THE APPLICATION OF ADVANCED CONTROL TECHNIQUES IN WATER TREATMENT AND SUPPLY SYSTEMS

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Water Research Commission

THE APPLICATION OF ADVANCED CONTROL TECHNIQUES IN WATER TREATMENT AND SUPPLY SYSTEMS

Report to the
WATER RESEARCH COMMISSION

by

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on the Project

" The optimisation of water and wastewater treatment and supply systems using advanced process control techniques "

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EXECUTIVE SUMMARY

The rapid increase in pressure on natural resources brought about by human economic activity has led to a growing realisation of the need to manage such resources on an integrated basis. In the water field, the growing importance of disciplines such as Integrated Catchment Management, Demand Management, Life Cycle Analysis and Pinch Analysis are evidence of this trend. The use of these and other, strategic tools which attempt to ensure that resources are consumed with the maximum benefit and minimum environmental degradation, inevitably leads to technological systems with many complex interactions and constraints. To realise the benefits of integrated planning and design, it is vitally necessary to be able to control these systems effectively. This basic problem is common to many issues beside water management, and process control technology continues to develop rapidly to provide the required capabilities. It is these capabilities which need to be assimilated by the water and wastewater industry if the benefits of integrated planning are to be fully realised.

This project aimed to build capacity within the industry by demonstrating and apply advanced control techniques to water treatment processes. Target levels were senior managers (an appreciation of the potential), middle managers (project identification and management) and engineers (implementers of the advanced control systems). The application of advanced process control techniques should reveal that opportunities exist for enhanced management and operation of water systems.

Following the recent replacement of analogue control systems in the industry with digital systems (SCADA and DCS), there has been little advantage taken of the ability of digital computers to work quite differently from the old analogue control loops. Instrument technicians have been inclined to translate the old loops one-by-one into the equivalent digital controllers. This ignores the ability of the computer systems to take many input variables into account in an algorithm, possibly manipulate many outputs simultaneously, and indeed to do far more complex calculations in the control algorithm itself.

The identification of suitable demonstration applications was considered best achieved in discussion with the key water industries, in this case the Umgeni Wiggins Water Works and the Ethekwini Municipality Water Services.

1. Project Objectives

(1) To demonstrate the use of advanced process control for integrated management of water and wastewater systems in the South African Water Industry.
(2) To develop advanced control systems for managing selected water and/or wastewater treatment systems.
(3) To develop advanced process control as a tool which will enhance the capacity of the South African Water Industry to achieve greater integration of the management of water and wastewater.

2. Overall course of the project

Two demonstration projects were initially proposed to explore the principle of Advanced Control in the water industry:

(a) Chlorine concentration control at the exit of Durban's Umgeni-Wiggins waterworks.
(b) Optimal distribution of water downstream of the waterworks in the municipal reservoir network of Durban.

During 2000, several meetings were held separately with staff at the Umgeni Wiggins Water Works (led by Dan Naidoo) and the Ethekwini municipality water services (led by Reg Bailey), in order to finalise the scope and objectives of these two studies:

(A1) chlorine composition to be controlled at the exit of the conditioning reservoir of the Umgeni Wiggins Waterworks

(A2) chlorine composition control to account for
- (i) varying chlorine loss rate
- (ii) varying reservoir level
- (iii) varying reservoir inflow and outflow
- (iv) flow peculiarities within the reservoir, to be identified through CFD modelling

(B1) target application for water distribution optimisation to be Durban's Southern Aqueduct.

(B2) cost of electrical power used in drinking water transfers was to be minimised.

(B3) chlorine usage (dosing and re-dosing) in the distribution system was to be minimised, yet the target chlorine concentration range at the consumer end was to be maintained.

The LIPE-INSA (Laboratoire d'Ingénierie des Procédés de l'Environnement, Institut National des Sciences Appliquées) in Toulouse, France, is a Chemical Engineering laboratory with extensive experience in the investigation of water treatment processes. One of their strengths is in the group of Prof Alain Liné, which is experienced in the CFD (Computational Fluid Dynamic) modelling of these processes. There was existing research collaboration between Prof Mike Mulholland of the School of Chemical Engineering in Durban, and Prof Marie-Veronique LeLann of INSA (Institut National des Sciences Appliquées, Département de Génie Electrique et Informatique) and CNRS/LAAS (Laboratoire d'Analyse et d'Architecture des Systèmes) in Toulouse, who has a close relationship with the LIPE laboratory. Professors LeLann and Mulholland were already collaborating in the areas of process modelling and control.

In order to utilise the various skills of Professors Mulholland, Lelann and Liné, two projects were set up to meet the above objectives using MScEng research students from Toulouse, Ms Amelie Pastre and Mr Cédric Biscos.

Both of these students were from Prof LeLann's original institution in Toulouse: ENSIGC (Ecole Nationale Supérieure d'Ingénieurs de Génie Chimique, later becoming ENSIACET: Ecole Nationale Supérieure d'Ingénieurs en Arts Chimiques et Technologiques). This Chemical Engineering School falls under the INP-T (Institut Nationale Polytechnique de Toulouse).

Professor Mulholland (School of Chemical Engineering, University of KwaZulu-Natal, Durban) supervised the work, and it was co-supervised by Professor Buckley and Mr Brouckaert of the Pollution Research Group in that School, as well as by Prof M-V LeLann from Toulouse. Prof. Alain Liné was able to assist during the course of the studies on CFD issues.

In January 2001, the two research students attended some instruction in CFD techniques given by Prof Liné at the LIPE laboratory in Toulouse. At the end of January they arrived in Durban to commence the projects. During the first 6 months they also undertook coursework in Process Control and Real-time Process Data Analysis. During this first year there was also a sequence of meetings with all concerned separately at the Umgeni Wiggins Water Works, and at the Ethekwini Water Services.

During 2002 it proved possible to bring the chlorine concentration control study (A) virtually to a conclusion. The CFD modelling of flow in the reservoir was completed, and it proved possible to simulate this with a simple compartment model. That model then formed the basis of a Kalman filter for real-time identification of the chlorine decay rate constant, which in term formed the basis of a Dynamic Matrix Control (DMC) predictive controller. By October 2002, the necessary algorithm was mounted and operating on the Umgeni Wiggins Water Works' Adroit SCADA system. However, there were some obvious problems with the algorithm. These stemmed from the way the filter-bed discharges upset the inlet chlorine measurement, and finally, the fact that operators were actually working according to an alternative chlorine monitor at the reservoir exit. Fair operation was ultimately obtained. The MScEng degree was awarded to Amelie Pastre *cum laude* in December 2003. Nevertheless, it had been observed that the predictive horizon of the controller should be extended, and tuning slackened. This was not done at the time, so it came as a surprise in May 2004, to learn that the algorithm was in regular use at the water works, in spite of the sub-optimal tuning.

In the meantime, the water distribution study (B) took longer. The Southern Aqueduct solution by MINLP (Mixed Integer Non-linear Programming) turned out to be a very large multivariable problem. Our original DICOPT-CONOPT solver in GAMS proved capable of solving only relatively small subsections of the Southern Aqueduct. The problem was simplified by taking the network pressure-balance out, just working within the constraints of maximum observed flows in any one piping section. Finally, we experimented with the DICOPT-CPLEX combination, which proved much more powerful at solving this type of problem. Cédric Biscos was faced with extensive data manipulation and lengthy computer executions, which took him to June 2003. He submitted his MscEng thesis later in the year, and was awarded the MScEng in May 2004. Cédric's work ended with an off-line simulation of a real-time optimisation of the Southern Aqueduct. It could not be implemented on-line in the course of this study, because the solution was too time-consuming and not 100% reliable, because of the lead-time that would be required to set up the software on the Metro's SCADA system managing the network, and because of possible additional instrumentation requirements. A thrust for future work would be to streamline the solution, and achieve 100% reliability.

At the outset of the project, attempts were made to engage with managers and operators of wastewater treatment plants to formulate a project involving control of a wastewater treatment process. However, the idea evoked little interest with prospective partners, and no suitable application was identified. In South Africa wastewater treatments tend to be designed to operate without automatic control; this might explain the lesser interest in wastewater applications. The situation tends to be similar elsewhere, although in Europe there is a growing interest in applying tighter control to wastewater treatment plants, often associated with space limitations which provide an incentive to obtaining greater throughput from given equipment. Nevertheless, this interest is still largely academic rather than practical. In 2001 the 1st International Water Association Conference on Instrumentation, Control and Automation (ICA2001), held in Sweden, was devoted to the management and control of water and wastewater treatment and transport systems. In reviewing the conference Olsson (2002) stated: *Plant wide and integrated control of sewer and wastewater treatment plants was announced as one of the key topics in ICA2001, yet there where hardly enough papers to cover a single session. The reason for this became clear during the session: the ground-breaking theoretical studies have been made and further theory development is no longer considered interesting. However, to make the next step and to implement - at least some part of - control in the total system in reality proves to be a very tedious task. The present obstacles seem to have more to do with data (assessment, management, analysis) and administration than with control algorithm issues. Moreover, since the overall objective can still not be defined in concrete terms, the operators seem to be reluctant to embark on the idea of integrated control approaches. ... In the survey reported by Jeppsson et at (2002) many* *countries replied that integrated control might be the main focus, but not until 2010.* This is essentially the same situation as encountered during this project.

3.1 Modelling and control of chlorine composition at the exit of the Umgeni-Wiggins waterworks

The Umgeni Wiggins water works gravitates potable water from its 124 ML storage reservoir to southern Durban and adjacent areas. Presently the design capacity of the waterworks is 350 ML/d of raw water. At the end of the treatment plant, a chlorine contact reservoir, divided into two sections, holds treated water. Because of the variable demand, the reservoir level and residence time vary over a range of values, making the control of the chlorine dosing difficult. The chlorine residual concentration declines with time, so that the outlet chlorine concentration will depend on the residence time in the reservoir. The chlorine concentration of final water should be kept between 0.8 and 1.2 mg/L: above 0.8 mg/L to ensure it maintains water disinfection within the distribution network and below 1.2 mg/L to prevent taste and odour complaints. Prior to this investigation, the chlorine dosage in the *inlet* of the reservoir has been manipulated manually based on the experience of the operators, in order to maintain the desired *exit* concentration.

The aim of this project was to develop an effective real-time control strategy to predict and control the chlorine concentration at the exit of the plant, and hence allow optimal targeting of the chlorine concentration in water delivered into the Durban Metro region.

3.2 Chlorine contact reservoir at the Umgeni-Wiggins waterworks

From the treatment plant, water proceeds through a pipe to the reservoir (Figure 1). The actual chlorine addition occurs shortly before this pipe splits to supply the two reservoir sections. The levels are equal in both sections due to the interconnection of the outlets. These reservoir sections are large: each is 116.5 m by 76 m. The wall height is 7 m. The inlets of 1 m in diameter are positioned 1.5 m above the floor and the outlets of 3 m in diameter draw from a trough in the floor. At the exit of this reservoir, the treated water is transferred to the Durban Unicity buffer reservoirs according to the consumers' demand.

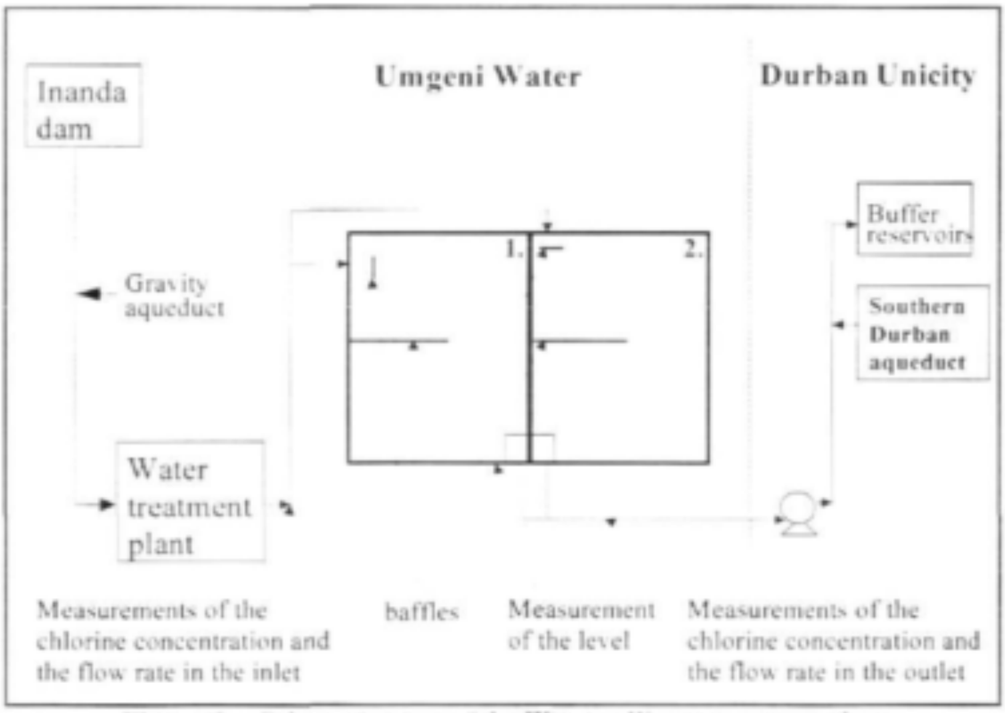


Figure 1 *Schematic view of the Wiggins Water treatment plant*

The Adroit SCADA system (Supervisory Control and Data Acquisition) allows storage of selected measured and derived variables at specified time intervals. The data records of interest in this study (inlet and outlet chlorine concentration, inflow rate, outflow rate and level for the entire reservoir) were saved at 5-minute intervals.

A CFD (Computational Fluid Dynamic) analysis was performed to obtain an understanding of the flow pattern through the reservoir.

3.3 Computational Fluid Dynamic modelling of flow in the chlorine contact reservoir

For each section of the reservoir, computational domains were built with the dimensions of the real section. The features included were the base, vertical walls and baffles, inlets and outlets. To avoid the complexities of a free surface model, the water surface was modelled as a rigid horizontal frictionless surface. A *deforming* mesh was used to represent the change in level as the reservoir filled or emptied with time. This model was not able to represent the situation when the water level dropped below the top of the baffle walls. An example of the *FLUENT* CFD solution for a 1.6 day period starting on 25/9/2001 is given in Figure 2. This flow field (speed) is shown at a height 3m above the base of the reservoir, whilst the actual reservoir level is at 3.5m. There is clear evidence of channeling and relatively dead zones.

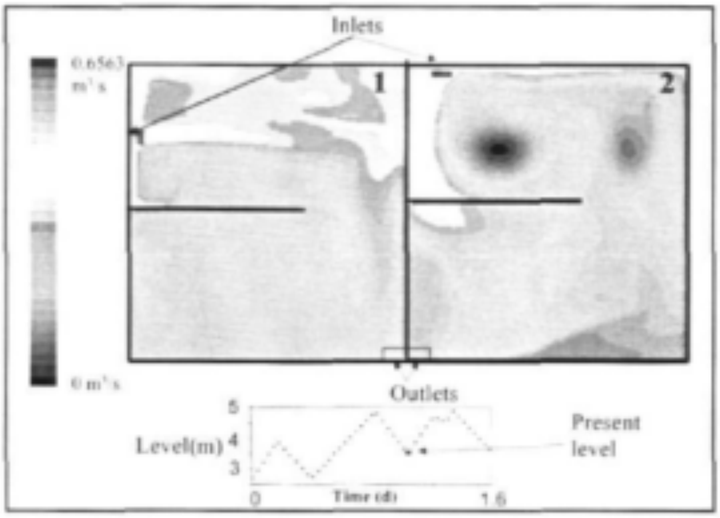


Figure 2 *Plan view of flow velocity magnitudes in the reservoirs (level = 3.5 m)*

The chlorine consumption was modelled using a first order decay with a kinetic factor of 2 d^{-1} (Figure 3).

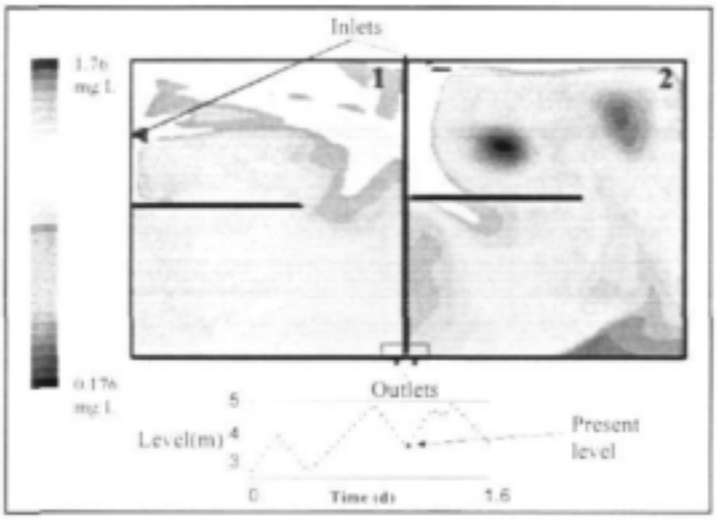


Figure 3 *View of chlorine concentrations in both sections (outlet chlorine concentration: section 1 = 0.779 mg/L; section 2 = 0.773 mg/L)*

As can be seen, the flows through the two sections have preferred pathways. In those parts of the volume which are off these preferred paths, the water experiences longer residence times, which causes the chlorine to be depleted. The outlet chlorine concentrations predicted by *FLUENT* are compared with the real measurement data in Figure 4. Best results were ultimately obtained with a kinetic factor of 1.5 d^{-1}, which was subsequently supported by measurements of Kandasamy and Govender (2002).

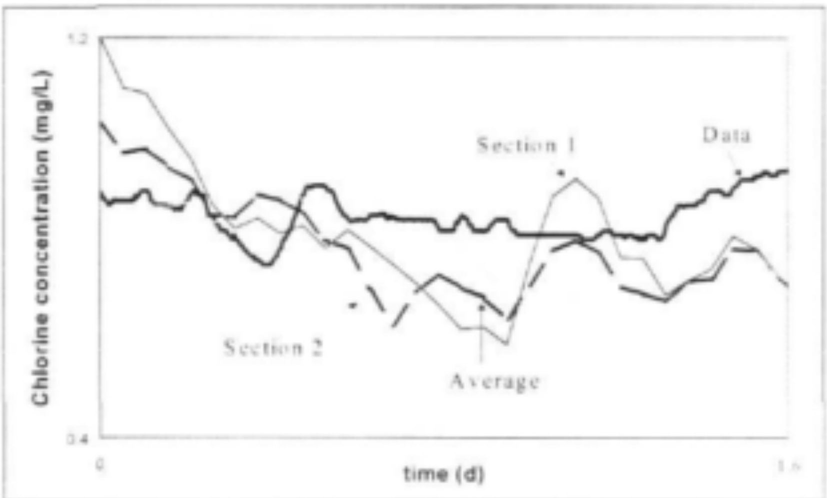


Figure 4 *Comparison between the predicted and measured chlorine outlet concentration*

To simulate a tracer test in FLUENT, a certain amount of a non-reactive species was added at t = 0, for just one time step, to give an effective pulse in the inlet concentration. The two reservoir sections produced a double peak in the outlet concentration response, which was subsequently simulated in MATLAB by fitting the compartment models depicted in Figure 5.

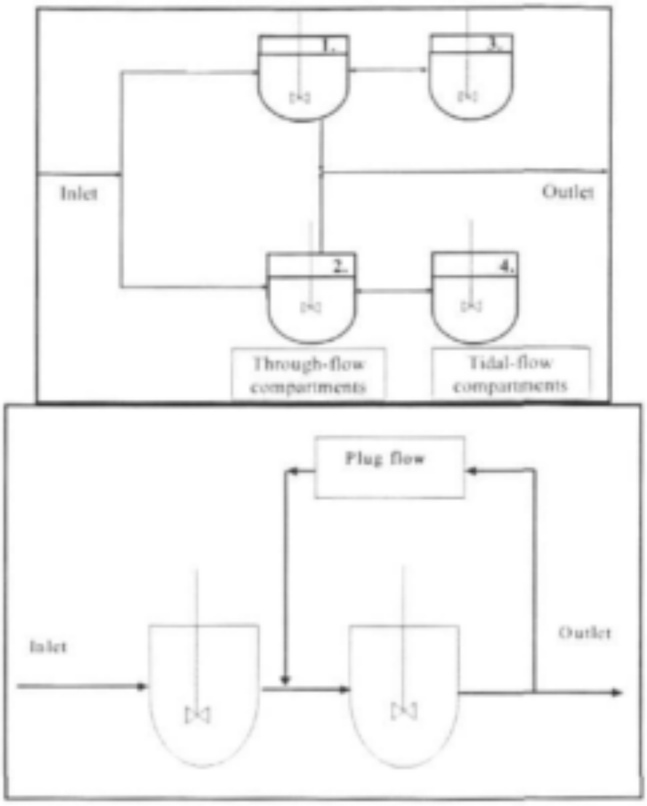
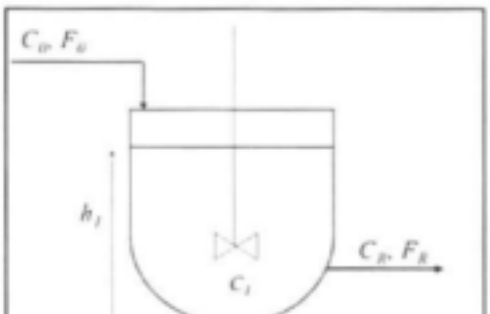


Figure 5 *Compartment models giving a good fit to the RTD predicted by CFD modelling*

3.4 Simulation of the chlorine contact reservoir using compartment models

Despite the better performance of the multi-compartment models in representing the reservoir RTD, a single mixed compartment (Figure 6) was found quite adequate for identification of the kinetic constant k, and as a basis for the adaptive predictive control using DMC (Dynamic Matrix Control).



Volume balance:

$$A_1 \frac{dh_1}{dt} = F_1 - F_0$$

Mass balance:

$$A_1 h_1 \frac{dC_1}{dt} = F_1(C_0 - C_1) - kA_1 h_1 C_1$$

Figure 6 *Single-compartment model*

Since the compartment model was solved in the form of an EKF (Extended Kalman Filter), it was a simple matter to supply the model with measured records of the exit chlorine concentration, and to use it to provide instead predictions of the necessary kinetic constant k variation in time (Figure 7) which would account for the these measurements (given also the records of flow, level and inlet concentration).

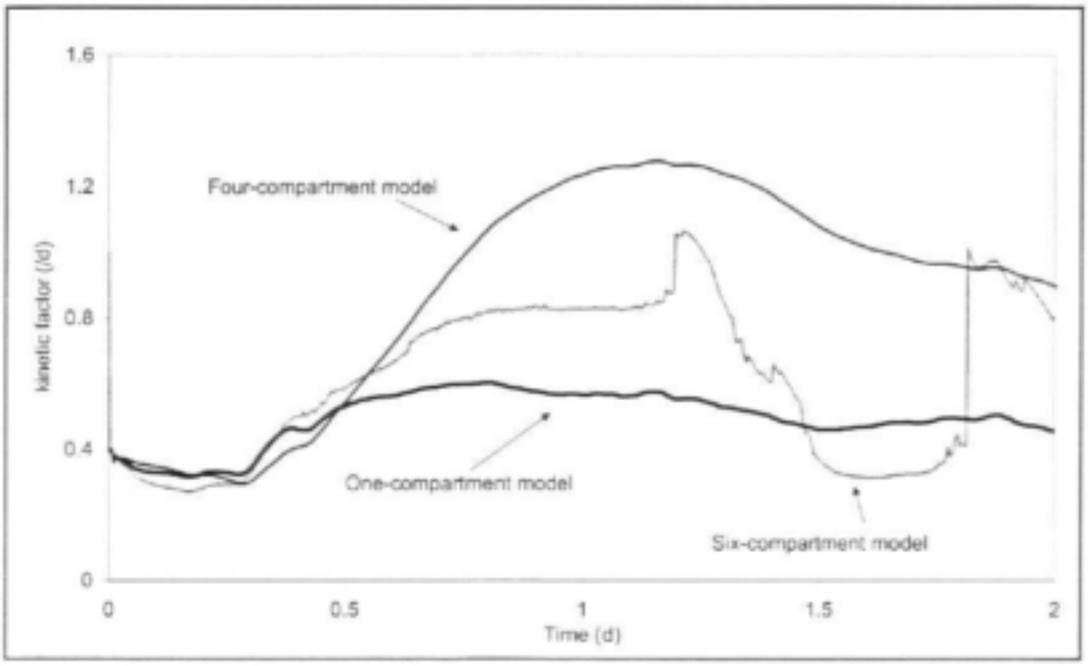


Figure 7 *Identified kinetic factor value for the three models (10/10/01 to 12/10/01)*

3.5 On-line implementation of an adaptive predictive controller for chlorine concentration

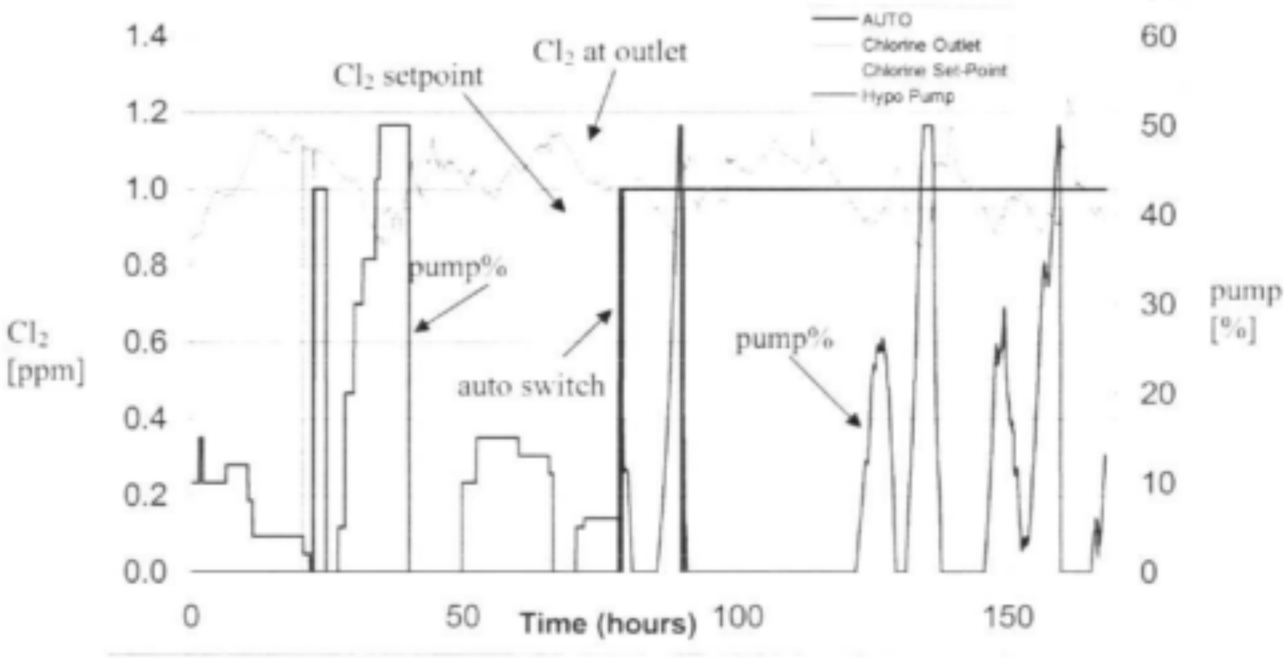


Figure 8 *Comparison of outlet chlorine control performance by manual operation (up to 80 hours) and by adaptive Dynamic Matrix Controller (after 80 hours)*

In the DMC algorithm, the dynamic matrix is dependent on the kinetic factor k, and can be updated each time the extended Kalman filter runs, allowing for better control of the outlet chlorine concentration. The Adroit SCADA program is called every 5 seconds, to give a reasonably fast response on the screen to operator changes of the settings, such as set-point or tuning parameters.

The first part of Figure 8 has no automatic control, as is evidenced by the occasional stepping of the hypo pump setting by the operator. Then the DMC controller (incorporating the adaptive adjustment of the identified chlorine loss rate constant from the Kalman filter) is switched on at the 80th hour (shown by the "auto line ", which goes from zero to one). The chlorine set point is fixed at 0.95 mg/L.

The automatic control is acceptable, perhaps a little better than the occasional adjustments of the operators. In fact, there is some confidence in the new controller amongst the operators. However, it may be possible to improve the tuning of this controller using a longer optimisation horizon (4 hours instead of the present 2 hours), and to increase the move suppression. On the plot, the main deviations from the set-point are all caused by excessive control action - each peak comes from an excessive hypochlorite pump adjustment - making the controlled variable a bit oscillatory. Other issues such as appropriate tuning of the Kalman filter, for a desirable rate of variation of the predicted chlorine decay rate constant, should also be examined further.

3.6 Conclusions concerning the modelling and control of the chlorine at the exit of the waterworks

This study had a number of distinct phases (CFD modelling, compartment modelling, identification, predictive control, real-time programming), which have been successfully integrated to produce an adaptive predictive controller that has gained acceptance by the operating staff. It is believed that the performance can be improved further by extension of the predictive horizon from two to four hours, and by increasing the move suppression.

This on-line implementation of advanced control in a water treatment process has shown that there is potential for further development in this area, especially in a case like this where long time constants, temporal changes in behaviour, and a multivariate dependency, conspire to make intuitive operator interventions less effective.

4. Optimal operation of a water distribution network by predictive control using MINLP

Pumping is one of the most important operational costs found in water distribution systems. This is particularly true in the Durban Metropolitan Area (DMA), which is a very hilly area, where many pumping operations are required to transfer water from one area to another. Consequently, eThekwini Water Services has identified pumping operations as one of the possible domains requiring operational optimisation. Although operating policies are applied so that pumps do not switch on during electricity cost peak periods as far as possible, no strategy is currently under use to restrain pumping operations during intermediate and off-peak periods.

The maximisation of the use of low-cost power for pumping operations is the main optimisation objective and can be achieved by using an optimal planning that will allow best use of gravitational flows, and restriction of pumping to low-cost power periods (e.g. overnight pumping) as far as possible. This objective is constrained by the maintenance of the minimum and maximum emergency volumes in all reservoirs, as well as a good water quality in every part of the distribution system. Longer hold-ups in reservoirs would increase chlorine loss, which must be accounted for in an optimisation strategy. The combination of dynamic elements (i.e. reservoirs volumes) and discrete elements (pumps and valves) makes this a challenging predictive control and constrained optimisation problem, which has been solved by Mixed Integer Non-Linear Programming (MINLP).

4.1 Literature relating to the operational optimisation of water distribution networks

Optimisation of water distribution systems has received enormous attention in the past few decades. Research in this field is still highly active, as the complexities associated with the operation of multiple interconnected reservoirs still exceeds the capabilities of existing optimisation tools in finding solutions easily (Teegavarapu and Simonovic, 2002). Minimisation of supply and pumping costs, maximisation of water quality and leak prevention are among the main targets (Cembrano *et al.*, 2000).

The first approach of dynamic programming applied successfully on a large-scale system can be found in Fallside and Perry (1975). In the 1980's, one of the most relevant applications was the implementation of the model developed by Carpentier (1983), for the southern areas of the Paris distribution network. His method was based on the use of dynamic programming techniques, which take into account the particular star structure of this supply system. Alla and Jarrige (1988) present economic results concerning the use, since 1985, of a similar system in Paris' western areas. They highlight that the benefits, which can be realised with the optimised sequence, lie between 0 and 5 % of the total operational cost of this particular part of the network.

The recent appearance of improved Non-Linear Programming (NLP) algorithms on the market, handling the underlying non-linear equations describing water distribution networks, convinced researchers to rather apply NLP techniques to solve this problem. Chase and Ormsbee (1989), Lansey and Zhong (1990) and Brion and Mays (1991) linked network-simulation models with non-linear optimisation algorithms to determine optimal operations. Yu *et al.* (1994) also proposed a method based on NLP to determine the optimal operation of general water distribution systems containing multiple sources and reservoirs. These researchers state that they could achieve a profit of approximately 10 % on the operational costs, for a 50-node case-study network.

4.2 Modelling of a water distribution network for operational optimisation

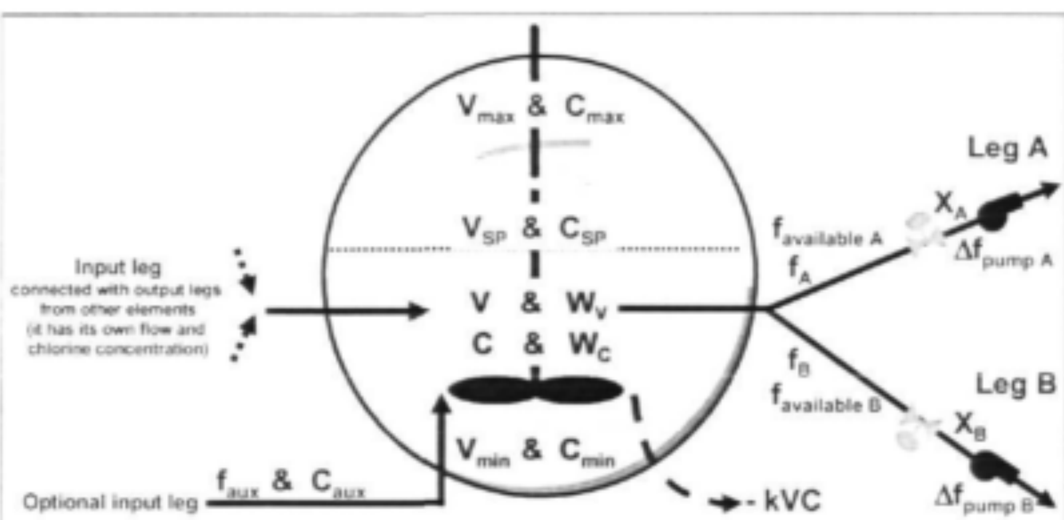


Figure 9 *Representation of the standard modelling element for the available flows strategy*

The modelling of the network of piping and reservoirs was based on a standard modelling element, *viz.* a mixed-flow vessel equipped with one input and two separate outputs. An optional auxiliary input can also be defined. Figure 9 shows the standard element with parameters defined for the *available flows* strategy. These elements were inter-linked to build up the full network description. The first modelling strategy attempted was based on *calculated flows*. Friction losses and available heads were accounted for in a hydraulic model that gave a detailed pressure-balance. Initial solutions were limited to rather small networks, and it was difficult to get true integer solutions for the binary variables – convergence was not guaranteed.

The second modelling strategy, known as the *available flows* strategy, was much easier to solve, since it is based on a model where the pipe network hydraulics have been simplified. Here, individual pipe sections are defined to have a maximum capacity – obtainable from the observed rates of change of level in the associated reservoirs. As more pipelines and reservoirs attempt to draw from any single pipeline, the total flow can become constrained by capacity. This new strategy was easier to solve, and with the acquisition of the "CPLEX" solver, it was possible to go to a full representation of Durban's Southern Aqueduct.

4.3 Operational optimisation of the distribution network based on Model Predictive Control

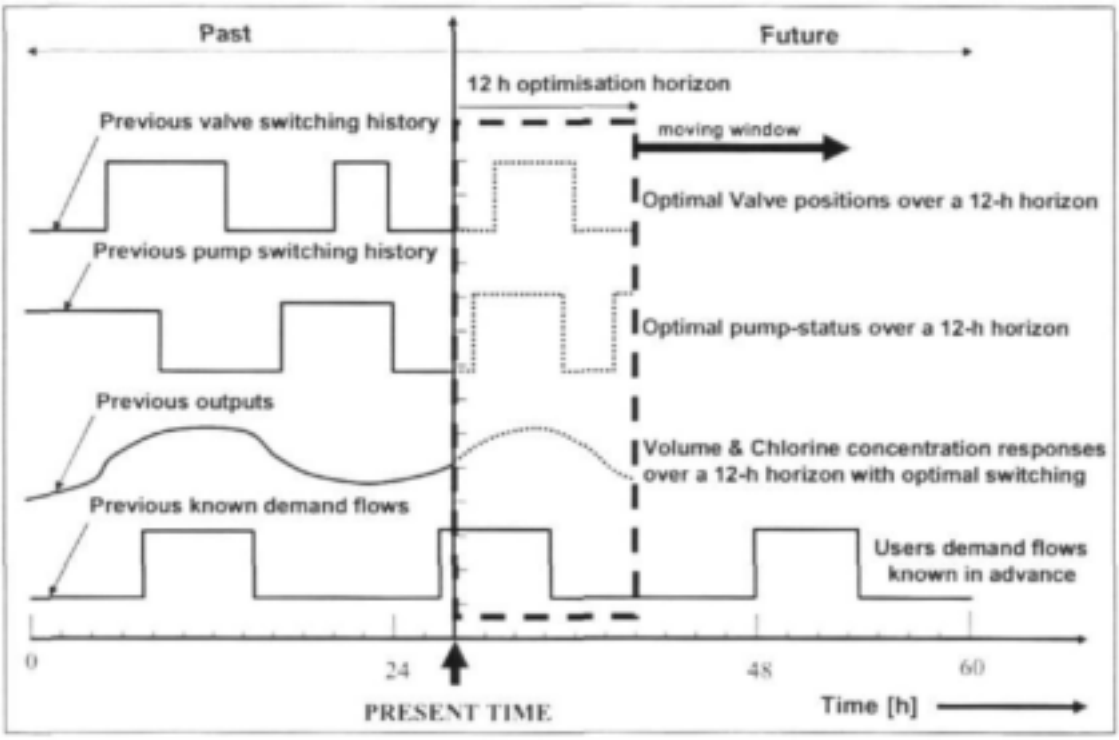


Figure 10 *Model predictive control over 48 hours with a 12-hour prediction horizon*

The objective function, to be minimised by choosing the best settings for all valves in the network, is based on a summation of set-point deviations over the prediction horizon ahead of present time. The normal move suppression factor of Model Predictive Control (MPC) is replaced by a term representing the cost of pumping at the current tariff.

$$J = \left(\bar{V} - \bar{V}_{SP}\right)^T \bar{W}_V \left(\bar{V} - \bar{V}_{SP}\right) + \left(\bar{C} - \bar{C}_{SP}\right)^T \bar{W}_C \left(\bar{C} - \bar{C}_{SP}\right)$$
$$+ \left(\bar{f}^T \, \overline{Pump_1} + \overline{Pump_2}\right) E_{COST}$$

In this expression, volumes and chlorine concentrations, and their set-points, over the prediction horizon, are gathered into vectors, and W_V *and* W_C are appropriate diagonal weighting matrices that respectively penalise squared deviations from volume set-points, and squared deviations from chlorine concentration set-points. $Pump_1$ and $Pump_2$ are vectors containing the slope and intercept of linear pump power relationships with flow, whilst E_{COST} is a scalar representing the relative penalisation for power consumption.

The optimisation strategy is formulated as an MPC problem, which uses a moving-horizon, ahead of present time, as presented in Figure 10. For the 48h solution period, the length of the control horizon used is 12h, so a total of 60h of known feed flow data and known user demand flow data has to be provided at the beginning of each new experiment. The sampling period is fixed to equal periods of 1h.

4.4 Application of operational optimisation to Durban's Southern Aqueduct

The Southern Aqueduct supplies more than a half million people and some large industrial areas in the southern environs of Durban. It links the Durban Heights Water Works to Northdene, Queensburgh, and Chatsworth, and allows the provision of water to the Umlazi and Amanzimtoti areas. eThekwini Water Services' telemetry system links more than 60 telemetry outstations at the various reservoirs and pump stations, and is used to monitor and control the complete Southern Aqueduct system from the central control room at eThekwini Water Services head office in Central Durban.

There are three modes of operation of the Southern Aqueduct, of which only one will be considered here as the basis of operational optimisation. This is the flow scheme in Figure 11, in which gravity flows are used as far as possible to reduce costs.

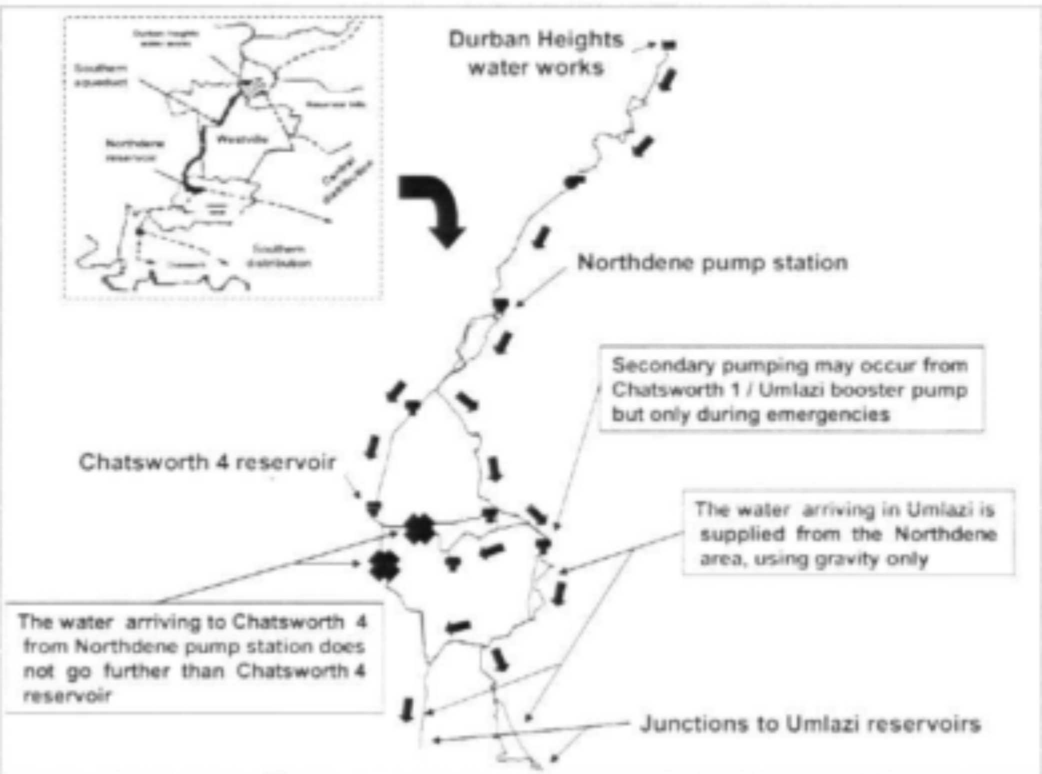


Figure 11 *Minimum pumping mode of the Southern Aqueduct*

The modelled version included 14 reservoirs, 4 pump stations equipped each with a single fixed-speed pump, 8 binary valves, 10 continuous valves and 15 connections with the reticulation system. No post-chlorination is required after the potable water has left the Durban Heights waterworks. Therefore, water quality objectives have no real impact in this particular example of operational optimisation, and therefore they were dropped in favour of objectives concentrating on the satisfaction of volume set-points and the respect of low / high emergency volumes.

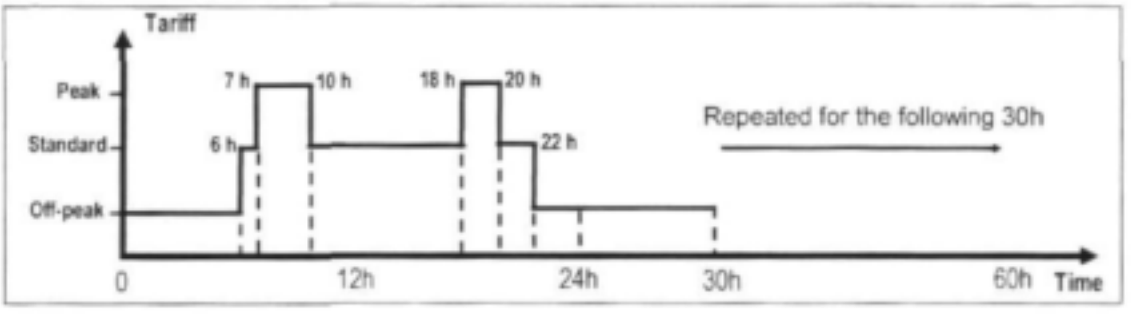


Figure 12 *Electricity cost structures used with the Southern Aqueduct configuration*

The electricity tariff structure in Figure 12 is based on the bulk supply agreement offered to industrial customers by Durban Metro Electricity. The optimisation has been performed with the MINLP solver DICOPT++, associated with the MILP solver CPLEX and the NLP solver CONOPT.

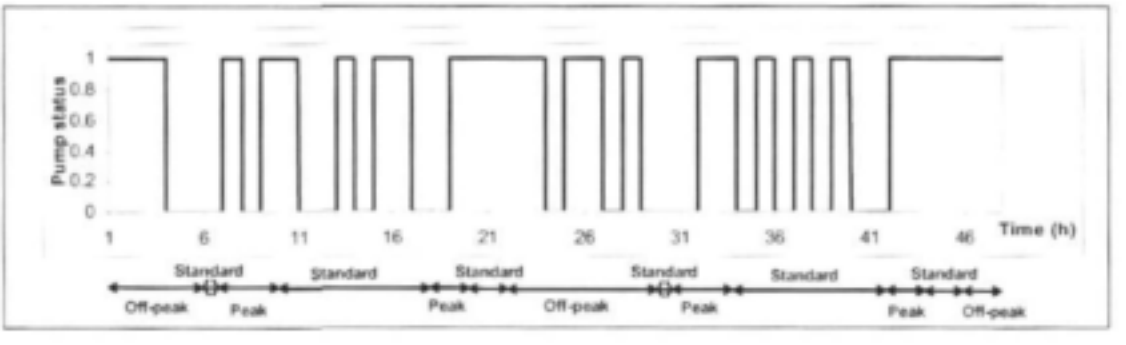


Figure 13 *Pump status at the Lea Drive pump station for the true electricity cost structure*

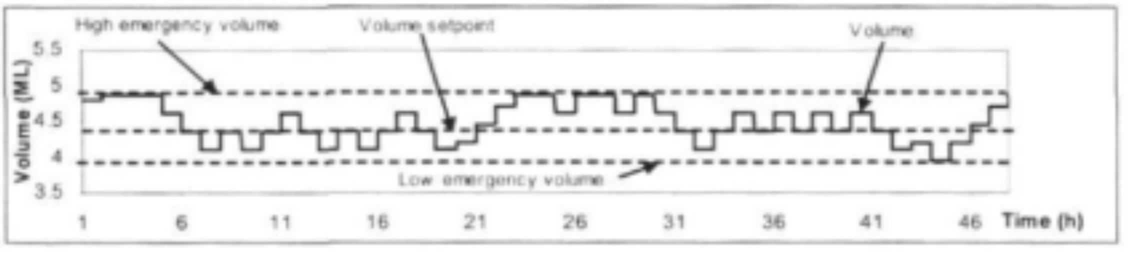


Figure 14 *Evolution of the volume in Lea Drive reservoir (true electricity cost)*

As an example of the optimised behaviour of the network, just the Lea Drive pump and reservoir will be examined (Figures 13 and 14). The pump is on for the first four hours of the simulation: this is to make best use of the off-peak electricity and fill the reservoir while the electricity is cheaper. However, since the volume reaches its top limit before the electricity tariff is switched to standard, the pump has to be turned off. The pump remains in the off position until the 7th hour, where it is used again for one hour. It can be observed at this point that the electricity tariff is charged at the peak rate. Therefore, turning the pump on at this time may appear odd, but there is unfortunately no choice since the volume has become very close to its low-emergency constraint. This action brings the volume closer to its set-point, which justifies stopping the use of the pump. However, the demand out of the reservoir is steady, and interrupting the reservoir feed, by turning the pump off, causes an immediate a drop in the volume. This makes it necessary to switch on the pump again at the next time-step, although the tariff is still at the peak rate. This on / off situation goes on until the 19th hour of simulation and, inside this interval, the optimiser needs to find a compromise between the cost of electricity consumed and the low-volume constraint. Then, the pump remains on until the end of the 23rd hour of simulation. This is to make best use of the cheaper electricity, which is charged in standard or off-peak rates. However, since the volume is quickly corrected by the use of the pump, and reaches its high emergency constraint, the pump must be turned off, although at the end of the 23rd hour the electricity is still at the off-peak tariff. When the second day of simulation starts, the situation in the reservoir is very similar to that at the beginning of the simulation. Therefore, the pump behaviour and volume evolution over the next 24 h are much the same as has just been described for the first day of simulation.

4.5 Conclusions concerning the operational optimisation of a water distribution network

Since operational data were not obtained for the final calibration of the model, no attempt was made at comparing the current pumping sequence employed by Durban Metro with the optimised one. Further, no economic figures could be given on the Southern Aqueduct to confirm the benefit of applying such a method. However, results such as the filling of reservoirs in advance of high demand and electrical tariff, suggest that application of these optimisation techniques on the Southern Aqueduct would lead to a significant reduction in the operational costs.

The modelling method developed needs some adaptation before it can provide *on-line* optimisation of such systems. The computational complexities escalate rapidly as the network size grows. It is now clear that the algorithms currently used, even with the most powerful PC technology available nowadays, would not allow the solving of such big problems, in amounts of time compatible with real-time application. Nevertheless, the existing solution could be used off-line to generate policy approaches for dealing with a series of typical scenarios. An interesting area of future research would be streamlining of the optimisation search, for example by searching for the best fit amongst multiple pre-solved optimal control sequences.

5. TECHNOLOGY TRANSFER

The project aimed to build capacity within the industry to appreciate and apply advanced control techniques, targetting senior managers (for an appreciation of the potential), middle managers (for project identification and management) and engineers (implementers of the advanced control systems

In the case of the chlorination control at the Umgeni Wiggins waterworks, the installed adaptive predictive controller has gained acceptance by the operating staff. In the process of setting up the system and solving a range of problems, it was necessary to work closely with managers, operators, and control/ instrumentation technicians. There is an appreciation of the method there, and some ability to do basic maintenance on it. In the case of the optimised operation of water distribution for the eThekwini Water Services, there were a number of meetings with management, who appreciated the objectives. However, the study was largely into feasibility, and was performed off-line, so no finished software product was handed over.

When the projects first started, there were frequent meetings with the management staff at each institution, to agree on the objectives and facilitate the work. In the case of the chlorination control, liaison progressively moved downwards to those most directly involved (operators and technicians). For the water distribution, the main contact remained at the manager level, with some input by other engineers/technicians, e.g. for Epanet data or SCADA operation.

Unfortunately, the project did not succeed in the aim of training local students to become advanced control engineers. We were not able to recruit local candidates with the requisite mathematical and computational skills at this time. However, two full-time MScEng students from Toulouse, France, were ideally suited from the modelling and control perspective, and were available. One benefit from this has been to strengthen our cooperation with (and access to water and control technology held by) the two home institutions in Toulouse, France: LIPE-INSA (Laboratoire d'Ingénierie des Procédés de l'Environnement, Institut National des Sciences Appliquées) and CNRS/LAAS (Laboratoire d'Analyse et d'Architecture des Systèmes). The project team members have benefitted greatly from the experience and expertise of the French partners.

The overall aim of the project was *to demonstrate the use of advanced process control for integrated management of water and wastewater systems in the South African Water Industry.* This was achieved directly through the implementation of the chlorination control system at the Wiggins Water Treatment Works, and indirectly through the following publications:

Biscos C, Mulholland M, LeLann M-V, Buckley CA, Brouckaert CJ Bailey R & Roustan M	Optimal operation of a potable water distribution network	**2001**	*International Water Association Conference on Water & Wastewater Management for Developing Countries, Kuala Lumpur, 29-31 October, 2001 Association Confer International Waterence on Water & Wastewater management for Developing Countries, Kua*
Pastre A, Mulholland M, LeLann M-V, Buckley CA, Brouckaert CJ, Roustan M & Mabeer V	Control of potable water chlorination	**2001**	*International Water Association Conference on Water & Wastewater Management for Developing Countries, Kuala Lumpur, 29-31 October 2001 Associament for Developing Countries, Kuala Lumpur, 29-31 October 2005*
Pastre A, Mulholland M, Brouckaert CJ, Buckley CA, Le Lann M-V, Roustan M, Naidoo D and Mabeer V	Modelling and Control of Potable Water Chlorination	**2002**	***Water Science and Technology***, **46**, 9, 103-108 (2002)
Biscos C, Mulholland M, Le Lann M-V, Brouckaert CJ, Bailey R, and Roustan M	Optimal Operation of a Potable Water Distribution Network	**2002**	***Water Science and Technology***, **46**, 9, 155-162 (2002)
Biscos C, Mulholland M, LeLann M-V, Buckley CA, Brouckaert CJ & Roustan M	Water Distribution Network Optimisation	**2002**	*WISA 2002 Conference (Water Institute of South Africa), Durban, May, 2002*
Pastre A, Mulholland M, Brouckaert CJ, Buckley CA, LeLann M-V, Roustan M & Naidoo D	Control of potable water chlorination	**2002**	*WISA 2002 Conference (Water Institute of South Africa), Durban, May, 2002*
Biscos C, Mulholland M, Le Lann M-V, Buckley CA, and Brouckaert CJ	Optimal Operation of Water Distribution Networks by Predictive Control using MINLP	**2003**	***Water SA***, **29**, 4, 393-404 (2003)
Biscos C, Mulholland M, LeLann M-V, Buckley CA and Brouckaert CJ	Optimisation techniques applied to a potable water distribution system	**2003**	*2003 South African Chemical Engineers Congress, Sun City, 3-5 September, 2003.*

6. CONCLUSIONS AND RECOMMENDATIONS

The field of control engineering is a wide one with many diverse aspects, such as measurement techniques, sensors, signal conditioners, signal transmission systems, information flow, actuators and control algorithms. In this project the focus was purely on advanced control algorithms. The conclusions drawn here must be seen in this context, they have a firm basis where they relate to issues within this rather narrow focus, but become somewhat more speculative when they relate to other issues.

6.1 Conclusions

Although the project was envisaged as considering both water and wastewater applications, the two studies that were chosen after consultation with Umgeni Water and the eThekwini Municipality concerned only potable water, not wastewater. In South Africa wastewater treatments tend to be designed to operate without automatic control; this might explain the lesser interest in wastewater applications. This is not to suggest that wastewater treatment systems cannot benefit from the application of advanced control systems; however a great deal of infrastructure needs to be in place before one can apply the kind of algorithmic methodology that was the focus of this project. For instance, the measurements that are routinely made at wastewater treatment works to monitor the biological processes do not lend themselves to automatic control.

The chlorination control study led to a successful application of the technique at the Wiggins Water works. However this study did not really address the issue of integrated system control: it amounted to the use of advanced techniques to a control a difficult single process. On the other hand, the water distribution network was very much a complex integrated system. In this case the study only reached the stage of developing the control algorithm and demonstrating the feasibility of the concept. Implementation would have required a much greater logistical effort than for the chlorination control because of the extent of the system, and this required a political commitment and a budget from the eThekwini City Council: something that was not within the power of the project team to secure.

The water industry has lagged in taking advantage of advanced control techniques which are becoming common in the chemical processing industry. Process control does not appear to be considered a high priority in the industry. There is definitely potential to benefit economically and in terms of safety through the use of these methods, as demonstrated in more accurate/appropriate chlorine dosing (minimising excesses), minimisation of the use of high-tariff electricity, or merely complying with the minimum and maximum emergency levels required in reservoirs.

The barriers to implementation are largely related to capacity within the water industry. The belief is quite often expressed that advanced control can compensate for lack of sophisticated operational personnel, however this is only true to a limited extent. Supporting and maintaining the advanced control system requires even more sophisticated personnel, though possibly a relatively small number of them. Thus advanced control is not likely to provide a solution for small, poorly operated systems in the near future. If it is desired to have treatment plants operating under remote control and monitored by a central core of expert staff, then the industry needs to attract control engineers and build up this capacity. However the large cities in South Africa are developing sophisticated integrated systems in response to the need to extend services to as many people as possible while using as few resources as possible. Advanced control clearly has an important role to play in making these systems as efficient as possible.

6.2 Recommendations

Regarding chlorine dosing control

The situation encountered at the Wiggins Treatment Plant must be similar to those at many water works around the country. There is therefore scope to replicate the control scheme in partnership with a commercial vendor of process control systems.

The reasons why the process is difficult to control arise to some extent from the hydraulic design of the system, which was clearly undertaken without controllability in mind. The design of the conditioning reservoirs needs to avoid dead volume and channelling. New installations should be designed to make their control as easy as possible. There may be other units in water treatment that could be better designed nowadays to take advantage of modern process control.

Nevertheless, despite the complex channelling and dead-volume behaviour predicted by CFD, the actual chlorine loss behaviour was quite adequately modelled using a single stirred reactor volume, albeit with a smaller active volume. Very significant variations in the effective chlorine decay rate constant were identified by the Kalman filter – by a factor of up to $2d^{-1}$. Although this was probably an artificial result of the simplified model, it was shown to control the process well in practice. Thus model based control is a viable option where circumstances make an awkward hydraulic design unavoidable.

Regarding water distribution control

The situation encountered in Durban's Southern Aqueduct is also similar in other large cities, so there is potential for the envisaged control system to be replicated widely. Because of the very large power requirements of water distribution systems, the peak load shedding method developed during the project could have significant implications for power distribution management as well as for water management. This could be a very beneficial line of research.

Further work needs to be done to make the control algorithm practically implementable. The main issues that still need to be addressed are reducing the computation time and incorporating feedback to account for deviations from anticipated water demands.

Regarding advanced control in the water industry

There is a growing need for integrated management of resources, of which water is one of the most important. Advanced control methods will be required as part of the management strategy and the water industry therefore needs to build technical capacity in this area. This will involve training and recruiting of control engineers and technicians, and promoting the application of advanced control through forums such as WISA. In the field of wastewater treatment, the philosophy of designing and operating wastewater treatment plants will have to be extensively revised in order to establish an environment which is conducive to the application of integrated control techniques so as to improve performance and reduce costs and environmental burdens.

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THE OPTIMISATION OF WATER AND WASTEWATER TREATMENT AND SUPPLY SYSTEMS USING ADVANCED PROCESS CONTROL TECHNIQUES

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Mr GN Steenveld	Water Research Commission (Chairman)
Ms S Chetty	Water Research Commission (Secretary)
Prof E Boje	University of KwaZulu-Natal
Mr R Rajagopaul	Umgeni Water
Mr R Bailey	eThekwini Water Services.

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Mr M Docrat	Umgeni Water
Mr P Bahrs	Umgeni Water
Mr M Pryor	Umgeni Water

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1. IDENTIFICATION & PREDICTIVE CONTROL OF CHLORINE DOSING

1.1 Introduction

This section presents an overview of the Umgeni Wiggins Water Works. It introduces the problem of controlling the chlorine concentration at the exit of the works in the light of a variable demand.

1.1.1 Umgeni Wiggins Waterworks

Umgeni Water is the largest water authority in Kwa-Zulu Natal. The area of supply covers some 24 000 square kilometres with the main boundaries being the Indian Ocean in the East, the Tugela and Mooi Rivers in the North, the Drakensberg Mountains in the West and the Mkomazi and Mzimkulu Rivers in the South (Figure 1.1).

Figure 1.1 : *Umgeni Water operational area (Umgeni Water, 2002)*

The Wiggins water works is situated in the Cato Manor district of Durban. The design of the plant commenced in September 1980. The works (Figure 1.2) is designed to treat water from the Umgeni River intake, and also water supplied from Inanda Dam. A system of five tunnels and pipelines transfers raw water by gravity. Inanda Dam is the last dam situated on the Umgeni River.

Figure 1.2 : *Wiggins Waterworks aerial view (Umgeni Water, 2002)*

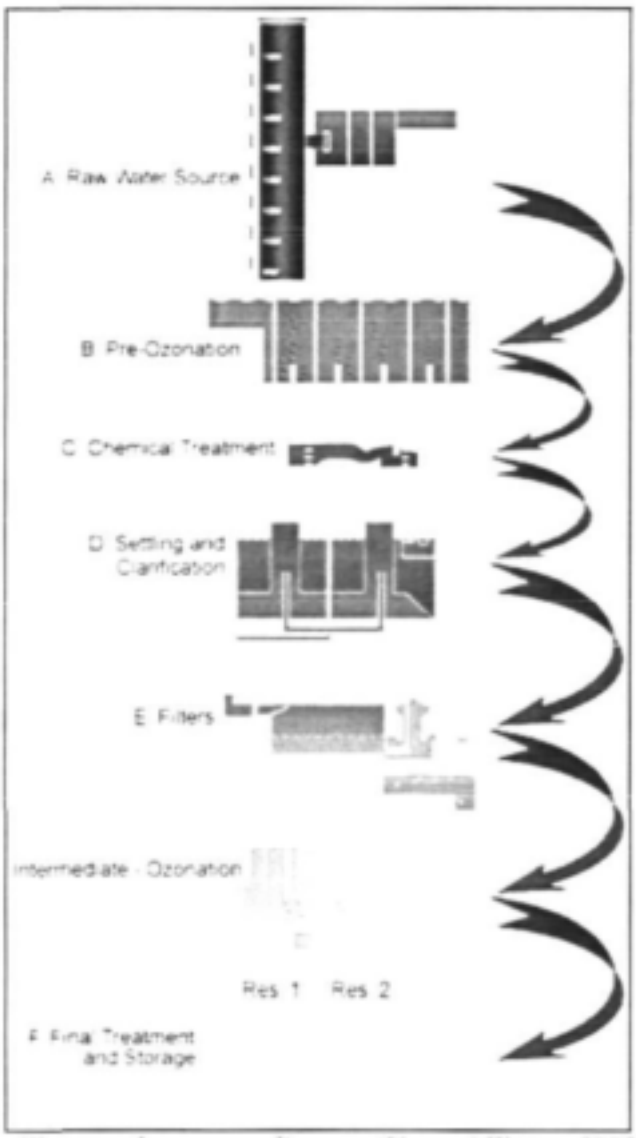

Figure 1.3 : *Wiggins Waterworks process diagram (Umgeni Water, 2002)*

There are several purification steps taken at the Wiggins water works to make water clean and safe for domestic and industrial consumption. The process diagram of the plant is shown in Figure 1.3. The water works gravitates potable water from its 124 ML storage reservoir to southern Durban and adjacent areas. Presently the design capacity of the waterworks is 350 ML/d of raw water. The final water has a pH ranging between 7.8 and 8.2 and a turbidity value below 0.8 NTU.

1.1.2 Chlorine concentration control problem

At the end of the treatment plant, a chlorine contact reservoir, divided into two sections, holds treated water (Step F, Figure 1.3). Because of the variable demand, the reservoir level and residence time vary over a range of values. The maximum combined capacity of the two reservoir sections is 120 ML (7.05 m level), and the minimum capacity is 25.5 ML (1.5 m level). The daily average demand is 140 ML/d, with a daily peak about 220 ML/d. However, this can occasionally be as high as 320 ML/d, making the control of the chlorine dosing difficult. The chlorine residual concentration declines with time, so that the outlet chlorine concentration will depend on the residence time in the reservoir. The chlorine concentration of final water should be kept between 0.8 and 1.2 mg/L: above 0.8 mg/L to ensure it maintains water disinfection within the distribution network and below 1.2 mg/L to prevent taste and odour complaints. Prior to this investigation, the chlorine dosage in the inlet of the reservoir has been manipulated manually based on the experience of the operators.

1.1.3 Aim of Umgeni-Wiggins chlorine control study

The aim of this project was to develop an effective real-time control strategy to predict and control the chlorine concentration at the exit of the plant, and hence allow optimal targeting of the chlorine concentration in water delivered into the Durban Metro region.

Firstly, a Computational Fluid Dynamic (CFD) simulation was undertaken to gain an understanding of the physical processes in the reservoir. However, this kind of modelling is too computationally intensive to be applicable for real-time control. Therefore, three compartment models were created to estimate the kinetic decay factor of the chlorine and to control the outlet chlorine concentration. These models were solved within an extended Kalman filter. Finally, a Dynamic Matrix Control algorithm was chosen to control the outlet chlorine concentration, using the continuously updated estimated kinetic decay factor.

1.2 Related work by other researchers

1.2.1 Chlorine contact reservoir

The World Health Organisation (1984) guidelines state that, to achieve virus free water, chlorinated water should receive 30 min contact time with a minimum free chlorine residual of 0.5 mg/L. On the other hand, the customer will complain about the strong taste of the chlorine if the concentration is above 1.2 mg/L. Therefore managers of drinking water supplies are concerned about the control of the free residual chlorine concentration in the water which leaves the plant.

One of the process approaches that has evolved since the introduction of chlorination is the chlorine contact reservoir. As van der Walt (2002) explained, this reservoir is at the end of the treatment plant and aims to achieve sufficient contact time between the dissolved chlorine and the water.

According to Laubush (1955) and Desjardins (1975), the products used most commonly to obtain disinfection by chlorine are chlorine gas (Cl_2), sodium hypochlorite (NaOCl), calcium hypo-chlorite ($Ca(OCl)_2$), mono-chloramines (NH_2Cl) and chlorine dioxide (ClO_2). Chlorine gas is the most widely used, however sodium hypochlorite is sometimes generated on-site because it is easy to manipulate and safe for the operator.

These two authors emphasise the definition of the different expressions:

- *Combined residual chlorine* is that residual chlorine existing in the water in chemical combination with ammonia or organic nitrogen compounds.
- *Free residual chlorine* is that residual chlorine existing in the water as hypochlorous acid (HOCl) or hypochlorite ion (OCl^-).
- *Total residual chlorine* is the sum of the free residual chlorine and the combined residual chlorine.

However, the term *chlorine* is generally used for *free residual chlorine concentration*. Indeed, it defines the chlorine which is available to disinfect the water, and on-line instruments are available to measure the free residual chlorine concentration. Thus, in the following work, free residual chlorine will be termed as chlorine.

1.2.2 Chlorine contact reservoir kinetics

The chlorine concentration decreases with time because it is consumed during disinfection and can be lost at free surfaces. The chlorine decay in tanks has not been investigated to the same extent as the investigation of chlorine decay in pipelines. Viljoen *et al.* (1997) studied the chlorine decay in pipelines for free chlorine and for monochloramines:

- the general formulation for an n order decay reaction is:

$$rate = -kC^n(t) \qquad (1.1)$$

where $rate$ = loss rate of chlorine (mg L^{-1} d^{-1})
C = chlorine concentration (mg L^{-1})
k = chlorine kinetic factor ($mg^{(1-n)}$ $L^{(n-1)}$ d^{-1})
t = time (d)
n = order of the reaction (-)

- free chlorine decay showed a large variation in decay rate constant (0.96 to 1.2 per day) and in reaction order (0.36 to 1.22)

- a first order decay formulation (equation 1.2) proved a good compromise between accuracy and simplicity

$$rate = -kC(t) \qquad (1.2)$$

Fang Hua *et al.* (1998), Powell *et al.* (1999), and van der Walt (2002) reached the same conclusion and found that the chlorine kinetic factor ranged between 0.01 per day and 6 per day for chlorine decay in pipelines. In reservoirs, water volumes are larger, so short-circuiting and re-circulation will cause deviations from plug-flow chlorine decay, not to mention the presence of air and different containment surfaces. The large surface area provided by pipe walls is considered to play a key role in chlorine reaction in pipelines.

1.2.3 Computational Fluid Dynamic (CFD) Simulation

As van der Walt (2002) explained, several attempts have been made to simplify the design of contact tanks. One way was to base the amount of disinfection on the product of disinfectant concentration and the theoretical residence time based on plug flow. Unfortunately, this conventional approach disregards possible channeling in the contact tank, and therefore often leads to overestimation of disinfection effectiveness, and undersizing of the system.

CFD (computational fluid dynamics) applies general conservation laws such as the conservation of mass and momentum to model fluid behavior. These general laws are applied to specific tank geometries. The solution of a CFD analysis is therefore uniquely defined by the boundary conditions, and provides insight into the internal flow structure, such as currents and stagnant zones. This overcomes the limitation of the conventional approach as highlighted above.

Leclerc and Grevillot (1998) and Chataigner *et al.* (1999) compared CFD results with experimental residence time distribution curves. This comparison showed that CFD results agreed remarkably well with the conventional turbulence models, with errors ranging between 6.7% and 9.3%. Regardless of the source of error, the agreement is sufficient to use CFD not only for qualitative, but also quantitative predictions.

Van der Walt's study (2002) used the FLO++ code (Le Grange, 1998) to simulate the turbulent flow patterns. The Reynolds averaged Navier-Stokes equations and the k-ε turbulence model were used to simulate turbulent flow. Chlorine transport was modeled by a convection-diffusion formulation with a sink term representing chlorine decay. The scalar transport equation is given by

$$\frac{\partial}{\partial t}C+\frac{\partial}{\partial x}(uC)+\frac{\partial}{\partial y}(vC)=\frac{\partial}{\partial x}[(D+\frac{\mu_t}{\sigma_{CX}})(\frac{\partial C}{\partial x})]+\frac{\partial}{\partial y}[(D+\frac{\mu_t}{\sigma_{CY}})(\frac{\partial C}{\partial y})]-kC \qquad (1.3)$$

where	μ_t	=	fluid turbulent viscosity ($m^2\ s^{-1}$)
	σ	=	turbulent Schmidt number (-)
	D	=	turbulent diffusivity ($m^2\ s^{-1}$)
	u, v	=	velocities in x- and y-directions ($m\ s^{-1}$)
	C	=	chlorine concentration ($mg\ m^{-3}$)
	k	=	rate of chlorine decay (s^{-1})

The first term represents the unsteady chlorine accumulation, the second and third terms represent the convection of chlorine, the fourth and fifth terms represent the diffusion of chlorine and the last term represents the first order chlorine decay sink term. The value of k (s^1) represents the rate of chlorine decay and can be adjusted to suit the actual chlorine decay rate. In his study, van der Walt concluded that the CFD is able to model the hydraulics and the chlorine decay accurately.

However this kind of modelling is too detailed and computationally intensive to be applicable for real-time control, so a simplified control model will be required. The Fluent software is also not designed for on-line application.

1.2.4 Control of water treatment processes

The most widely practised disinfection control is to inject an overdose of chlorine at the inlet to the contact tank and adjust this to the desired free residual chlorine level in the effluent stream. However, this method of control may not be optimal. Johnson *et al.* (1997) described reliable predictions of residence time distributions and the use of programmable logic controllers (which are not detailed), coupled with an understanding of chlorine disinfection kinetics, and so offered a potential for a more efficient chlorine dosing algorithm.

Other authors, Sérodes *et al.* (2001), created a methodology for developing decision-making tools for chlorine disinfection control: Chlorocast©. This methodology is based on the creation of a database based on typical situations and the use of an artificial neural network.

These methods work with a well known fixed kinetic factor and residence time distribution curve. However the kinetic factor depends on the temperature and the quality of the water, as well as the initial concentration of free residual chlorine in the flow, which enters the chlorine contact tank (Powell *et al.*, 1999).

The problem is that, during a year, the kinetic factor has to be recalculated, depending on the situation (dry season, rain season, winter, summer). Furthermore, depending on the plant, it is not always possible to obtain a residence time distribution curve.

1.2.5 Extended Kalman filter and parameter identification

The equations describing the contact tank are multivariable and nonlinear with a mixture of differential and algebraic equations (DAE), with state variable outputs (chlorine concentration), inputs (flow, level), associated variables (consumer demand) and physical parameters (chlorine decay factor).

An obvious target here for on-line identification (for the purpose of real-time updating in a control algorithm) is the kinetic rate costant for chlorine loss, which is known to vary with a number of conditions. There is a large number of recursive identification algorithms described in the literature. Ogunnaike and Ray (1994), Åström et al. (1995) and Ljung (1999) overview these techniques. The most popular recursive technique for this on-line identification is the *Kalman filter*. This approach, attributable to R.E Kalman, first appeared in March 1960, and solves the recursive estimation problem. It has been described by Catlin (1980) and Balakrishnan (1984). Chui et al. (1987) used a Kalman filter to estimate the state vector in a linear model, but it can equally be used to estimate the physical parameters in the modelling equations.

If the model is non linear, a local linearisation of the equations can be performed at each time-step. A Kalman filter based on this local linear model is called an *extended Kalman filter* (EKF). The EKF has found many important real-time applications, one of which is adaptive parameter identification. The EKF thus can be used in this way to identify, in real time, the chlorine decay factor.

1.2.6 Model predictive control

The term Model Predictive Control (MPC) describes a class of computer control algorithms which are used to obtain new settings for the manipulated variables based on the predicted future output of the plant (Garcia *et al.*, 1989). MPC technology was originally developed for power plant and petroleum refinery applications, but is now applied in a wide variety of manufacturing environments including chemical, food processing, automotive, aerospace, metallurgy, and pulp and paper. Its popularity can be attributed to three important factors:

- Incorporation of an *explicit* process model into the control calculation. This allows the controller to deal directly with all significant features of the process dynamics.
- The plant behavior is considered over a period which extents to a future time horizon. This means that the effects of feedforward and feedback disturbances can be anticipated and removed, allowing the controller to drive the plant more closely along a desired future trajectory
- The process input, state and output constraints are directly considered in the control calculation, so constraint violations are less likely.

Hence MPC designs have the ability to yield high performance control systems capable of operating without expert intervention for long periods of time.

Clarke *et al.* (1987) as well as Qin *et al.* (1997) described this algorithm. The future moves of the manipulated variables are determined by minimizing the predictive set-point error subject to operating constraints. This optimization is repeated at each sampling time based on update information (measurements) from the plant.

1.2.7 Dynamic Matrix Control

Garcia *et al.* (1989) reviewed different techniques emanating from MPC: Dynamic Matrix Control, Model Algorithmic Control, Inferential Control and Internal Model Control. The DMC algorithm is currently one of the most popular and widely used MPC algorithms, because it is simple, intuitive and allows a formulation of the prediction vector in a natural way. It is based on a linearised step response model called the *convolution* model to predict the effect of possible control actions. Such a strategy enables the model-based control to anticipate where the process is heading.

Successful applications of DMC have been reported in the literature. Cutler and Ramaker (1980) described the DMC algorithm and reported an application to a fluidised bed catalytic cracker. A control algorithm based on the DMC has been developed by Mulholland and Prosser (1997) following the methods of Chang and Seborg (1983), as well as Morshedi *et al.* (1985). This algorithm has been applied to the control of the top and the bottom temperature of a semi-industrial distillation column.

In recent work, Guiamba (2001) examined modelling and control issues for a complex multivariable industrial operator training plant, and developed and applied methods for integrating processes, and for adapting the controller on-line to account for non-linearity.

1.3 The chlorine contact reservoir at the Umgeni-Wiggins waterworks

This section presents the chlorine contact reservoir and available measurements. It considers also the viability of using a tracer test to predict the residence time distribution of the chlorine contact reservoir.

1.3.1 Physical arrangement

In the last stage of the water treatment process (Figure 1.3), the water is chlorinated with NaOCl and is stored in a reservoir, which is divided into two sections. This permits a sufficient contact time between the water and the chlorine for water disinfection. In addition, it provides a holding-time to allow any water quality problem to be corrected before the water enters the distribution network. Figure 1.4 is a view on the top of the reservoir, showing the inspection hole and the instrumentation room.

Figure 1.4 : *View of the top of the reservoir*

Flow through the reservoir

The flow configuration is illustrated in Figure 1.5. From the treatment plant, water proceeds through a pipe to the reservoir. The actual chlorine addition occurs shortly before this pipe splits to supply the two reservoir sections. The ratio of the flow split is unknown. The levels are equal in both sections due to the interconnection of the outlets. For operational reasons, the flow rate through the water treatment process preceding the reservoir is kept as steady as possible. At the exit of this reservoir, the treated water is transferred to the Durban Unicity buffer reservoirs according to the consumer demand.

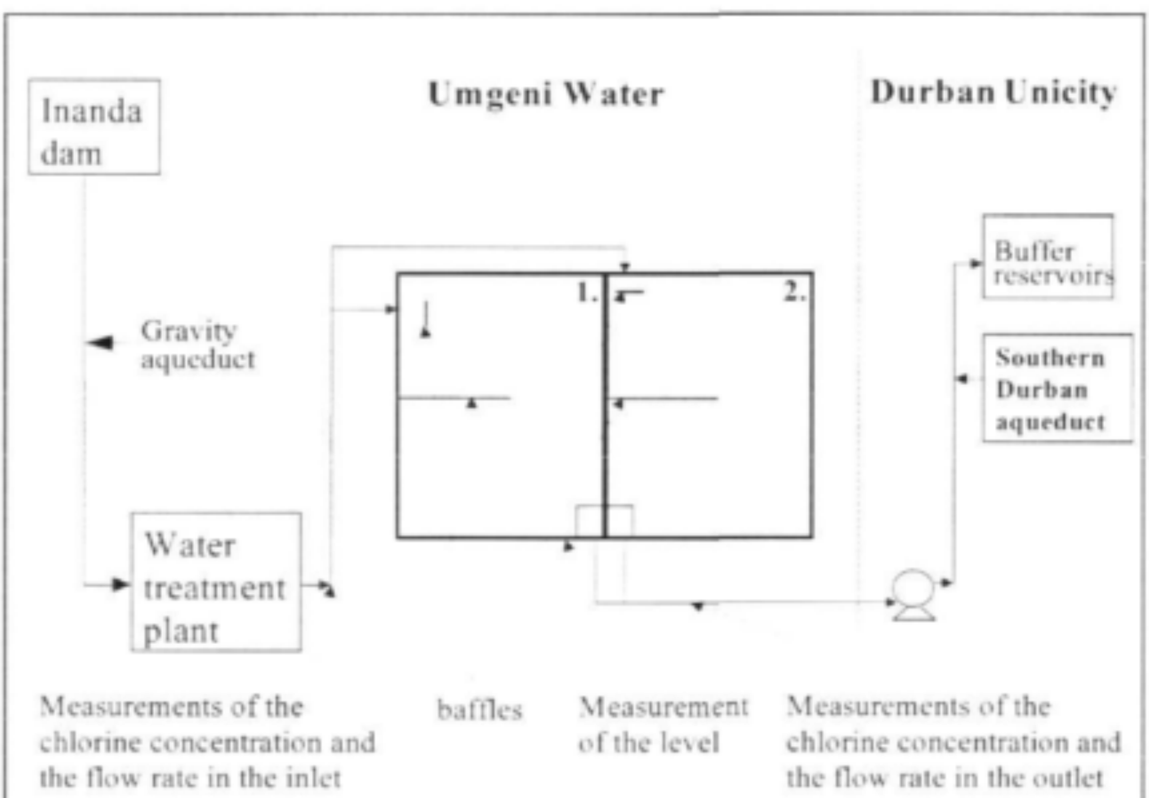

Figure 1.5 : *Plan view of the Wiggins Water treatment plant*

The buffer reservoirs supply water directly to consumers. When any of the buffer reservoirs reaches the minimum level, individual pumps and valves drawing from the main aqueduct are switched on. With many buffer reservoirs drawing water on demand, the water drawn from the Wiggins reservoir will follow a user demand profile with typical diurnal characteristics (Figure 1.6).

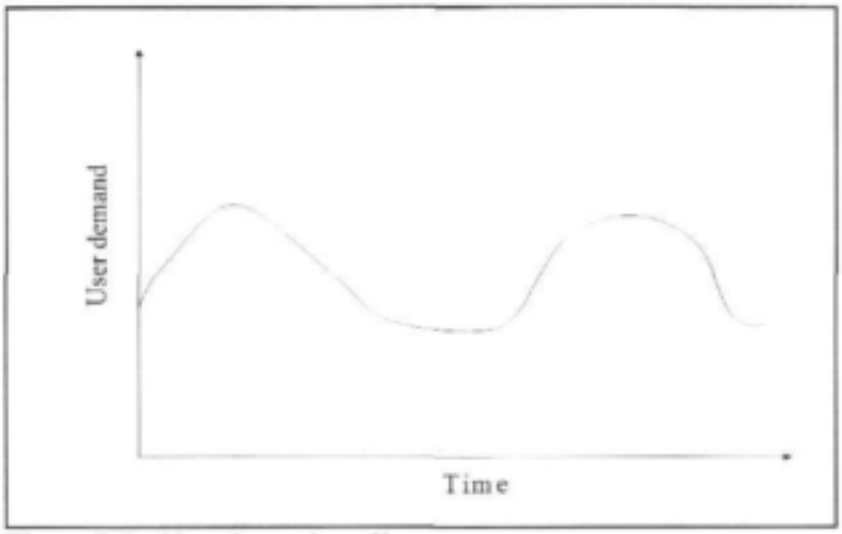

Figure 1.6 : *User demand profile*

However, several large pumps at the Wiggins works are also switched on periodically to transfer water to the Durban Heights Waterworks, which serves a different distribution area due to its greater altitude. When these pumps switch on, there is a noticeable step impact on the Wiggins outflow, with consequent decrease in level (Figure 1.7).

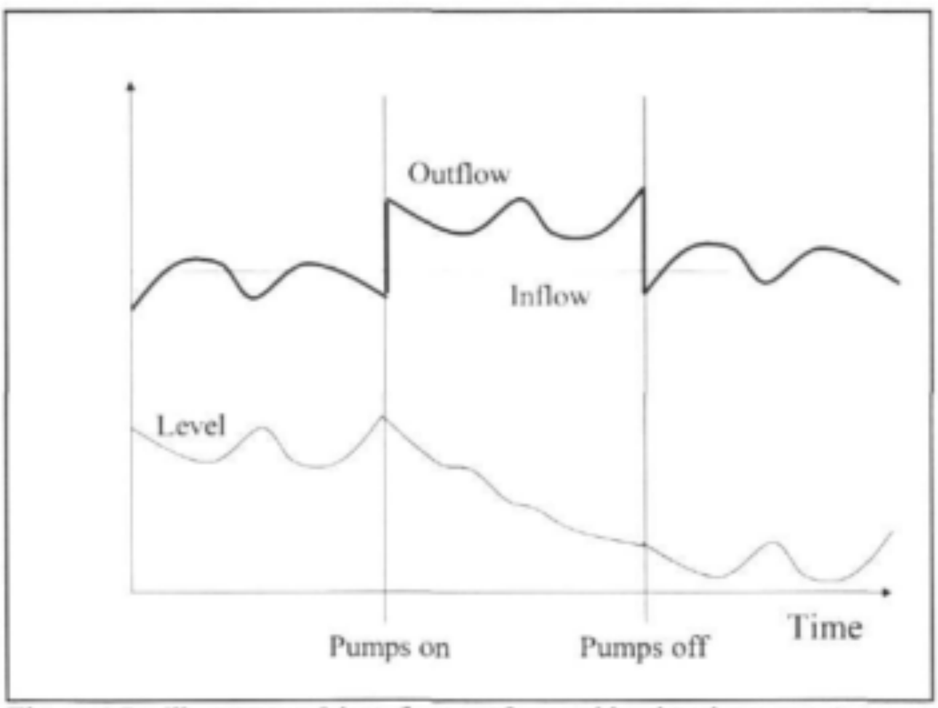

Figure 1.7 : *Illustration of the inflow, outflow and level in the reservoir*

Depending on the difference between inflow and outflow, the water level inside the reservoir may rise or fall. The amount of chlorine required is dependent on the quantity of water present in the reservoir, and the flow rate. The longer the residence time, the more chlorine required per unit of flow out of the reservoir.

Geometry of the reservoir sections

These reservoir sections are large: 116.5 m by 76 m. The wall height is 7 m but the baffles shown in Figure 1.5 are 2 m in height. The smaller baffle is 5 m in length, placed directly in front of the inlet; whereas the larger baffle is 48 m long situated in the middle of each section. The inlets of 1 m in diameter are positioned 1.5 m above the floor and the outlets of 3 m in diameter draw from a trough in the floor.

Instrumentation

The inflow of the reservoir is measured by a crump weir and ultrasonic level device (Milltronic Multiranger Plus, accuracy: ± 0.25% of range, range is set to 900 mm). The total outflow of the reservoir is measured by magnetic flowmeters (Kent ABB, calibrated by positive displacement tests on the storage reservoir, accuracy: ±0.5%). The inlet chlorine concentration to the reservoir is measured by an on-line chlorine analyser (Wallace & Tiernan, calibration against DPD method, accuracy: repeatable to 0.1 mg/L). The outlet chlorine concentration from the reservoir is also measured by an on-line chlorine analyser (Endress & Hauser, calibration against DPD method, accuracy: repeatable to 0.1 mg/L) (Figure 1.8). An ultrasonic level measurement device measures the level of the reservoir (Milltronic Multiranger Plus, calibrated against the survey detail of the reservoir and on empty distance, accuracy: ±0.25% of range, range used equal to 7.5 m).

Figure 1.8 : *On-line pH meter and Chlorometer at the exit of the reservoir*

A 4 to 20 mA driver board drives these circuits and provides an input voltage for an A/D converter scanned by a PC SCADA (Supervisory Control and Data Acquisition) system (Adroit). This system processes the data, executes control loops and stores data at regular intervals to assist with short-term operational decisions as well as long-term planning.

1.3.2 Data collection

The Adroit SCADA system allows storage of selected measured and derived variables at specified time intervals. The data records of interest in this study (inlet and outlet chlorine concentration, inflow rate, outflow rate and level for the entire reservoir) were saved at 5 minute intervals. Each month (from August 2001 to April 2002), at least one week of data was collected from the plant, analysed using Microsoft Excel and then directly imported to a Matlab file.

Figure 1.9 presents a typical set of data:

- The inflow is maintained as steady as possible
- The outflow exhibits the step characteristic forms that were explained in section 1.3.1
- The level rises when the outflow rate drops, and falls when the outflow is greater than the inflow.

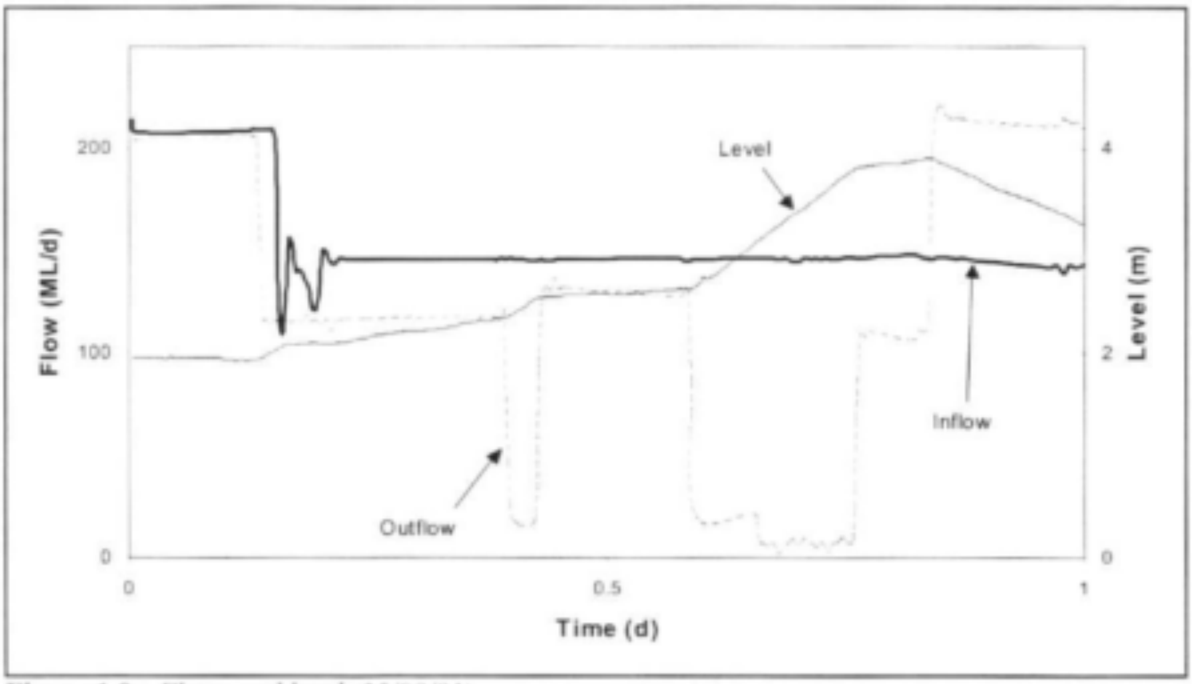

Figure 1.9 : *Flows and level (25/08/01)*

Figure 1.12 shows the chlorine concentration and the level plotted for the same period of time.

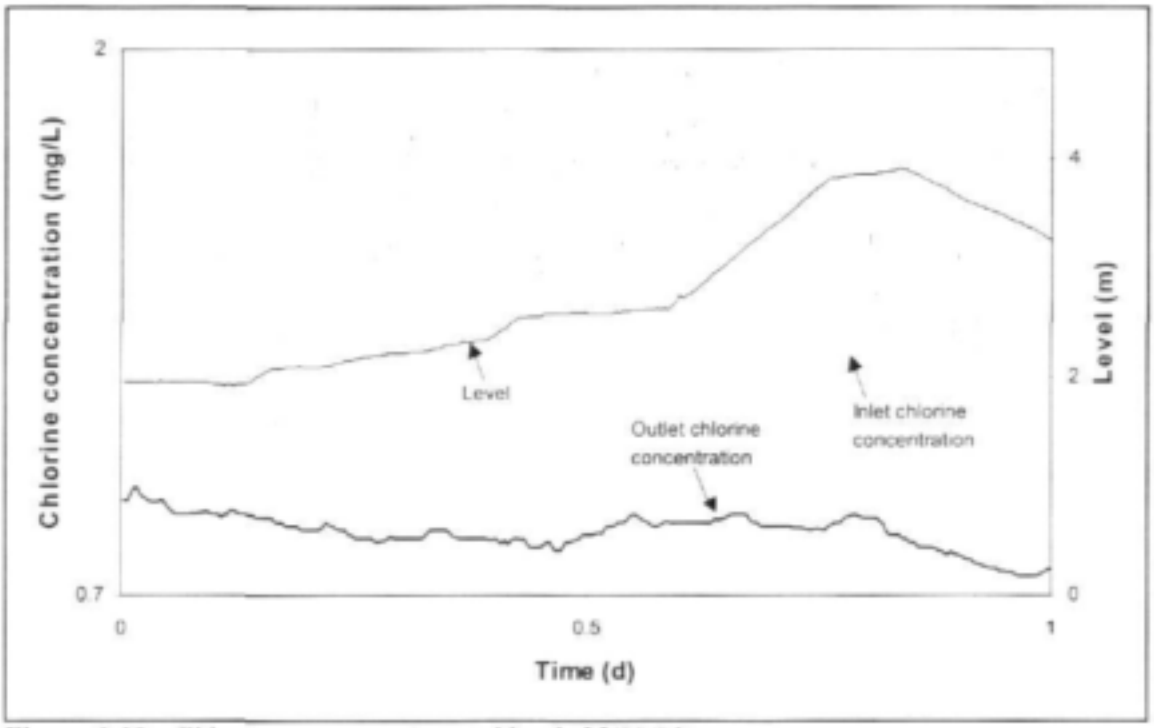

Figure 1.10 : *Chlorine concentration and level (25/08/01)*

Some of the treated water is used to back wash the filters every 2 h. As the flow is measured at the head of works, this will not show on the flow meter. However, the chlorine dosing does not account for this fluctuation in flow, therefore *spikes* occur in the reservoir inlet chlorine concentration as shown in Figure 1.10.

The main problem is that there is no apparent correlation between the inlet chlorine concentration variations and the outlet chlorine concentration variations. Indeed, with a fixed inlet flow rate and a certain range of level and inlet chlorine concentration, the outlet chlorine concentration shows variations which seem unpredictable. Furthermore, the fact that available data are for the whole reservoir and not for the separate sections (which do not have the same geometry) adds to difficulties in predicting the outlet chlorine concentration.

Considering that August is during the cold season in Durban, another set of data during January (representing the hot season) is shown in Figures 1.11 and 1.12.

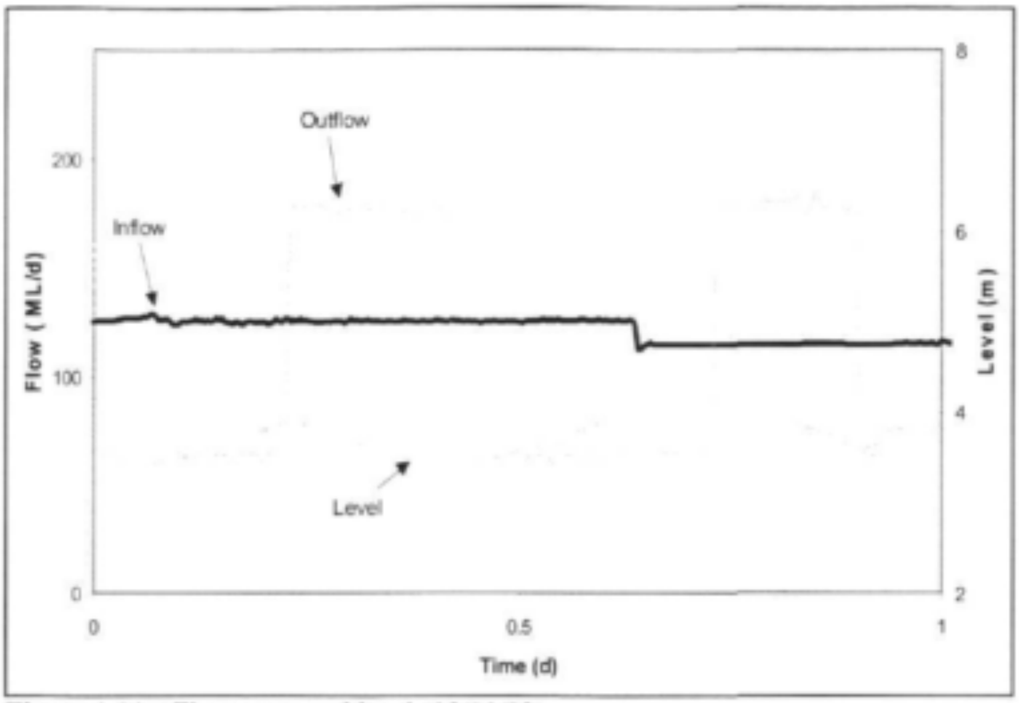

Figure 1.11 : *Flow rates and level (15/01/02)*

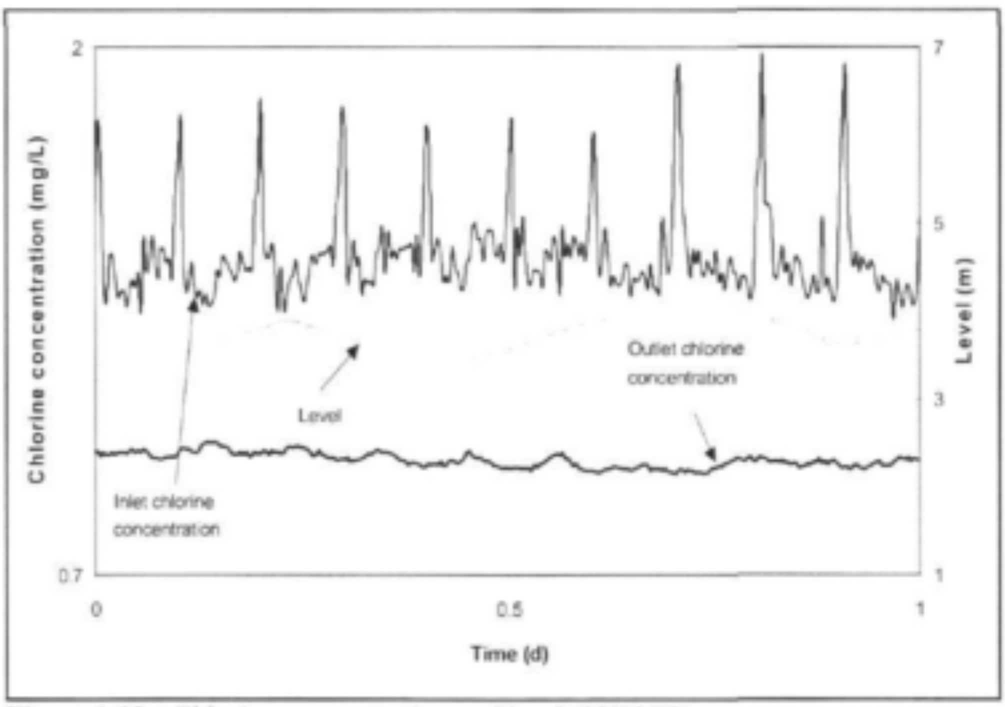

Figure 1.12 : *Chlorine concentration and level (15/01/02)*

The data from the hot season, which is also the rainy season are much more stable. This can be explained by the fact that, as it is the rainy season, Inanda dam is full and a constant inflow can be taken from this dam. As it is the summer and the holiday season, people are using more water but always at the same period of the day so the outflow is subject to constant variations. On the contrary, in winter, people used less water and not regularly so the outflow is much more variable.

1.3.3 Considerations for tracer test

The reservoir sections are poorly designed from the point of view of chlorine concentration control: they are large with just two baffles 2 m in height, whereas the level can reach 7 m. Moreover, they do not have the same geometry and the measurement available for the inlet flow is before the division to the two reservoir sections, the actual split in the flow being unknown. To gain an understanding of the hydrodynamic behaviour of these two sections, tracer tests were considered.

Two methods are available as explained by Environment Quebec (2002)

- Constant rate injection
- Pulse injection

The most stable fluorescent dye in chlorinated potable water is phloxine B, but it is very expensive. Given that the reservoir sections are very large, a large amount of tracer would have been required in carrying out a tracer test. In addition, with changes in the outflow being made frequently, thereby making it impossible to hold steady state for the necessary duration of the test, the idea of doing tracer tests was finally abandoned. To gain an understanding of the physical processes occurring in both reservoir sections, a Computational Fluid Dynamic simulation was thus considered, since this is now able to give results close to reality.

1.4 Computational Fluid Dynamic modelling

In this section, the behaviour of the reservoir is simulated using the Computational Fluid Dynamic, software FLUENT. This study led to the creation of compartment models, which are not computationally intensive but are likely to give a good approximation of the system.

Computational Fluid Dynamics (CFD) seeks to find a solution to the fundamental equations, which describe the motion of fluids. Early CFD code was often written for specialist applications and its applicability was limited to the specific problem for which it was created. Many general purpose CFD packages are now commercially available. This investigation used FLUENT (V4.5 and V5.5) from Fluent Inc (Lebanon, New Hampshire, USA) .

However the FLUENT software is not actually designed to be suitable for on-line control. Within the framework of this project, the aim of doing CFD modelling was to gain an understanding of the physical processes in the chlorine contact reservoir, and not to create a very accurate model for predicting the outlet chlorine concentration at the exit of the reservoir. The insight gained from the CFD model was subsequently used to guide the development of a simplified compartment model for control purposes.

1.4.1 Equations solved

The chlorine contact reservoir is a turbulent system. Turbulent flows are represented by the partial differential Navier-Stokes equations for mass, momentum, energy, and species conservation. To solve these equations, FLUENT divides the space in which the problem is posed into a solution mesh, which consists of a large number of cells, which fill the entire space. The partial differential equations are discretized over the mesh and then solved iteratively. A FLUENT simulation is considered converged when all governing equations are balanced within an acceptable tolerance at each point in the solution domain. In this case, the simulation was considered well converged when the normalized residuals (velocities, pressure, turbulence, eddy dissipation, viscosity, chlorine concentration) were of the order of 1×10^{-5}. In this work, a k-ε turbulence model was chosen. This model is a semi-empirical one that has been proven to provide engineering accuracy in a wide variety of turbulent flows.

This involves two additional partial differential equations for the turbulent kinetic energy (k) and its rate of dissipation (ε). The water level in the reservoir is time dependent. Hence the time dependent option in FLUENT V4.5, that allows a simulation of varying fluid depth using a deforming mesh, was used.

1.4.2 Computational geometry

The reservoir

The dimensions of the reservoirs were provided in section 1.3.1. For each section of the reservoir, computational domains were built with the dimensions of the real section. The features included were the base, vertical walls and baffles, inlets and outlets. To avoid the complexities of a free surface model, the water surface was modelled as a rigid horizontal frictionless surface. A *deforming mesh* was used to represent the change in level as the reservoir filled or emptied with time. This model was not able to represent the situation when the water level dropped below the top of the baffle walls.

To create the deforming mesh, FLUENT needs different meshes with different heights, assuming that the level of water is always equal to the height of the modelled section. An intermediate time calculates an intermediate level by linear interpolation. Therefore, 9 computational domains were built with the same area and different heights for each, which were extracted for the maxima and the minima of the level during 1.6 days. Figure 1.15 shows the real variation of the level during 1.6 days in August 2001 and the linear interpolation. The inflow is taken constant at 100 ML/d.

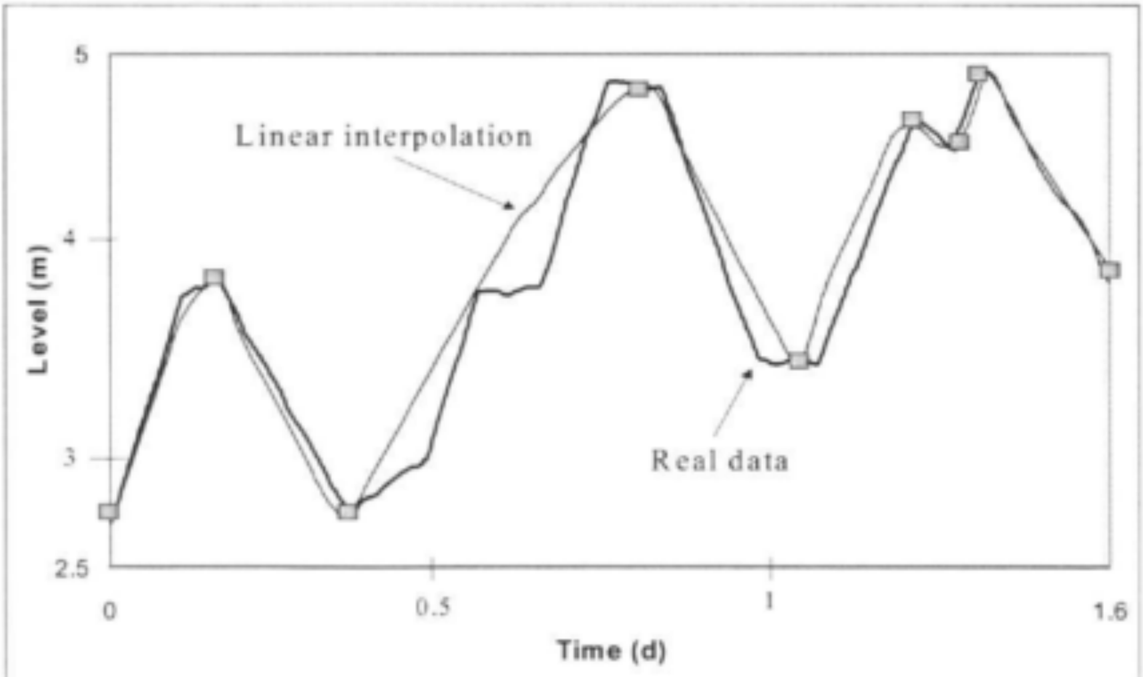

Figure 1.13 : *Level linearisation from 9h00 of 25th August 2001 to 20h00 of 26th August 2001*

Mesh of the computational domain

In order to obtain a converged solution to the problem posed, the computational mesh accuracy has to be considered. In general, the finer the mesh, the more accurate the model is expected to be. However the dimensions of each section are very large, and increasing the number of cells in the mesh will increase the computational time, which must be considered in the light of the limited objectives of the CFD model.

A simple mesh was built with a spacing of 1 m between each node, which produced a mesh containing 55 932 cells, which could be solved in a reasonable time on the available computer. Figure 1.14 shows the mesh for section 1 and the Figure 1.15 shows it for section 2.

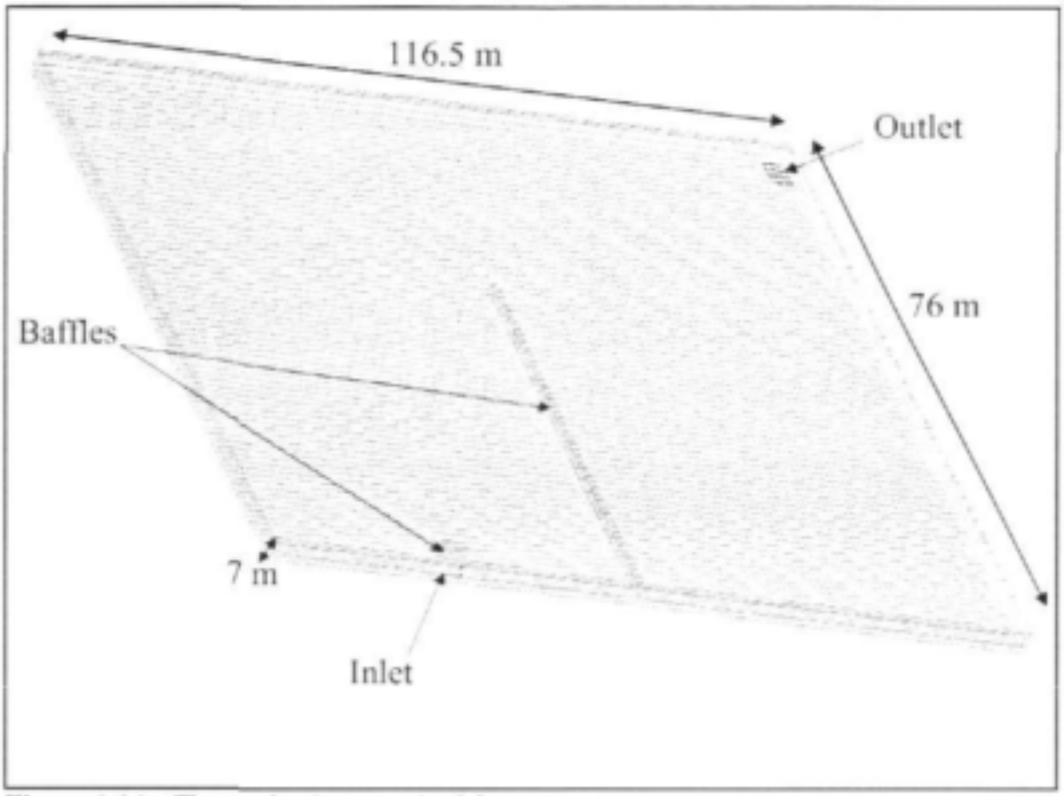

Figure 1.14 : *The mesh of section 1 of the reservoir*

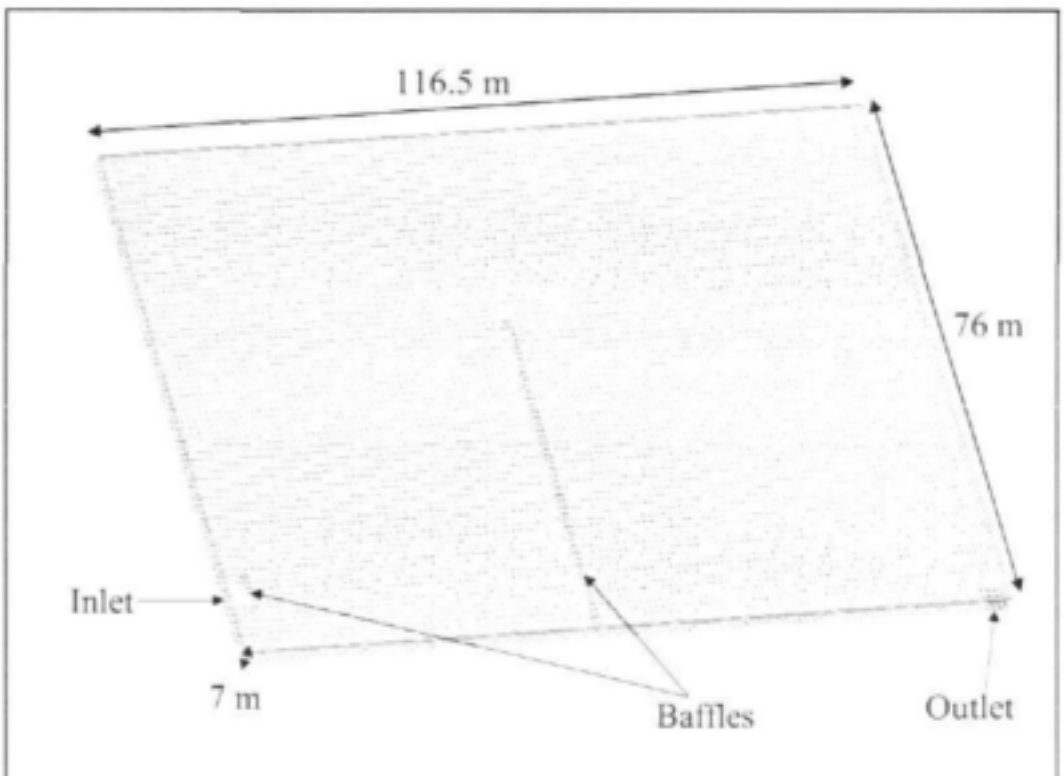

Figure 1.15 : *The mesh of section 2 of the reservoir*

1.4.3 Boundary Conditions and Initial Conditions

Boundary conditions
Fluent provides a standard set of boundary conditions, which can be used to represent inlets, outlets and walls.

Inlet
The FLUENT *inlet velocity* boundary condition was used to define the flow at the inlet. The area of the two reservoirs are equal, the outlets are connected and hence the water levels in the two sections are equal, it was assumed that the flow entering each section was equal to half of the overall flow. The normal flow rate is equal 100 ML/d for this set of data, which means 50 ML/d to each section, i.e. 0.6365 m/s entering through 1 m diameter pipes.

Outlet
The outlets (3 m by 3 m) are pits sunk into the floor of the reservoir. The outlet flow rates were calculated by mass balance bearing in mind the rate of change of volume in the sections.

Wall
The height of the containment walls is 7 m, but the baffle walls are 2 m in height. A *no slip* boundary condition was applied to the walls.

Initial conditions
Time dependent partial differential equations require specifications of initial conditions throughout the domain, and boundary conditions for the entire solution period; but initial conditions of the problem are unknown because they are determined by the entire previous history.

To get around this problem, a *steady state* solution was obtained for the initial height and inflow rate. Thus the steady state was considered as the initial condition for the simulation. Although this steady state was not an true representation of conditions in the actual reservoir at the start of the simulated period, the influence of the initial conditions decreases as the solution time increases.

1.4.4 Simulation results

A simulation of the flow pattern was first undertaken, then FLUENT was used to simulate the chlorine decay inside both sections of the reservoir.

Flow patterns
The section height of all the plan views shown was 3 m (ie. the view is a slice through the reservoir at the height of 3 m independently of the water level; if the water level is below 3 m, the velocity field at the free surface is shown). Since the reservoir sections never operate at steady state, the solution shown in Figure 1.16 cannot be taken as a detailed representation of the flow pattern in practice. However, it represents some kind of average behaviour of the flow. It indicates the presence of preferred pathways (the light areas), particularly in section 1, for flow through the reservoir, creating relatively stagnant volumes (dark areas). The numbers 1 and 2 refer to the sections 1 and 2 of the reservoir respectively.

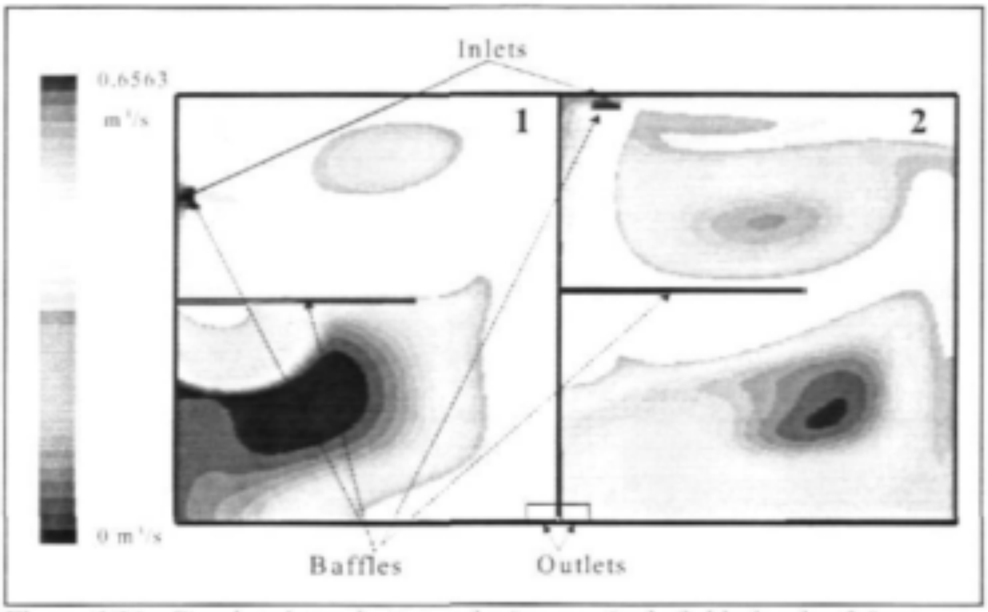

Figure 1.16 : *Simulated steady state velocity magnitude field (level = 2.8 m)*

Starting from the initial steady state solution, the time dependent simulation evolved over a period of 1.6 d as indicated in Figure 1.13. Thus, it was hoped that the dynamic state would approach the state of the reservoir in reality. Figures 1.17 to 1.21 show the variation of flow patterns (at the height of 3 m, or less) in the sections when the level increases with time: from 2.75 m (0.5 d) (Figure 1.17) then 3.83 m (0.7 d) (Figure 1.18) to 4.77 m (1.3 d)(Figure 1.19) and then decreases to 4.09 m (Figure 1.20) to finally reach 3.5 m (Figure 1.21). This particular period of time has been chosen because the simulation has already calculated one rise and one drop of level and so the influence of the initial condition should have largely disappeared.

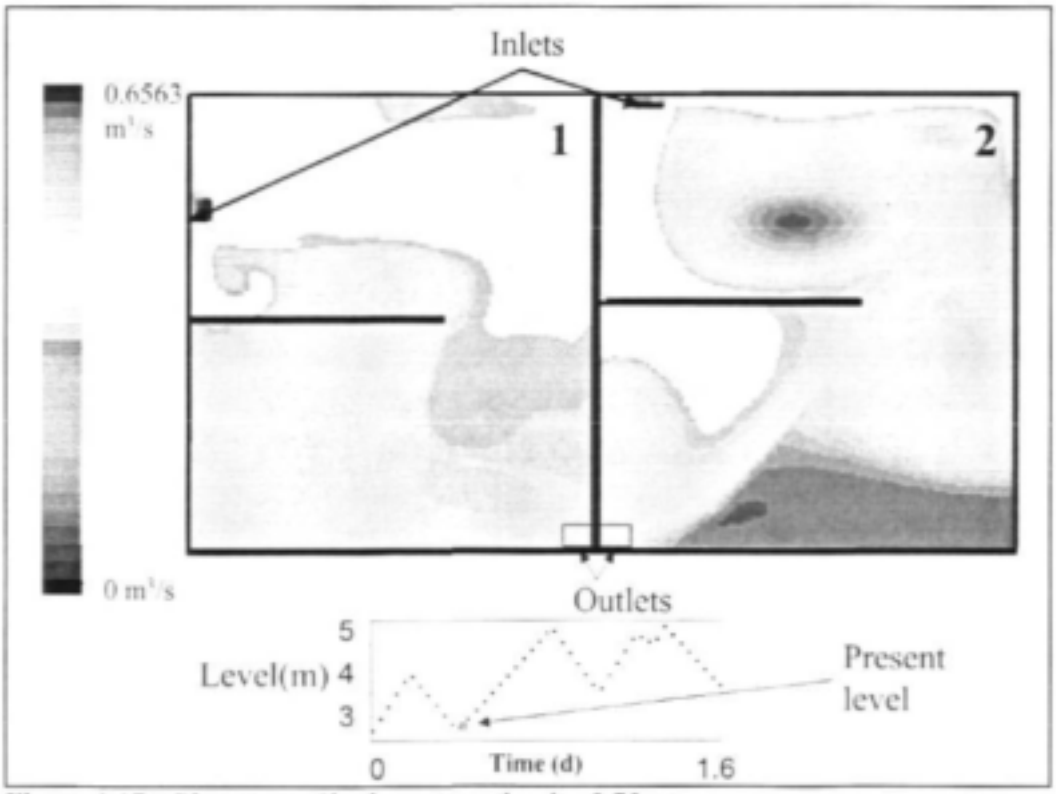

Figure 1.17 : *Plan view of both sections (level = 2.75 m)*

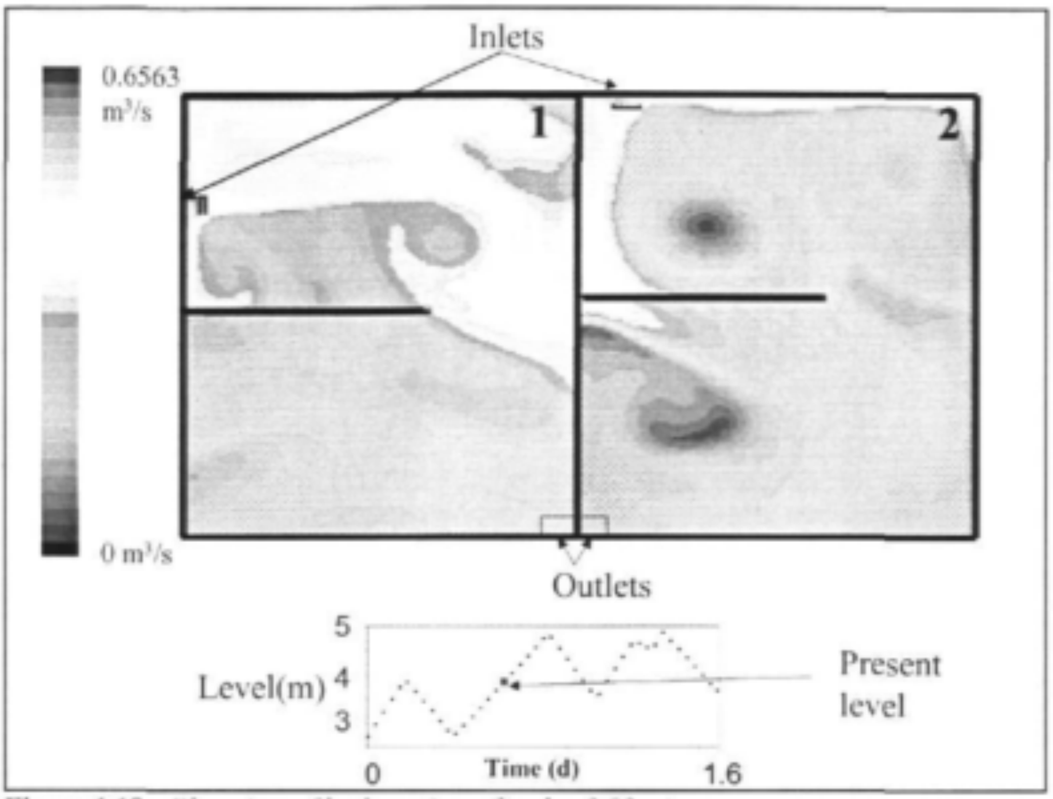

Figure 1.18 : *Plan view of both sections (level = 3.83 m)*

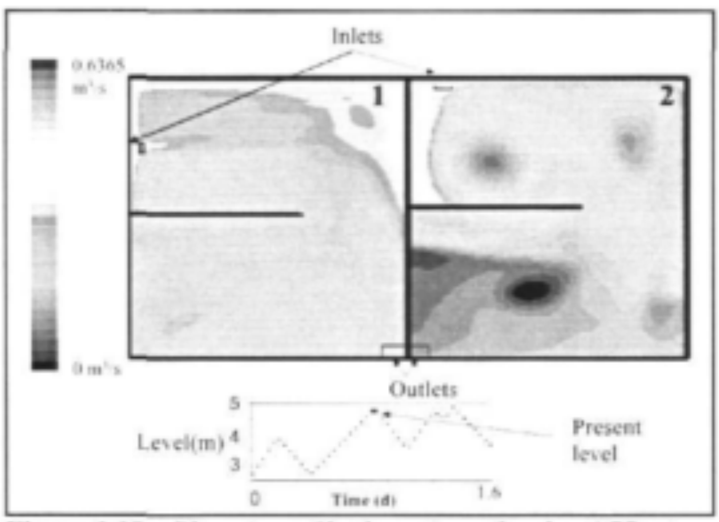

Figure 1.19 : *Plan view of both sections (level = 4.77 m)*

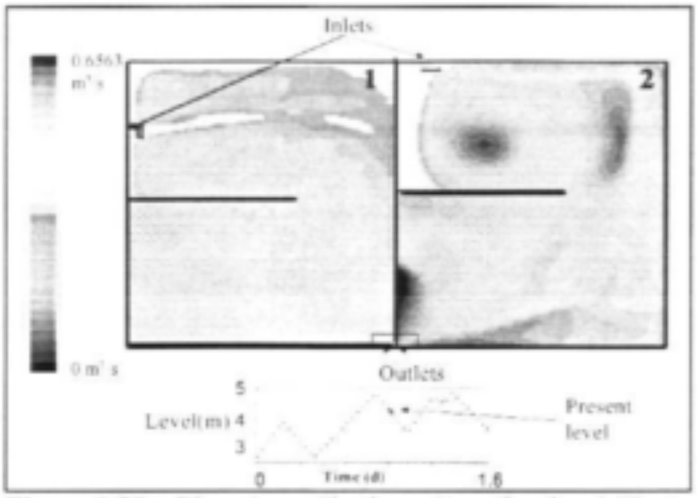

Figure 1.20 : *Plan view of both sections (level = 4.09 m)*

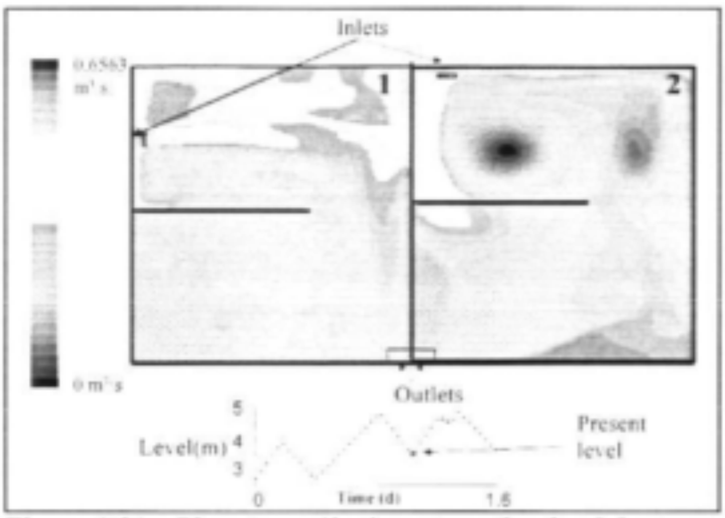

Figure 1.21 : *Plan view of both sections (level = 3.5 m)*

As shown in these figures, when the level increases, the stagnant volumes increase in certain areas, whilst over the whole area the flow rate decreases. That is the consequence of the rising of the level: as the inflow is constant, the outflow is smaller so the water experiences a longer residence time, and the flow rates drop. As the level decreases, the reverse happens.

Chlorine decay simulation

The chemical reaction modelling facilities of FLUENT were used to simulate the behaviour of the chlorine in the reservoir sections. At the inlet, two species were defined: water and chlorine. The inlet chlorine concentration has been approximated by a piecewise linear representation as shown in Figure 1.22 (in a similar way to the level in Figure 1.13).

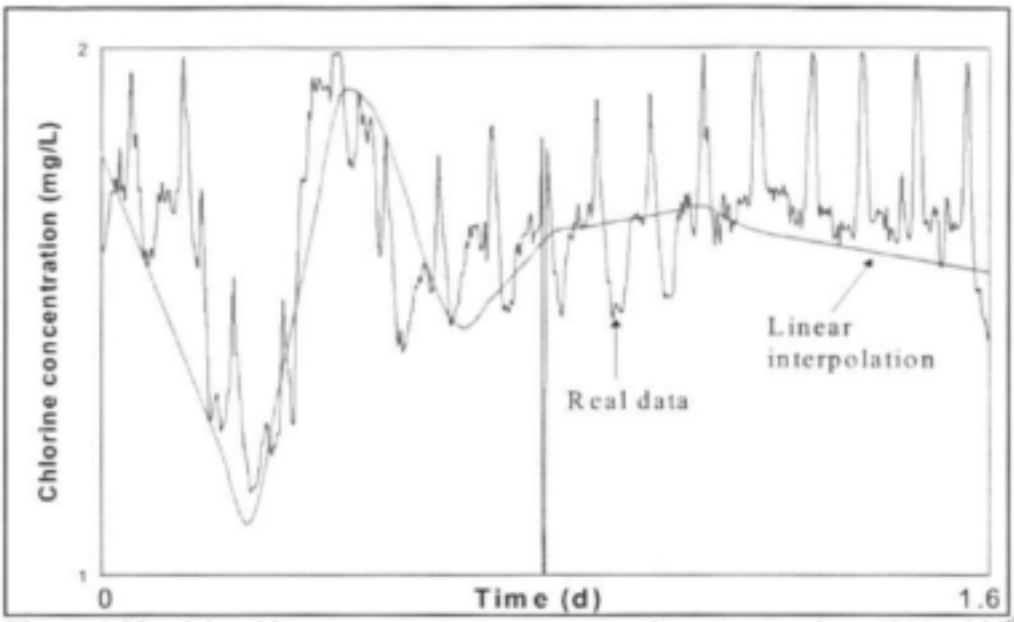

Figure 1.22 : *Inlet chlorine concentration piecewise linearisation from 9h00 of 25[th] August 2001 to 20h00 of 26[th] August 2001*

For the CFD simulation, the chlorine decay consumption was modelled using a first order decay with a kinetic factor equal to 2 d^{-1}. The CFD simulations were repeated with kinetcs factor of 0.5, 1, and 1.5 d^{-1}. Figures 1.23 to 1.27 illustrate the FLUENT simulation with k = 2 d^{-1}.

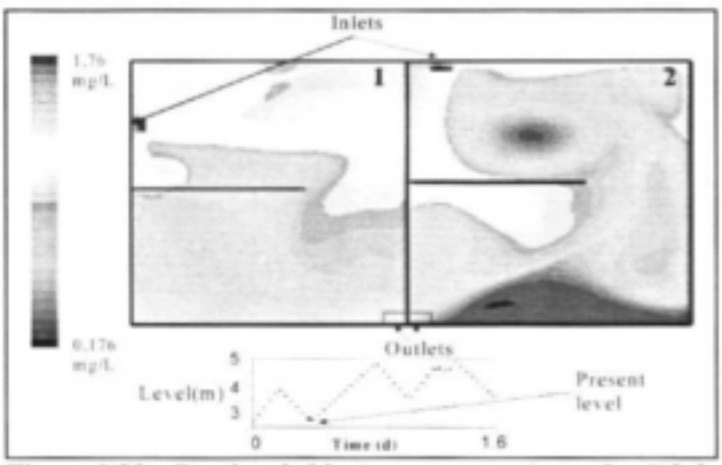

Figure 1.23 : *Simulated chlorine concentrations after 0.5 d*

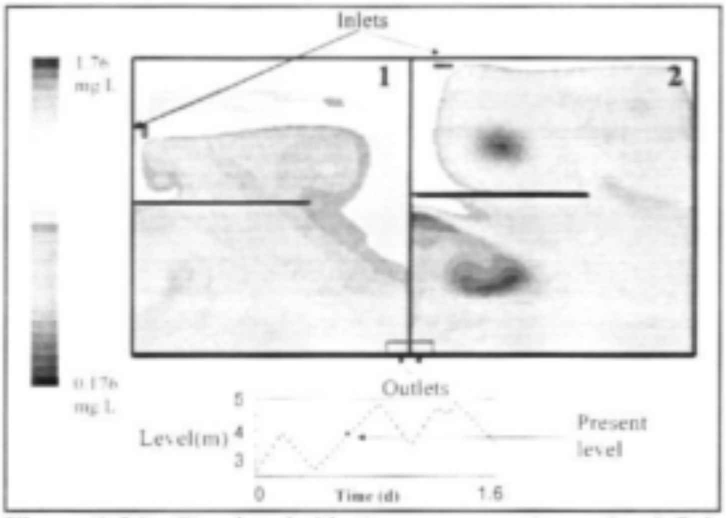

Figure 1.24 : *Simulated chlorine concentrations after 0.7 d*

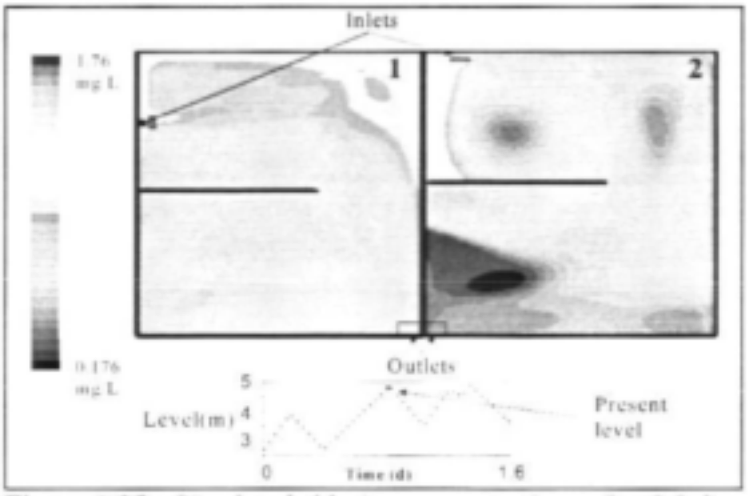

Figure 1.25 : *Simulated chlorine concentrations after 0.8 d*

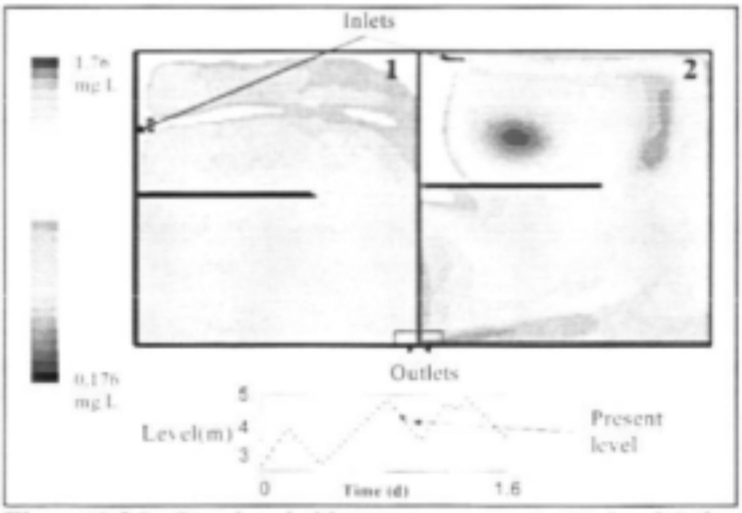

Figure 1.26 : *Simulated chlorine concentrations after 0.9 d*

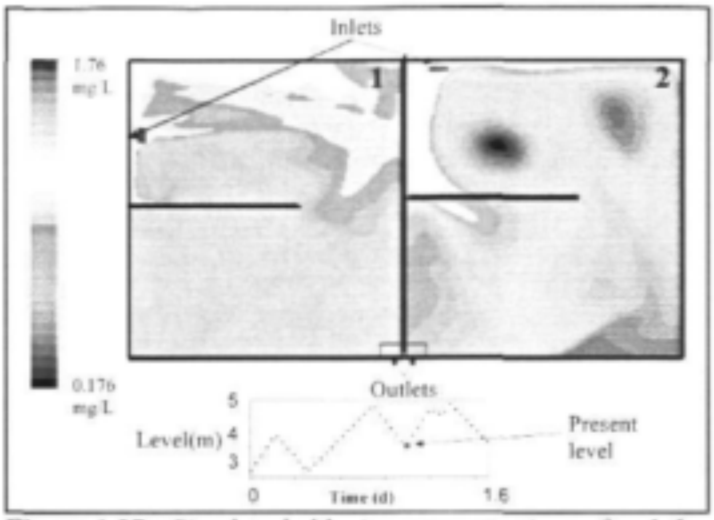

Figure 1.27 : *Simulated chlorine concentrations after 1 d*

As can be seen, the flows through the two sections have preferred pathways. In those parts of the volume which are not on these preferred paths, the water experiences longer residence times, which cause the chlorine to be depleted. The graphs describing the flow pattern or the chlorine concentration inside the sections consequently show similar features. The same conclusion can be reached regarding level: as the level increases, the chlorine concentration decreases in certain areas, whilst the average of the chlorine concentration also decreases (as the level increases, the residence time of the water increases, giving the chlorine more time to react, following the first order decay model). As the level decreases, the reverse happens. The outlet chlorine concentrations predicted by FLUENT is compared with the real measurement data in Figure 1.28.

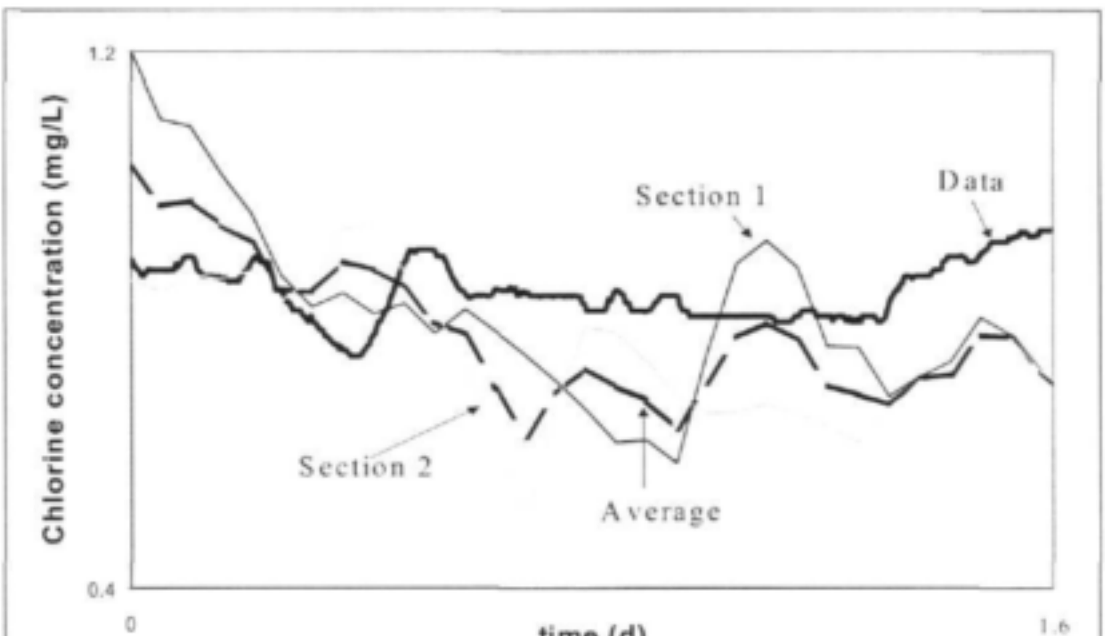

Figure 1.28 : *Comparison between the predicted and measured outlet chlorine concentration*

On these curves, it was observed firstly that the two sections are not giving the same outlet chlorine concentration. At 0.5 d and at 0.9 d of the simulation, the two curves follow the same trend but at 1.3 d, they are completely divergent. These differences are certainly due to the different geometry of the two sections.

As it has been assumed that the inflow is equally divided between the two sections, the average outlet chlorine concentration is the average of the two values obtained for the outlet from each section. The second observation was that the kinetic factor *k* is probably too high (the predictions are below the measurements, which means than too much chlorine has disappeared in the calculation).

Simulations were conducted with different values for the kinetic factor *k* (0.5; 1; 1.5 and 2 d^{-1}). Figure 1.29 shows the average outlet chlorine concentration for each value of the kinetic factor.

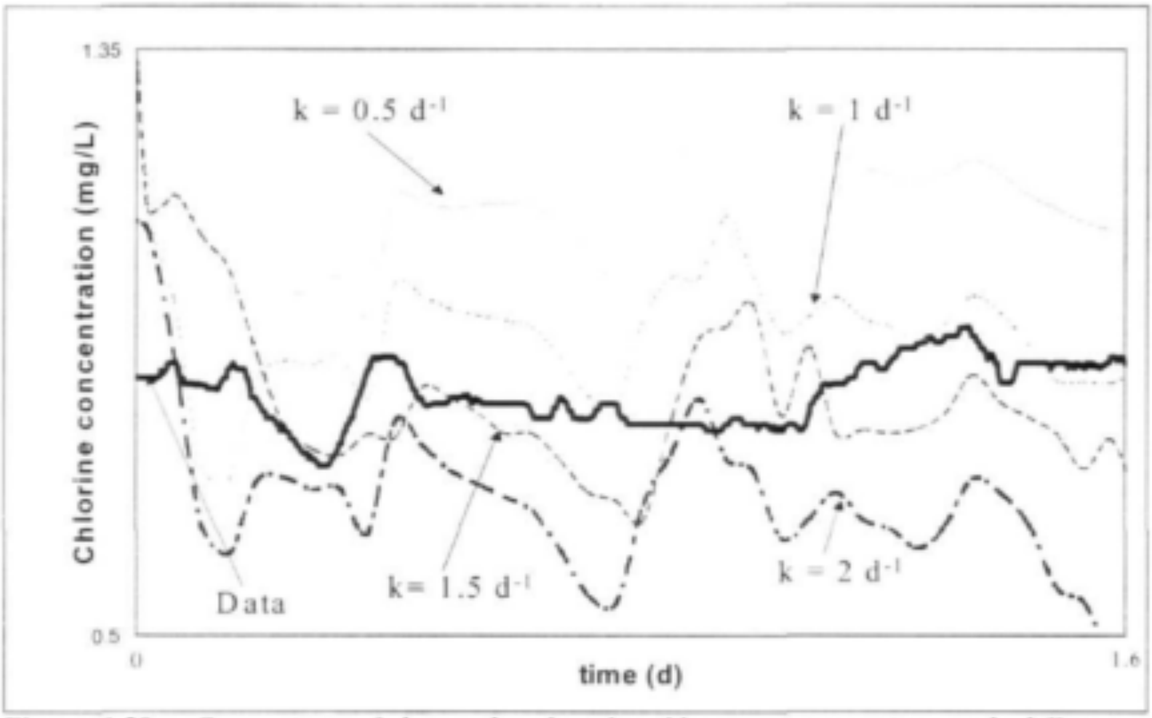

Figure 1.29 : *Comparison of the predicted outlet chlorine concentrations with different values of k*

The best results are obtained with the kinetic factor equal to 1.5 d^{-1} (which was the average value found in a laboratory study, which was conducted after these simulations had been performed (Kandasamy and Govender, 2002)). Although the predicted outlet chlorine concentration is of the same order as the measured data, it does not match perfectly. However a perfect match was not required for the purposes of the study.

1.4.5 Simplified compartment model

Even if CFD modelling gave correct results and a good prediction of outlet chlorine concentration, this kind of modelling was too detailed and computationally intensive to be applicable for real-time control, so a simplified model was required. The FLUENT software is also not designed for on-line application. Thus it was planned to create compartment models using the MATLAB software.

Because of the difficulties associated with undertaking tracer tests (Section 1.3.3), tracer tests were simulated by CFD modelling to obtain residence time distribution curves. By interpreting these curves, different combinations of compartments (well mixed and plug flow), likely to give a good approximation of the system, were found.

Tracer test simulation using CFD

To simulate a tracer test in FLUENT, a certain amount of a non-reactive species was added at t = 0, for just one time step, to give an effective pulse in the inlet concentration. The chlorine inlet concentration was set to 1g/L for the first time step only. The result of the test is presented in figure 1.30. Initial and boundary conditions are as defined in Section 1.4.3.

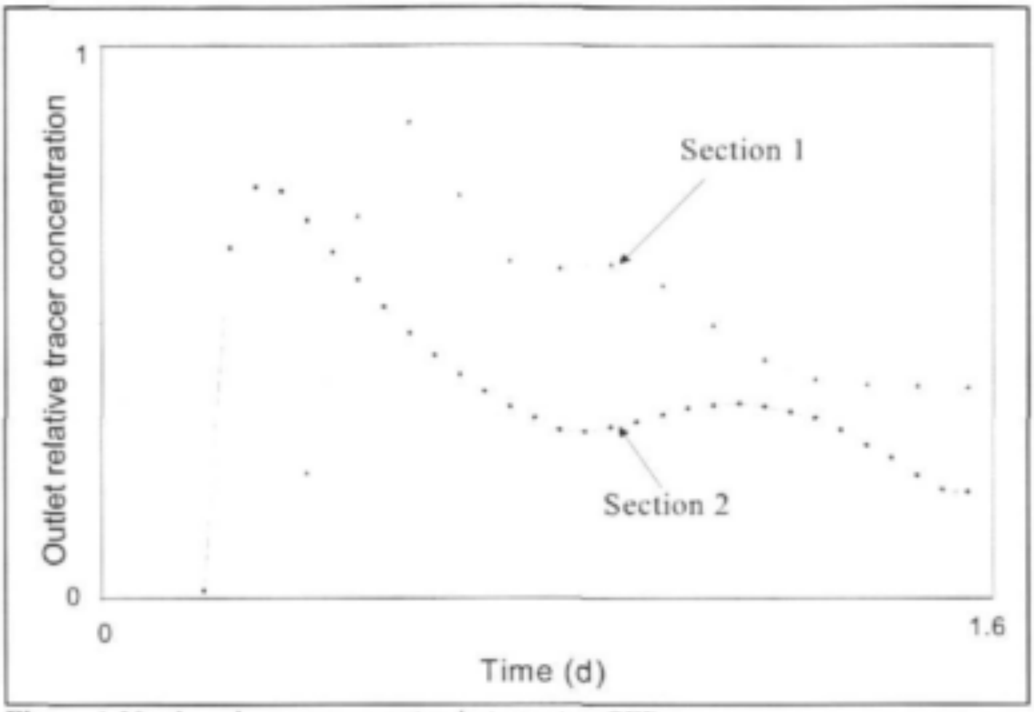

Figure 1.30 : *Impulse tracer test simulation using CFD*

Interpretation

The two peaks shown in Figure 1.30, one for each section, were explained with two different combinations of well-mixed and plug flow compartments.

The first combination is based on dividing each section into two types of compartments. The first type lies on the preferred path of flow, and will be termed the *through-flow compartment* or volume. The second type falls off the preferred path. Thus, it does not interchange water directly with the inlet and outlet, but rather with the through-flow volume. This situation is similar to flow experienced by the backwater of a coastal lagoon, where water flows in and out as a result of the action of the tides, but there is no through flow. For this reason, this type of compartment will be called a *tidal-flow compartment* or volume (Figure 1.31).

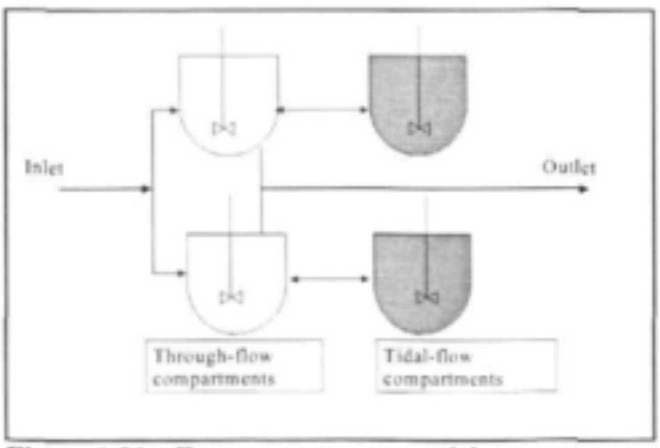

Figure 1.31 : *Four-compartment model*

The second combination is a compartment model with a plug flow re-circulation (Figure 1.32). In practice the plug flow region was approximated as 4 four equal stirred vessels in series, giving a total of six compartments.

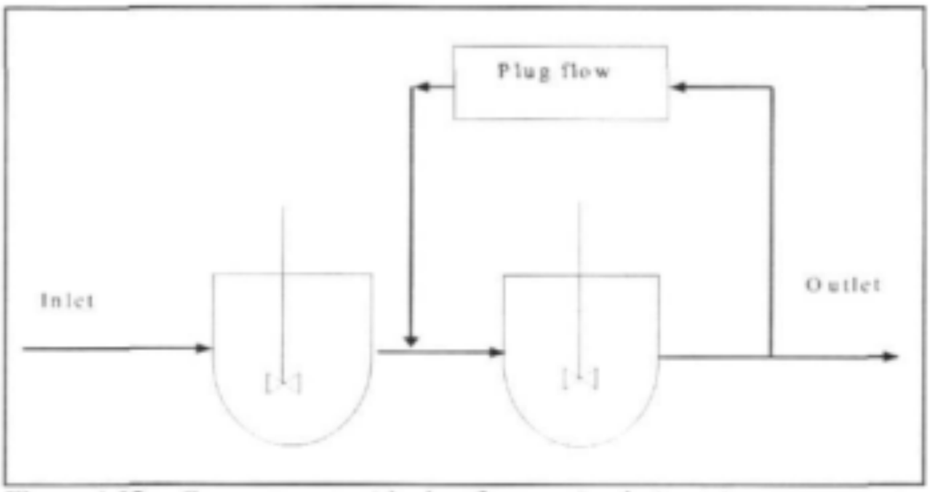

Figure 1.32 : *Compartment with plug-flow re-circulation (six-compartment model)*

These two models were created using MATLAB programming to predict the outlet chlorine concentration, and to test the control thereof. The six-compartment model (Figure 1.32) proved best able to simulate the double-peak RTD arising from summation of the predicted RTD's of the two reservoir sections shown in Figure 1.30. However, it was soon found that a single stirred vessel, though unable to produce the double peak, proved quite adequate for identification and control purposes, and subsequently formed the basis of the on-line system.

1.5 Process modelling

This section describes the single-compartment model, and how it is solved within an extended Kalman filter. Then the extended Kalman filter is tuned, and the results concerning the open-loop simulation are presented.

1.5.1 Single-compartment model

If a mathematical model could be developed to represent the variation of the chlorine concentration in the reservoir, accounting also for the first-order chlorine loss, it could be used to calculate the chlorine dose required to provide the correct residual concentration. In order to implement the model on-line, the reservoir has been considered as a single compartment (Figure 1.33).

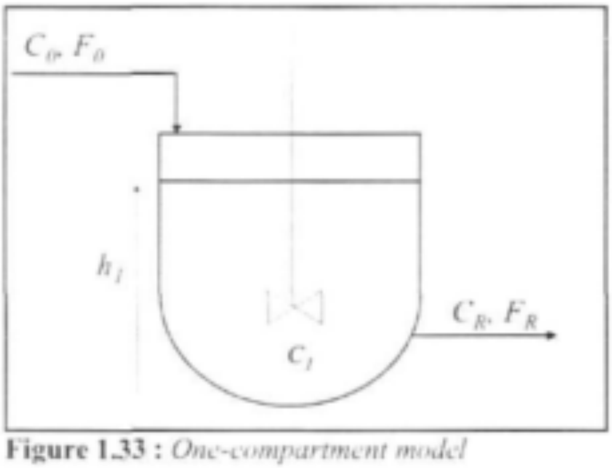

Figure 1.33 : *One-compartment model*

- **Model differential equations**

Volume balance:

$$A_1 \frac{dh_1}{dt} = F_1 - F_0 \tag{1.4}$$

Mass balance:

$$A_1 h_1 \frac{dC_1}{dt} = F_1(C_0 - C_1) - kA_1 h_1 C_1 \tag{1.5}$$

- **Model algebraic equation**

Mass balances:

$$F_R = F_1 \tag{1.6}$$

$$C_R = C_1 \tag{1.7}$$

1.5.2 Extended Kalman filter

The mathematical model described above is non-linear with a mixture of differential and algebraic equations (DAE). The kinetic factor k is an unknown parameter, and it is needed by the model to represent the outlet chlorine variation. The Kalman filter is a stochastic filter that allows the estimation of the states of the system based on a linear model. The extended Kalman filter (EKF) uses local linearisation to extend the scope of the Kalman filter to systems described by non-linear ordinary differential equations (ODE). This scheme has been applied to the state and parameter estimation using models described by ODEs.

Linearisation of DAEs

By applying a Taylor series expansion and truncating after the first order term, the process model is linearised taking into account the DAE nature of the models. The differential equations can be re-written in this form:

$$\frac{dy}{dt} = f(y, z) \tag{1.8}$$

Similarly the algebraic equations are expresses as:

$$0 = g(y, z) \tag{1.9}$$

where y is the vector of state variables, and z the vector of additional variables used in the equations. The differential equations express the unsteady volume and chlorine balances in each compartment, while the algebraic equations express the summation and splitting of flows between vessels, and the dependence of flows on levels.

The EKF algorithm for solution of this system is detailed by Pastre (2003) and Mulholland *et al.* (2004). The Jacobians of f and g, ie. the derivatives of all of their elements with respect to the arguments of these functions, are obtained locally on each time-step by perturbation of the argument variables. Equations (1.8) and (1.9) are expressed as a combined set of differential equations by re-casting (1.9) as a set of differential equations with a fast time-constant.

1.5.3 Use of off-line compartment models to identify k from plant data

Since the compartment models are solved in the form of a Kalman filter, it is a simple matter to supply the model with measured records of the exit chlorine concentration, and to use it to provide instead predictions of the necessary kinetic constant k variation in time which would account for the these measurements (given also the records of flow, level and inlet concentration.

The results are presented in Figure 1.34 for the data period 10/10/01 to 12/10/01, for the three models (1,4 & 6 compartments). In this period the level undergoes a lot of variation but the inlet and outlet chlorine concentrations are fairly steady.

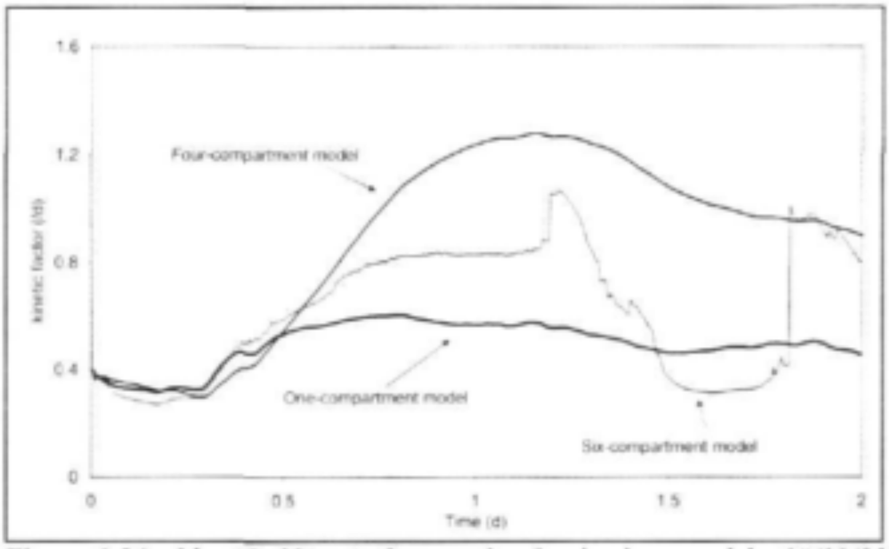

Figure 1.34 : *Identified kinetic factor value for the three models (10/10/01 to 12/10/01)*

To explain the difference between the three curves, it is important to remember that the kinetic factor *k* is directly linked to the volume. The total volume for the three models is the same but it is not shared out in the same way, and hence the water experiences different residence times, resulting in the deduced kinetic factors being different.

It can be seen that the one-compartment model (which is the simplest one) has a kinetic factor, which does not vary much. On the contrary, the kinetic factor for the six-compartment model, which is the most complicated, shows a wider variation. Figure 1.35 presents the identified kinetic factor for the period 15/03/02 to 17/03/02. Here, conditions differ to Figure 1.34: the level is almost steady but the inlet and outlet chlorine concentrations vary a lot.

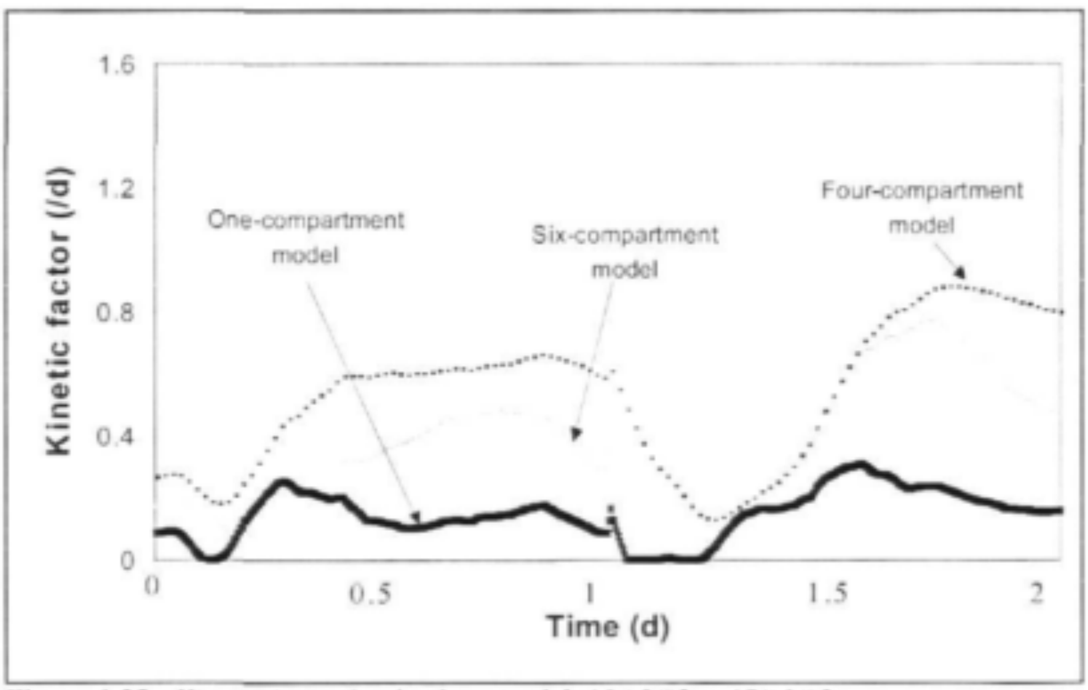

Figure 1.35 : *Kinetic factor for the three model (15/03/02 to 17/03/02)*

Although the relationship between the identified values is similar, the value range of each kinetic factor for the March data is below that for October, which suggests that irrespective of the types of model, the kinetic value changes depending on the period of the year. This is probably due to the quality of the water.

1.5.4 Use of off-line compartment models in a Kalman filter with fixed *k*

Although the model is not required in this form to meet the project objectives, it is instructive to note the performance of the three different compartment models, in Kalman filter format, as they attempt to predicted the outlet chlorine concentration with a fixed kinetic constant, but taking the actual measured outlet chlorine concentration as feedback to the Kalman filter. Fixed *k* values chosen from the above identifications were:

1-compartment	:	k=0.20 d^{-1}
4-compartments	:	k=0.55 d^{-1}
6-compartments	:	k=0.45 d^{-1}

The tuning of the filter is based on the expected error in outlet chlorine predictions compared to the expected error in the outlet chlorine chlorine measurements. Best results were obtained by setting the ratio of these two errors, the *measurement sensitivity* ε, to unity.

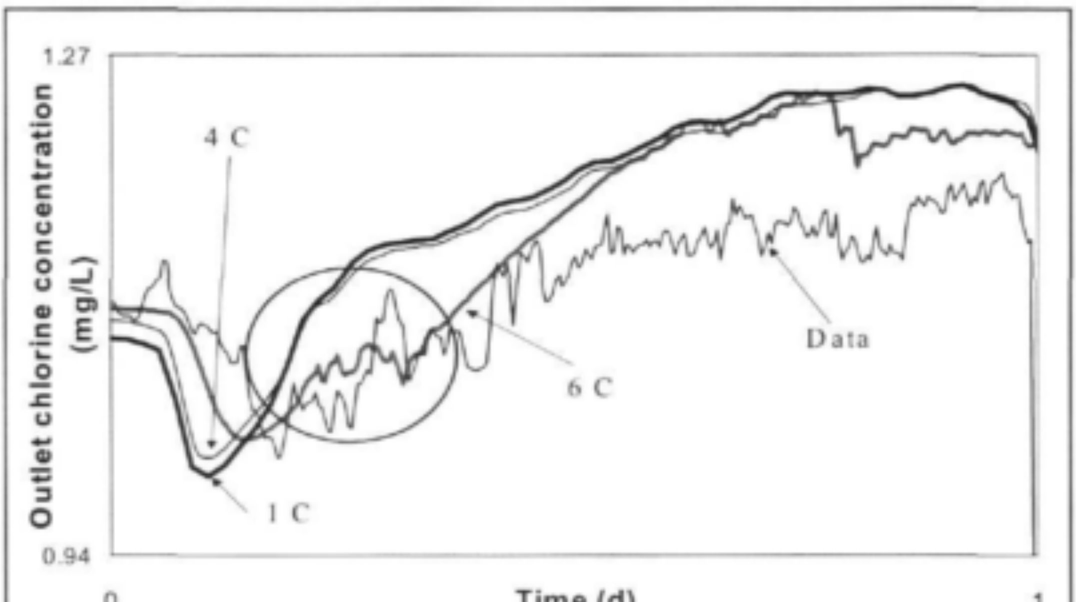

Figure 1.36 : *Comparison of the Kalman-filtered predicted outlet chlorine concentration for the 1.4 and 6-compartment models*

The three models have the ability to predict the outlet chlorine concentration quite closely (Figure 1.36). The six-compartment model (C = 6) is the best one, considering that it predicts the detailed variation of the observed outlet chlorine concentration (circled in Figure 1.36). On the other hand, because it is the simplest one, the one-compartment model is more robust than the others. The one-compartment algorithm was therefore selected to be used online, for its simplicity and its ability to predict the outlet chlorine concentration adequately.

1.5.5 Dynamic Matrix Control of 1-compartment model

Before going on-line with the algorithm, the dynamic matrix control (DMC) was first attempted off-line, using the 1-compartment model to simulate the process. The background to DMC has been presented in section 1.2.7. Its main advantage in this application is its ability to deal with dead-time, and arbitrarily long response times in the process. In the case of the single compartment, the necessary step-response for construction of the dynamic matrix is obtained as below by stepping the inlet chlorine concentration (Figure 1.37).

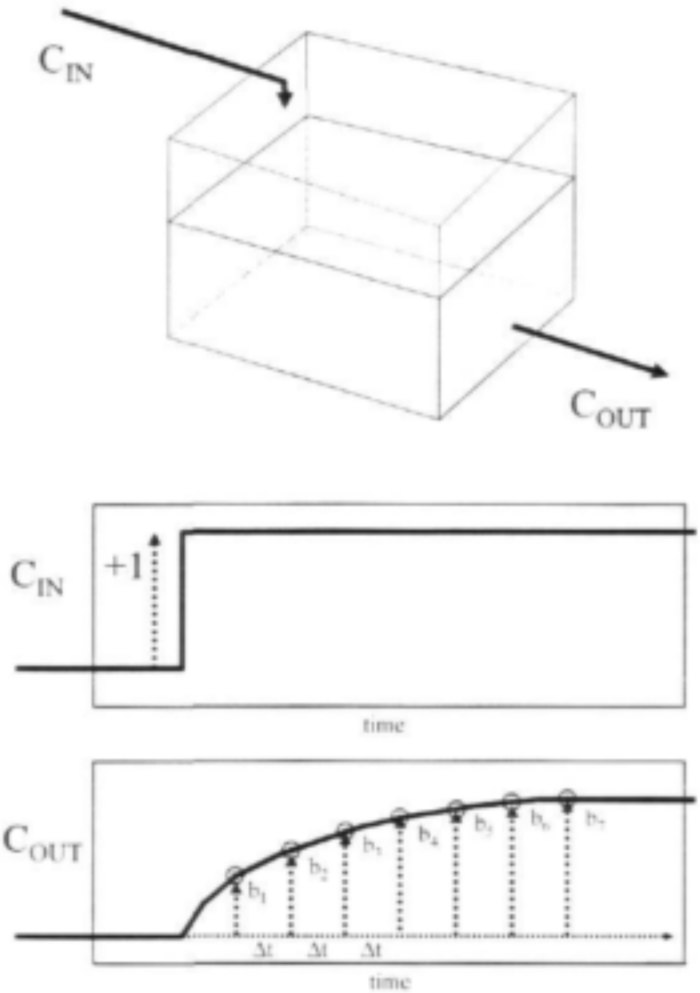

Figure 1.37 : *Inlet chlorine concentration step to obtain outlet step response for DMC*

Then the outlet concentration variation for a series of "moves" ΔC_{INi} in the inlet concentration is predicted using

$$\begin{pmatrix} C_{OUT1} \\ C_{OUT2} \\ C_{OUT3} \\ C_{OUT4} \\ C_{OUT5} \\ C_{OUT6} \\ C_{OUT7} \end{pmatrix} = \begin{bmatrix} b_1 & 0 & 0 & 0 & 0 & 0 & 0 \\ b_2 & b_1 & 0 & 0 & 0 & 0 & 0 \\ b_3 & b_2 & b_1 & 0 & 0 & 0 & 0 \\ b_4 & b_3 & b_2 & b_1 & 0 & 0 & 0 \\ b_5 & b_4 & b_3 & b_2 & b_1 & 0 & 0 \\ b_6 & b_5 & b_4 & b_3 & b_2 & b_1 & 0 \\ b_7 & b_6 & b_5 & b_4 & b_3 & b_2 & b_1 \end{bmatrix} \begin{pmatrix} \Delta C_{IN1} \\ \Delta C_{IN2} \\ \Delta C_{IN3} \\ \Delta C_{IN4} \\ \Delta C_{IN5} \\ \Delta C_{IN6} \\ \Delta C_{IN7} \end{pmatrix} \qquad (1.10)$$

The DMC algorithm accounts for the contribution of past moves to the *open-loop* future values of the controlled variable (C_{OUT}), and calculates the optimum sequence of *future* control action moves ΔC_{INi} which will minimise deviations from a planned setpoint trajectory up to a given horizon (in this illustration, 7 steps ahead). Only the first move is actually implemented, and the entire computation is repeated on the next time-step. Industrial versions of DMC also handle contraints on manipulated and controlled variables optimally.

The DMC algorithm minimises the weighted square deviations from set-point together with the weighted square moves (move suppression). Experiments in tuning of the control of the 1-compartment model suggested a *gain* of 10000 on the squared set-point deviations, compared to 1 on the pump flow rate *moves*, for best control (ie. 10000×(chlorine range 0-1)^2 + 1×(pump range 0-100)^2). Figure 1.38 shows the quality of control using a 0.035 day (10-step) and 0.070 day (20-step) optimisation horizon.The observed plant *Inlet Cl_2* Concentration (manipulated variable: MV) and observed plant *Outlet Cl_2* Concentration (controlled variable: CV) are shown up to time = 1day, at which point the contoller is switched to automatic control, and thereafter the model-predicted CV is shown in response to the MV strategy determined by the DMC algorithm, as it controls the model.

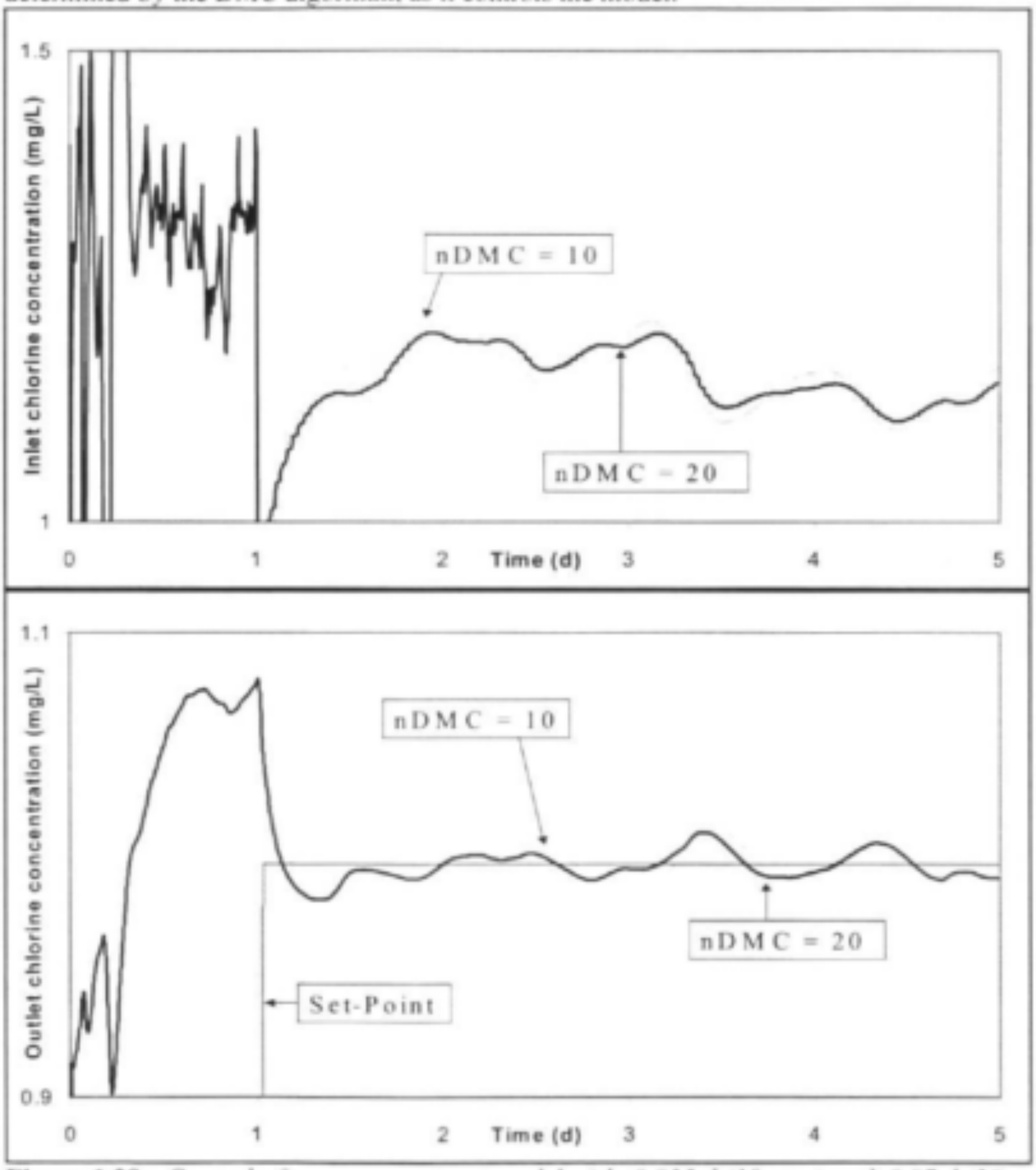

Figure 1.38 : *Control of one-compartment model with 0.035 d (10-step) and 0.07 d (20-step) optimisation horizons (control from time = 1d onwards)*

1.6 On-line implementation

Isermann (1982) explained that adaptive control systems adapt their behaviour to the changing properties of controlled processes and their signals. In the feedback adaptation (closed loop adaptation, Figure 1.39) information on the process behaviour is gained by measuring process input and output signals, from which parameters determining the behaviour can be identified. Then, based on this information, the controller output signal can be calculated and adapted.

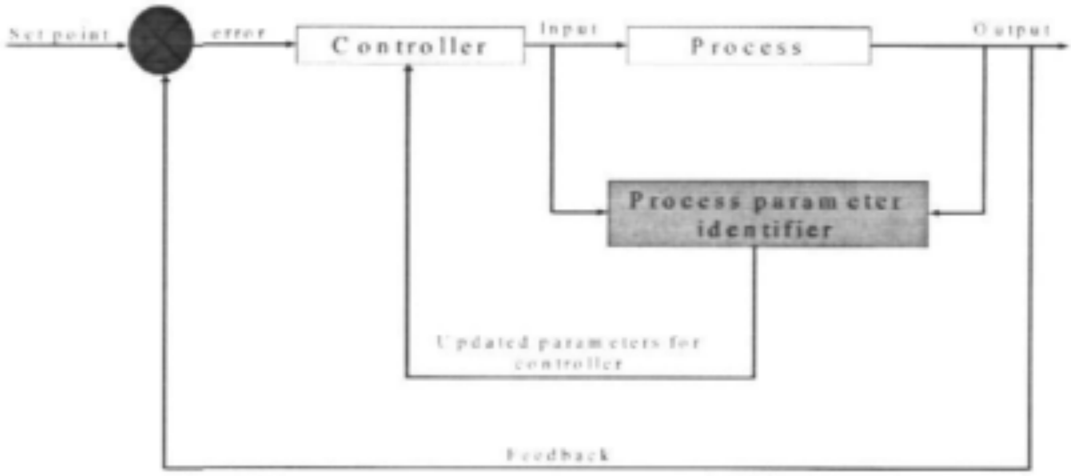

Figure 1.39 : *Adaptive control structure*

1.6.1 1-Compartment model as a basis for *k* identification

The one-compartment model was chosen to be implemented on-line. However, some modifications were made. There is evidence that the k value for chlorine loss is not constant, depending inter alia on the time of the year. The extended Kalman filter is simplified and used to find the kinetic factor k on-line as described in the following equations using the measured inlet and outlet chlorine concentration, the level and the inlet flow rate of the reservoir.

Equation 1.5 can be re-written in the form:

$$\frac{dC(t)}{dt} = \frac{1}{\tau}C_0(t) - \left(\frac{1}{\tau} + k\right)C(t) \quad \text{with } \tau = \frac{\mathrm{A} \times \mathrm{h}}{\mathrm{F}_0} \qquad (1.11)$$

where C_0 = inlet chlorine concentration (mg/L)
C = outlet chlorine concentration (mg/L)
t = time (d)
k = kinetic factor (d^{-1})
A = area of the reservoir (m^2)
h = water level (m)
F_0 = inlet flow rate of the reservoir ($m^3.d^{-1}$)
τ = theoretical residence time (d)

If the first order decay is not included, Equation 1.11 can be integrated as

$$C(t+\Delta t)=\left(e^{A^*\Delta t}\right)C(t)+[1-e^{A^*\Delta t}]A^{-1}BC_0(t) \quad \text{with } A=\frac{1}{\tau} \quad \text{and } B=-\frac{1}{\tau} \qquad (1.12)$$

Now, including the first order decay with a simple Euler integration, equation 1.13 is obtained from equation 1.12:

$$C(t+\Delta t)=A_*C(t)+B_*C_o(t)-k(t)C_t(t)\Delta t \quad \text{with } A_*=e^{A\Delta t} \quad \text{and } B_*=e^{A\Delta t}-1 \qquad (1.13)$$

Thus, the main equation of the Kalman filter is obtained:

$$k(t+\Delta t)=k(t)+K\left[A_*C(t)+B_*C_0(t)-k(t)C(t)\Delta t-C(t+\Delta t)\right] \qquad (1.14)$$

Then, in the DMC algorithm, the dynamic matrix (equation 1.10) is dependent on the kinetic factor k, and can be updated each time the extended Kalman filter runs, allowing for better control of the outlet chlorine concentration. The number of optimisation steps to the horizon for the DMC is set to 40 points spaced at 3 minute intervals (2h horizon). For the 2-4 hour response times observed under the typical reservoir volumes and flows that apply here, this was a compromise bearing in mind the computational load.

1.6.2 On-line script program

The MATLAB test program (section 1.5) was translated into VISUAL BASIC, which is the mathematical language read by the Script Agent of ADROIT, the SCADA system operating at the waterworks. The observed outlet chlorine concentration $C_m(t)$ was filtered (to remove rapid measurement noise) using the following relationship:

$$C_F(t+\Delta t) = \gamma C_m(t)+(1-\gamma)C_F(t) \qquad \ldots\ldots (1.15)$$

where C_F = filtered outlet chlorine concentration (mg/L)
C_m = observed outlet chlorine concentration (mg/L)
t = time (d)
γ = smoothing factor

This smoothed chlorine concentration is then used as feedback to the DMC, and, of course, the DMC output is sent to the plant if the DMC. Figure 1.40 is a simplified flow diagram of the ADROIT program.

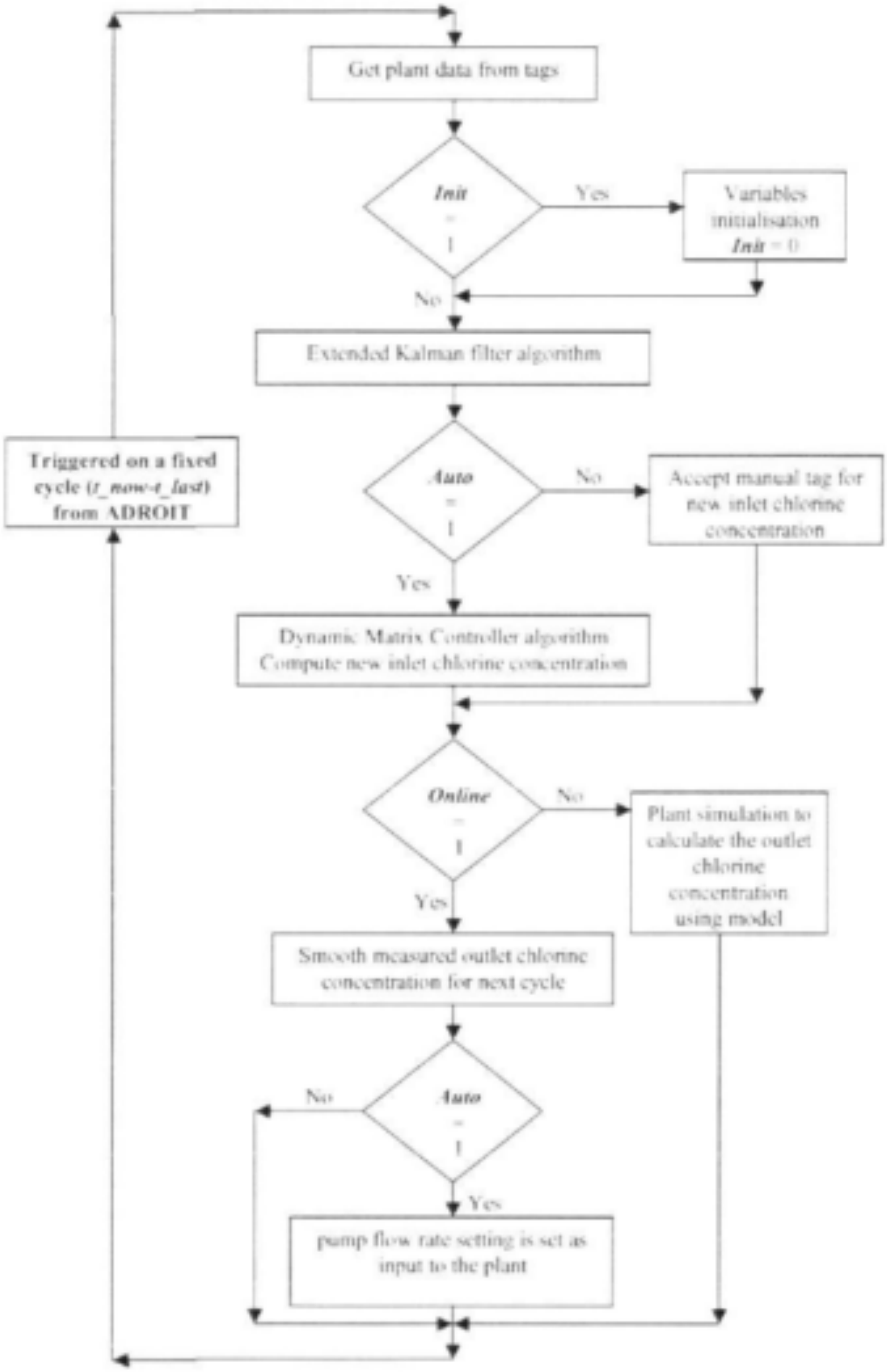

Figure 1.40 : *Simplified flow diagram of the ADROIT program*

1.6.3 Operation of the adaptive controller in the Adroit system

All of the data in Adroit are referenced by *tags* and stored in different categories (analogue, digital, script, text, etc.) in the Configuration Agent. In the Script Agent, it is possible to import the tags values directly to the program, to modify them, and then to set them back into the Configuration Agent. Thus, the inlet and outlet chlorine concentration, the inlet flow rate and the level of the reservoir data are directly imported in the program, as well as the time elapsed since midnight in seconds. The time t_p between two runs of the program is set in the Script Agent. The program is protected from wrap-around at midnight.

It is not possible to store variables in the program itself, and some of them need to be permanently updated accounting for the previous moves. Where previous values are required on each call of the algorithm, these variables are defined as tags. It is then possible to obtain, modify and relay them to the Configuration Agent between two runs of the program, thus allowing them to be stored. Figure 1.41 shows the User Interface of Adroit.

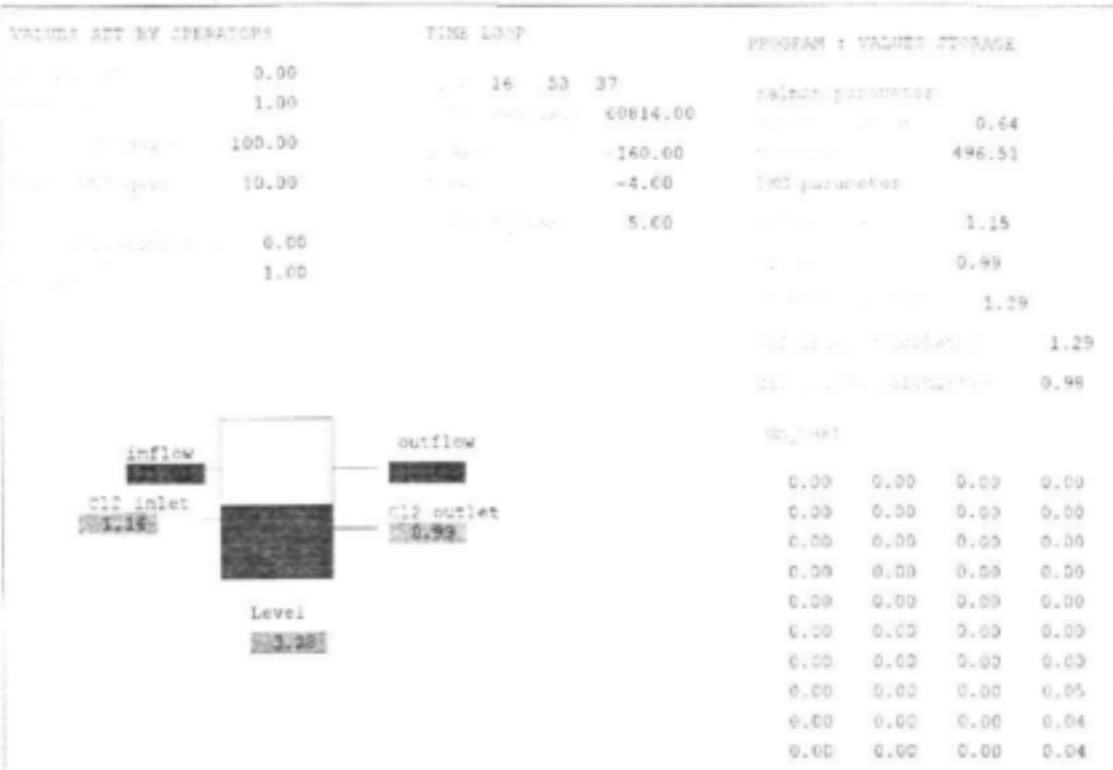

Figure 1.41 : *User Interface screen of ADROIT*

1.6.4 Tuning of the on-line system

Extended Kalman filter tuning

Firstly, the extended Kalman filter gain ε was set to 100 and the Dynamic Matrix Controller gain β was set to 10, both of which had been found optimal in testing the on-line system using a model to represent the plant. However, the estimated kinetic constant k responded too quickly on the actual plant. To obtain a smoother value, the extended Kalman filter gain ε has been decreased to 10. The Dynamic Matrix Controller gain β was kept at 10.

Hypochlorite pump for inlet chlorine

The operators previously controlled the chlorine concentration in the inlet flow based on their experience. They did this by varying the pump flow rate of NaOCl (sodium hypochlorite or *hypo*) Table 1.1 shows typical observed behaviour. Depending on the water inflow rate, and on the inlet chlorine concentration desired, they set different values for the hypo pump flow rate.

Table 1.1 : *Inlet chlorine concentrations for different water inflow rates and different pump flow rates*

Inflow rate (ML/d)	Pump (%)	Inlet chlorine concentration (mg/L)
110	5	1.4
110	6.5	1.6
140	7	1.4

In order to apply the required inlet chlorine concentration calculated by the DMC algorithm, an open-loop predictor was added that sets the hypo pump flow rate to achieve the specified inlet chlorine concentration, which in this case is the manipulated variable of the DMC. It was found impossible to use the feedback measurement of the actual chlorine inlet concentration, due to large swings in this value as water is drawn to, or discharged from, the filter beds (See Figures 1.12 and 1.22).

Based on Table 1.1, the following relations have been used:

$$\text{Predicted Inlet Chlorine Concentration} = \frac{(6.15 \times \text{Pump Setting}) + 95.5}{(\text{Water Flow} + 2)} \qquad (1.16)$$

$$\text{Required Pump Setting} = \frac{(\text{Required Inlet Chlorine Concentration}) \times (\text{Water Flow} + 2) - 95.5}{6.15} \qquad (1.17)$$

The offset of the Water Flow by +2 is merely a protection against *division-by-zero*.

1.6.5 On-line performance of the adaptive DMC

The first part of the following graph (Figure 1.42) has no automatic control, as can be seen from the occasional stepping of the hypo pump setting by the operator. Then the DMC controller (incorporating the adaptive adjustment of the identified chlorine loss rate constant from the Kalman filter) is switched on at the 80th hour (shown by the *auto switch*, which goes from zero to one). The chlorine set point is fixed to 0.95 mg/L.

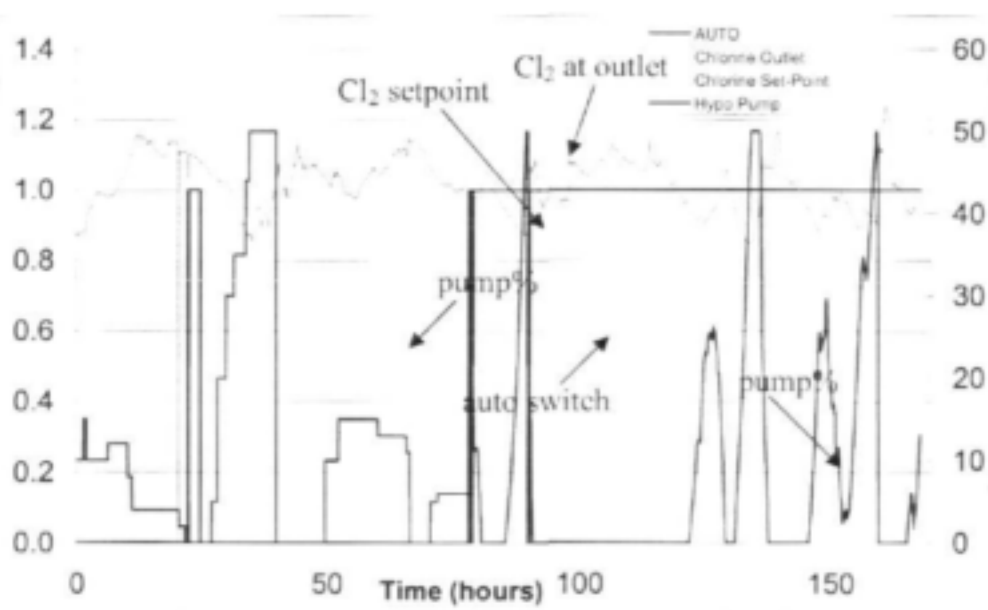

Figure 1.42 : *Comparison of outlet chlorine control performance by manual operation (up to 80 hours) and by adaptive Dynamic Matrix Controller (after 80 hours)*

The automatic control is acceptable, perhaps marginally better than the occasional adjustments of the operators. In fact there is some confidence in the new controller amongst the operators. However, it is intended to improve the tuning of this controller using a longer optimisation horizon (4 hours instead of the present 2 hours), and to increase the move suppression (lower β). This will reduce the gain of the controller, which appears to be over-compensating. Bearing in mind that this version of the DMC controller does not treat constraints optimally (it simply *clips* hypo pump settings at maximum and minimum values), one problem is that the hypo pump cannot be set lower than 0 % (whereas hypo is still being added at 0 %). On the plot, the main deviations from the set-point are all caused by excessive control action - each peak comes from an excessive hypo pump adjustment - making the controlled variable a bit oscillatory. Other issues such as appropriate tuning of the Kalman filter, for a desirable rate of variation of the predicted chlorine decay rate constant, should also be examined further.

2. OPTIMAL OPERATION OF A WATER DISTRIBUTION NETWORK BY PREDICTIVE CONTROL USING MINLP

2.1 Considerations in water distribution operational optimisation

Pumping is one of the most important operational costs found in water distribution systems. This is particularly true in the Durban Metropolitan Area (DMA), which is a very hilly area, where many pumping operations are required to transfer water from one area to another. Consequently, eThekwini Water Services has identified pumping operations as one of the possible domains requiring operational optimisation. Generally, pumps are electrically driven and fall into two categories: the fixed-speed pumps and the variable-speed pumps. While variable-speed pumps have several continuous operating points; fixed-speed pumps, on the contrary, are operated according to binary choices: they are either on or off. Although operating policies are applied so that pumps do not switch on during electricity cost peak periods as far as possible, no strategy is currently under use to restrain pumping operations during intermediate and off-peak periods.

It is believed that filling storage capacities by means of pumping when the electricity is less expensive, would allow cutting operational costs by a significant amount. This research project was created to tackle this issue and it has been conducted as a case study.

General principles have been validated on a small-scale application. Once validated, the methods have been implemented on an existing large-scale application, representing a particular part of the water distribution network of the DMA. The maximisation of the use of low-cost power for pumping operations is the main optimisation objective and can be achieved by using an optimal planning that will allow best use of gravitational flows, and restriction of pumping to low-cost power periods (e.g. overnight pumping) as far as possible. This objective is constrained by the maintenance of the minimum and maximum emergency volumes in all reservoirs, as well as a good water quality in every part of the distribution system. Even if pumping operations are optimised, the new approach must ensure that adequate chlorination is maintained in the treated water as it is distributed.

In the DMA the water is treated by means of chlorination. As the network covers a wide area, there is a need for re-chlorination at some stages. The quality of water can therefore be guaranteed, even at final delivery points. Chlorine is a cost efficient, easy-to-use disinfectant, effective in killing bacteria and having residual properties. Unfortunately, it produces odours that are easily recognised by customers. When solving the problem of operational optimisation of the water distribution network, efforts are also made to find a compromise between sufficient chlorination (to ensure bacteriological quality) and providing the network with water which consumers find pleasant to drink.

The combination of dynamic elements (i.e. reservoirs volumes) and discrete elements (pumps or valves status, routing) makes this a challenging predictive control and constrained optimisation problem, which has been solved by Mixed Integer Non-Linear Programming (MINLP). The MINLP algorithm is used for its ability to handle integer choices. In this optimisation problem, the presence of integer choices arises from the use of binary valves, and fixed-speed pumps in the distribution network. Integer choices are associated with the status of these elements: valves open or shut / pumps on or off in this particular case.

A non-linear penalty function and several constraints are defined with respect to the equations describing the dynamic behaviour of the system. This model is based on a standard modelling element, having one input and two outputs linked through a vessel of variable volume. The physical properties of an element can be defined in such a way as to allow representation of any item in the actual network: pipes, reservoirs, valves and pumps. The overall network is defined by inter-linking a number of standard modelling elements. Once the network is represented within the model, the penalty function is minimised over a defined time horizon, with input and output variables obeying the constraints.

This non-linear optimisation requires the anticipation of a consumer demand profile, which is obtained from historical data stored by the SCADA system monitoring the network. Valve positions, pump characteristics, reservoir volumes, and electricity cost patterns are some of the parameters required to achieve the operational optimisation of the system.

2.2 Water distribution in the Durban Metropolitan Area

The Durban Metropolitan Area (DMA) covers an area of approximately 1 370 square kilometres (km^2) and has an estimated population of 2.77 million inhabitants residing within its boundary. It has been extended at the end of the year 2000 and now includes large areas of former neighbouring councils as presented on Figure 2.1.

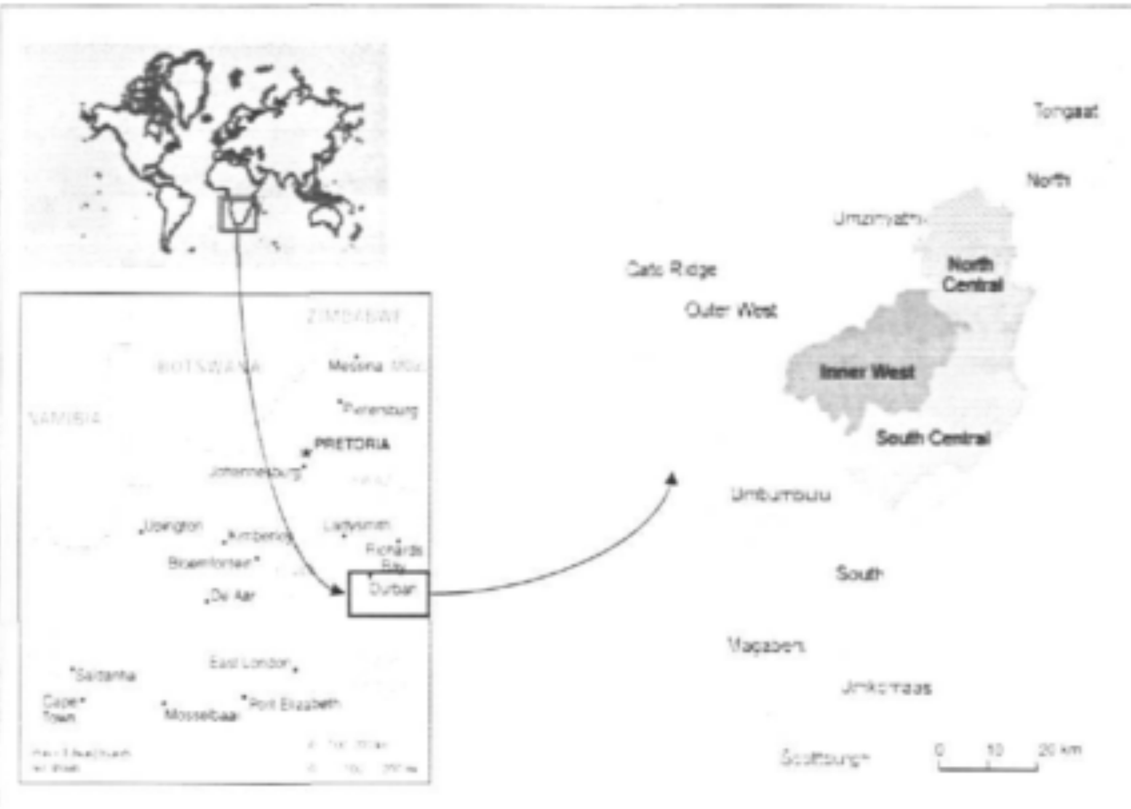

Figure 2.1 : *Durban Metropolitan Area (DMA)*

The raw water purchased by eThekwini Water Services from Umgeni Water originates from Inanda, Nagle and Midmar dams. The total capacity of these supply sources represents 800 million megalitres (ML) per year. Durban's water distribution network is provided with water arriving mainly from two waterworks: Durban Heights Waterworks and Wiggins Waterworks. Durban Heights Waterworks is the oldest and the biggest water treatment plant of the DMA, capable of providing up to 720 ML/day of treated water. Wiggins waterworks has been designed in the mid-eighties at a time when Durban Heights waterworks was utilised at near full capacity. The initial capacity of this new plant was 175 ML/day, extended to 350 ML/day in 1995.

Durban Heights and Wiggins Waterworks are capable of providing to the DMA up to 1070 ML/day, of the total 1540 ML of treated water that Umgeni Water can produce each day. During normal consumption days, only 790 ML/day are purchased from Umgeni Water. This is supplemented by approximately 23 ML/day from three small water-treatment plants that eThekwini Water Services owns and operates.

Approximately 800 km of bulk mains convey treated water to 220 storage reservoirs within the DMA. Water is further distributed via 4500 km of pipes, generally with a diameter less than 300 mm. These secondary pipes radiate outwards from the storage reservoirs or water treatment works. Approximately 160 test points in the piped potable water distribution system are continuously monitored by eThekwini Water Services to ensure compliance with quality standard.

2.3 Literature survey of relevant techniques

2.3.1 ptimisation applied to water distribution systems

Whereas much effort has gone into the *optimal design* of water distribution systems, the present work concerns their *optimal operation*. The objective for this second family of problems is mainly to generate control strategies ahead of present time, using predictive techniques, to guarantee a competent service in the network, while achieving certain quality objectives at the same time. Minimisation of supply and pumping costs, maximisation of water quality and leak prevention are among the main targets (Cembrano *et al.*, 2000).

The first approach of dynamic programming applied successfully on a large-scale water distribution system can be found in Fallside and Perry (1975). However, this attempt did not achieve satisfactory results in the long term, which lead Coulbeck (1977) to offer a more general and improved version of the original decomposition techniques developed by Fallside and Perry (1975). Once the initial method became more robust and reliable, Fallside (1988) conducted a study to identify what would be the requirements to implement the same methodology on water distribution systems not as highly monitored as the one they used initially. They reached the conclusion that without the same degree of telemetry as the one used in the Cambridge (UK) area, it would be difficult to successfully apply the initial method they developed.

In Spain, almost simultaneously with Fallside and Perry's research, Solanas (1974) created an approximation of an existing dynamic programming algorithm, which was improved until 1988, and finally successfully implemented on the Barcelona distribution network (Solanas and Montoliu, 1988). This approach was preferred to existing decomposition techniques because of the highly interconnected nature of Barcelona network, which makes inefficient the use of these techniques.

In the 1980's, one of the most relevant applications was the implementation of the model developed by Carpentier (1983), for the southern areas of the Paris distribution network. His method was based on the use of dynamic programming techniques, which take into account the particular star structure of this supply system. Carpentier's method has since been used successfully on other areas of Paris distribution system. For instance, Alla and Jarrige (1988) present economic results concerning the use, since 1985, of this system on Paris' western areas. On the positive side, they highlight that the benefits, which can be realised with the optimised sequence, lie between 0 and 5 % of the total operational cost of this particular part of the network. However, on the negative side, these savings are expensive in terms of safety, since reservoir levels often reach their lower emergency levels during periods where the electricity tariffs are high. Furthermore, pump and valve operations occur very often, which increases operator workload significantly, since this tool does not provide automatic closed-loop control yet.

About this time, a new trend arose due to improvement of computational resources, which enabled the use of LP (Linear Programming) to solve the pump-scheduling problem. Researchers became interested in LP because of the availability of new robust and efficient methods to solve this class of problems. For example, Jowitt *et al.* (1988) described the use of LP for the real-time forecasting and control of water distribution systems. Jowitt and Germanopoulos (1992) continued to work in this particular field and combined the use of LP with an extended-period network-simulation approach. Their research was aimed at determining the minimum cost schedule of pumping, on a 24 hours basis, for a given distribution network. The method was applied successfully to an existing network, for which substantial savings on the pumping costs were achieved.

In the long term however, it appeared that the resulting accuracy and reliability of such models could be quite poor. Consequently, one of the latest approaches consists in taking fully into account the non-linear relationships, which are part of any pump-scheduling problem. As for LP, the appearance of improved Non-Linear Programming (NLP) algorithms on the market, convinced researchers to rather apply NLP techniques to solve for this problem. Chase and Ormsbee (1989), Lansey and Zhong (1990) and Brion and Mays (1991) linked network-simulation models with non-linear optimisation algorithms to determine optimal operations. Yu *et al.* (1994) also proposed a method based on NLP to determine the optimal operation of general water distribution systems containing multiple sources and reservoirs. Their method does not require any network simplification, since full network simulations are performed on the basis of optimal strategies calculated by the NLP algorithm they used. These researchers state that they could achieve a profit of approximately 10 % on the operational costs, for a 50 node case-study network.

As NLP techniques are relatively recent, few references are available in the literature demonstrating successful application of these methods to existing installations. The work conducted in this field is still at the stage of research. For instance, recently, a number of European research programmes have attempted to produce tools applicable to a variety of water utilities across Europe. The most advanced one, WATERNET, aims to demonstrate the benefits of introducing optimisation techniques to the water distribution sector. With their work, Cembrano *et al.* (2000) are part of this research programme. Their work proves the advantages of optimal control methods on the water distribution network of a Portuguese district. However, although they integrated their model successfully within the existing supervisory control system, they have not been able to obtain integer strategies for pumping operations, which is the final objective of the WATERNET programme.

It is nowadays clear that recent developments in the field of optimisation allow the resolution of water distribution problems, without applying major simplifications or decomposition methods on the elements they include. Among researchers who began to explore this new path, relatively few have reached a point where it is possible to use predictive techniques to obtain optimal operation; using one optimisation step only, in a multi-reservoir system fitted with several binary valves and pumps, keeping the binary nature of some variables as an essential part of the formulation. The present project proposes an original approach to address this important issue.

2.3.2 Model predictive control (MPC)

The methodology of all of the controllers belonging to the MPC class of algorithm is characterized by the strategy summarized in Figure 2.2. As presented in Camacho and Bordons (1995), the future outputs for a determined horizon N (called the prediction horizon) are predicted at each instant t using the process model. These predicted outputs depend on the known values for past inputs and outputs, up to the instant t, and on the future control signals, which are those to send to the system and to be calculated. The set of future control signals is calculated by optimising a determined criterion, in order to keep the process as close as possible to the reference and conform to constraints, when they exist. This criterion usually takes the form of a quadratic function of the errors between the predicted output signal and the specified reference trajectory (or setpoint), over an N-step horizon. Including the control effort in the objective function is optional, and often this term is rather replaced by an economic objective.

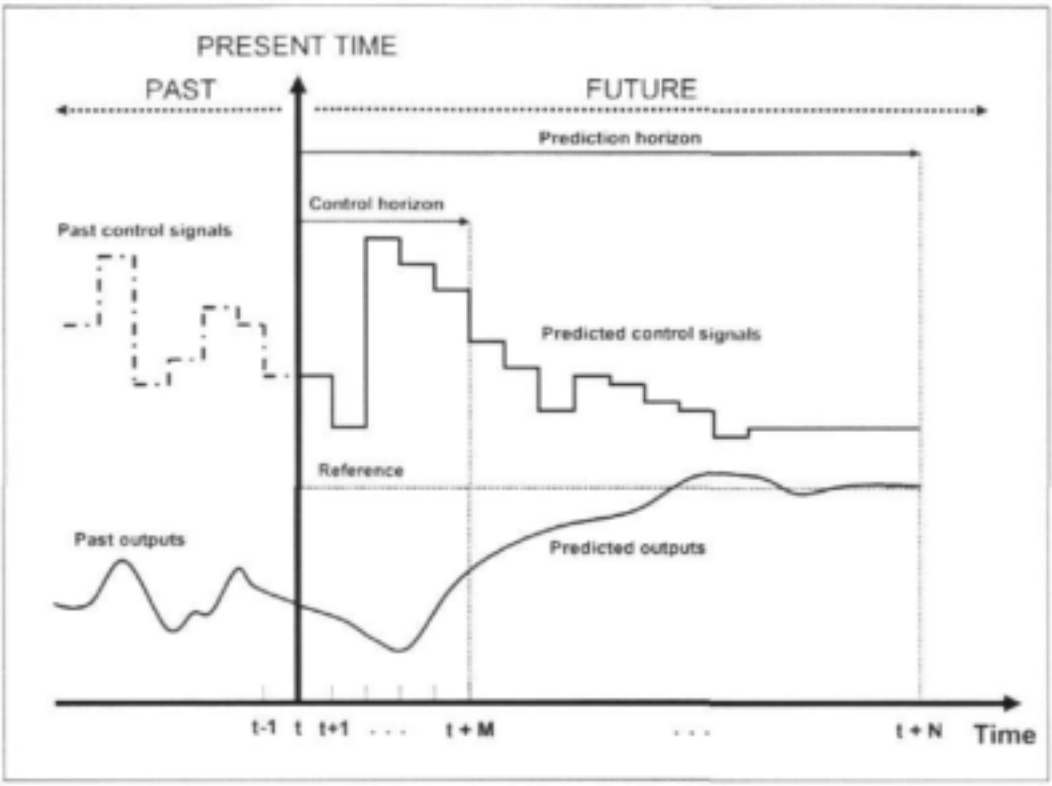

Figure 2.2 : *Model predictive control strategy*

The length M of the manipulated variables sequence represents the control horizon. When the horizon is moved forward, only the first element of this predicted control sequence of M steps is sent to the real process. Since the rest of this control sequence was calculated for, now outdated, plant conditions, it is not adequate anymore and needs to be brought up to date with the new plant conditions. This explains why the rest of control signals stored in the control horizon are rejected.

The whole sequence described above is repeated on each time-step. The period between controller intervention is commonly referred to as a time step or sampling instant. When this strategy is applied to a constrained non-linear model, an iterative optimisation method has to be used for each time-step to solve for the set of future control signals.

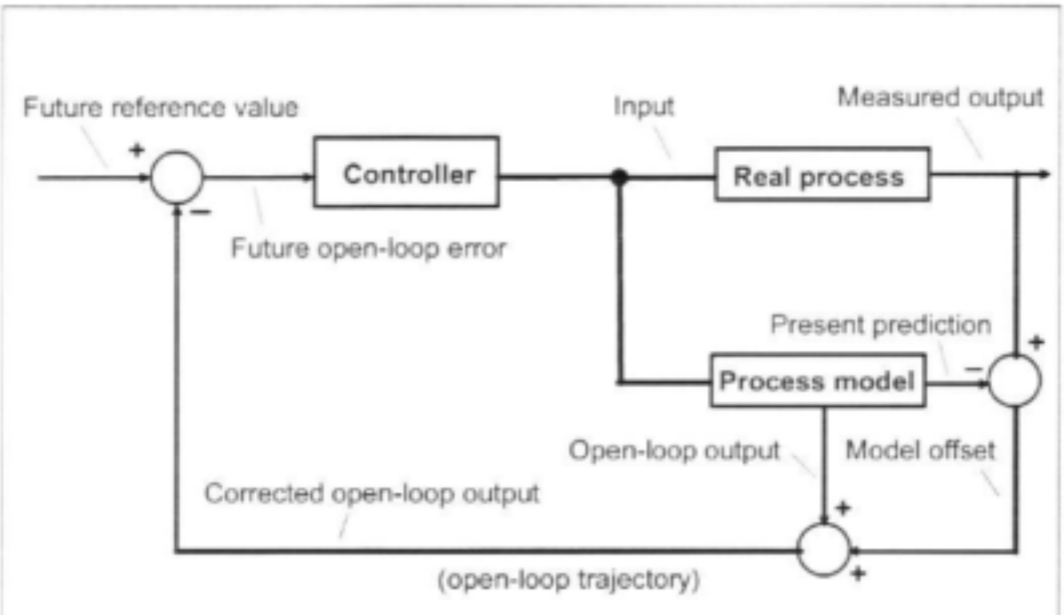

Figure 2.3 : *Basic structure of Model Predictive Control controllers*

The structure in Figure 2.3 shows how to implement this strategy with a controller. This concept is behind most of the model predictive controllers. The controller acts to reduce the gap between the openloop trajectory and the setpoint trajectory up to the horizon.

2.3.3 Mixed integer non-linear programming (MINLP)

Mixed Integer Non-Linear Programming deals with optimisation techniques in which an objective function is minimised (or maximised), while subjected to both equality and inequality constraints, and where two types of variables can be specified: continuous or integer variables. Continuous variables can take any real value within given bounds, while integer variables can either take 0 – 1 values or access a range of integer values. The unique feature of the method is precisely the capability of handling binary variables, which makes it particularly suited to represent discrete decisions in combinatorial problems. MINLP is a class of these problems for which the functions involved in the problem are non-linear.

Grossman (1996) gives an extensive overview of mixed-integer techniques for algorithmic process optimisation. The information presented about the different algorithmic families used to solve MINLP problems has been extracted from Geoffrion (1972), Gupta and Ravindran (1985), Duran and Grossman (1986a), Grossman (1989) and Quesada and Grossman (1992).

The most basic form of an MINLP problem, when represented in algebraic form is as follows:

$$\begin{aligned} &\min Z = \underline{f}(\underline{x}, \underline{y}) \\ &\text{subject to } \underline{g}_j(\underline{x}, \underline{y}) \leq 0 \quad j \in J \\ &\underline{x} \in X \; ; \; \underline{y} \in Y \end{aligned} \tag{2.1}$$

In (2.1), $\underline{f}(\cdot)$ and $\underline{g}(\cdot)$ are convex, differentiable constraint functions, while $\underline{x}$ and $\underline{y}$ are the continuous and discrete variables respectively. The set X is commonly assumed to be compact, while Y is generally discrete. These sets can be defined as shown below:

$$X = \left\{ \underline{x} \mid \underline{x} \in \mathbb{R}^n,\ D\underline{x} \le d,\ \underline{x}^L \le \underline{x} \le \underline{x}^U \right\} \tag{2.2}$$

$$Y = \left\{ \underline{y} \mid \underline{y} \in \mathbb{R}^m,\ A\underline{y} \le a \right\} \tag{2.3}$$

In (2.2), the continuous variables $\underline{x}$ are generally represented by process parameters, such as flow rates, pressure levels, temperature, or even equipment sizing characteristics. The inequalities $D\underline{x} \le d$ represent linear equalities or inequalities that involve these process parameters. As represented by $\underline{x}^L \le \underline{x} \le \underline{x}^U$, the set X is generally constrained by known lower and upper bounds on the continuous variables (respectively $\underline{x}^L$ and $\underline{x}^U$).

In (2.3), the discrete variables are generally restricted to 0 – 1 values in most applications, which means that the corresponding set can be transformed as follows:

$$Y = \left\{ \underline{y} \mid \underline{y} \in \{0, 1\}^m,\ A\underline{y} \le a \right\} \tag{2.4}$$

Numerous algorithms have been developed to solve the problem presented in equation (2.1). However, with the application of MINLP techniques to large-scale systems, only three major approaches have proved successful: the Generalized Benders Decomposition, the family of Outer Approximation approaches and finally, the LP / NLP-based Branch and Bound. All of these algorithms have in common the fact that the optimisation problem is partitioned into a MILP master problem and several NLP sub-problems. The approaches differ by the way the MILP and NLP problems interact with each other.\

2.4 Modelling of a water distribution network for operational optimisation purposes

The modelling of the network of piping and reservoirs was based on a standard modelling element, *viz.* a mixed-flow vessel equipped with one input and two separate outputs. An optional auxiliary input can also be defined. This modelling element is presented in Figure 2.4.

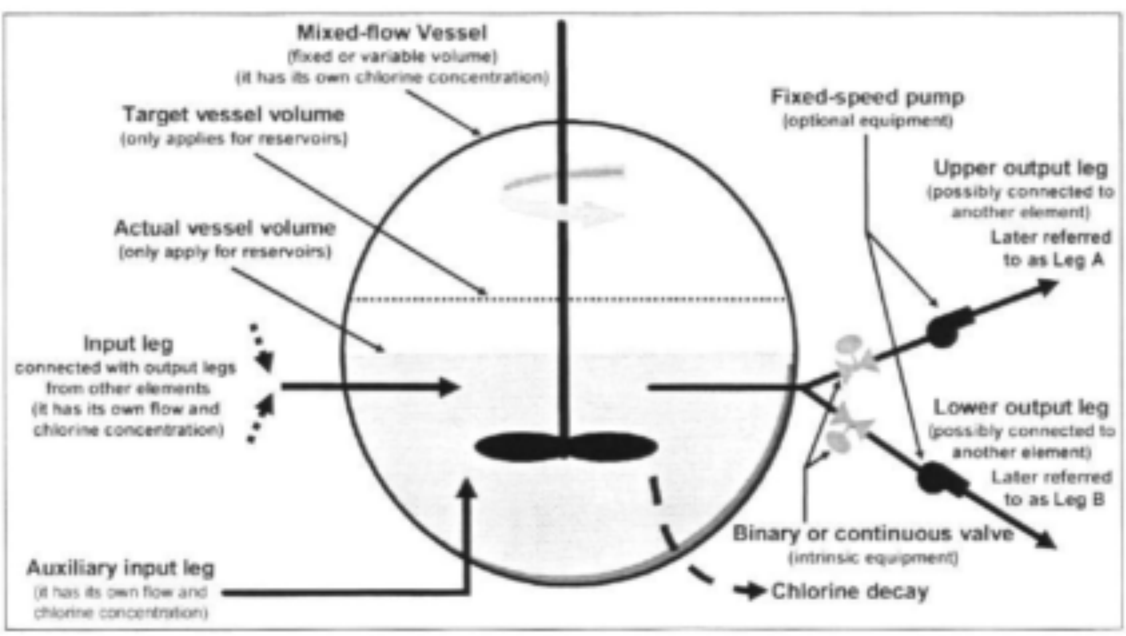

Figure 2.4 : *General representation of the standard modelling element*

2.4.1 Mixed-flow vessel

The inputs and the outputs of the standard modelling element are linked through a mixed-flow vessel, which represents the most important part of the modelling element. The vessel's main parameter is its volume, which can either be fixed or variable. Depending on how this volume is defined, the vessel can represent an open storage tank, a reservoir under pressure or a pipe offtake.

When an open storage tank or an under-pressure reservoir is modelled with a standard element, an initial volume has to be given. For an open storage tank, the volume is a variable of time, while for an under pressure reservoir this volume is fixed. In both cases, this volume must be different from zero.

Setpoint, low-emergency and high-emergency volumes must also be provided. Beside the reservoir volume, the chlorine concentration in the tank also requires initialisation. A setpoint, a low-emergency level, a high-emergency level and a chlorine decay constant also need to be defined for the chlorine property.

Within the framework of a modelling element, a pipe offtake (ie. flow split) is equivalent to a reservoir under pressure with a zero volume. Consequently, when a pipe offtake is defined, the volume of the mixed-flow vessel is taken equal to zero. No other parameter of the vessel requires definition for this particular configuration

2.4.2 Application example

Consider the example in Figure 2.5. The lower pipe offtake of Element 1 (just a pipe split) is linked to the input of the open reservoir Element 2, while the upper output of the open reservoir is connected to the input of the pipe offtake. This configuration is equivalent to having a pipe offtake linked to an open reservoir via a pipe in which water can flow in either direction. It represents a good example of a complex configuration that can be achieved with this model, since it is possible to feed the reticulation system linked to the upper output leg of Element 1 with water coming from the open reservoir represented by Element 2.

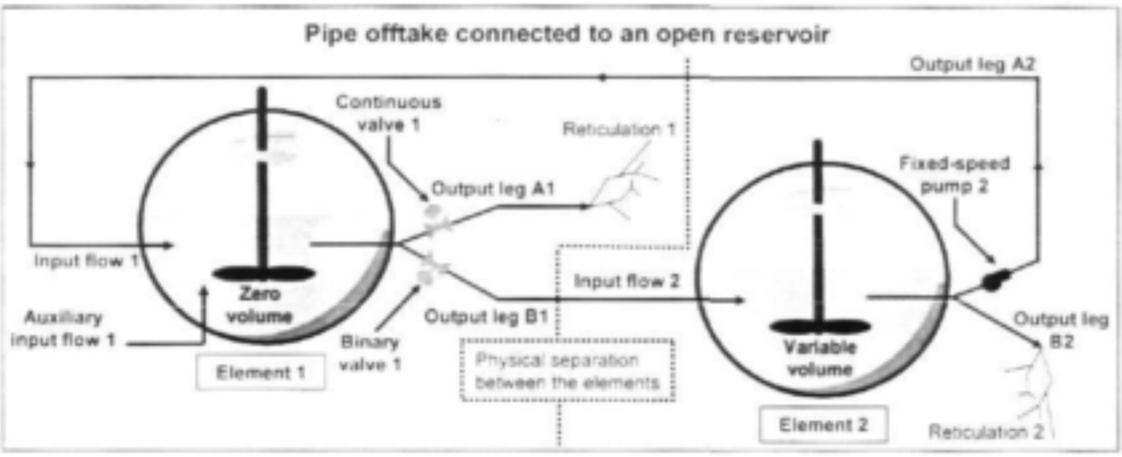

Figure 2.5 : *Illustration of a possible linkage between two modelling elements*

On the first element, the main change occurs by the definition of an input leg, which is in fact an extension of the upper output leg of the second element. An input leg only appears on a modelling element when there is an interconnection between the current element and one or serveral other elements. The main function of an input leg is to transmit the data from one element to another, decoding and summing these data from the multiple legs which can attach to the input of the current element.

The input leg is the one part of the standard element that does not need the definition of any data in the main data table, since its properties are directly deduced from those set for the output legs of preceding elements from which the current element is fed. Time-dependent variables, like flows or chlorine concentration are the main variables that are transmitted between elements for the calculation of the new volumes or chlorine concentration in the reservoirs.

Still on the first element, the new feed flow arriving from the second element is defined as variable. Furthermore, the lower output leg is not connected to the reticulation system, but to the input of the second element.

For the second element, an input leg is also defined: it represents an extension of the lower output leg coming from the first element. Furthermore, the upper output leg of Element 2 is linked with the input of the first element. It can be observed that the auxiliary input flow has been removed for this particular element. Typical data describing this system are presented in Table 2.1. Negative items indicate variables which can be manipulated by the solver, the absolute value being a *starting value* for the solution.

Table 2.1 : *Topology, main, and auxiliary tables for the 2 modelling elements in Figure 2.5*

(a) Topology table

	Element number to which leg A is connected	Element number to which leg B is connected
Pipe offtake (element 1)	0	2
Open reservoir (element 2)	1	0

(b) Auxiliary data table

	T-V coef. for feed flow 1	T-V coef. for demand flow on leg A1	T-V coef. for demand flow on leg B2
Element number	1	1	2
Position in the main table	5	3	4
Time step 1	0.521	1.0	1.0
2	0.558	1.0	1.0
3	0.000	1.0	0.5
4	0.000	1.0	0.5
5	0.409	1.0	0.5
6	1.0	1.0	0.0
7	0.980	1.0	0.0

(c) Main data table & initialisation values

Physical property	(1) X_A	(2) X_B	(3) f_A	(4) f_B	(5) f_{aux}	(6) C_{aux}	(7) C	(8) C_{So}	(9) C	(10) C	(11) K	(12) V	(13) V_{So}	(14) V	(15) V	(16) Pump, 1st param	(17) Pump, 2nd param	(18) Pump, 1st param	(19) Pump, 2nd param
Pipe offtake	0.3	3	8.5	-5.66	15.75	0	0	0	0	0	2	0	0	0	0	0	0	0	0
Open reservoir	4	-1	-0.01	5	0	2.0	-1.8	1.8	1.3	2.4	2	-17.6	20.3	6.8	32.1	6.6	434.6	0	0

2.4.3 Solution strategy

The optimisation strategy is formulated as a Model Predictive Control problem (MPC – see section 2.3.2), which uses a moving-horizon, ahead of present time, as presented in Figure 2.6. The sampling period is fixed at 1 h. For each simulation, the length of the total period of solution is fixed at 48 h. In other words, the solution begins at 0 h and proceeds to 48 h. The model is configured so that the algorithm looks ahead at immediate future demands over a 12h horizon, ie. the control horizon 12 steps ahead, and a total of 60 h of known feed flow data and known user demand flow data has to be provided at the beginning of each new experiment.

This parameterisation of the optimisation problem gives a good compromise between the accuracy of the results, the amount of data to be provided and the computational time needed to achieve a result.

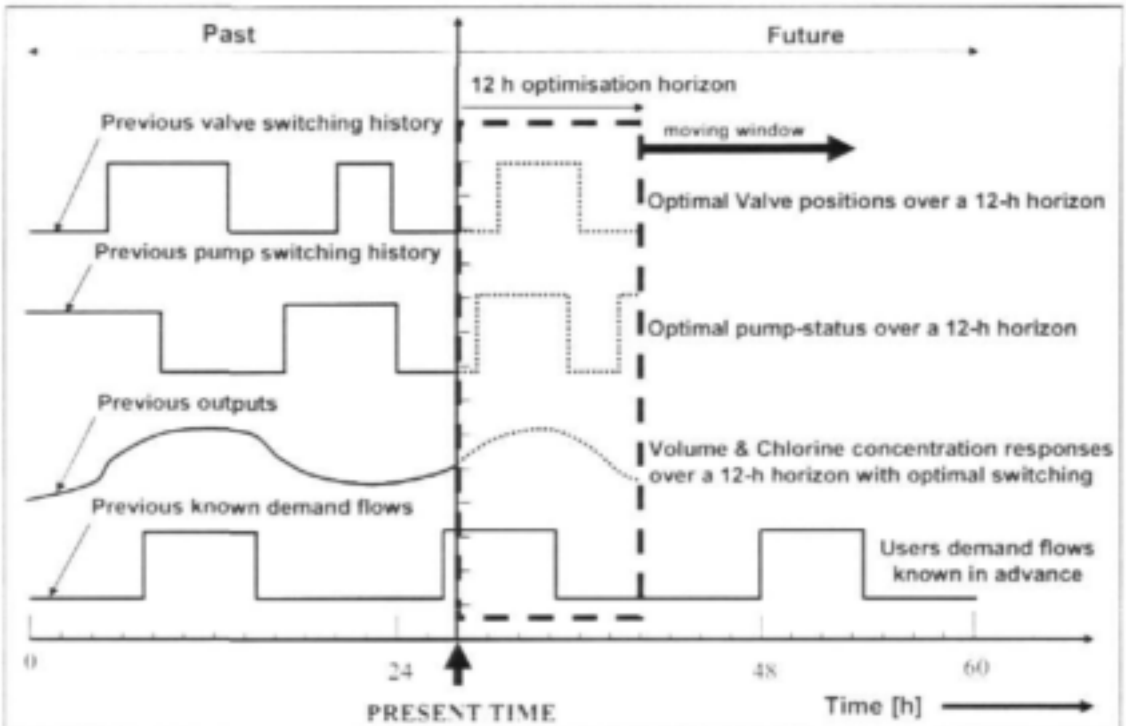

Figure 2.6 : *Model predictive control over 48 hours with a 12 hour prediction horizon*

2.4.4 Modelling strategies

The above features are common to the two different modelling strategies successively developed in the course of this project. However, although these strategies share a common background, they are based on modelling concepts that do not use the same physical principles to reach the solution of the optimisation problem.

The first modelling strategy is referred to as the "calculated flows" strategy. It is based mainly on friction losses and available heads. The resulting hydraulic model gives a detailed pressure-balanced solution, which is used to get a comprehensive representation of the physical phenomena taking place in the network. Although accurate, this modelling approach is rather complex to solve. Initial solutions were limited to rather small networks, and it was difficult to get true integer solutions for the binary variables – convergence was not guaranteed. This situation would have improved had the "CPLEX" module been available earlier in the project (this handles the MIP part – Mixed Integer Programming) of the overall MINLP solution).

The second modelling strategy, known as the "available flows" strategy, is much easier to solve, since it is based on a model where the pipe network hydraulics have been simplified. Here, individual pipe sections are defined to have a maximum capacity – obtainable from the observed rates of change of level in the associated reservoirs. As more pipelines and reservoirs attempt to draw from any single pipeline, the total flow can become constrained by capacity. This new strategy was easier to solve, and with the arrival of the "CPLEX" solver, it was possible to go to a full representation of Durban's Southern Aqueduct.

Since the "available flows" strategy was ultimately used in the solution of the Southern Aqueduct, it will be presented in reasonable detail. The details of the "calculated flows" strategy, entailing the full pressure balance, are obtainable from Biscos (2004). Although this presentation concentrates on the "available flows" strategy, some case studies using the "calculated flows" strategy will also be presented.

2.4.5 Available flows strategy

The main feature of the available flow strategy is the definition of an hydraulic model which simplifies the pipe network hydraulics by removing all of the pressure balances. All of the network components are still represented using a standard modelling element, but the flows in these elements are determined in a linear way, using a valve position or a pump status. Essentially, the system hydraulics are solved using a mass balance approach without taking into account the pipe frictions. This new approach removes all of the problems of non-linearity linked to the use of pressure balances in the equations and simplifies the amount of data to be provided in the model.

The standard modelling element on which this strategy is based is presented on Figure 2.7. The mixed-flow vessel has a liquid volume V, a volume setpoint V_{SP}, a chlorine concentration C, and a chlorine concentration setpoint C_{SP}.

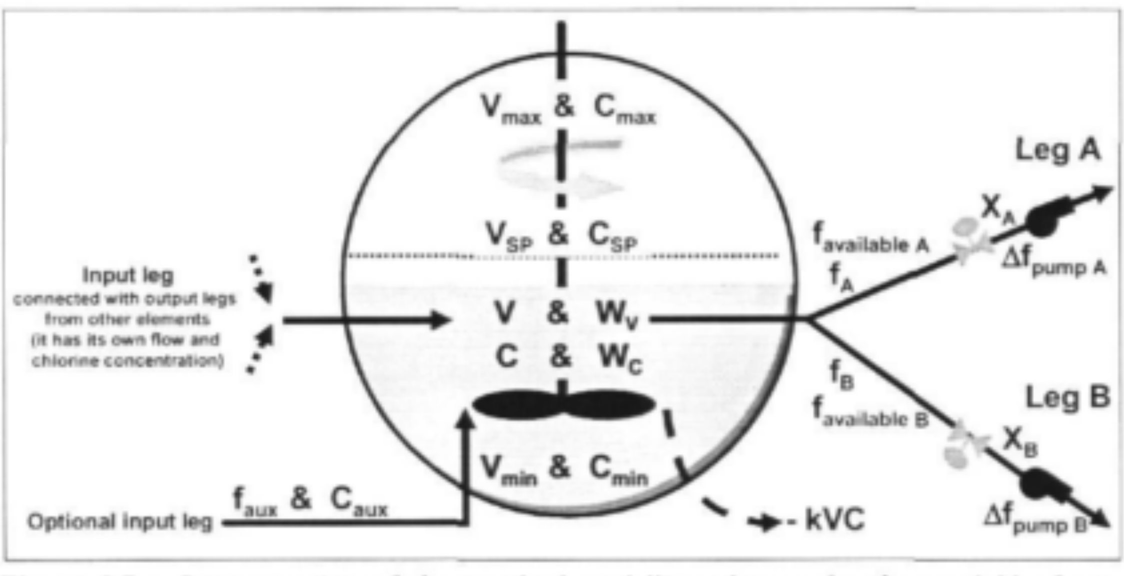

Figure 2.7 : *Representation of the standard modelling element for the available flows strategy*

It is constrained by the low-emergency and high-emergency volumes V_{min} and V_{max}, as well as by the low-emergency and high-emergency chlorine concentration C_{min} and C_{max}. A first-order chlorine decay constant k is also defined. The auxiliary input leg is represented by its flow f_{aux} and its chlorine concentration C_{aux}, while for the output legs the upper output flow is represented by f_A and the lower output flow by f_B. X_A and X_B are used to represent the valve position or pump status, depending on which type of equipment is fitted on the upper and lower output legs.

A maximum available flow, referred to as $f_{available\ A}$ or $f_{available\ B}$ depending on the line considered, is defined for each line. The current output flows are then calculated at each new time-step, up to the prediction horizon, as the product of the maximum available flow of the current pipeline and the actual valve position in this particular line. In this model, the maximum available flows are equivalent to the flows provided by each output leg, once the valve fitted on the leg is fully opened. If there is an online pump in the same pipeline, its contribution is translated into an additional flow, added to the initial maximum available flow. The pump contribution is represented by $\Delta f_{pump\ A}$ and $\Delta f_{pump\ B}$ depending on which of the output legs is considered.

Output flows calculation

Output flows are obtained by multiplying the current valve position for the output leg by the maximum available flow defined for that pipeline. However, the balance equations differ slightly depending on the valve or pump configuration chosen for the current output leg.

For instance, when a fixed or a binary valve is defined on the upper output leg of the current modelling element, the balance equation can be written as follows:

$$f_A \leq X_A f_{available\ A} \tag{2.5}$$

Similarly, for any lower output leg:

$$f_B \leq X_B f_{available\ B} \tag{2.6}$$

It is remembered that maximum available flows are fixed parameters corresponding to maximum possible flows when valves are fully opened. These are obtained in practice from measurements of the rates of change of reservoir volumes. Upstream limits may well mean that sections of pipelines work below their maximum flows. When a pipeline is fitted with an inline pump, the flow contribution of the pump must be added to the maximum available flow defined with the pump offline.

$$f_A = f_{available\ A} + \Delta f_{pump\ A} \tag{2.7}$$

$$f_B = f_{available\ B} + \Delta f_{pump\ B} \tag{2.8}$$

The inflow of any modelling element is defined by:

$$Inflow = f_{aux} + \sum^{nel_{connect}} f_{in} \tag{2.9}$$

The parameter $nel_{connect}$ in equation (2.9) refers to the number of standard modelling elements which are connected to the input of the current modelling element.

To select the inputs that need to be included in the summing term of equation (2.9), use is made of a flow interconnection parameter, represented by the notation m_f. This parameter may only take values of 0 or 1, depending on whether or not a connection is present between two elements.

The values of the flow interconnection parameter are automatically determined by the model using the topology table described above. Therefore, considering the standard element i, the expression represented by equation (2.9) can be transformed as follows:

$$Inflow_i = f_{aux_i} + \sum_{j=1}^{2*nel} m_{f_{ij}} f_{in_j} \tag{2.10}$$

The index j used in equation (2.10) allows identifying the output legs of the other elements of the network to which the input of the element i is possibly connected.

Concerning the output zone of a standard element, this is represented by the sum of both flows leaving the current element via its output legs. It is expressed by:

$$Outflow = f_A + f_B \tag{2.11}$$

The volume balance is

$$\frac{dV}{dt} = f_{aux} + \sum_{i=1}^{nel_{connect}} f_{in_i} - f_A - f_B \tag{2.12}$$

When integrated over the period t to $t+\Delta t$, the volume balance is expressed by:

$$V(t+\Delta t)_{ij} = V(t) + \frac{dV}{dt}\Delta t \tag{2.13}$$

The expression of the volume balance used at time t is deduced from equations (2.12) and (2.13):

$$V(t+\Delta t)_{ij} = V(t) + \left[f_{aux}(t) + \sum_{i=1}^{nel_{connect}} f_{in_i}(t) - f_A(t) - f_B(t) \right]\Delta t \tag{2.14}$$

When the modelling element represents a pipe offtake or an under-pressure reservoir, a constant volume has to be specified in the main data table (represented by a positive value). From this, it is deduced that the term dV/dt is equal to 0. Hence, the volume balance becomes:

$$V(t+\Delta t)_{ij} = V(t) \tag{2.15}$$

Chlorine concentration balance

The accumulation is equivalent to the rate of variation of the chlorine inventory $\frac{dVC}{dt}$.

$$Inflow = f_{aux}C_{aux} + \sum_{i=1}^{nel_{connect}} (f_{in}C_{in})_i \tag{2.16}$$

$$Outflow = f_A C + f_B C \tag{2.17}$$

The consumption is represented by the natural chlorine decay inside the vessel, represented here by first-order kinetics. This may be expressed by the term:

$$Consumption = kVC \tag{2.18}$$

In this equation, k represents the first order chlorine-decay constant.

The chlorine concentration balance can then be deduced from equations (2.16) to (2.18):

$$\frac{dVC}{dt} = f_{aux}C_{aux} + \sum_{i=1}^{nel} (f_{in}C_{in})_i - f_AC - f_BC - kVC \tag{2.19}$$

The change in inventory can be approximately expressed using an average gradient term

$$V(t+\Delta t)C(t+\Delta t) = V(t)C(t) + \left(\frac{dVC}{dt}\right)\Delta t \tag{2.20}$$

Using the arithmetic average of the gradient, the chlorine concentration balance becomes:

$$\begin{aligned}
&V(t+\Delta t)C(t+\Delta t) = V(t)C(t) \\
&+ f_{aux}(t)C_{aux}(t)\Delta t + \sum_{i=1}^{nel}\left[f_{in}(t)_i\left(\frac{C_{in}(t+\Delta t)+C_{in}(t)}{2}\right)_i\right]\Delta t \\
&-\left[f_A(t)+f_B(t)\right]\left[\frac{C(t+\Delta t)+C(t)}{2}\right]\Delta t \\
&- k(t)\left[\frac{V(t+\Delta t)C(t+\Delta t)+V(t)C(t)}{2}\right]\Delta t
\end{aligned} \tag{2.21}$$

Reorganisation of data into vectors and matrices

For convenience, all of the equations of this modelling strategy have been presented until now for a particular standard element isolated from the rest of the system to which it belongs. These equations still apply when several modelling elements are linked together.

However, the equations are more conveniently dealt with in matrix-vector form. In this section, the notation used to represent all of the variables defined earlier as vectors and matrices is introduced. The organisation of the data inside the vectors and matrices is also detailed.

$\overline{C}$ represents the vector of chlorine concentrations in the mixed-flow vessel, while the volumes in each vessel are stored in $\overline{V}$. These vectors are composed of nel rows, since a value is specified for these variables on each modelling element. They are organised as follows:

$$\overline{C} = \begin{pmatrix} C_1 \\ C_2 \\ \vdots \\ C_{nel} \end{pmatrix} \quad \overline{V} = \begin{pmatrix} V_1 \\ V_2 \\ \vdots \\ V_{nel} \end{pmatrix}$$

The other vectors of variables organised similarly are: $\overline{f_{aux}}$, $\overline{C_{aux}}$, $\overline{C_{SP}}$, $\overline{C_{min}}$, $\overline{C_{max}}$, $\overline{k}$, $\overline{V_{SP}}$, $\overline{V_{min}}$ and $\overline{V_{max}}$.

For other variables, the organisation of the data can be slightly different, since some of the parameters defined on the output legs are represented by pairs (one value for each leg). Therefore, some vectors must be constituted with $2 \times nel$ rows. This rule applies for instance to

$\overline{X}$, used to represent the vector of valve positions and pump status, or to $\overline{f}$ which is the vector of output flows. These vectors are organised as follows:

$$\overline{X} = \begin{pmatrix} X_{A_1} \\ X_{B_1} \\ X_{A_2} \\ X_{B_2} \\ \vdots \\ X_{A_{nel}} \\ X_{B_{nel}} \end{pmatrix} \quad \overline{f} = \begin{pmatrix} f_{A_1} \\ f_{B_1} \\ f_{A_2} \\ f_{B_2} \\ \vdots \\ f_{A_{nel}} \\ f_{B_{nel}} \end{pmatrix}$$

Certain other variables are organised similarly.

Objective function

The objective function for this optimisation problem was defined according to the work presented about MPC techniques in section 2.3.2. In the particular example of the operational optimisation of water distribution networks, the move suppression factor was replaced by an economic term associated to the cost of using fixed-speed pumps during pumping operations.

$$\begin{aligned} J = &\left(\overline{V} - \overline{V}_{SP}\right)^T \overline{W}_V \left(\overline{V} - \overline{V}_{SP}\right) + \left(\overline{C} - \overline{C}_{SP}\right)^T \overline{W}_C \left(\overline{C} - \overline{C}_{SP}\right) \\ &+ \left(\overline{f}^T\, \overline{Pump_1} + \overline{Pump_2}\right) E_{COST} \end{aligned} \tag{2.22}$$

In this expression, W_V and W_C are appropriate diagonal weighting matrices which respectively penalise squared deviations from volume set-points, and squared deviations from chlorine concentration set-points. $Pump_1$ and $Pump_2$ are vectors containing the slope and intercept of linear pump power relationships with flow, whilst E_{COST} is a scalar representing the relative penalisation for power consumption.

Available flows strategy problem specification

The overall problem specification for the available flows strategy is summarised in Figure 2.8.

$$Min\ f = \sum_{k=1}^{N}\Big[\left(\overline{V}-\overline{V_{SP}}\right)_k^T \overline{W_V}\left(\overline{V}-\overline{V_{SP}}\right)_k + \left(\overline{C}-\overline{C_{SP}}\right)_k^T \overline{W_C}\left(\overline{C}-\overline{C_{SP}}\right)_k + \left(\overline{f}_k^{\,T}\ \overline{Pump_1} + \overline{Pump_2}\right)\overline{E_{cost}}\Big]$$

$$\text{subject to: } \begin{cases} \overline{f} \leq \overline{X}^T\ \overline{f_{available}}\ \left(+\ \overline{\Delta f_{pump}}\right) \\ \overline{V}(t+\Delta t) = \overline{V}(t) + \overline{f_{ext}}(t)\Delta t + M_{flow}\ \overline{f}(t)\ \Delta t \\ \overline{V}(t+\Delta t)_i \overline{C}(t+\Delta t)_i = \overline{V}(t)\overline{C}(t) \\ \qquad + \overline{f_{ext}}^T(t)\overline{C_{ext}}(t)\Delta t \\ \qquad + \overline{f_{in}}^T(t)\left(\dfrac{\overline{C_{in}}(t+\Delta t)+\overline{C_{in}}(t)}{2}\right)\Delta t \\ \qquad + M_{flow}\overline{f}^T(t)\left[\dfrac{\overline{C}(t+\Delta t)+\overline{C}(t)}{2}\right]\Delta t \\ \qquad - \overline{k}^T(t)\left[\dfrac{\overline{V}(t+\Delta t)\overline{C}(t+\Delta t)+\overline{V}(t)\overline{C}(t)}{2}\right]\Delta t \end{cases}$$

$$\begin{cases} \overline{X} \geq 0 \\ \overline{X} \leq 1 \\ \overline{V} \geq \overline{V_{min}} \\ \overline{V} \leq \overline{V_{max}} \\ \overline{C} \geq \overline{C_{min}} \\ \overline{C} \leq \overline{C_{max}} \end{cases}$$

Figure 2.8 : *MINLP problem to solve for the operational optimisation of water supply systems (available flows strategy)*

Shortcoming of the available flows strategy

It is obvious that the available flows strategy, while simpler and easier to solve, does not provide an accurate model of the complex hydraulic behaviour of a water distribution system. Such a simplified model can only be relied upon if validated using a full hydraulic model of the system. It is recommended to combine the use of the modelling strategy presented in this paragraph with the public domain software package ePANET (Rossman, 2000). ePANET is widely accepted as the world standard in hydraulic and water quality modelling of water distribution systems. However, ePANET cannot be used alone to meet the objectives of this project, since it is not an optimisation tool.

2.5 Tests of the operational optimisation using small-scale case-studies

Before applying the model on a real target application, it was decided to develop several case studies of limited scale to check that the two modelling strategies gave logical and reliable results.

2.5.1 Case-study using the *calculated flows* strategy

This case study involved four standard modelling elements. Its main objective was to fulfil chlorine concentration set-points in two demand reservoirs by mixing water arriving from two different storage reservoirs. Since all of these reservoirs were interconnected, water could be routed from storage reservoirs to demand reservoirs using several valve arrangements.

As this case study concentrated on the satisfaction of chlorine setpoints, no specifications were required on volume setpoints. However, constraints relating to reservoir volume limits and demand flows of users were still defined. It was hoped to provide an integer strategy for all of the binary variables over the complete prediction horizon.

The water distribution system is represented by the 4-element configuration shown in Figure 2.9.

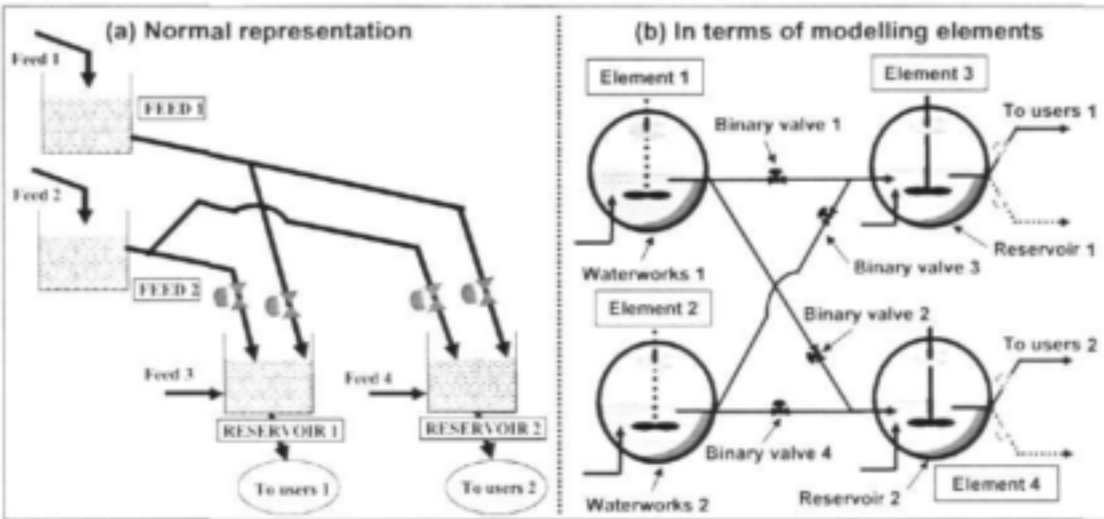

Figure 2.9 : *Chlorine case study – 4-element configuration*

The first feed reservoir has been designed so that there is always a high chlorine concentration available at any time: therefore, a high volume and a chlorine concentration of 1.5 $mg.L^{-1}$ have been fixed. On the contrary, the second feed reservoir was used to supply low chlorine concentration water to the network (chlorine concentration fixed at $0.5 mg.L^{-1}$). The initial capacity of both feed reservoirs was fixed at a relatively large volume of 50 ML, so that the volume constraints of these reservoirs had no effect. For each feed reservoir, two binary valves were available, one for each output leg.

At the other side of the network, the two demand reservoirs have been deliberately designed with a small initial volume (5 ML each) to speed up the concentration response in these reservoirs. The setpoint for the concentration in both demand reservoirs was fixed to an intermediate value (1 $mg.L^{-1}$). The first demand reservoir was initialised with a high chlorine concentration, while the second one was initialised with an low chlorine concentration.

In both reservoirs a first order chlorine decay constant was used with a value of $2.d^{-1}$ (Powel *et al.*, 2000). All demand flows were set to 40 $ML.d^{-1}$, and 10 $ML.d^{-1}$ was provided in each demand reservoir by the auxiliary feed flow.

This example was solved with the MINLP solver DICOPT++, still associated with the MILP solver OSL and the NLP solver CONOPT, at a time when a license fo the CPLEX solver was not yet available. The optimised sequence was achieved with respect to the security constraints specified on the volume in the reservoirs and on the chlorine concentration in the reservoirs. A good example of this can be seen on Figure 2.10 and Figure 2.11.

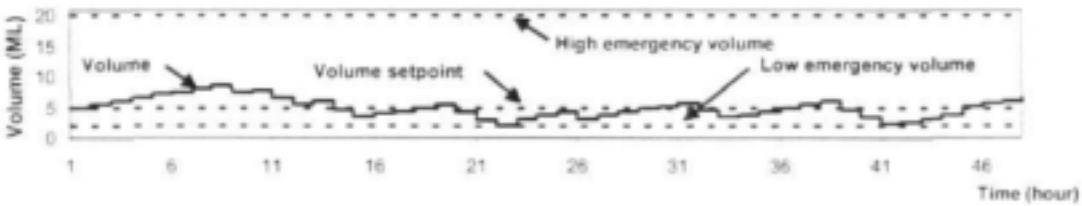

Figure 2.10 : *Evolution of the volume in Reservoir 1 (4-element configuration)*

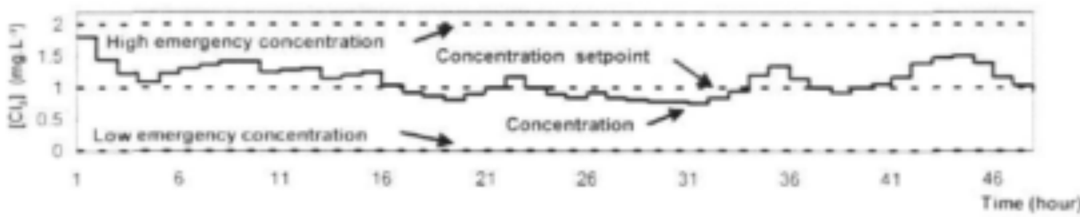

Figure 2.11 : *Evolution of the chlorine concentration in Reservoir 1 (4-element configuration)*

The optimised sequence of valve switching does not correspond rigorously to a binary one, since the solution had to be *relaxed* for some of the time-steps of the prediction horizon. This can be seen on Figures 2.12 and 2.13, which represent respectively the valve status at the inlet of Reservoir 1 and Reservoir 2.

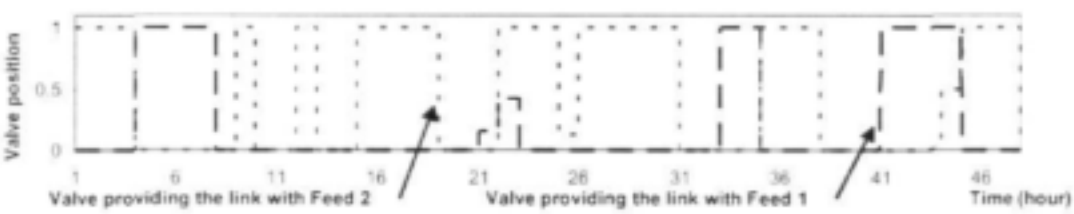

Figure 2.12 : *Valve positions of the binary valves positioned at the input of Reservoir 1 (4-element configuration)*

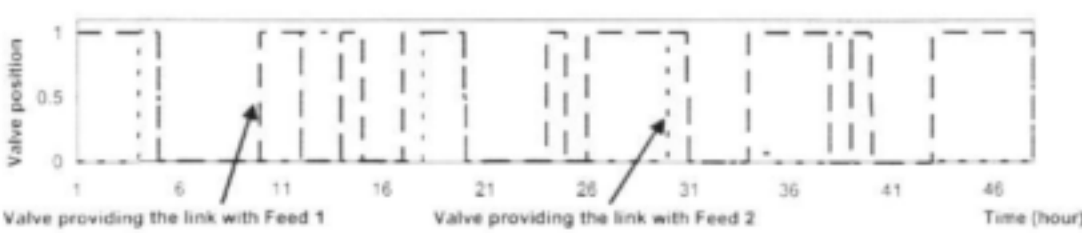

Figure 2.13 : *Valve positions of the binary valves positioned at the input of Reservoir 2 (4-element configuration)*

This result was disappointing since, in this particular configuration, the number of standard elements used was very small. Therefore, it was obvious that the complex pressure-balanced approach, which represented the core of the calculated flows strategy, had to be simplified.

2.5.2 Case-study using the *available flows* strategy

The 10-element system considered is shown in Figure 2.14. This represents a water distribution system which includes a waterworks composed of one storage reservoir, connected to a network consisting of several pipelines linking one pump-station (composed of a fixed-rate pump) with four storage reservoirs (preceded each by a binary valve). This network has seven different connections with the reticulation system to users.

The purpose of this case study was to highlight the ability of the modelling approach referred to as the available flows strategy to produce the operational optimisation of a typical water distribution network. The main objective here was to reduce the pumping cost to a minimum, satisfying simultaneously volume setpoints in storage reservoirs and user demand flows out of the reservoirs, using valve arrangements and flow redirections. Water quality objectives are also considered, but this optimisation objective was secondary. A typical electricity cost pattern was also defined and user demand flows were specified on output legs connected to reticulation. A simulation period of 24 h was used, with a control horizon of 8h. As a result, 32 h of data had to be provided. (Later, with the purchase of a licence granting access to a more powerful MILP solver (known as CPLEX), rather than the OSL solver used here, it became possible to tackle the water and chlorine objectives simultaneously, and over a full 60h period with 12h control horizon).

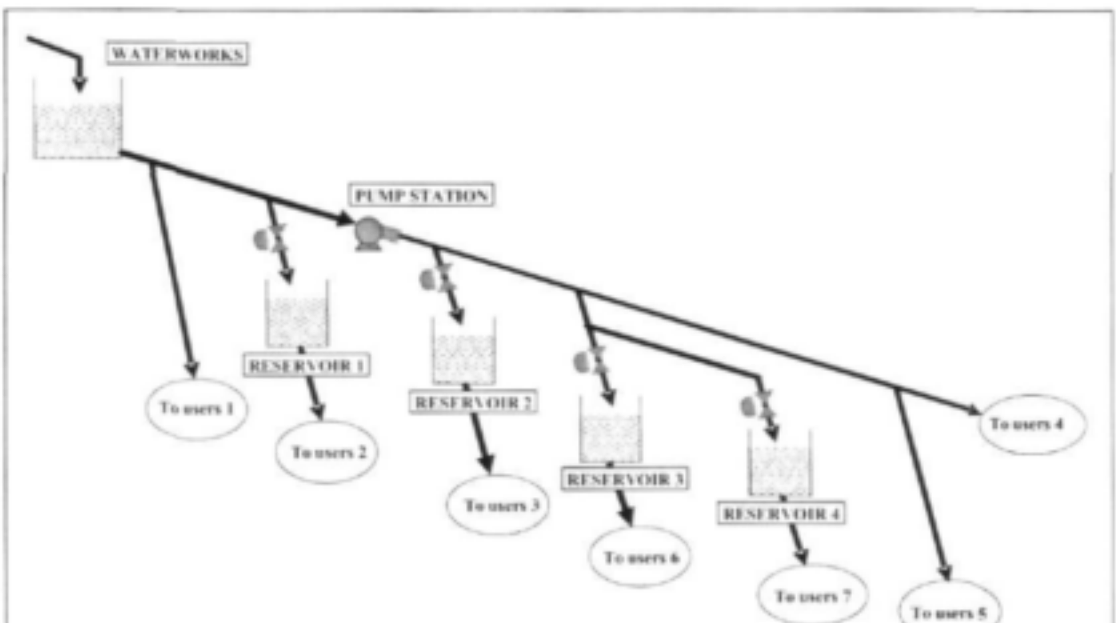

Figure 2.14 : *Operational optimisation of water supply systems - 10-element configuration*

Two separate simulated experiments were conducted with different electricity cost structures, as represented on Figure 2.15.

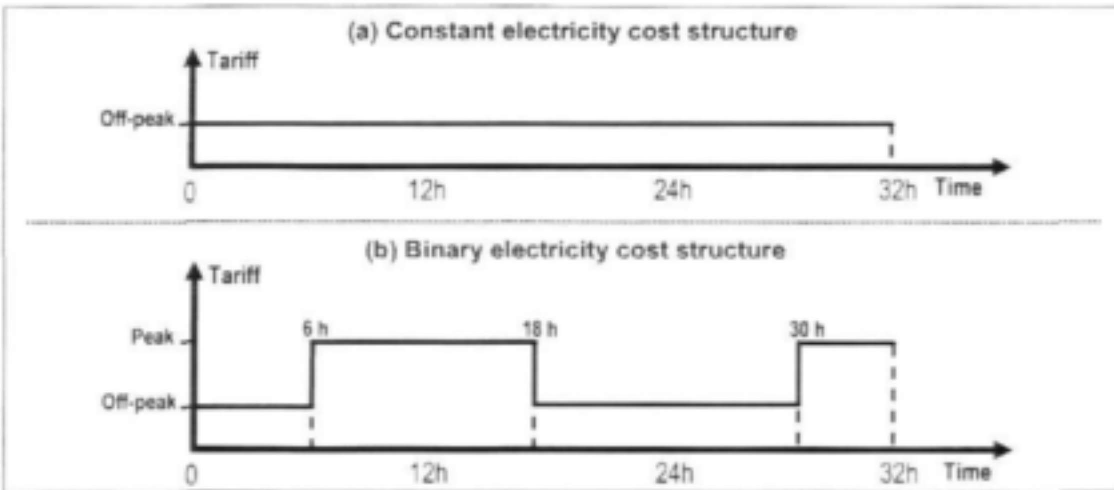

Figure 2.15 : *Available flows strategy/OSL – 10-element configuration: electricity cost structure*

The first pattern is referred to as *constant electricity cost*: it uses the same electricity cost factor over the whole prediction horizon. The second structure uses time-varying parameters for the electricity cost.

The results show that the optimised sequence has been achieved for the two electricity structures, while observing the security constraints applicable to reservoir volumes. A good example of this can be seen on Figure 2.16 for the configuration based on a constant cost, and on Figure 2.17 for the configuration using the binary (two-level) cost structure. These graphs represent the evolution of the volume in Reservoir 2 for the two electricity structures.

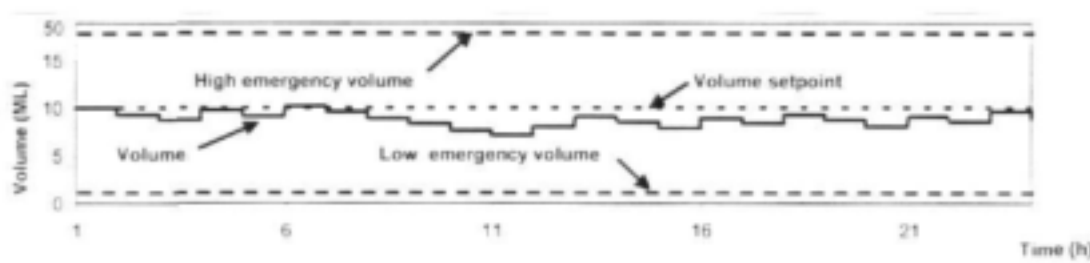

Figure 2.16 : *Evolution of the volume in Reservoir 2 (10-element configuration – constant cost)*

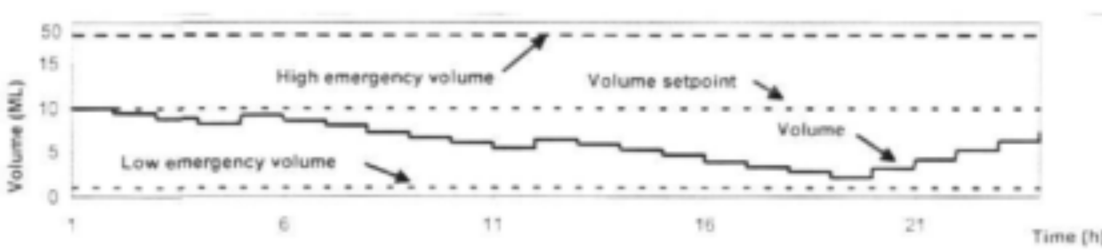

Figure 2.17 : *Evolution of the volume in Reservoir 2 (10-element configuration – binary cost)*

The change to the *available flows* modelling strategy has allowed the provision of an integer solution for all of the binary variables, over the whole prediction horizon. This can be seen on Figure 2.18, which represents the pump status for the different electricity cost structures.

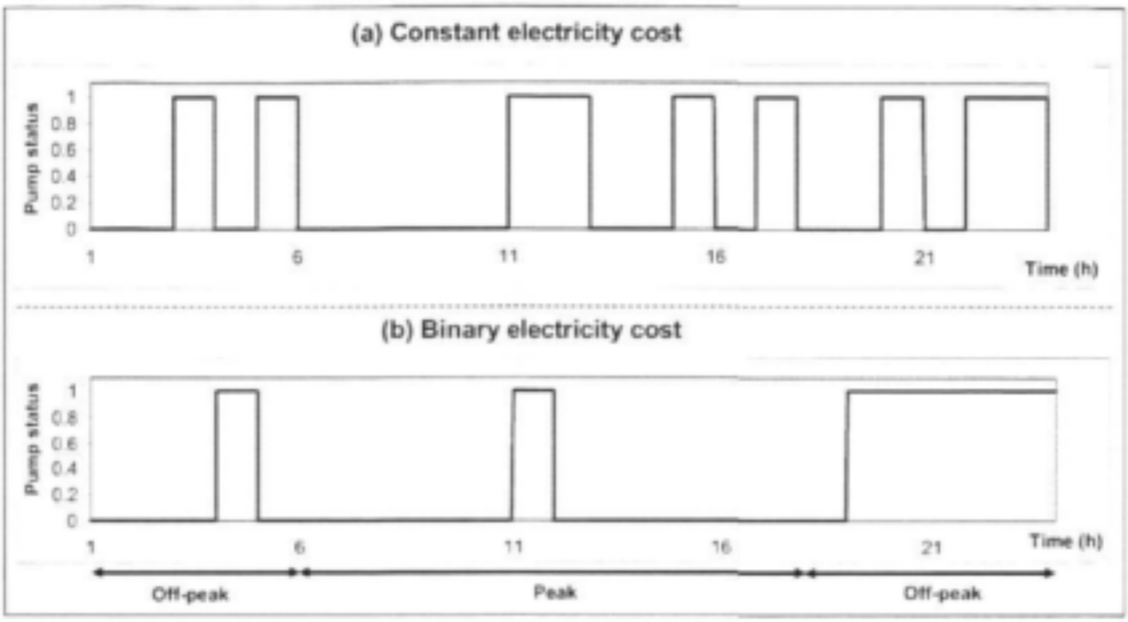

Figure 2.18 : *Pump status at the pump station for different electricity cost structures (10-element configuration)*

The electricity tariff structure has a strong influence on the optimised sequence of valve and pump switching resulting from the operational optimisation of the system. As Figure 2.18 shows, while a constant electricity structure clearly results in a unrestricted use of the pump, it is obvious on the contrary that a variable electricity cost leads to a restriction of the use of the pump. The use of several tariff levels also results in larger deviations from the volume setpoint in the demand reservoir. In this example, for the particular weights chosen, the penalty incurred from the electricity cost during the peak periods clearly outweighs any benefit of attempting to keep the reservoir volumes close to their setpoints. A good example of this can be seen on Figure 2.16 and figure 2.17, by looking at the differences occurring in the evolution of the volume in Reservoir 2 when different electricity patterns are chosen. However, the use of the pump during peak periods can become necessary to avoid infringing one of the emergency volume constraints of one of the reservoirs.

This case study is also very interesting in the way that it highlights once again the anticipation phenomenon. This can be seen on Figure 2.18 (configuration (b)), from the 11th hour of simulation, when the pump switches on in the middle of a period when the electricity is billed at its peak rate. This preventive action occurs to respond to a problem of volume in Reservoir 2, which would only occur from the 19th hour of simulation if nothing had been done earlier.

This study made it clear that the problems of convergence encountered with the *calculated flows* strategy were not only due to the complex modelling choices that were made, but also probably to the less appropriate optimisation tools deployed initially. With the subsequent purchase of a licence granting access to a more powerful MILP solver (known as CPLEX), reliable solutions were obtained for the *available flows* strategy, for simultaneous water and chlorine optimisation, for larger systems over a full 48h period with a 12h optimisation horizon.

2.6 Large-scale application: The operational optimisation of Durban's Southern Aqueduct

The potable water supply to the Durban Metropolitan Area (DMA) is mainly provided from two purification works, which are controlled by Umgeni Water. These waterworks, known as Durban Heights and Wiggins, abstract water from the Umgeni River. While Wiggins is located in the Cato Manor area, Durban Heights is situated in Reservoir Hills, above Durban. Durban Heights is the largest purification plant operated by Umgeni Water and it supplies Durban via the Northern, Central and Southern Aqueduct systems. The purified water is purchased in bulk from Umgeni Water and is distributed throughout the city by eThekwini Water Services.

2.6.1 Southern Aqueduct

The Southern Aqueduct (Figure 2.19) supplies more than a half million people and some large industrial areas in the southern environs of Durban. It links Durban Heights to Northdene, Queensburgh, and Chatsworth, and allows the provision of water to the Umlazi and Amanzimtoti areas. The original aqueduct, which was installed in the early 1960's, is equipped with a pre-stressed concrete pipeline, having a 965 mm internal diameter. It passes mainly through suburban residential areas, with a section traversing the Paradise Valley Nature Reserve. It is 9 km long.

The throughput of this main was originally 128 $ML.d^{-1}$. Since sections of the southern zone of supply can also be fed by the central aqueduct system via a circuitous route, the initial configuration could give a total peak capacity of 189 $ML.d^{-1}$ for the south of Durban (1988 figures).

Figure 2.19 : *Schematic representation of the Southern Aqueduct*

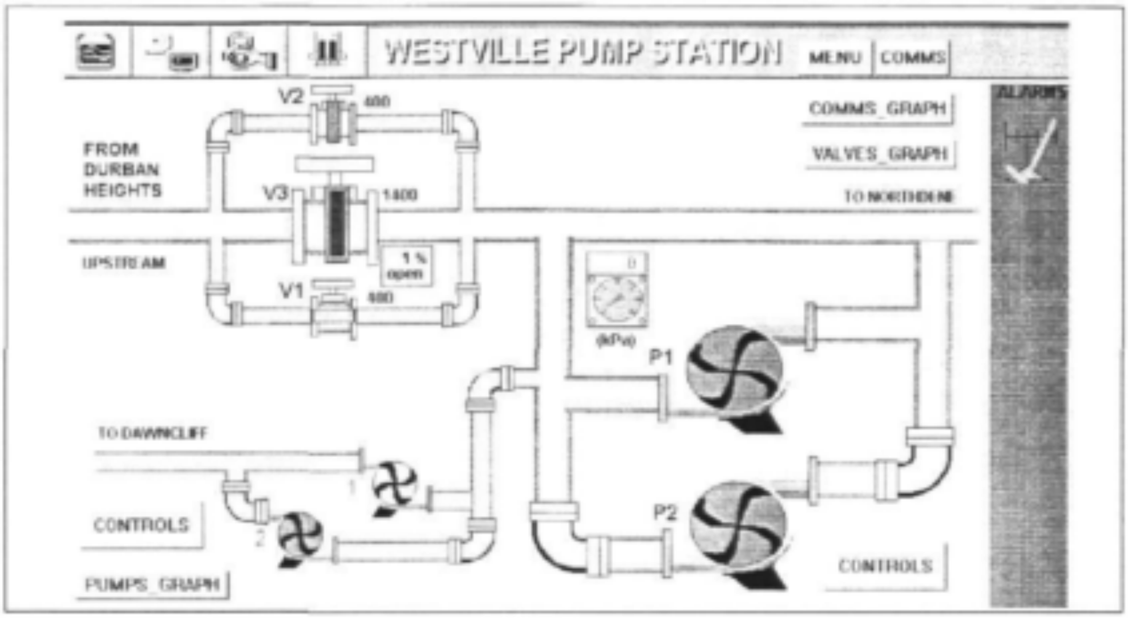

Figure 2.20 : *Representation of Westville pump station within the supervisory control system (courtesy of eThekwini Water Services, a non-return valve appears to be missing on the main line)*

eThekwini Water Services' telemetry system links more than 60 telemetry outstations at the various reservoirs and pump stations, and is used to monitor and control the complete Southern Aqueduct system from the central control room at eThekwini Water Services head office in Central Durban. All of the major elements constituting the Aqueduct are represented by a mimic graphic display on the SCADA system. Figures 2.20 and 2.21 are examples of the interface used for representation of the Southern Aqueduct within the supervisory control system.

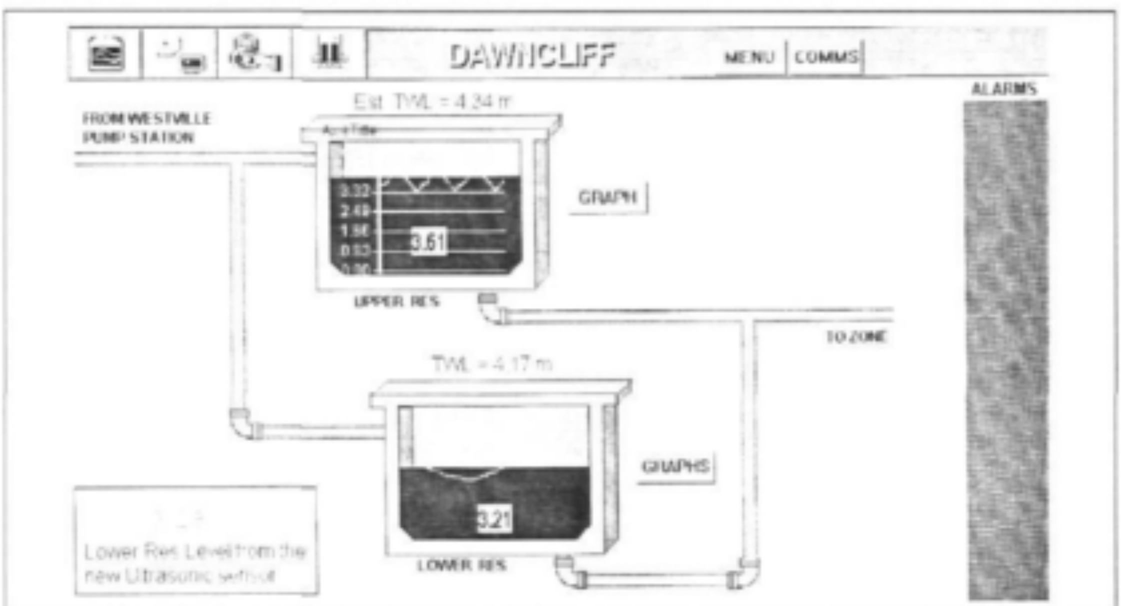

Figure 2.21 : *Representation of Dawncliff reservoirs and reticulation system within the SCADA system (courtesy of eThekwini Water Services)*

The current control strategy applied to the Southern Aqueduct is relatively simple. Hydraulic considerations have led to the creation of a set of rules that prevent non-desirable configurations. The relevant rules and constraints are now part of the supervisory control system, which means that there is a pre-defined response for all of the known behaviours of the Aqueduct. For instance, when an excessive pressure drop is detected in the Umlazi area, the response of the supervisory system is to switch on one of the pumps of the Chatsworth 1 / Umlazi booster pump. Similarly, when the volume in Chatsworth 4 reservoir reaches its low-emergency level, one of the pumps located at Northdene pump station is started until the volume in the reservoir reaches a satisfactory level again. It is obvious that the current way of operation is not optimal.

On Figure 2.19, it is seen that the water can be directed to the different reticulation zones connected to the Southern Aqueduct via several routes. Each route represents a different way of operation of the scheme: currently, there are three different scenarios to operate the Southern Aqueduct. The first scenario consists in maximising the use of the gravity between the Durban Heights waterworks and the Umlazi reticulation system. The second scenario conveys the water from Durban Heights to Umlazi by transferring it via the Chatsworth 4 reservoir. Finally, the third scenario consists in simultaneously feeding Chatsworth 1 and Chatsworth 4 reservoirs with the water arriving from Northdene.

Only the first scenario will be considered here, as it also formed the basis of the operational optimisation in this work. The use of the pumps is reduced as far as possible in favour of the use of gravity, where and when possible. To achieve this objective, the pump station located in the Northdene area is only used to supply Shallcross and Chatsworth 4 reservoirs with potable water. The water entering Chatsworth 4 reservoir does not go further than this reservoir, as shown on Figure 2.22. In this scenario, the pumping taking place at the Chatsworth 1 / Umlazi booster pump only occurs in case of unexpected consumption or during emergency situations. This first scenario is interesting in the way that it reduces the number of pumps that must be used to convey the water as far as the Umlazi area. This explains why this scenario, represented by Figure 2.22, was the reference for the operational optimisation.

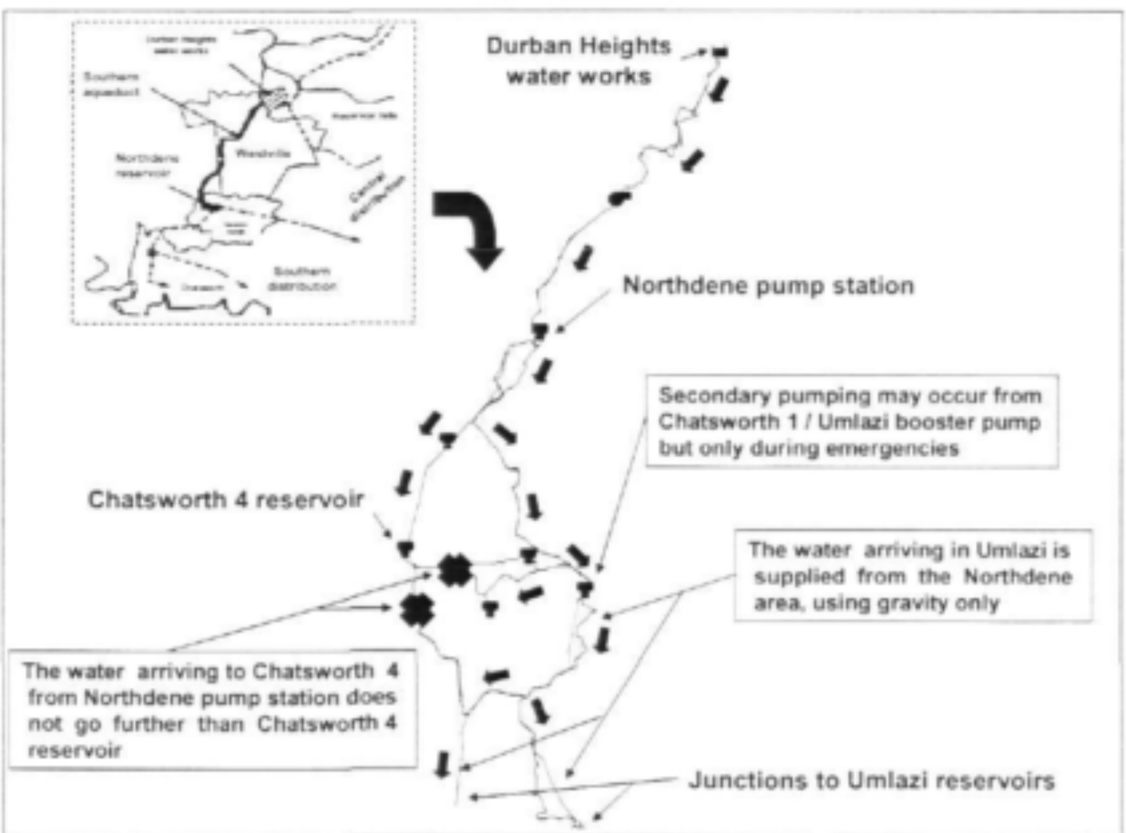

Figure 2.22 : *First scenario of operations of the Southern Aqueduct*

2.6.2 Operational optimisation of the Southern Aqueduct

To set up all of the data tables required for the operational optimisation of the system, a large amount of data collection had to be undertaken. Fortunately, an electronic version of the Southern Aqueduct, in the ePANET format, was available from eThekwini Water Services. This file gives a very accurate representation - it is mainly used to test the hydraulic feasibility of the different operating scenarios of the scheme, or to test the influence of any maintenance operations on the hydraulics of the system. In the present work the modelled version included 14 reservoirs, 4 pump stations equipped each with a single fixed-speed pump, 8 binary valves, 10 continuous valves and 15 connections with the reticulation system. This version was kept as a reference for the construction of the data tables required by the available flows strategy.

No post-chlorination is required after the potable water has left the Durban Heights waterworks. Therefore, water quality objectives have no real impact in this particular example of operational optimisation, and therefore they were dropped in favour of objectives concentrating on the satisfaction of volume setpoints and the respect of low / high emergency volumes. In other words, the results given by the optimisation software only had to comply with users' demand flows, as well as all of the constraints specified for low and high emergency volumes, and at the same time minimise the pumping cost.

All of the simulations were conducted over a period of 48 h, using a control optimisation horizon of 12 h. Therefore, according to the Model Predictive Control (MPC) theory presented in section 2.3.2, it was necessary to provide data tables with 60 h of known time-varying rates for electricity cost and demand flows. This allowed the algorithm to look ahead at future demands over the full period.

The electricity cost structure for this particular example is represented in Figure 2.23.

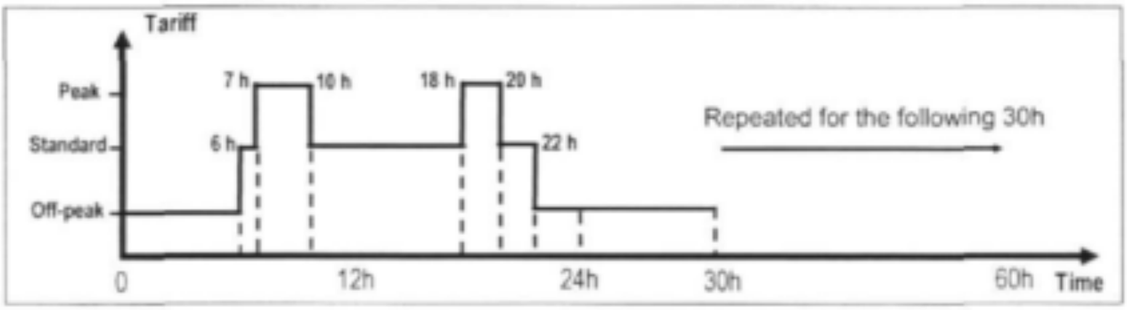

Figure 2.23 : *Electricity cost structures used with the Southern Aqueduct configuration*

This tariff structure, referred to as the *true electricity cost*, was based on the bulk supply agreement offered to industrial customers by Durban Metro Electricity. It is a 3-zone tariff structure divided into off-peak, standard and peak periods. While the off-peak power is charged at R 0.09610.kWh^{-1}, the standard power costs R 0.150009.kWh^{-1} and the peak power is billed R 0.268030.kWh^{-1} (2002 figures, courtesy of Durban Metro Electricity). Thus, with this tariff structure, the normal power is one and a half times more expensive than the off-peak power and the peak power is almost three times more expensive than the off-peak power.

The optimisation was performed with the MINLP solver DICOPT++, associated with the MILP solver CPLEX and the NLP solver CONOPT. An optimised sequence was achieved with respect to the constraints specified on the emergency volumes in the reservoirs, as well as the user demand flows.

The behaviour of the Lea Drive pump for the "true electricity cost" tariff structure (Figure 2.23) is presented in Figures 2.24 and 2.25.

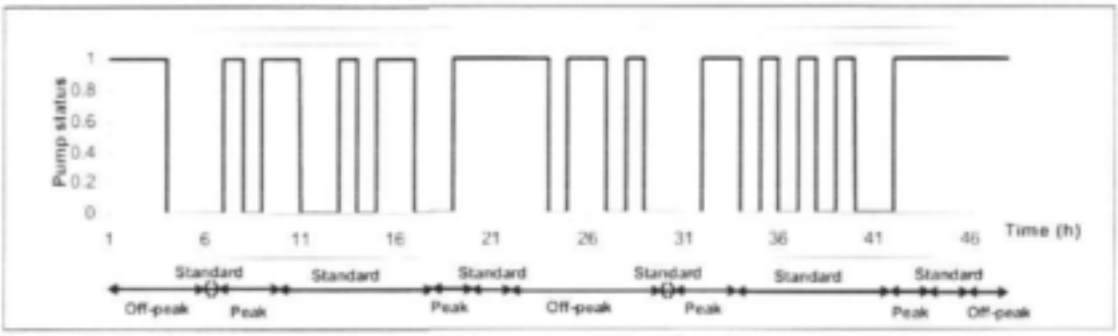

Figure 2.24 : *Pump status at the Lea Drive pump station for the true electricity cost structure*

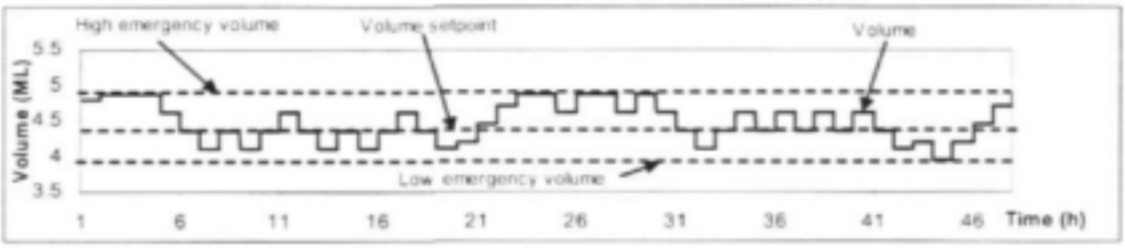

Figure 2.25 : *Evolution of the volume in Lea Drive reservoir (true electricity cost)*

The pump is on for the first four hours of the simulation: this is to make best use of the off-peak electricity and fill the reservoir while the electricity is cheaper. However, since the volume reaches its upper limit before the electricity tariff is switched to standard, the pump has to be turned off to respect this particular constraint. The pump remains off until the 7th hour, where it is used again for one hour. It can be observed at this point that the electricity tariff is at the peak rate. Therefore, turning the pump on at this particular time appears undesirable, but there is unfortunately no choice since the volume has reached its low-emergency constraint. This action brings the volume closer to its setpoint, which justifies stopping the pump. However, the demand out of the reservoir is steady, and turning the pump off causes an immediate a drop in the level. This makes it necessary to switch on the pump again at the next time-step, although the tariff is still at the peak rate. This on / off situation goes on until the 19th hour of simulation and, inside this interval, the optimiser needs to find a compromise between the amount of electricity consumed and the respect of the low-volume constraint. Then, the pump remains on until the end of the 23rd hour of simulation. This is to make best use of the cheaper electricity, which is charged in standard or off-peak rates. However, since the volume is quickly corrected by the use of the pump to reach its high emergency constraint, the pump must be turned off, although at the end of the 23rd hour the electricity cost still belongs to the off-peak tariff. When the second day of simulation starts, the situation in the reservoir is very similar to that at the beginning of the simulation. Therefore, the pump behaviour and volume evolution over the next 24 h are much the same as has just been described for the first day of simulation.

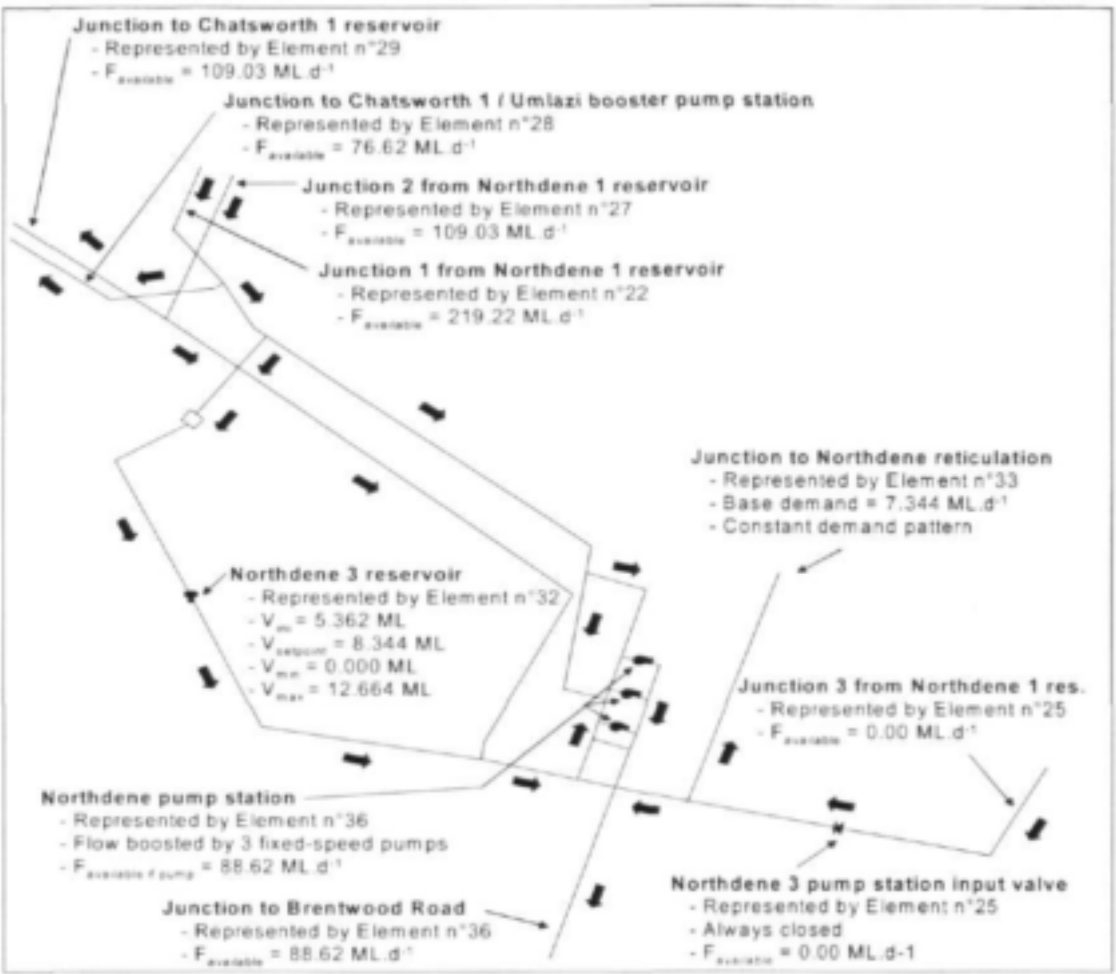

Figure 2.26 : *Northdene 3 reservoir, pump station and reticulation system*

Let us now describe the pump behaviour at the next pump station located in the Northdene area (Figure 2.26). This behaviour is quite different from that occurring at Lea Drive, as shown in Figure 2.27.

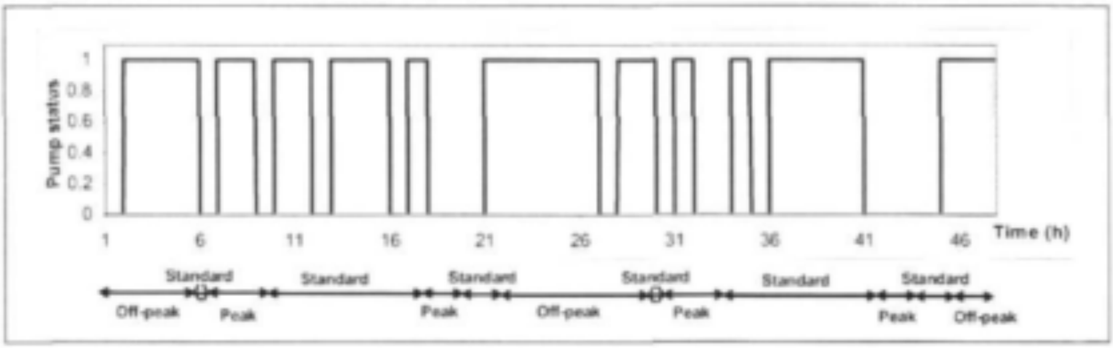

Figure 2.27 : *Pump status at the Northdene pump station for the true electricity cost structure*

The results obtained here are indeed less oscillatory. This can be explained, since the pump here is an in-line pump that is used to boost the flow leaving the Northdene reservoir system, for the remote areas of the water supply system (Shallcross, Harinager drive, Klaarwater and Chatsworth 4 reservoirs). In constrast, the pump used at Lea Drive was used to abstract water from the main distribution trunk and to convey it to a single relatively small reservoir.

Furthermore, the water flowing from the Northdene pump station can be directed to several different areas. As a result, when one of the reservoirs reaches its high emergency volume, there is no need to switch off the pump if the electricity is still charged at the off-peak rate. The water may indeed be conveyed to one of the other reservoirs of the area. Therefore, for this particular pump station, the results show that the proposed optimal pumping sequence makes best use of overnight pumping, with the pump rarely switching on during peak and standard periods. From an operational point of view, since the retention time in all of the reservoirs of the Southern Aqueduct is low, the reservoirs empty rather quickly; therefore, this explains why the pump in the facility of Northdene pump station has to be working for 30 h over the 48 h of the simulation.

The following pump station is located in the facility of Shallcross, just before the input of Klaarwater reservoir, as presented in Figure 2.28. Once again, the behaviour of this pump is in Figure 2.29.

The use of the tariff structure conforming to the bulk supply agreement gives a pumping sequence that is even less oscillatory than that of Northdene pump. Since this particular behaviour is linked to having the Klaarwater pump draw water out of a main line to convey it to a storage reservoir, the analysis of the optimised sequence of operation in the facility of Klaarwater can only be conducted by looking simultaneously at the evolution of the volume in Klaarwater reservoir (Figure 2.30).

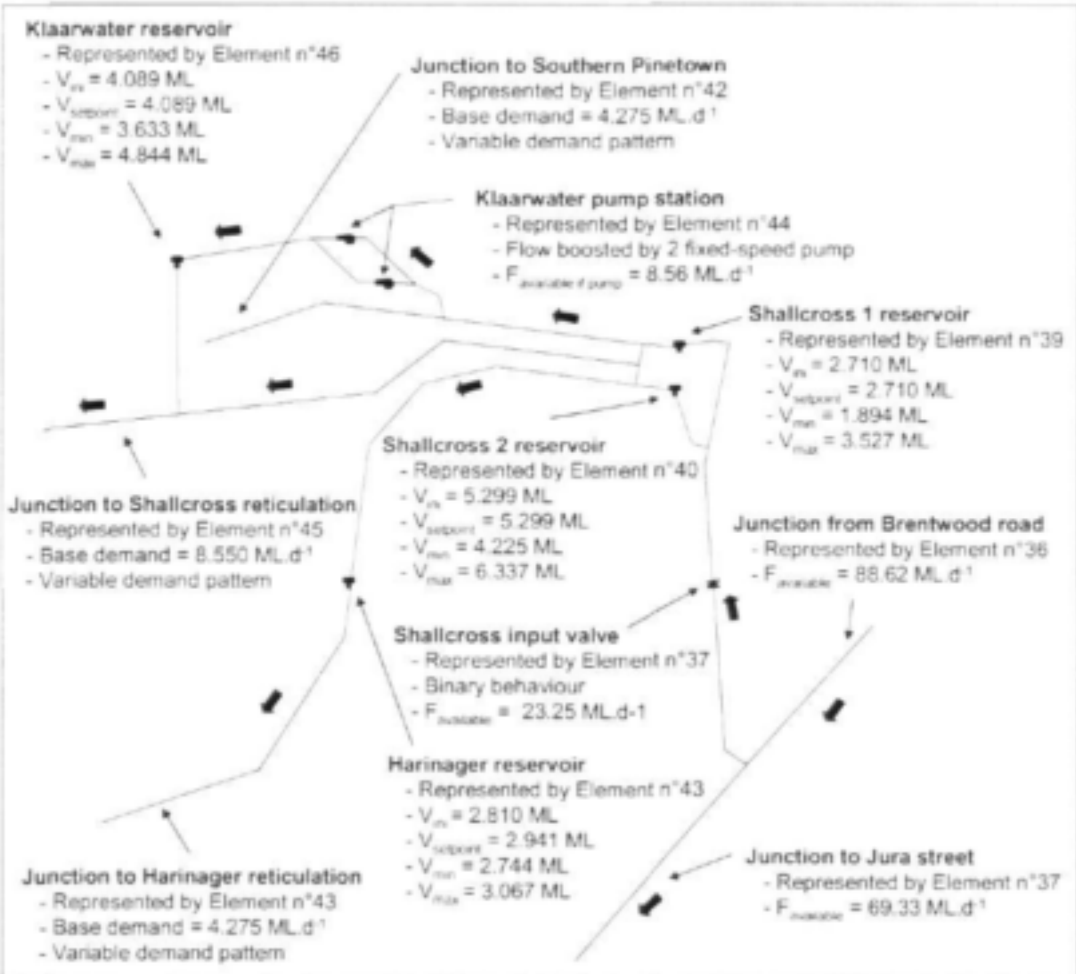

Figure 2.28 : *Shallcross area reservoirs and reticulation system*

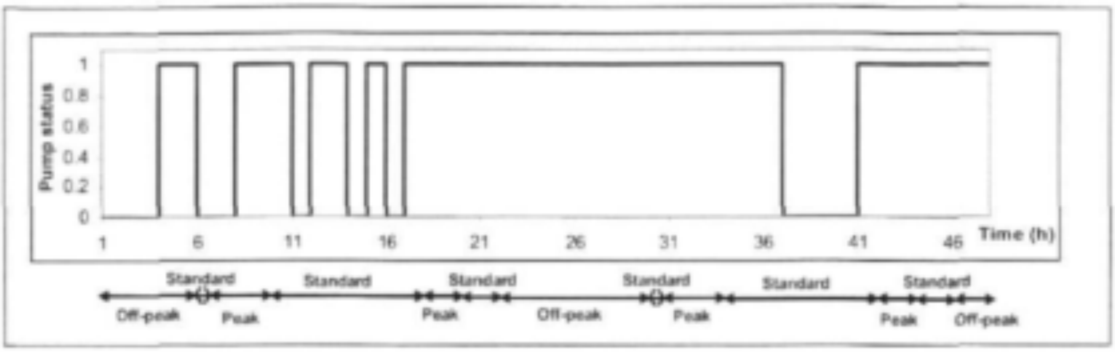

Figure 2.29 : *Pump status at the Klaarwater pump station for the true electricity cost structure*

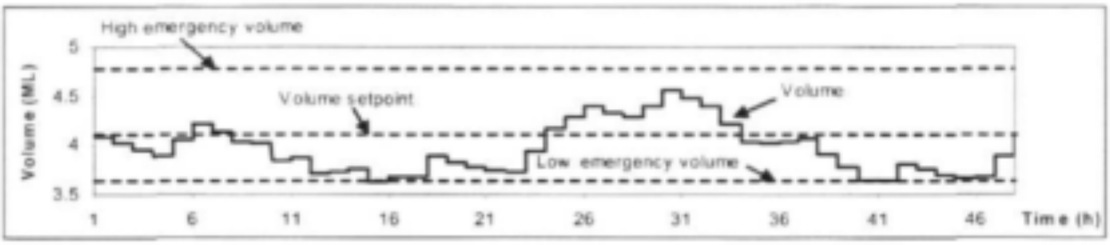

Figure 2.30 : *Evolution of the volume in Klaarwater reservoir (true electricity cost)*

The optimised pumping sequence starts with the pump off for the first four hours of the simulation, although the electricity is billed at the off-peak rate. Since Klaarwater reservoir volume is initialised at its setpoint, the optimiser prefers to distribute the water arriving from Northdene pump station to the Shallcross reservoirs and reticulation system where it is most needed. However, once water is again available for Klaarwater reservoir, it is sent to the reservoir to try to correct the deviation from the setpoint that resulted from not feeding the reservoir for the first four hours. Therefore, the Klaarwater pump is switched on for two hours, until the tariff switches to the normal rate, which reduces the gap between the current volume in the reservoir and its setpoint. From this point, it could have been expected that the pump would not switch on before the next off-peak period occurring from the 22[nd] hour of simulation onwards. However, this is not the case, and as Figure 2.29 shows, the pump is switched on almost for the rest of the simulation, not taking into account anymore the different tariff zones. Furthermore, despite the action of the pump, there is a lot of difficulty in reducing the gap between the reservoir volume and its setpoint.

The explanation for this unexpected behaviour can be found in Figure 2.31, representing the evolution of the user demand flow out of the Shallcross reticulation system, to which the output of Klaarwater reservoir is connected.

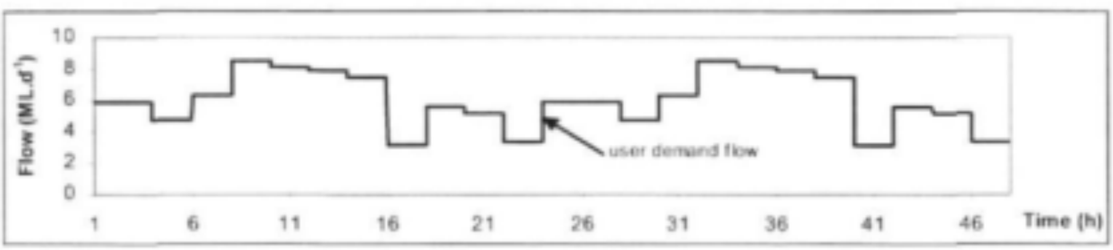

Figure 2.31 : *User demand flow out of the link to Shallcross reticulation system*

This figure shows clearly that at the time there is a need of water in Klaarwater reservoir, there is also a strong demand in water from the users located in the Shallcross area. Therefore, all of the water pumped into the Klaarwater reservoir is immediately sent to the reticulation system, which explains why the volume remains steady in spite of the action of the pump. In other words, the action of the pump is not sufficient to correct the deviation of the volume from its setpoint, and the level is not far from the lower constraint, which explains why the pump remains on during periods of peak electricity cost.

The situation in the Klaarwater reservoir only gets back to normal at the end of the 25^{th} hour of simulation, with the volume crossing over its setpoint. This is made possible by the use of the pump, in conjunction with a strong decrease in the consumption by the Shallcross reticulation system. Since at this time, the tariff is charged at the off-peak rate, the pump remains on for some time after the setpoint has been crossed. This is to make best use of overnight pumping, which allows storing water for further use. Unfortunately, the relief on this particular reservoir does not last for long, with the increase in the Shallcross user demand which materialises again from the 30^{th} hour of simulation. With the reservoir volume dropping quickly towards it low-emergency constraints, there is no other choice but to use the pump once again to prevent the volume crossing this low limit, with no regard to the current cost of energy.

To summarise the analysis conducted on the Klaarwater reservoir, it can be said that once again these results suggest that the emergency volume limits that have been applied to the Klaarwater reservoir are not optimal. Similarly to what was presented on the Lea Drive reservoir, important savings would be achieved on the pumping costs by allowing a more flexible range of operation on this particular reservoir.

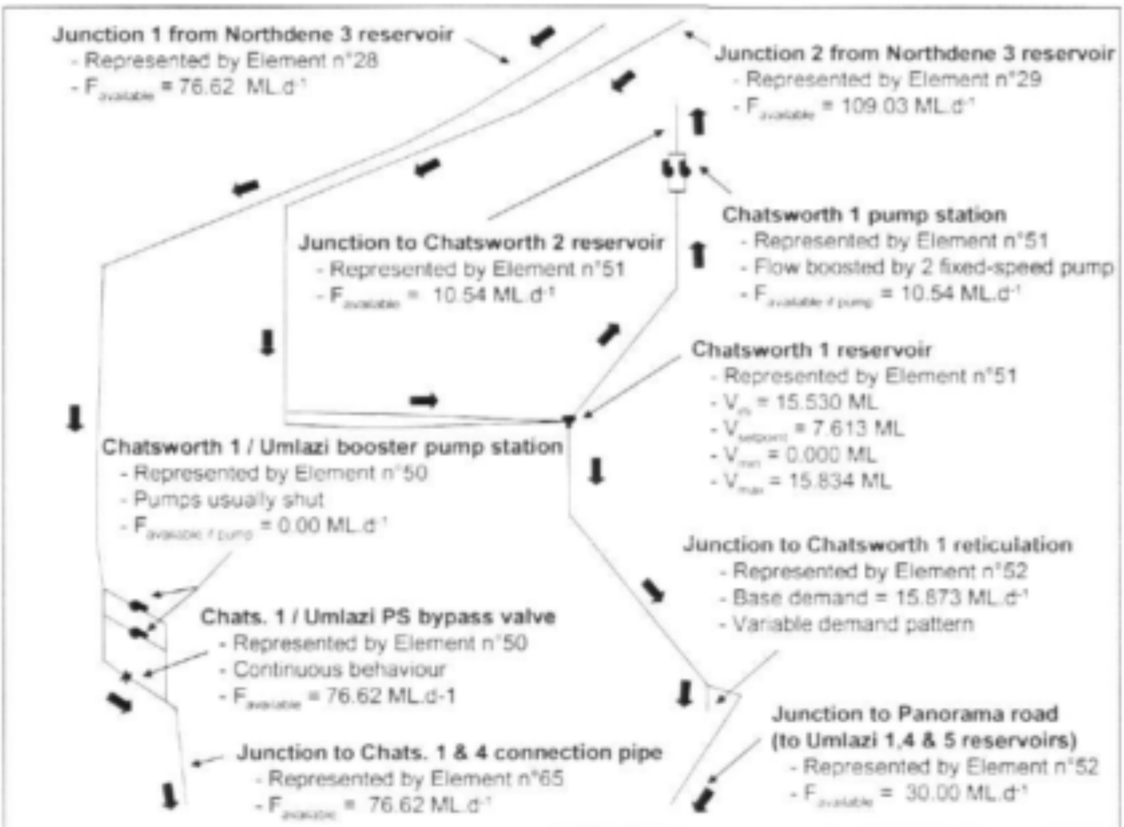

Figure 2.32 : *Chatsworth 1 reservoir, pump stations and reticulation area*

Finally, the last pump station integrated in the optimisation problem is in the area surrounding Chatsworth 1 reservoir. It is used to boost the flow used to feed Chatsworth 2 and Chatsworth 3 reservoirs (Figure 2.32). (The pumps located in the facility of Chatsworth 1 / Umlazi booster pump station are never switched on in this first scenario of operations).

For this particular pump station, the use of the electricity cost structure of the bulk supply agreement gives results that are more complex to analyse, as shown in Figure 2.33.

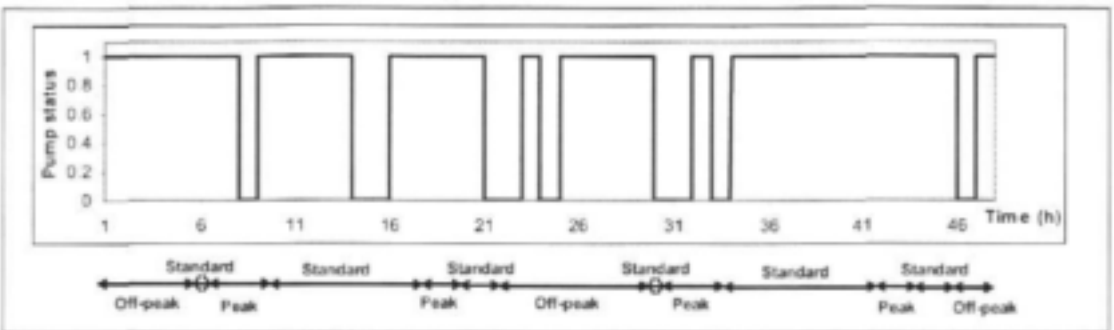

Figure 2.33 : *Pump status at the Chatsworth 1 pump station for the true electricity cost structure*

The pump is indeed switched on over almost the full length of the simulation (38 hours of use for a 48 h simulation), whereas the volume in Chatsworth 2 reservoir (reservoir to which this pump station is linked) never reaches its low-emergency constraint, as shown in Figure 2.34.

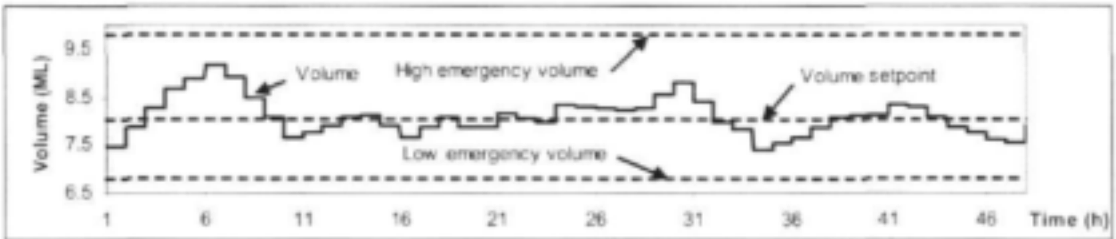

Figure 2.34 : *Evolution of the volume in Chatsworth 2 reservoir (true electricity cost)*

Furthermore, despite the continuous action of the pump, the volume of this reservoir remains relatively steady and around its setpoint. It could be thought that this comes from the strong user demand requested in the Chatsworth 2 area at the time, but as a combined look at Figure 2.33, Figure 2.34 and Figure 2.35 shows, the use of the pump does not allow any correction in the volume of the reservoir, even during low consumption periods.

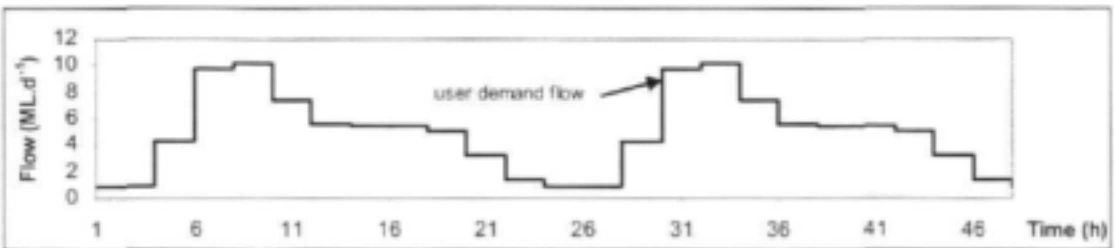

Figure 2.35 : *User demand flow out of the link to Chatsworth 2 reticulation system*

In fact, the explanation resides in the fact that there is a lack of water in this part of the Aqueduct at the time the pump tries to correct the volume in Chatsworth 2 reservoir. Therefore, the flow entering Chatsworth 2 reservoir by the action of the pump is far below its maximum available flow and does not succeed in bringing the volume in the reservoir much above its setpoint. This behaviour is an important characteristic of the available flows modelling strategy: the maximum available flow defined for each pipeline cannot be guaranteed at each-time step. This is another example, which shows that in general not all of the objectives of an optimisation problem can be satisfied simultaneously. Unfortunately, this behaviour will have strong consequences on the overall electricity cost.

Although it has not been possible to compare the results given by the optimiser with the current strategy of operation, it has nevertheless been possible to highlight some interesting behaviours of the optimiser. Several examples of the presence of the anticipation phenomenon have been given. This is one of the most important features of the optimiser regarding to the operational optimisation objective, since it makes best use of overnight pumping by storing water in the reservoir when the electricity is cheaper. Furthermore, it was shown that, by extending the modelling strategy to a large-scale application, there is even more chance of having to compromise between conflicting objectives.

Besides the final validation of the modelling choices presented earlier in this document, the results obtained on the approximate version of the Southern Aqueduct have also opened a new field of application for the particular modelling tool developed here. For instance, to improve the economic savings, it is clear that the software may be used to test the sensitivity to the emergency constraints chosen for the reservoirs. Furthermore, it could be envisaged to use this software tool to test the influence of different electricity patterns on the operational costs, therefore helping the management during their negotiations with electricity providers. Finally, this software could also be used for the development of new scenarios of operation that are more efficient from an economic point of view, by testing several configurations, on the same supply system. This would however require that the hydraulics of the new scenario be validated before by software similar to ePANET for instance (to obtain the "available flows").

However, before initiating further developments, like a commercial version of the modelling tool for instance, the next phase would be to calibrate more closely the model used to represent the Southern Aqueduct. This step would be necessary to estimate how large the savings on the operational costs could be on the Southern Aqueduct, with the new sequence of operations provided by the modelling tool. To check that this calibration is reliable, a good test would be to check that the rates at which the reservoirs fill are similar in both the model and the actual network.

It is indeed true that the available flows strategy on which this optimisation tool was based, while easier to solve, does not provide a complete representation of the hydraulic behaviour of a complex distribution system. Such a simplified model can only be relied upon if it is validated using a full hydraulic model of the system. This is why it is recommended to use this modelling tool in combination with the public domain software ePANET, to determine all the required maximum available flows, as explained in the above sections.

Nevertheless, it is now clear that the modelling method developed especially for the operational optimisation of an existing water distribution system needs some adaptations before it can provide on-line optimisations of such systems. The computational complexities escalate rapidly as the network size grows. It is now clear that the algorithms currently used, even with the most powerful PC computer technology available nowadays would not allow the solving of such large problems, in amounts of time compatible with an on-line application.

3. TECHNOLOGY TRANSFER

The project aimed to build capacity within the industry to appreciate and apply advanced control techniques, targetting senior management (for an appreciation of the potential), middle management (for project identification and management) and engineers (implementers of the advanced control systems

In the case of the chlorination control at the Umgeni Wiggins waterworks, the installed adaptive predictive controller has gained acceptance by the operating staff. In the process of setting up the system and solving a range of problems, it was necessary to work closely with management, operators, and control/ instrumentation technicians. There is an appreciation of the method there, and some ability to do basic maintenance on it. In the case of the optimised operation of water distribution for the eThekwini Water Services, there were a number of meetings with management, who appreciated the objectives. However, the study was largely into feasibility, and was performed off-line, so no finished software product was handed over.

When the projects first started, there were frequent meetings with the management staff at each institution, to agree on the objectives and facilitate the work. In the case of the chlorination control, liaison progressively moved downwards to those most directly involved (operators and technicians). For the water distribution, the main contact remained at the management level, with some input by other engineers/technicians, eg. for Epanet data or SCADA operation.

Unfortunately, the project did not succeed in the aim of training local students to become advanced control engineers. We were not able to recruit local candidates with the requisite mathematical and computational skills at this time. However, two full-time MScEng students from Toulouse, France, were ideally suited from the modelling and control perspective, and were available. One benefit from this has been to strengthen our cooperation with (and access to water and control technology held by) the two home institutions in Toulouse, France: LIPE-INSA (Laboratoire d'Ingénierie des Procédés de l'Environnement, Institut National des Sciences Appliquées) and CNRS/LAAS (Laboratoire d'Analyse et d'Architecture des Systèmes). The project team members have benefitted greatly from the experience and expertise of the French partners.

The overall aim of the project was *to demonstrate the use of advanced process control for integrated management of water and wastewater systems in the South African Water Industry*. This was achieved directly throught the implementation of the chlorination control system at the Wiggins Water Treatment Works, and indirectly through the following publications:

Biscos C, Mulholland M, LeLann M-V, Buckley CA, Brouckaert CJ Bailey R & Roustan M	Optimal operation of a potable water distribution network	**2001**	*International Water Association Conference on Water & Wastewater Management for Developing Countries, Kuala Lumpur, 29-31 October, 2001 AssociationConferInternational Water Conference on Water & Wastewater Management for Developing*
Pastre A, Mulholland M, LeLann M-V, Buckley CA, Brouckaert CJ, Roustan M & Mabeer V	Control of potable water chlorination	**2001**	*International Water Association Confer ence on Water & Wastewater Manage ment for Developing Countries, Kuala Lumpur, 29-31 October 2001 Associament for Developing Countries, Kuala Lumpur, 29-31 October 2001*
Pastre A, Mulholland M, Brouckaert CJ, Buckley CA, Le Lann M-V, Roustan M, Naidoo D and Mabeer V	Modelling and Control of Potable Water Chlorination	**2002**	***Water Science and Technology*, 46**, 9, 103-108 (2002)

Biscos C, Mulholland M, Le Lann M-V, Brouckaert CJ, Bailey R, and Roustan M	Optimal Operation of a Potable Water Distribution Network	**2002**	***Water Science and Technology*, 46**, 9, 155-162 (2002)
Biscos C, Mulholland M, LeLann M-V, Buckley CA, Brouckaert CJ & Roustan M	Water Distribution Network Optimisation	**2002**	*WISA 2002 Conference (Water Institute of South Africa) , Durban, May, 2002*
Pastre A, Mulholland M, Brouckaert CJ, Buckley CA, LeLann M-V, Roustan M & Naidoo D	Control of potable water chlorination	**2002**	*WISA 2002 Conference (Water Institute of South Africa) , Durban, May, 2002*
Biscos C, Mulholland M, Le Lann M-V, Buckley CA, and Brouckaert CJ	Optimal Operation of Water Distribution Networks by Predictive Control using MINLP	**2003**	***Water SA*, 29**, 4, 393-404 (2003)
Biscos C, Mulholland M, LeLann M-V, Buckley CA and Brouckaert CJ	Optimisation techniques applied to a potable water distribution system	**2003**	*2003 South African Chemical Engineers Congress, Sun City, 3-5 September, 2003.*

4. CONCLUSIONS AND RECOMMENDATIONS

The field of control engineering is a wide one with many diverse aspects, such as measurement techniques, sensors, signal conditioners, signal transmission systems, information flow, actuators and control algorithms. In this project the focus was purely on advanced control algorithms. The conclusions drawn here must be seen in this context, they have a firm basis where they relate to issues within this rather narrow focus, but become somewhat more speculative when they relate to other issues.

Conclusions

Although the project was envisaged as considering both water and wastewater applications, the two studies that were chosen after consultation with Umgeni Water and the eThekwini Municipality concerned only potable water, not wastewater. In South Africa wastewater treatments tend to be designed to operate without automatic control; this might explain the lesser interest in wastewater applications. This is not to suggest that waterwater treatment systems cannot benefit from the application of advanced control systems; however a great deal of infrastructure needs to be in place before one can apply the kind of algorithmic methodology that was the focus of this project. For instance, the measurements that are routinely made at wastewater treatment works to monitor the biological processes do not lend themselves to automatic control.

The chlorination control study led to a successful application of the technique at the Wiggins Water works. However this study did not really address the issue of integrated system control: it amounted to the use of advanced techniques to a control a difficult single process. On the other hand, the water distribution network was very much a complex integrated system. In this case the study only reached the stage of developing the control algorithm and demonstrating the feasibility of the concept. Implementation would have required a much greater logistical effort than for the chlorination control because of the extent of the system, and this required a political commitment and a budget from the eThekwini City Council: something that was not within the power of the project team to secure.

The water industry has lagged in taking advantage of advanced control techniques which are becoming common in the chemical processing industry. Process control does not appear to be considered a high priority in the industry. There is definitely potential to benefit economically and in terms of safety through the use of these methods, as demonstrated in more accurate/appropriate chlorine dosing (minimising excesses), minimisation of the use of high-tariff electricity, or merely complying with the minimum and maximum emergency levels required in reservoirs.

The barriers to implementation are largely related to capacity within the water industry. The belief is quite often expressed that advanced control can compensate for lack of sophisticated operational personnel, however this is only true to a limited extent. Supporting and maintaining the advanced control system requires even more sophisticated personnel, though possibly a relatively small number of them. Thus advanced control is not likely to provide a solution for small, poorly operated systems in the near future. If it is desired to have treatment plants operating under remote control and monitored by a central core of expert staff, then the industry needs to attract control engineers and build up this capacity. However the large cities in South Africa are developing sophisticated integrated systems in response to the need to extend services to as many people as possible while using as few resources as possible. Advanced control clearly has an important role to play in making these systems as efficient as possible.

Recommendations

Regarding chlorine dosing control

The situation encountered at the Wiggins Treatment Plant must be similar to those at many water works around the country. There is therefore scope to replicate the control scheme in patnership with a commercial vendor of process control systems.

The reasons why the process is difficult to control arise to some extent from the hydraulic design of the system, which was clearly undertaken without controllability in mind. New installations should be designed to make their control as easy as possible.

Nevertheless, despite the complex channelling and dead-volume behaviour predicted by CFD, the actual chlorine loss behaviour was quite adequately modelled using a single stirred reactor volume, albeit with a smaller active volume. Very significant variations in the effective chlorine decay rate constant were identified by the Kalman filter – by a factor of up to $2d^{-1}$. Although this was probably an artificial result of the simplified model, it was shown to control the process well in practice. Thus model based control is a viable option where circumstances make an awkward hydraulic design unavoidable.

Regarding water distribution control

The situation encountered in Durban's Southern Aquaduct is also similar in other large cities, so there is potential for the envisaged control system to be replicated widely. Because of the very large power requirements of water distribution systems, the peak load shedding method developed during the project could have significant implications for power distribution management as well as for water management. This could be a very beneficial line of research.

Further work needs to be done to make the control algorithm practically implementable. The main issues that still need to be addressed are reducing the computation time and incorporating feedback to account for deviations from anticipated water demands.

Regarding advanced control in the water industry

Both these studies made use of receding-horizon predictive controllers to achieve an optimal control. Such computer-based approaches have proven themselves in other industries (e.g. the oil and chemical industries), and the present study confirms that the water treatment and distribution industries would benefit in the same way.

There is a growing need for integrated management of resources, of which water is one of the most important. Advanced control methods will be required as part of the management strategy, and the water industry therefore needs to build technical capacity in this area. This will involve training and recruiting of control engineers and technicians, and promoting the application of advanced control through forums such as WISA. In the field of wastewater treatment, the philosophy of designing and operating wastewater treatment plants will have to be extensively revised in order to establish an environment which is conducive to the application of integrated control techniques so as to improve performance and reduce costs and environmental burdens.

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