid. Therefore, it is possible to build the development base of the cloud and water resources in Northwest China, and other management measures can also increase the available water resources for Northwest China. At the same time, it is necessary to improve the conscience of the people for protecting and saving water using scientific knowledge, technology, and laws while improving population qualities to maximize the economic benefits derived from the limited water resources.

5.5 *Establishment of the price of water through the market mechanism*

The price of water resources is very low, making it hard to use price in a regulatory capacity. In addition wastage of water resources is very severe. Based on calculations of operating costs, water fees accounts for only 0.1–0.3% of the cost of industries and for 0.23% of consumer expenditures. The mean price of water used for agriculture in China is only 50–60% of the actual cost. The low water prices make it difficult to effectively restrain the overall number of units and individual water use to produce an efficient mechanism for managing the available discharge and reducing the amount of water waste. Some enterprises use tap water to dilute polluted water in order to satisfy sewage drainage standards. Studies from the United States of America show that when water price is very low, water demand per 1 kWh produced by a power plant is 50 gallons, but when water price increased to 5 cents per 1000 gallons, the water demand decreased to 2.8 gallons. In view of this, the establishment of a market mechanism for water can effectively reduce the gap between water supply and demand. This may represent a heavy burden for the local people, but water price within the market mechanism is an effective tool for developing a consciousness about water use, and at the same time, ensuring sustainable development of water resources.

6 SUGGESTIONS

The Water shortage of Northwest China is an undoubted fact. Severe waste and pollution, soil sanility, and overexploitation and utilization of water resources have been causing environmental deterioration further sharpening the problems of the water shortages. In order to realize sustainable development in society and the economy in the northwest region, it is essential to thoroughly solve the existing water problems now, to pay attention to the environment and to take measures for developing new water fields while economizing on expense. It is also important to distribute water resources equitably, to carry out unified management of water resources within basins, to use the water price as a management tool for water distribution, to improve the conscience of water saving, to establish a water-saving society, and to ensure ecological water use provides a beautiful water environment.

REFERENCES

Keshan, P. 2000. Analysis again on the basic thinking on returning the reclaimed farmland to forest (or grassland) in the loess plateau region. *Water resources planning and design*: 29–31

State Statistic Bureau. 2001. *2000 yearbook*. Chinese Statistic Press

Wenlai, J. 1999. *Water resources values. Beijing*: Science Press: 17–24

Xin, L. 2001. The hydrological effects under human activities in the inland watersheds of Xinjiang, China. *New problems, new techniques and new methods in hydrological search for 21st century in China*. Beijing: Science Press: 96–103

Demand side water management in Los Angeles and San Diego: in search of sustainable water supply

C. Pezon
Conservatoire National des Arts et Métiers, Paris, France

ABSTRACT: Los Angeles and San Diego have always been challenged by water resource scarcity. They have dealt with this problem by adding imported water to their local resources. They purchase imported water from the Metropolitan Water District of Southern California, which owns and operates the Colorado River Aquaduct and is entitled to control half the water the State of California routes from the north to the south through the California aquaduct. Since the worst drought that ever occured in California (1987–1992), Los Angeles and San Diego Water Departments have worked on reducing water demand. Demand side management has become a strategic dimension of their water supplies.

1 INTRODUCTION

From 1987 to 1992, California faced the worst drought of its history. The usual 5 months of rainfall that supply 35 million person water demand had been insufficient for 5 consecutive years. The result of which ended up in water restrictions in direct proportion to each major city's dependance on imported water.

California rainfalls are both geographically and timely inequal. They occur north of the Sacramento River where semi-desertic southern California holds a third of the water consumed by its 17 millions inhabitants. Only 10% of the water used by the cities of Los Angeles and San Diego has local origin. The remainder comes through long distance conveyed water, or the 'California way' of supplying water. Since the beginning of the twentieth century, the distance and the volume of routed water have deeply increased, making it possible to sustain the fastest demographic growth in the USA.

Los Angeles and San Diego have developed different water supply strategies in respect to their demographic profiles.

Los Angeles built up the first aquaduct in California in 1913 (385 km, 200 millions m^3 capacity) for its own usage. Then the Colorado River Aquaduct (390 km, 1.5 billion m^3 capacity), built and run by Metropolitan (the Metropolitan Water District of Southern California) since 1941, introduced regional water systems to Southern California. Metropolitan knew its system would become insufficient as early as the late fifties. Water transfers were planned on a huge scale by the State of California to meet ultimate statewide needs. The Northern abundant and unused resources would have to offset current and coming water deficits, in particular in Southern California. In 1961, the DWR (Department of Water Resources) committed to sell 50% of the 5 billions m^3 of water it would convey every year through the State Water Project, down to Metropolitan territory, 1000 km away, from 1971. A year earlier, Los Angeles completed its second aquaduct, as big as the first one, in order to limit up to 5% its dependancy on Metropolitan imported water.

Conversely, 95% of San Diego water demand depends on the water wholesaled by the city from Metropolitan through SDCWA (the San Diego County Water Authority). In 1940, San Diego's population was equal to the population of Los Angeles in 1910. During the 1940s though, due to its strategic position during the Second World War, San Diego's population increased by two-third, leading

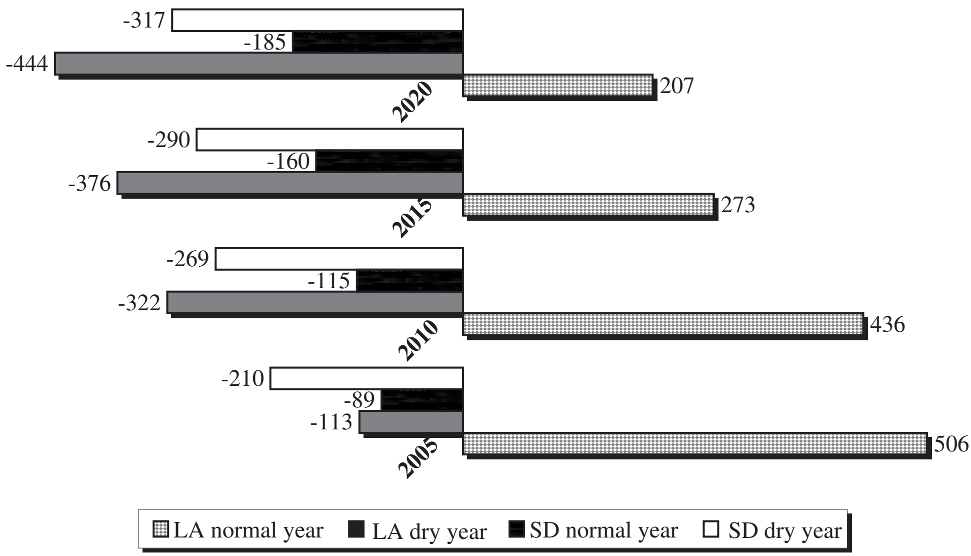

Figure 1. Los Angeles & San Diego water deficits from 2005 to 2020 (millions m^3).

to the need for imported water. San Diego became a Metropolitan client in 1946. Along with SDCWA, it has become Metropolitan's largest purchaser.

The 1987–1992 drought has led to a major change regarding the way Los Angeles and San Diego among other cities would have to consider future water supply. Increased water transfers to comply with never-ended increased demand are no longer feasible. This one-century solution was designed to meet agricultural, industrial and urban needs but did not take into account the environmental damages endured by export areas. Today, environmental protection comes first and transfered water users must deal with it by reducing their dependancy on imported resources.

New limits were set up between 1997 and 2001. They help to figure out the water deficits that Los Angeles and San Diego would incure by 2020 if they do not work on reducing water demands and on developing new water supply (Figure 1).

The 1987–1992 drought has also found expression in increased cost of transfered water. With given fixed costs, a reduced volume of transported resources rises the water unit cost. During the drought, when Los Angeles Aquaducts capacity was as reduced as it will be in the next 20 years, the unit cost rised from $0.08/m^3 to $0.4/m^3. In the same time, the water routed by the DWR was as limited as the level that has been set up until 2020 and its cost doubled, from $0.12/m^3 to $0.24/m^3. Imported water has lost its position as cheapest water supply. Los Angeles and San Diego started to redesign water supply strategies according to the cost profiles of alternative resources and their reliability in case of drought.

Last, the 1987–1992 drought has brought to light a major conflict between Los Angeles and SDCWA, regarding the way Metropolitan water is allocated amongst its 26 members. Los Angeles, as the founder member, supports the in force allocation regime, based on each member's historical contribution to Metropolitan tax income, which provides Los Angeles with 23% of Metropolitan water. SDCWA, as the first Metropolitan customer, seeks to bring into operation an allocation regime based on each member's historical contribution to Metropolitan tax and watersales incomes. This new regime would double the SDCWA entitled volume, and hence San Diego imported water supply, but reduce by two third the volume Los Angeles gets with the preferential rights regime.

Both Los Angeles and San Diego work simultanously in three directions in order to meet increasing population water demand: water demand reduction, reallocation of the water wholesaled by Metropolitan and development of cost-effective water.

If Southern California water demand was as low as in Europe (180 l/c/d), Metropolitan would be able to supply enough water until 2020, even during dry years, and regardless of the local water available within its 26 members' territories.

Los Angeles and San Diego water demands, respectively 580 l/c/d and 640 l/c/d, raised by 6% and 8% in dry years, can be partly explained by outdoor uses which make these cities almost look like tropical places despite their semi-desertic climates. In Southern California, the water consumption levels are a paradoxal consequence of the climate that anywhere else would conversely lead to a careful use of the water resource. The indoor consumption is as high as US average water demand and can grow up over 1 m^3/c/d in single-unit because of outdoor use.

In other words, the forecasted water shortage in Southern California is firstly the result of high water demand even though the long-distance water drop hastens it.

The demand-side management aims at reducing water consumption per capita in order to release enough resources to meet demand induced from the demographical growth. Urban water services provide their customers financial incentives to set up conservative domestic equipment such as ultra-low-flush toilets and high-efficiency washing machines. They also increase water rates to promote conservative behaviours.

These policies have already given strong results, as shown in table 1.

In Los Angeles, water demand in 2000 was similar to that of 1985, despite a population growth of 600,000 people. This spectacular result comes from the very voluntarist politics led by the LADWP (Los Angeles Department of Water and Power) which invested $100 millions over 10 years, of which 75% was spent on the fittings of 950,000 ultra-flosh toilets, responsible for 90% of the water conserved in this category. But the consumption decrease (60 l/c/d) has been mainly attributable to the progressive rate structure applied since 1995.

Its main characteristics are:

- No base fee.
- A low rate ($ 0.3999/m3) is applied to basic uses. For superior demand higher rates depend on the season (winter, summer), the kind of house (multi or single unit), and the zone of residence (3 different micro climates).
- Basic uses are quantified according to the house surface (the bigger the higher, on a scale from 1 to 3), and to the average temperature of the zone one lives (20% difference). Basic uses are also bigger in summer.
- In single unit, incremental demand is charged double during winter (November 1st to May 1st). In summertime, incremental demand is charged 2,5 times more ($1.0524/$m^3$) to all kind of units.

The very progressive rate structure of Los Angeles translates a discontinuous operating costs structure, according to the type of water supplied in 1995. In raising order, the Los Angeles Aquaducts water comes first, which costs $0.08/$m^3$ when operated at full. Next are local resources, $0.12/$m^3$ that meet from 5 to 10% of demand. The remainder is supplied through wholesales from Metropolitan, $0.28/$m^3$. The progressive rate structure aims at limiting wholesales water: a 10% variation in demand makes the full supply cost of Los Angeles three times bigger and increases its operating costs by 15%.

In San Diego, the water supply incremental and average costs are the same as long as 95% is wholesaled from Metropolitan through SDCWA, which adds to Metropolitan rate ($0.28/$m^3$) its

Table 1. Annualy conserved water in 2000 (millions m^3).

	Los Angeles	San Diego	Metropolitan
Conserved water	161	31.2	817
Conserved water due to new domestic equipment	52.4	21.4	557.7
Conserved water due to rate increase	108.6	9.8	259.3

transportation cost, \$0.07/m^3. Therefore a progressive rate structure aims first at easing the pressure on SDCWA whose wholesales from Metropolitan exceed far more the volume of water secured by its preferential rights. But as far as the water supply cost is continuous and represents half the operating costs of the San Diego Department of Water, a very progressive rate structure with a base rate below the average supply cost may quickly turn out in an operating deficit. Indeed, a 10% decrease in demand leads only to a 5% decrease in the operating costs.

Therefore, the rate structure voted in San Diego in 1999 is not as an incentive conservative as in Los Angeles. Restricted to single units, the San Diego Department of Water rate structure has a base fee and distinguishes three different rates: the first 20 m^3 are 10% more expensive than in Los Angeles but the highest rate is 40% below the highest LADWP rate. Since July 1st 2002, the San Diego Department of Water rate structure has been made less progressive: the base fee has increased by 11%, up to \$10.68 monthly, and each rate by 4%. A big consumer bill (over 1 m^3/day) is 5% more expensive whereas a small consumer bill (below 9 m^3/month) has raised by 8%.

By increasing its fixed income, the San Diego Department of Water is protecting itself against the impact of decreasing demand on its budget balance which was in deficit in 1999. By doing so, the San Diego Department of Water has decided to manage without the conserved water that is anyway considered not to be sufficient to offset the forecasted water supply deficit.

For the next 20 years, the goals assigned by the LADWP and the San Diego Department of Water regarding conservation are equivalent to those achieved in the last 10 years (Table 2).

The conserved water will make it possible to supply up to 30% of the Los Angeles and San Diego demographic growths. None of the two urban water services took into account in 2000 the impact of the water rate elasticity of demand. Though, the demand level targeted gives room to further conservation. At the level of its territory, Metropolitan forecasts that 36% of the 441 millions m^3 conserved from 2000 to 2020 would be due to rate increases. San Diego has already raised its rates and set up equivalent annual raises until 2007. Conversely Los Angeles has not yet adjusted its rate structure to the supply cost structure that comes with its new water supply composition, mostly made up with the most expensive water wholesaled from Metropolitan. Nevertheless, the LADWP water supply average cost is from now higher than its former incremental cost (Figure 2).

Both wholesaled water from Metropolitan and new supply water that the two cities are about to develop will be more expensive. This means that water rates are going to increase soon since both

Table 2. Annual water conservation goals by 2020.

	Los Angeles	San Diego
Conservation due to new domestic equipment	55.4 millions m^3	19.7 millions m^3
Equivalent in number of people supplied	330,000	90,500
Water demand in 2020	560 l/c/d	610 l/c/d

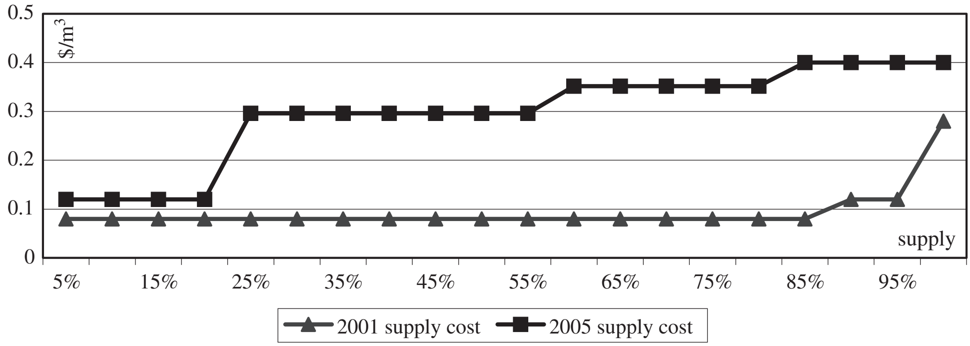

Figure 2 LADWP supply cost structure in 2001 and in 2005.

urban water departments pass on the cost of water supply to the drinking water rate. This also means that the volume of conserved water is likely to be more important than forecasted.

One can consider that Los Angeles and San Diego water rates are so low that both water departments may increase their rates to push their consumers to moderate their consumption. One may even think that the low water rates levels bring about the high demand level.

But beyond rate levels, it is interesting to notice that Los Angeles and San Diego water bills are higher than same sized French services bills. A San Diego consumer already pays \$384/month, an average bill that exceeds the affordability index of \$328/year working out by the US Environmental Protection Agency for a Californian family. Assuming that water consumers are more sensitive to bill level than to rate level, the main questions are: are they able to behave in a much more conservative way as quickly as required by the water supply restrictions, in order to keep their bills to the same level? If not, how would they react to big rate increases?

These questions deal with the social and equity side of sustainable developpment. Even though conservation is the cheapest way to increase water supply (\$0.12/m^3), it requires to raise rates to be efficient.

2 REALLOCATION OF METROPOLITAN RESOURCES

Metropolitan rate policy substantially changed in 2002. Forecasted supply drop is going to double its water supply cost (\$0.32/m^3 in 2010 versus \$0.18/m^3 in 1995), since Metropolitan has to absorb the same fixed costs with a volume of resource 40% lower than expected.

In order to balance a budget whose 90% expenses is fixed and 80% income is variable, Metropolitan has to increase its rates, thus to reduce the competitive advantage of its water. To get sufficient incomes, Metropolitan has set up new rates since January 1st 2003 (Table 3). Its members may take advantage of a lower rate (rate 1) applied to 90% of their highest historical purchases if they commit to buy at least 60% of their highest historical volumes until 2018. A higher rate (rate 2) is charged to their additionnal purchases. The members who do not commit for 15 years would see their purchases charged on rate 2 basis, as soon as their purchases exceed 60% of their historical highest volumes. However, Metropolitan stopped subsidizing local resources development, in order to provide its members with a price signal, according to which each of them has to set up a competitive water supply strategy.

Metropolitan's members commitment to buy 60% of their highest historical volumes is equivalent to Metropolitan commitment to supply those volumes even during dry years. In this case, Los Angeles commitment would be worth a volume, 305 millions m^3, pretty close to the volume secured by its preferential rights. SDCWA commitment would be worth 495 millions m^3, much more than the 220 millions m^3 secured by the preferential rights regime. The new water allocation regime appears like the most clever way Metropolitan has choosen to solve the conflict between Los Angeles and San Diego regarding the former regime. In case of drought, Los Angeles would get the same volume of water in either regime and SDCWA a volume as high as the volume that would come out of the reform it has been claiming for. SDCWA understood it very well: it had never wholesaled so much water from Metropolitan as in 2002, in order to boost its historical volume of reference.

The new Metropolitan rate policy outlines the increasing water supply cost that its members will have to face. Under the best conditions, Los Angeles and San Diego will pay in 2007 their purchases 10% more than in 2002.

Table 3. Metropolitan rate structure (\$/m^3).

	1995–2002	2003	2004	2005	2006	2007	
Raw water	0.2829	0.2643	0.2821	0.2959	0.2975	0.3016	Rate 1
		0.33	0.3381	0.3518	0.3535	0.3575	Rate 2
Treated water	0.3494	0.3308	0.3567	0.3624	0.3964	0.4232	Rate 1
		0.3964	0.4126	0.4426	0.4426	0.4791	Rate 2

Last, this progressive and incentive rate policy shows that Metropolitan's mission to offset Southern California water deficit is over. It is now in charge of providing its members with basic supplies and lets them be responsible for building up their remaining supplies. In case of drought, neither Los Angeles nor San Diego would be able to meet water demand with Metropolitan water. Consequently, both have to find out cost effective additionnal supplies whose levels will help them to figure out their commitment to Metropolitan.

Like many urban water services in California, Los Angeles and San Diego have seen the water market as a promising way to be provided with the additional resources that Metropolitan is no longer able to supply.

The water market was born in 1991 when the State of California created the Drought Water Bank in order to moderate the restrictions that cities were putting up with the drought. The Drought Water Bank is in charge of organizing transfers between the DWR partners. Agricultural districts sell part of their annual entitlements to urban services through an auction based mechanism sponsored by the Drought Water Bank. Water sellers get the income relative to the volume they give up and water purchasers pay in addition to this price the incremental transportation cost that the DWR charge them.

These one year transfers make up the spot water market. Longer term contracts are so far suspended from water rights modifications.

A first step was completed in 1995. The rights were equalized amongst the DWR partners. The former regime gave a priority to urban services, in case of drought, at the expense of agricultural districts. The 1995 Monterey Agreements stated that available resources would be allocated in direct proportions to contractual volumes, regardless of their use. Potential transfers would thus become predictable. Urban services gave up their priority as a compensation for a lower global agricultural entitlement. This means that the water right of use hold by the State of California since 1959 has to be modified to comply with the new volume that each partner would be entitled to buy.

A water right of use is being modified as it is being created, according to the terms of the EIR (environmental impact report), conducted by the SWRCB (State Water Resources Control Board). The concerned use must be useful and reasonable. Urban water supply has always been considered as both useful and reasonable. The concerned use must also not reduce prior water uses which used to be agricultural districts, urban services and industrial companies.

Since the 1970 California Environmental Quality Act, the environmental use has become as useful and reasonable as traditional uses. No permit can be delivered should the concerned water be likely to spoil the environment. In such case, the potential water user is asked to pay for the environmental damages his use may produce, in order to let the environment as it was before. These considerations have raised the cost of water permit and the time required to get an answer from the SWRCB up to 10 years. Actually any environmental protection association is able to represent environment interets to the SWRCB. As many associations are opposed to the DWR water transfers, many have challenged the EIR on which the Monterey Agreements enforcement depends. They call the attention of the SWRCB on to unoticed injuries in order to raise the negative impacts up to a point where water transfers would no longer be cost effective.

Water transfers from the Colorado River are symptomatic of the raising water permit cost. In 1998, SDCWA and IID (Imperial Irrigation District) came to a 45 to 75 year agreement to exchange 247 millions m^3 per year. Last December, the temporary EIR results stated that this exchange would dangerously increase the salt content of the Salton Sea which harbors endangered species. The injuries would need $1 billion work to be corrected. SDCWA and IID have already reconsidered their exchange down to 98,7 millions m^3 for 5 years.

The worst is that once settled, long term transfers are neither pricely nor timely secured. Since the Public Trust doctrine has been qualified as being relevant to fix the water right litigations in 1983, not only requested water permits but also the in force ones are subjected to environmental rules. According to the environment regulation the Federal State keeps on developping, water permits may be either reduce or cancel to comply with new regulations, or suspended from expansive repair works.

Los Angeles has been the first urban service to deal with such a court decision. The city was condemned to half reduce its aquaduct water supply and has to fund dozens of millions of dollars to repair the injuries.

Table 4. Water market rates ($/m^3).

	Water rates	Transportation cost to Metropolitan	Water market rate for LA	Transportation cost to SD	Water market rate for SD
1991	0.1013	0.04	0.1043	0.0446	0.1859
2001	0.18	0.04	0.22	0.073	0.2924
2003	0.2027*	0.073	0.2757**	0.073	0.3487

* SDCWA-IID rate agreement without the EIR impact.
** virtual rate that LA would pay according to SDCWA-IID rate agreement and the transportation cost charged by Metropolitan.

Table 5. Recycled water goals by 2020 (millions m^3/year).

	Los Angeles	San Diego	Metropolitan
Recycled water in 2000	51.5	4	333.2
Additional recycled water by 2020	74.75	20.3	222.1
Equivalent in number of people supplied	368,700	92,000	838,000

The increasingly competing water usages have so far led to substantial price raises on the water market (Table 4).

Between 1991 and 2001, the water marketed by the DWR had increased up to 80%: it costs more before being routed than it would cost to Metropolitan if it gets its contractal amount ($0.12/m^3) from the DWR. The content is worth more than the technical system of transportation. In other words, the market water that flows to Los Angeles and San Diego and which is only charged with the incremental transportation cost is as costly as the water wholesaled by Metropolitan from the DWR and which is charged at full transportation cost.

Recent court decisions have brought into light the cost that urban services would have to face for their uses to be environmentally respectful. The way this cost is directly charged to the involved users fits into a dynamic that makes sense to sustainable development. Since urban water services have to deal with high demographic growth, and increase their water supplies in a cost-effective way, this dynamic leads them to think their development under environmental protection constraints.

3 THE DEVELOPMENT OF COST-EFFECTIVE WATER

The alternative options are recycled water and desalination ocean water.

Recycled water has been experimented for many years in Southern California. In 2000, Los Angeles and San Diego planned to speed up this type of local water supply (Table 5).

These levels of production would allow Los Angeles and San Diego to lower their deficits in case of drought to respectively 313 and 277 millions m^3 by 2020.

Recycled water is not used for drinking purpose. But the resources that used to be consumed for irrigation and some industrial purposes become available for others consumers. Thus, recycled water provides both cities with reliable additionnal resources since recycling is not dependant on climate conditions.

On the other hand, Los Angeles and San Diego will not keep on recycling water under the financial conditions they expected in 2000. From 1995 to 2002 Metropolitan refunded its members the difference between recycling cost and its treated water rate ($0.35/m^3) capped to $0.2027/m^3. Since January 1st 2003, local water services must face the full cost of this kind of supply that ranks from $0.259 and $0.678/m^3.

At the same time, desalination technique has become more cost-effective water supply. In 2000, desalination was still too expensive (from $1.0539 to $1.7835/m^3). But since the Tampa Bay water department has set up a desalination plant and is provided with drinking water that costs $0.608/m^3, it has become clear that desalination would interest Southern Californian urban services. This technique combines many advantages:

- This process increases the amount of available resources, conversely to the water market that only aims at reallocating resources, and raises for this reason political issue regarding agricultural activity. California produces 60% of the US fruits and vegetables and finds it difficult to let its producers give up their activity in order to rely on the watersales incomes.
- Desalination produces drinking water, conversely to recycled water. More precisely, recycled water may be drinkable, but is still unfavourably considered by the population. Los Angeles was about to start distributing such a water to its consumers but had to abandon this project during the municipal elections, under a media denunciation campaign whose slogan was 'From the toilet to the tap!'.
- Desalination is providing a water supply security that is lacking to long distance routed water. Desalinated water does not depend on neither climate hazards nor surface water regulations.
- Desalination process is as competitive as recycling.
- Desalination is becoming as competitive as water marketed regarding the increasing cost of the water resource and its treatment. In 1985, water treatment was worth 12.5% of the treated water that Metropolitan wholesaled. In 2001, it was worth 20% and had to increase up to 22% by 2010. In fact, this increase would be achieved as soon as 2007. The treated water will be worth $0.42, and $0.5/m^3 once routed to San Diego.

In November 2002, SDCWA laid the foundations of an agreement with the Poseidon Resources Corporation to build and operate the biggest desalination plant of the western world. This 69 millions m^3 capacity plant would be able to meet 112,000 people demand, at $0.6437/m^3 full cost. Though this plant will not offset the country water deficit, it augurs well for this process as a solution.

The SDCWA desalination strategic choice could also lead to significant change regarding the Authority's activities. The first desalination cost driver lies in energy consumption. In October 2001, our interlocutors from SDCWA were studying closely to enter the electricity market in order to lower the desalination cost and to put San Diego free from electricity deregulation negative impact.

4 SAN DIEGO AND LOS ANGELES WATER SUPPLY STRATEGIC CHOICES

In San Diego, conservation and recycling will release 40 millions m^3 by 2020, enough to meet demand of 180000 people or 45% of the forecated demographic growth. The SDCWA initiatives (Metropolitan water allocation regime, IID agreement on the water market, and Poseidon agreement) are likely to half reduce the forthcoming San Diego deficit by 2010 in case of drought, and lower it up to one third under normal pluviometric conditions. In other words, one out of three San Diego inhabitants would be out of water in 2010 and at least one out of 15, even though SDCWA is successful in all its enterprises.

In economical terms, the San Diego water supply cost will not be as continuous as it has been ($0.35/m^3). From 2005, it will range from $0.3487/m^3 (35 millions m^3 – IID agreement) up to $0.6437/m^3 (34 millions m^3 – Poseidon agreement), going through Metropolitan basic rate, $0.3689 (742 millions m^3), Metropolitan higher rate, $0.4248 (55 millions m^3), and recycling (24 millions m^3 at $0.4685 in average). This new cost structure allows the San Diego Department of Water to set up a more progressive rate structure that may increase conservation.

Los Angeles is only at risk in cases of drought. Besides, the LADWP is ready to come onto the water market and to undertake desalination: it is connected to both the Colorado River and the California Aquaducts and produces enough electricty to operate a desalination plant. For the moment,

Table 6. Los Angeles rates in case of drought.

Water deficit	10%	15%	20%	25%
Rate $/m^3	1.3066	1.5680	1.8293	2.1365

Los Angeles has made the bet to develop the cheapest water that comes out of conservation and expects huge rate increases in case of drought to offset its deficit.

A 4.5 l lower consumption per capita per day would be enough to offset the expected 2005 deficit and a 142 l lower consumption per capita per day to offset the expected 2020 deficit. The highest rate that would be applied in such a case, \$2.1365/m^3, may be big enough to reduce the average consumption down to 420 l/c/d.

5 CONCLUSION

Since the 1987–1992 drought, the water uses competition has intensified and enhanced the value of a scarcer resource. Urban water services internalise this value in order to figure out most cost-effective water supply to develop. The economic value of the water resource is passed on to consumers through progressive rate structures related to the incremental water supply cost.

The economic value of water as a water management tool has been efficient in California. The rate adjustments that have resulted from the increasing water supply costs have led to major conservation. While the former demand levels left room for substantial conservation, it is important to note that the economic value is relevant:

In a region where inhabitants can afford a resource whose protection depends on increasingly sophisticated technics. The spreading of market mechanisms all along the drinking water supply chain leads to full cost local rates that may be socially and politically unsustainable;

For local services which already pass on their full cost to their consumers. In Los Angeles and San Diego, water rates adjustments have not been as severe as they would have been if both services had to simultanously pass on water resource, transportation, treatment and distribution costs to their consumers. This is probably one of the reason why the economic value of water resource only concerns drinking use. The farmers that consume 80% of the Californian water have never been charged the full cost of the water they use. The aquaducts that were built up for agricultural purpose (Central Valley Project, All-American Canal and Coachella Canal) were funded by the Federal State. Agricultural districts just have to deal with the maintenance costs. The water lost by the agricultural aquaducts reveals the wastings that a resource considered as inexhaustible as almost free may be subjected to. The first IID-SDCWA agreement in 1998 was about the annual transfer of 247 millions m^3 that could be conserved from the All-American Canal repairs. This volume would have met demand of 1 million people in Southern Californian.

REFERENCES

City of Los Angeles, Department of Water and Power, 2000, *Urban Water Management Plan*, LA

City of San Diego, Water Department, July 1997, *The Strategic Plan for Water Supply 1997–2015*, SD

City of San Diego, Water Department, 2000, *The 2000 Urban Water Management Plan,* SA

Department of Water Resources, Octobre 1995, *Implementation of the Monterey Agreement,* Sacramento

Department of Water Resources, 1998, *The California Water Plan Update, Bulletin 160-98*, Sacramento

Department of Water Resources, mars 2001, Management of the California State Water Project, Bulletin 132.99, Sacramento

Metropolitan Water District of Southern California, December 2000, The Regional Urban Water Management Plan for the Metropolitan Water District of Southern California, LA

MWDSC, February 11, 2002, *Report on Metropolitan's water supplies*, LA

MWDSC, December 2002, *Annual budget 2002–2003*, *executive summary,* LA

Pezon C, mars 2002, La gestion de l'eau en Californie entre marché et développement durable – le Metropolitan District of Southern California, rapport pour l'OTAN, Paris, CNAM

San Diego County Water Authority, 2000, *The 2000 Urban Water Management Plan*, SD

State Water Resources Control Board, Division of Water Rights, 1999, *A Guide to Water Transfers*, California Environmental Protection Agency

State Water Resources Control Board, Division of Water Rights, 2001, *A Guide to California Water Right Appropriations,* California Environmental Protection Agency

Urban water demand management – The city of Thessaloniki-Greece case study

Y. Mylopoulos, E. Kolokytha, A. Mentes & D. Vagiona
Aristotle University of Thessaloniki, Faculty of engineering, Department of civil engineering, Division of Hydraulics & Environmental Engineering

ABSTRACT: This paper presents the main results of a research project performed by the Division of Hydraulics and Environmental Engineering, AUTH in cooperation with the Water Utility of the city of Thessaloniki, Greece. It deals with the examination of urban demand management aspects in the city of Thessaloniki. Residential and industrial water uses are examined and analyzed. Variables that influence and modulate water demand, water demand function and price elasticity for residential water demand, water recycling options for the industrial sector, the possibility of construction of dual supply system and the evaluation of different pricing policies are presented. Finally, public preferences concerning pricing policies, information and education are being incorporated in the formulation of a demand oriented water policy.

1 INTRODUCTION

Rapid population growth, sprawling urbanization and the changing life styles to more water intensive ones have led to a constant increase in water demand.

Satisfying water demand has been traditionally viewed as a technical and managerial problem of increasing supply to meet demand. However, supply oriented policies have severe environmental and economic impacts and had led to water resources contamination and depletion. Thus, a re-examination of current approaches to water resources planning and management is necessary. Urban management strategies should incorporate the notion of limited water resources, the need for conservation, innovation and renovation of contemporary technologies to cope in a flexible and responsible manner with the short and long-term water problems. Urban sustainability implies policy integration, a shift from structural to non structural solutions, cooperation and partnership and sharing of goals with emphasis to public involvement and participation (Maksimovic et al 2001, Mylopoulos et al 2003b). Water utilities face the challenge of developing new comprehensive water policies by adopting advanced technologies for demand management through a series of incentives and disincentives, reuse of treated wastewater, installation of water saving equipment, renewed water conservation efforts through consumer education and introduce changes in pricing procedures and cost recovery. Emphasis should be given to the integration of engineering, economic, political and social aspects of water supply (Biswas 1997, Baumman et al 1998, Grigg 1986).

Taking into account the challenges that a contemporary water utility is facing the Water Utility of Thessaloniki, in cooperation with the Aristotle University of Thessaloniki, Department of Civil Engineering, have tried to evaluate various aspects of current water policy, investigate the perspectives of water saving and explore new approaches towards sustainable water management in the water supply sector. The emphasis was given to economic and social aspects of the new water policy, having water conservation as the ultimate goal.

2 STUDY AREA

Thessaloniki metropolitan area, with a population of over 1 million inhabitants, enjoys a Mediterranean climate. The city is divided in 17 municipalities. Thessaloniki's Municipal Water Utility (TMWU) today delivers water to approximately 400,000 households. Daily water consumption reaches up to 250,000–280,000 cubic meter of water coming from groundwater resources.

Concerning residential water use, the average annual increase of consumers (as reflected by the number of water meters) was during the last 33 years, 9860 consumers. Total charged consumption increases on the average by 184.96 m^3 each year, as a result of a unit increase of the number of the consumers (as reflected by the number of water meters).

Concerning the industrial water use, most industries are located in the nearby industrial area and the average metering devices served by the Water Utility are 253.

3 RESIDENTIAL WATER USE

2171 questionnaires corresponding to equal number of households were collected.

For the better representation of the population the Thessaloniki area was divided into 17 districts. Probability sampling was the method applied. It refers to any sampling procedure that relies on random selection according to which there is a known and equal probability for every unit to be chosen as a unit of the sample (Baumann et al 1998). Personal interviews of the respondents were taken. Data analysis was made with the method of "the construction of double entrance matrices of absolute and relative frequencies". From the double entrance matrices we derive the qualitative relationships among all the variables under examination. In an effort to incorporate public preferences into future planning and management aspects of the current pricing policy, the WTP levels of the users and their perspectives towards demand management were evaluated.

According to the results (Fig. 1), the majority of the respondents (68.13%), think that the price of water is acceptable. 1 out of 3 people think that a more accurate water pricing will help in water conservation. More than half of the respondents don't believe in the effectiveness of the measure of water pricing indicating either that residential water use is inelastic to price or that the current water rate structure is far from acting as an incentive for water conservation. The current price of water in the city of Thessaloniki (0.35 \$/m^3) (Mylopoulos et al. 2000) doesn't help its conservation since only 1 out of 5 persons control their consumption in relation to its price.

The low reliability of the utility's infrastructure and services, as found by the survey, is strongly connected with a significant 44.08% of the respondents who are not willing to pay extra for service improvements (Fig. 2). Half of the respondents are accepting a 10–20% raise of water price, while only 4.56% approve a 20–30% raise and even a smaller percentage of 1.61% a 30–40% raise.

Concerning the perspectives of demand management only 1 out of 4 respondents believe that current and future water problems would be solved by adopting demand management options through water conservation. Construction of new water projects to satisfy demand is the most popular answer.

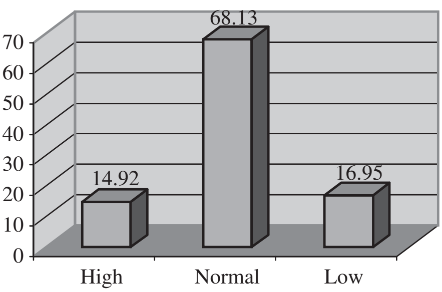

Figure 1. Valuation of water price.

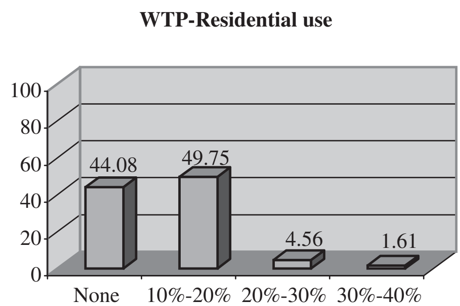

Figure 2. WTP levels in residential water use.

In estimating water demand, studies have used a variety of methods and econometric models depending upon the nature and availability of data (Danielson 1979, Billings & Agthe 1980, Weber 1989).

In this study, a cubic functional form for a residential water demand model is used in order to accommodate different price elasticities estimates for different volumes of water demand (Mylopoulos et al 2003a). The model is developed using both average and marginal price specifications. This was done in order to derive different price elasticities for different consumption intervals.

Data on household characteristics and community-specific variables, (number of residents in each household, type of dwelling, water uses, families with more than four children, income) collected in the field survey were matched with administrative data (price structure, changes in pricing policy, water consumption, public awareness and information) from TMWU records. The data cover the period from January 1994 until the first four months of 2000 (19 time series observations in total). Cross section data obtained from 1356 households. Monthly climatic data were collected from the Institute of Climatology and Meteorology of the Aristotle University of Thessaloniki. In total there were 25,764 pooled, time series and cross-section observations.

The residential water demand equation that was estimated is:

$$C_{it}= \alpha+\mathbf{X}_{it}\beta+\mathbf{Z}_{i}\gamma+\nu_{i}+\epsilon_{it}, \tag{1}$$

where
C is the water consumption in cubic meters, in household i at time t.
X_{it} is the vector of price variables.
Z_i is a vector of community-specific variables.
α, β, γ, coefficients to be estimated.
ν_i is the unexpected water consumption regime or the unit-specific residual.
ε_{it} is the error term.

In order to derive direct price elasticity estimates of demand for water a log transformation of equation (1) was used.

Among other variables, a dummy variable capturing the well educated and well informed consumers was introduced in the demand function, namely the water utility employees. It was found that although the water authority employees are not charged for their water consumption, they have a negative effect on consumption (elasticity $= -0.193$). A 10 percent increase in well-informed consumers can lead to the reduction of water consumption by 1.93%. This can be interpreted as implying that individuals, well informed about water problems, can save water even though there is no economic incentive in doing so.

In Thessaloniki the water bill is a combination of a two-part rate structure made up of a minimum fixed charge and a use charge proportional to the water consumption. The water demand evaluated at the mean of the remaining variables using the marginal price specification is as follows (marginal price specification gave better statistical results, hence will be discussed below):

$$\mathrm{Ln(C)}=0.2\cdot(\mathrm{LnP})3-0.356\cdot(\mathrm{LnP})2-0.51\cdot\mathrm{LnP}+4.3 \tag{2}$$

where P is the marginal price of water

Marginal price elasticity of demand is found equal to -0.51.

As price falls consumption increases at an increasing rate up to price of LnP $= 0.59$ (in Figure 1 the functions inflection point C) and thereafter increases at a decreasing rate. The point price elasticities take the value of -0.113 when price equals 1.6 (Fig 3-point A $= 29\,\mathrm{m}^3$), then decreases up to the value of -0.721 (C $= 50\,\mathrm{m}^3$), and thereafter decreases down to -0.176 for P $= -0.36$ (point E $= 84\,\mathrm{m}^3$). The negative values of price axes must not confuse the reader since one should realize that these values are the logs of real prices. The minimum and maximum logged real prices for the study period are -0.36 and 1.6 respectively. For quantities equal to $34\,\mathrm{m}^3$ (point B $= 34\,\mathrm{m}^3$) and $74\,\mathrm{m}^3$ (point D $= 74\,\mathrm{m}^3$) point elasticities take the same value of -0.51.

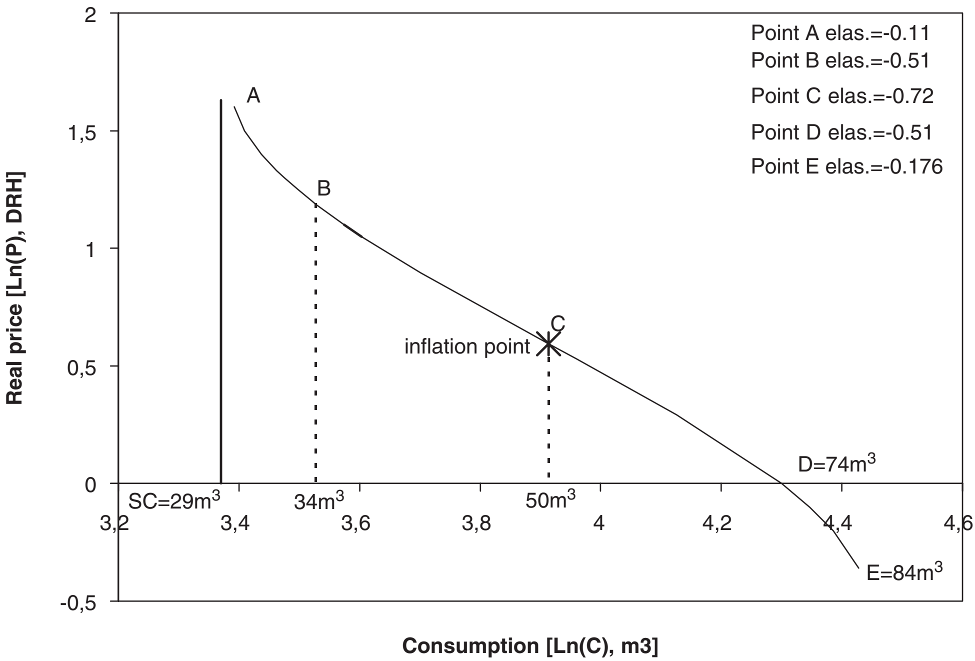

Figure 3. Demand function using the marginal price specification model.

It can be seen (Fig. 3) that the demand curve becomes asymptotic for a specific consumption of SC = 29 m^3 per four months. Thus, the demand for water on the average cannot fall beyond 238 liters per household per day (or 95.2 l/cap/day). For log real price equal to -0.36, equation (4) implies a consumption of E = 84 m^3. This value falls in the third block of the rate structure (60–120 m^3), illustrating that only the first three out of six blocks of the increasing rate structure, which is implemented by the water authority, are actively being used.

5 INDUSTRIAL WATER USE

123 industries getting water from the Water Utility of Thessaloniki were examined. 2/3 of the total stand in the industrial region of Sindos. Most of them occupy more than 50 employees in order to cover the needs of production. Only 10 out of 123 use both private and public water sources, while the others use solely the water provided by the water supply system.

It is well known that water conservation reaches 90% in some cases in the industrial sector (Postel 1997). Imposing recycling and reuse methods improves water efficiency and should be considered as a way of cutting costs in the long-run.

In the city of Thessaloniki only 26% of the industries that were examined (Fig. 4) perform recycling methods and most of them claim that such methods promote their financial situation. According to the recycling process that each industry follows, there is the possibility to reuse either all the quantity of water again in the production (9 out of 32 industries that perform recycling methods) or a small part of the total quantity, less than 20%. Industries that do not perform recycling methods claimed various reasons. They alleged that there is not any appropriate method, that implementing recycle demands extremely experienced staff as well as that recycling methods require increased cost. It is obvious that it is attainable to force industries to adopt such systems.

Most of the industries, which were examined, declare that the price of water is acceptable. Regardless of the price of water, most of them are reluctant to pay for the amelioration of water

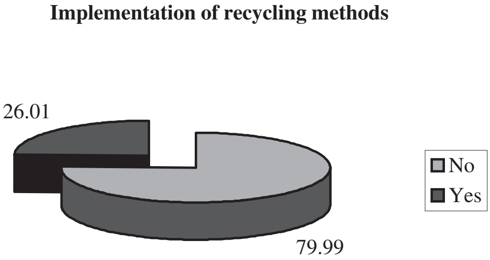

Figure 4. Implementation of recycling methods.

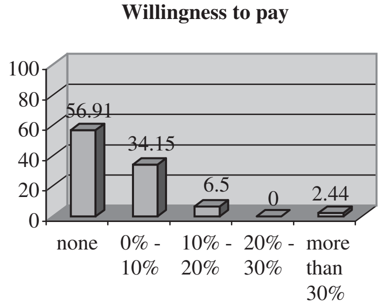

Figure 5. WTP levels in industrial water use.

services (Fig. 5). This is due to the fact that most people, even in our days, consider water as a public good, which should be offered for free. Undoubtedly, water for industrial use should be considered as an economic good. In some cases, according to the production process, water is a basic input to the production function and consequently, it contributes in the cost of production. The refusal for paying more should also be explained by the high percentage (60%), which expresses its complaints for the services of the Water Utility.

85% of the industries examined claim that an increase in water price will not act as an incentive to reduce water consumption, as they claim that they use just the quantity needed for the production process. This attitude is due to a combination of factors, which includes the fact that the price of water is low enough to represent a financial incentive in order to reduce demand and to the fact that there is no other input to be used as a substitute for water. Until 2001, industries had been charged with uniform tarrifs. From 2001 the Water Utility of Thessaloniki introduced the increasing block rates which had been applied until today. Thus, industries, which consume up to 2000 cm in a four months time period (72%), are charged with 0.44 Euros/cubic meter, while those which exceed this amount are charged with 0.73 Euros/cubic meter. Of course, the implementation of a different pricing policy, with considerably higher rates will lead to the deduction of the consumption of water and consequently to the saving of water.

6 CONCLUSIONS – SUGGESTIONS

In order to form new demand oriented water policy the Water Utility of the city of Thessaloniki should give special attention to the following remarks and suggestions:

- As water is considered to be more a social good than a commodity in Thessaloniki, a sole water pricing reform would not lead to remarkable changes in water consumption. Water conservation can only be achieved by the combination of economic and social measures in order to control water demand.
- The social character of water in combination with the low elasticity of demand in the price of residential water, are two decisive factors for the formulation of the pricing policy. The results indicate that the current pricing policy is generally accepted by the users, but it doesn't act as an incentive for water conservation. The fixed charge should be discarded because it is not properly justified for low consumptions. The current water policy should better focus on adopting conservation-oriented structures that will protect "lifeline" consumptions while on the other should differentiate the prices for prodigal water users. The introduction of an inverted-block rate structure in residential use (higher prices for high water consumptions-small scaling in the upper scales), would help water conservation. In this rate structure the water users would have the incentive to remain in lower levels of consumption where the price is significantly lower.

- Concerning the industrial use the results of the new pricing policy will be seen in the near future. But the possibility of a pricing policy, with considerably higher rates will lead to the deduction of the water consumption and consequently to the saving of water.
- There is a great possibility of saving water from the industries. The potential of a dual system with water of lower quality and lower price would be a good solution to the problem achieving two goals. First, the network will be deloaded and second the water that will be used will be cheaper since it will not be potable.
- The low percentage of industries that use recycling and reuse methods indicate the need for the implementation of incentives in the industrial users to adopt "clean" technologies and promote conservation in this way.
- Another implication of this study is that a well-informed consumer, aware of issues of water conservation and techniques for water efficient use, may be more inclined to reduce his or her water consumption than otherwise. Education programs aiming at informing individuals about the importance of water conservation may be effective tools for saving water. Therefore, the Water Utility should place great emphasis in promoting education and information programs, in order to augment public participation and awareness. People ask for more information nevertheless.

REFERENCES

Baumann, D. Boland, J. & Hanemann, M. 1998. *Urban Water Demand Management and Planning*: USA, McGraw Hill.

Billings, R.B. Agthe, D.E. 1980. Price elasticities for water: a case of increasing block rates, *Land Economics*., 56(1), 73–84.

Biswas, A. 1997. *Water resources Environmental Planning, Management and Development*. Mc Graw Hill.

Danielson, L.E. 1979. An analysis of residential demand for water using micro time-series data, Water Resources. Research., 15(4), 763–767.

Grigg, N. 1986. *Urban water infrastructure: Planning, Management and Operations.* New York: J. Wiley and Sons.

Maksimovic, C. & Tejada-Guibert J.A. 2001. *Frontiers in Urban Water Management. Deadlock or hope* U.K.: IWA Publishing.

Mylopoulos, Y.A. & Mentes, K.A. 2000. Perspectives of a demand oriented water policy in the city of Thessaloniki, *Proc. International Conference: Protection Restoration of the environment V.* Vol. I, pp. 289–295. Thassos: Greece.

Mylopoulos, Y.A. Mentes, A.K. & Theodossiou, I. 2003a. Residential Water Demand Modeling Using Household Data: A Cubic Approach. *Water International Submitted for publication.*

Weber, J.A. 1989. Forecasting demand and measuring elasticity. *Journal of American Water and Wastewater Association* 77(5): 57–65.

Mylopoulos Y., Kolokytha E. Tolikas D. 2003b. Urban water management in Greece. Present conditions and Perspectives of sustainability, IWRA, *Water International,* Vol. 28, pp. 43–51, March 2003.

Postel S. 1997. *Last Oasis: Facing Water Scarcity*. W.W. Norton & Company.

Drought: fighting or adopting? A report about three years experience of drought in Khorasan province (north east of Iran)

Seyed Hossein Sanaei-Nejad
Ferdowsi University of Mashhad, Iran

Hossein Ansari
University of Tarbiat Modarress, Tehran, Iran

ABSTRACT: The latest three years consecutive spell of drought creates a harsh condition in Khorasan province (north east of Iran). Since available fresh water is very restricted in this region, the long period of drought caused many difficulties in different aspects.

We proposed a master plan to officials and persuaded them to consider it for drought management. In this plan, it is explained how to face deficit water availability, how could we mitigate drought harms and how fighting drought. However, finally we realized that we need adopting ourselves with drought as a natural fact that will be with us in a periodical return.

Fortunately, local government accepted the master plan and we began the work. We advanced a few steps forward and acquired some experiences, which are used in drought management programs.

We established a web-based meta-data of drought-related articles, hydroclimatologic data, reports and other related items. This meta-data is used to help managers defining their priorities and help experts doing their drought related projects.

Drought indices were also calculated and developed for the province and the results are published in a web page, which is established for Drought Information and Documents Center for Khorasan Province.

In this paper, we explain general aspect of drought in the region over the period, the main frame of the master plan for drought adaptation and also the results obtained so far.

1 INTRODUCTION

A drought is a prolonged, abnormally dry period when there is not enough water for users' normal needs (Commonwealth Bureau of Meteorology, 2003). There are different and complicate causes for drought; human activities are believed to affect or strengthen the procedure as well. Drought impacts are widely assessed in different sectors. The impacts can be classified as economic, environmental, social and in some cases political. Knutson (1998) presented a detailed check list of impacts that can affect a region or location.

In the last few decades, interest in planning for drought has increased at all levels in different countries. (NDMC, 2003; INDC, 2003,). However, so far, there has not been a drought management plan for Khorasan province, north of Iran. As Knutson (1998) pointed: " unfortunately, we tend to focus on drought when it is upon us. We are then forced to react – to respond to immediate needs, to provide what are often more costly remedies, and to attempt to balance competing interests in a charged atmosphere…". This was the case in khorasan province. The local government was taking action to compensate the damages, only when drought was upon the province. The compenstations were used to be loans (and in some cases not refundable) for farmers and stakeholders. It is clear that the policy is not good and not effective. Therefore, the public perception was that the government was not doing its job, even though many efforts were directed and a lot of money was spent to mitigate drought.

At the first beginning of this case study, it was clear that we needed a master plan to consider different aspects of drought. Then a long bureaucratic procedure was followed to legislate the master plan in the Disaster Council of Khorasan local government. For some bureaucratic and grants restrictions, the whole programs proposed in the master plan could not be put into practice. However, three main projects which had more priority were successfully performed. We will describe the master plan, the advances of succeeding the projects and the effects of this case study in drought management in Khorasan.

2 DROUGHT MASTER PLAN FOR KHORASAN PROVINCE

The plan provides a framework for coordinated pro-active approach to reduce the effect of drought on Khorasan province different sectors. The main stage in this plan are as following:

– Establishment of Drought and Flood Information Center (DFIC)
– Assessment of drought in Khorasan province
– Investigation into finding some ways to adopt with drought in the region
– Preparing drought management plan
– Renovating the results of the plan to legislated rules to be implemented in different sectors.

3 ESTABLISHMENT OF DROUGHT AND FLOOD INFORMATION CENTER (DFIC)

It is obvious that information is the basis of any study, planning and decision making procedure, specially in drought management and studies which are multidisciplinary and inter sector subject. We needed to provide our researchers, experts and managers in different levels by useful and up dated data and information related to drought and flood. As we found, most of the data and information are common between drought and flood. Therefore we decided to consider the information center for the both subjects. It should be noticed, that there was not any similar information service, neither officially nor in any other forms in the province. As the fisrt stage we tried to prepare a meta data for the related data and information which might have prepared and stored in different organizations and institutions. We planned a questioner form, collected the data by filling the questioner and extract the data.

A web based data base system was developed and the data was transformed to the database. At the moment the data base is in Farsi and we hope that we develop an English version in near future.

The following items are included in the meta data base system:

– Books, journals and other scientific documents
– Legislation, rules and law which can be used for drought management
– The official and other drought related project reports
– The reports of performing projects in relation of drought and flood in the province
– A full meta data for hydroclimatological data
– Other socio-economic information and documents related to drought.

We collected all of the data, information and documents. The collected information was edited, processed and classified properly to meet our goals in the meta data. We analysed the content of documents to discover if any of them could be used for supporting the drought plan. We also determined the needs for research programs, rules and legislation, which should be considered for complementing the plan. Eventually, the stages to establish DFIC are as following:

– Preparing a web based meta data system for all of the data, information and documents
– Planning a procedure to collect the results of researches and programs which will be done in the future by all of the ingaged organizations

– Developing a system to provide hydrolclimatological data in a web based frame for end users, who are working in drought and flood related disciplines
– Developing a measurment systems to collect necessary data and helping the ingage organizations by complementary equipments
– Developing a remote sensing data collection system for drought monitoring.

4 DROUGHT ASSESSMENT

Based on the data, information and documents which had been collected, we could make a reasonable assessment of drought in Khorasan. At the first stage, we needed a good drought index to evaluate drought duration and severity. We examined different drought indices such as: Palmer Drought Index, Standard Precipitation Index(SPI), Percent Normal Index and some others for the region and used SPI for drought assessment.

We then used Geographic Information Systems(GIS) to prepare a spatial distribution model of the index for Khorasan. The zoning map of SPI for the province was successfully accepted and used by different organizations such as insurance companies, Bank of Agriculture and also local government for their supporting programs in recently drought period.

We developed a software to calculate SPI, prepared zoning maps of the index and put the maps in a web page for end users involving in drought mitigation program over the province.

Drought assessment in the master plan includes other stages that we hope to complete in the future:

– Investigation into local and global atmospheric parameters which cause or intensify drought in the region
– Preparing a drought pre-warning system (A working group is working on this project and the reslts will be published in near future)
– Study of after drought changes in environmental parameters
– Study of drought influences on economic, social and ecological activities
– Assessment of drought related crisis in different economic, social and ecological aspects
– Study of global climate change on drought in the region.

5 DROUGHT MITIGATION PROGRAMS

We had to start taking action to mitigate drought in the region as it was upon us and something should have been done. Obviously we could not change cultivation plan to modify major economic and social activities in the province in short term. We do need to work hardly to achive these goals and we have planed to do so, as I explained in this section.

As an emergency and effective action, we prepared a public education program and tried to give people warning of shortage of water and ask them to use water as economic as it is possible. We used TV, Radio, Papers and even Billboards for this and arranged different seminars and workshops.

We advised local government to support and even subsidise the projects which could save water or suggest ways to consume water efficiently.

We considered to do the following works to facilitate our drought mitigation programs:

– Study of flood control projects for drought mitigation
– Study of altering cultivation plan in the region to adopt drought condition
– Study of finding appropriate methods to optimize water consume and increase water efficiency for different sectors
– Investigation into better pasture management to achive a livestock-pasture balance state in drought condition
– Study of using abnormal water, such as recycling sewage in drought conditions
– Optimization of ground water and surface water suplies for integrated water management.

6 DROUGHT MANAGEMENT AND PLANNING

Obviously all of the programs, studies and efforts should be directed and ended to management, plans and finally taking action to reduce drought damage and adopt activities with the harsh condition of the disaster. This is a long way to go. We began the way by preparing a proposal for implementation of some rules and programs in different organizations. The proposal is proceeded to be legislated by the authorities and will be implemented by the related organization afterward.

In the master plan we considered some other programs in order to implement a complete drought management and planning in Khorasan, such as:

- Determining risk factor for the region, based on probability of drought occuring and in danger resources
- Implementing of drought risk reduction methods such as:
  - Using local adopted technologies for water consuming
  - Public education programs for drought adoption
  - Modifying of related rules and law for supporting producers in order to make them ready in drought condition
  - Improving insurance industry to prepare an effective supporting system for producers in drought
- Drought monitoring and improving on time warning system for different sectors.

7 CONCLUSIONS

Recuring droughts in Khorasan and vulnerable economic structure of the region, which strongly depends on water, dictated us a multidisciplinary plan to face more shortage of water in drought condition.

Local government traditionally fight with drought by compensation drought damages and spending a lot of money each time drought occurs. However, we believe that we need to optimize economic and social activities with available water resources restrictions considering drought risk management. Consequently, we need to know drought conditions and occurrence, then quantify the parameters affecting drought in the region. Afterward, we should find effective methods, programs and ways to coordinate the risk management plan.

We examined some ways and realised that we need more supports from local government and also more data and information to monitor drought and explore different management aspects of the disaster to be considered in our master plan.

So far, the steps we put forward have been successful and we achived some results which are fundamental steps to complete our master plan in the future.

The most important results we achieved are as following:

- Establishment of drought and flood information center (web based)
- Monitoring drought in the region by calculation of standard precipitation index (SPI)
- Spatial modelling of drought index for the province
- Public education for economic consuming water under drought condition
- Proposing some rules and programs to be used for drought management.

REFERENCES

Commonwealth Bureau of Meteorology, 2003, http://www.bom.gov.au/climate/drought/livedrought.shtml

Drought Information Center, 2002, NOAA Web Page; http://www.drought.noaa.gov/

FSA, 2001, Farm Service Agency Web Page; http://www.fsa.usda.gov/pas/default.asp

Knutson (1998), How to Reduce Drought National Drought Mitigation Center 239L. W. Chase Hall, Lincoln, Nebraska 68583-0749, USA

NDMC, 2001, National Drought Mitigation Center Web Page; http://drought.unl.edu/