

Water Quality Modelling in Distribution Networks

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ABSTRACT

The thesis is a treatise of the quantity and quality aspects of potable water in distribution systems. The privatisation of the UK Water Industry in 1989 has seen the requirement for the Water Companies in England and Wales to be responsible for the delivery of good quality water that meets the demand of all consumers. In respect of the quantity of supply, there have been many previous studies that have examined the hydraulic performance of distribution systems and there are now many proprietary mathematical models that have been successfully used in this study. However, in respect of water quality the literature review has highlighted that the modelling approach is not so well advanced, as water quality is a function of many concepts, processes and parameters that include the source and age of water, the condition and deterioration of the assets in the system, the microbiological, chemical and physical processes and the network hydraulic performance, including pressure transients. These processes are highly interactive and complex.

In an attempt to better understand these processes a programme of research has been completed that has involved a field evaluation of the performance of a live system, including the development of instrumentation to continually measure water quality, and the development of a mathematical model to describe the processes associated with the age of water and the propagation of conservative and non-conservative substances. An initial attempt has also been made to develop a micro-biological model and a sediment transport model.

New original concepts developed by the author include age, biological and diagnostic models that may be used to identify the source of any incident (hydraulic or pollution) and the application of the model in near real time.

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Chapter 1 - Introduction

1.1 Background

The Romans introduced the first water supply systems into the UK with the extensive construction of aqueducts to supply clean water to their fortresses with the subsequent construction of simple but effective sanitary waste disposal systems. However, the engineering feats of the Romans were not paralleled again until some 1500 years later when the religious communities, through their concern for personal hygiene, supplied their “lavatoriums” with water. For example, in Cambridge in 1325, the Franciscan monks developed a pipe and channel system to supply clean water from a remote spring to avoid having to use the contaminated water in the River Cam, and it was this system that remained the source of supply to Trinity College for 300 years – a testament to their engineering skills and forward thinking. (*King & Angel, 1992*).

As the population in the UK continued to grow, more water was required and, on behalf of whole communities, enterprising individuals established water supply companies with the development of water supply systems. However, the increased use of water created a new problem: that of waste disposal. Flushing toilets, baths and showers were all developed and the rapid increase in the amount of wastewater and waste material caused widespread pollution problems. This was most famously highlighted in 1858 by the “big stink” in London when the River Thames became unbearably odorous due to large quantities of decaying waste material. In addition, engineers and scientists began to prove the links between illness, the water supply and waste disposal. It was also observed that even clean water when passing through certain pipe systems could become contaminated by the pipe material itself or by ingress from the surrounding ground where faecal waste material was buried. Cholera, typhoid and diarrhoea epidemics were the result of waterborne disease organisms and it was recognised that there was a need to isolate sewage from drinking water.

The ‘potability’ of water first became an issue in 1827 when the Government appointed Thomas Telford to report on the status of London’s water supply. He reported that:

“The growth of the population and with it pollution, the establishment of gas works and factories, the hopeless disorganisation of supply and distribution, the ruinous rivalry between rival water companies, have brought matters to an impossible state”.

(Sir Alexander Gibb, The Story of Telford, 1935).

It was the perennial subject of petition and complaint and came before every session of Parliament. Certainly the situation was unsatisfactory and a series of legislative steps followed in an attempt to resolve the problem.

1.1.1 The Royal Commissions

Many Royal Commissions were established to review the problem but it was not until the Public Health Act of 1936 and the disastrous Cholera outbreak in Croydon in 1937 that substantial progress was made to isolate sewage from potable water. The early systems have now been substantially developed to form a complex system of sewer pipes with relief overflows, storage and sewage treatment facilities, and water treatment plants supplying complex looped water distribution networks supported by pumps, valves and water storage facilities protected from pollution.

1.1.2 Local Government Reorganisation

In 1974 Local Government reorganisation resulted in the development of Regional Water Authorities. Many small water undertakings within regions were amalgamated and made responsible for the management of the supply and distribution of drinking water, waste disposal and all water resources within each region.

The National Rivers Authority was responsible for the condition of the river systems and had the power to prosecute polluters. However, the Regional Water Authorities of England and Wales were responsible for the collection and analysis of river samples and hence were both policeman and poacher when it came to pollution of rivers and watercourses. The Regional Water Authorities were also responsible for the analysis of the quality of the drinking water that they supplied.

1.1.3 Privatisation of the Water Industry and Legislation

In 1989 the water industry in England and Wales was privatised with the Environment Agency assuming responsibility for water resources and the quality of rivers, estuaries and coastal waters.

Water Companies concentrated on the supply of drinking water and in some cases the treatment and disposal of wastewater. The newly formed Water Companies were controlled by new legislation, policed by OFWAT, the Office of Water Services, to ensure:

- The economic regulation of the water industry
- (Setting limits on what water and sewerage companies can charge customers)
- Water and sewerage companies carry out their responsibilities under the Water Act 1991
- Inter company comparisons
- Protection of customers' standards of service
- Encouraging companies to be more efficient
- Undertaking activities to allow effective competition to develop

The most important statutory instruments included the Water Act (1989) and the Water Supply Regulations (1989).

1.1.3.1 The Water Act (1989)

The Water Act outlines the various powers given to Local Authorities, the National Rivers Authority and the Secretary of State. The Act requires that *a water company should supply customers' premises with water at a minimum pressure and flow all times*. The Act also stated that *the water should be of an appropriate quality as well as quantity*.

Areas of distribution networks may suffer from low-pressure problems because of their elevation, a combination of poor mains condition and sudden rises in demand resulting in high friction losses, or the inability of the network to support demand at times of peak flow. This may be brought about, for example, by maintenance of the system or bursts/leaks.

The minimum standards of service as defined by the Regulators State that areas where the pressure falls below 18 metres water column (mwc) at any time during a 24-hour period are deemed to be failing. It is also required to provide a continuous water supply 24 hours per day and financial penalties are imposed for leaving customers without water for periods of time without prior warning (to permit repair and maintenance work to be undertaken on the system).

The water quality criteria in the Water Act are dictated by and detailed in The Water Supply (Water Quality) Regulations.

1.1.3.2 The Water Supply (Water Quality) Regulations 1989

For England and Wales, the drinking water quality criteria are set out in the Water Supply (Water Quality) Regulations 1989. They stipulate the maximum concentration or acceptable level of a large number of substances. These standards are shown in Table 1.1.

Item	Parameters	Units of Measurement	Concentration or Value (maximum unless otherwise stated)
1.	Colour	mg/1 Pt/Co scale	20
2.	Turbidity (including suspended solids)	Formazin turbidity units	4
3.	Odour (including hydrogen sulphide)	Dilution number	3 at 25°C
4.	Taste	Dilution number	3 at 25°C
5.	Temperature	°C	25
6.	Hydrogen ion	pH value	9.5 5.5 (minimum)
7.	Sulphate	mg SO ₄ /l	250
8.	Magnesium	mg Mg/l	50
9.	Sodium	mg Na/l	150(i)
10.	Potassium	mgK/l	12
11.	Dry residues	mg/l	1500 (after drying at 180°C)
12.	Nitrate	mg NO ₃ /l	50
13.	Nitrite	mg NO ₂ /l	0.1
14.	Ammonium (ammonia and ammonium ions)	mg NH ₄ /l	0.5
15.	Kjeldahl nitrogen	mg N/l	1
16.	Oxidizability (permanganate value)	mg O ₂ /l	5
17.	Total organic carbon	mgC/l	No significant increase over that normally observed
18.	Dissolved or emulsified hydrocarbons (after extraction with petroleum ether); mineral oils	µg/l	10
19.	Phenols	µg C ₆ H ₅ OH/l	0.5
20.	Surfactants	µg/l (as lauryl sulphate)	200
21.	Aluminium	µg Al/l	200
22.	Iron	µg Fe/l	200
23.	Manganese	µg Mn/l	50
24.	Copper	µg Cu/l	3000
25.	Zinc	µg Zn/l	5000
26.	Phosphorus	µg P/l	2200
27.	Fluoride	µg F/l	1500
28.	Silver	µg Ag/l	10(ii)

Table 1.1a Prescribed concentrations and values Table A

Note (i) See regulation 3(5).

(ii) If silver is used in a water treatment process, 80 may be substituted for 10.

	<i>Parameters</i>	<i>Units a/Measurement</i>	<i>Maximum Concentration</i>
1.	Arsenic	µg As/l	50
2.	Cadmium	µg Cd/l	5
3.	Cyanide	µg CN/l	50
4.	Chromium	µg Cr/l	50
5.	Mercury	µg Hg/l	1
6.	Nickel	µg Ni/l	50
7.	Lead	µg Pb/l	50
8.	Antimony	µg Sb/l	10
9.	Selenium	µg Se/l	10
10.	Pesticides and related Products:		
	(a) Individual substances	µg /l	0.1
	(b) total substances(i)	µg /l	0.5
11.	Polycyclic aromatic hydrocarbons(ii)	µg /l	0.2

Table 1.1b Prescribed concentrations and values Table B

Notes (i) The sum of the detected concentrations of individual substances.

(ii) The sum of the detected concentrations of fluoranthene, benzo 3.4 fluoranthene, benzo 11.12 fluoranthene, benzo 3.4 pyrene, benzo 1.12 perylene and indeno (1,2,3-cd) pyrene.

<i>Item</i>	<i>Parameters</i>	<i>Units of Measurement</i>	<i>Maximum Concentration</i>
1.	Total coliforms	Number/ 100 ml	0(i)
2.	Faecal coliforms	Number/ 100 ml	0
3.	Faecal streptococci	Number/ 100 ml	0
4.	Sulphite-reducing clostridia	Number/ 20 ml	<=1 (ii)
5.	Colony counts	Number/ 1 ml at 22°C or 37°C	No significant increase over that normally observed

Table 1.1c Prescribed concentrations and values Table C

Notes (i) See regulation 3(6) to(8).

(ii) Analysis by multiple tube method.

Item	Parameters	Units of Measurement	Maximum Concentration or Value
1.	Conductivity	µS/cm	1500at20°C
2.	Chloride	Mg Cl/l	400
3.	Calcium	mg Ca/l	250
4.	Substances extractable in chloroform	mg/l dry residue	1
5.	Boron	µg B/l	2000
6.	Barium	µg Ba/l	1000
7.	Benzo 3,4 pyrene	ng/l	10
8.	Tetrachloromethane	µg /l	3
9.	Trichloroethene	µg /l	30
10.	Tetrachloroethene	µg /l	10

Table 1.1d Prescribed concentrations and values Table D

Note: (i) See regulation 3(3)(d).

Item	Parameters	Units of Measurement	Minimum Concentration (l)
1.	Total hardness	mg Ca/l	60 (Ed note: equiv 150 as CaCO ₃)
2.	Alkalinity	mg HCO ₃ /l	30 (Ed note: equiv 25 as CaCO ₃)

Table 1.1e Prescribed concentrations and values Table E

Note: (i) See regulation 3(2).

In addition to quality standards, the regulations also stipulate the minimum water sampling frequencies and describe the methodology for the creation of sampling 'zones'. (Section 3.1).

Since the Water Supply Regulations came into force, amendments and a number of EU Directives have supplemented them. The most important of these are the Water Supply (Water Quality) (Amendment) Regulations 1989 and 1999 that take account of the need for protection from Cryptosporidium, Nitrates and Pesticides.

The newest EU legislation pertaining to drinking water is the EC Drinking Water Directive 1998. In order to implement the requirements of the EU Directive, the UK Government produced new drinking water quality standards, The Consolidated Water Quality Regulations that came into force in December 1999. The Directive concerns not only drinking water quality standards, but also requirements for sampling and analysis, reporting, approval of materials for use in contact with drinking water, and necessary actions if the standards are breached.

The Drinking Water Inspectorate has the power to prosecute those who fail to meet the required standards and it is feasible that a water company could lose its operating license for serious

breach of the water quality standards of service. It is required therefore to understand the reasons why water quality failures occur and there is a clear need to better understand how the quality of water changes as it moves through the distribution network.

1.1.3 Water Quality

Changes in water quality are a function of many parameters but most processes are related to the age of water, the assets within the system (*Jones, 1993*), the microbiological, chemical and physical processes and the network hydraulic performance characteristics (*Kroon et al., 1990*). Some of the many network interactions are shown in Figure 1.1.

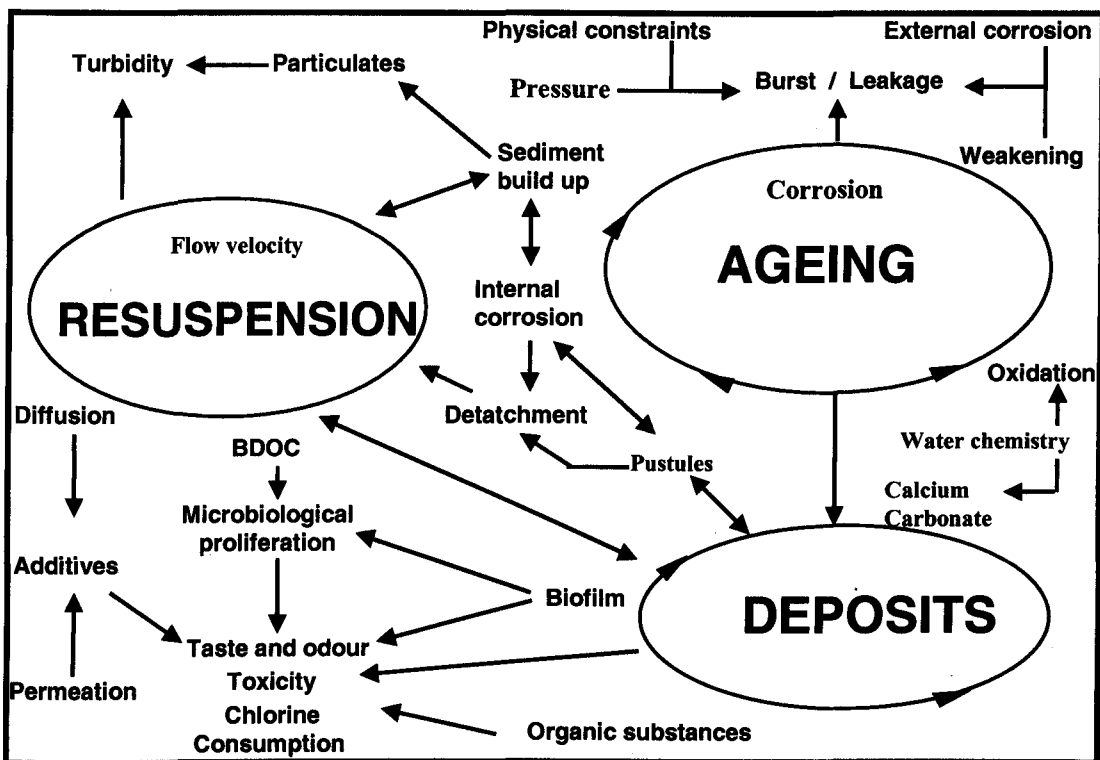


Figure 1.1 Some of the interactions within a water distribution network

The diagram shows the importance of understanding the relationships between hydraulic operation and water quality if these are to be managed effectively. This was recognised by the Drinking Water Inspectorate in 1999 as the following statement demonstrates:

“Companies have, or indeed should have, accurate simulation models of their distribution systems and these should be in use as an efficient tool for planning their operations” (Rouse, (DWI) 2000).

1.1.4 Simulation Models

The best of the current suites of mathematical models that may be used to predict such changes are lacking in that they cannot, for example, be applied to large complex networks (EPANet). Others have a lack functionality, for example, can not simulate certain dynamic network elements or do not have water quality simulation functionality, (EPANet, Stoner, LICwater, Picollo), or they do not take account of important factors (EPANet, Stoner, LICwater, Picollo).

Where models *are* adequate for purpose, they tend to be applied in a manner designed to resolve a specific network issue, for example, to resolve a pressure problem. Rarely do they simultaneously take account of other important and related aspects of network performance such as water quality, leakage or surge effects all of which are subject to regulatory control and are important factors for effective financial management of distribution networks.

This thesis therefore details the development of a mathematical model to obtain a better understanding of water quality within a distribution network by accurately calculating the age of water, and as to how the concentration of conservative substances change as the age related water travels through the network. As the change in water quality is clearly related to the hydraulic operation of the distribution network, the model that has been developed links the hydraulic and water quality functionality into a single entity.

The focus of attention to improve and maintain the quality of water delivered to the point of use requires an understanding of changes in the quality of treated water as it is transported through the distribution network. Hence the model developed may also be used to assist Scientists and Engineers to ensure the pipe networks used to deliver drinking water to the customer does not cause the treated water quality to deteriorate during transportation and that the system is operated in an efficient manner in order to provide appropriate service at minimum cost.

1.2 What is a Drinking Water Distribution Network?

The function of today's distribution system is to convey drinking water from the site of treatment to the customer. There is now a statutory obligation on the water companies of England and Wales to provide users in all parts of their geographical areas with an adequate water supply, 24 hours a day, which meets regulatory and industry water quality standards. (*Water Act, 1989*) (*Water Supply (Water Quality) regulations, 1989*), (*Water Industry Act, 1991*)

Following treatment, water is dispatched to the customers through a network of pipes called a distribution system or network.

A distribution network consists of the following components:

Storage devices such as service reservoirs and/or water towers

Water mains

Service pipes

Pumping stations

Valves and other fittings necessary to operate and control the system

1.2.1 Service Reservoirs.

The purpose of a service reservoir is threefold.

It provides storage to balance fluctuations in user demand during the day that can reach peak flows of approximately twice the average.

It provides strategic storage to safeguard supplies in the event of a system failure upstream of the reservoir. (This is usually twenty-four hours supply for the area it serves, but may be more in remote rural or strategic locations).

It provides a facility for blending and balancing waters from different sources.

Service reservoirs vary in size from a few cubic metres to over one hundred thousand cubic metres. They are constructed in a variety of materials including masonry, reinforced concrete, rigid plastic and coated steel.

Service reservoirs are located as close as possible to the population area that they are designed to serve at an elevation that will provide sufficient pressure to provide an adequate supply. Where the topography of the countryside does not permit this, a water tower fed by pumps may be used in conjunction with a service reservoir.

Service reservoirs must be covered to prevent pollution and must be water tight not only to inhibit leakage but also to prevent contamination by ingress water. Anti pollution measures such as secondary disinfection using chlorine or ultraviolet light are often used at service reservoir sites.

1.2.2 Water Mains

Water mains are categorised into trunk mains and service or distribution mains.

Trunk mains are usually defined as the strategic mains that supply storage facilities such as service reservoirs with water from treatment sites, and the larger mains downstream of service reservoirs that feed the service / distribution mains.

In urban areas trunk mains are frequently up to 1200mm in diameter. They are often arranged as ring mains to permit flow in more than one direction which, with suitable cross connections and inter linking, can be of great value in maintaining supplies when pipe failures occur and at times of excessively high demand.

Distribution mains range from 50mm diameter upward. The older main materials include cast iron, spun iron, asbestos cement and galvanised steel, and the newer materials are ductile iron, rigid plastics and medium density polyethylene.

This network of pipes is expanded continuously to meet the needs of new developments. Replacement or relining to maintain hydraulic and water quality performance requires continuous rehabilitation of the older mains. Iron mains may suffer internal and /or external corrosion caused by inadequate water treatment and lack of internal or external protection.

Modern techniques for rehabilitation include narrow trenching, pipe bursting, moling, slip lining, and pipe coating using concrete or epoxy resin linings.

1.2.3 Service Pipes

The water companies are responsible for that section of a service pipe to a property known as the communication pipe, which usually extends from the distribution main to the property boundary. The cost of maintenance and repair of these pipes can be almost as high as that of mains in some areas.

Materials include lead, galvanised iron or steel, polythene and copper. Replacement strategies tend to favour medium density polyethylene.

Many older properties are fed from joint service pipes where responsibility is shared between the various properties receiving a supply.

Pipe sizes vary from 12mm for individual domestic property connections to large pipe connections for industrial users.

1.2.4 Pumps

Water is often transferred to high-level service reservoirs and / or water towers using pumps. Energy conscious management of networks usually dictates the use of cheaper night energy and this is a continuous process.

Booster pumps, normally operating only when demand requires, are sometimes employed to maintain a minimum pressure in a network. Such systems would be used to service high-rise buildings and local geographical high points.

1.2.3 Valves and other fittings

Valves, pumps and other facilities are used to control the operation of the network. For effective operation and control due regard is given to the following parameters:

Flow

Pressure

Water quality

Leakage & unaccounted for water

Zone boundary adjustments

The planning / design of new supplies

Planning and carrying out main rehabilitation schemes

Repairing mains and renewing / repairing services, valves and other fittings

Cleaning, swabbing and flushing pipes

Maintenance of fire hydrants

Cleaning and repair of service reservoirs

As well as being the vehicle with which to deliver drinking water to customers the nature of a water supply / distribution network makes it an additional stage of the water treatment process. It is necessary therefore to understand the complex relationships between the component parts and how they interact to affect hydraulic and water quality performance. Figure 1 shows some of the complexity of the interactions within a typical distribution network.

Many of the processes have an influence on each other and hence an understanding of these interactions is required in order to ensure regulatory performance targets are met. Some are directly related and have to be understood and controlled if efficient operation is to be maintained.

For example internal corrosion may result in high roughness coefficients and hydraulic gradients and these may adversely affect the flow and subsequently the water quality at the customers tap.

It is clear that with a high degree of understanding of each process / parameter / interaction there is a better chance of effecting efficient hydraulic operation and simultaneous water quality management.

Mathematical models have traditionally been used to ensure that customers receive the required flow and pressure at their properties. In general however, modelling approaches that combine the use of hydraulic and water quality models in a holistic manner to facilitate network operation and control has not been implemented. The model developed in this thesis highlights how such an approach can be implemented to provide multiple operational benefits.

1.3 Leakage and unaccounted for water

Distribution networks suffer from leaks. Leakage may be defined as the water that is “lost” between two flow measurement points situated at the inlet and outlet of a network zone after legitimate use has been accounted for. Loss may be due to illegal or unaccounted for use or genuine loss through leaking joints or bursts. Leakage levels of 30% were common.

In 1995, Britain suffered its worst drought for many years and some of the water companies in England and Wales had difficulty in maintaining a water supply. Consequently, the Water Companies, the Environment Agency and the Office of Water Services collaborated to produce a ten-point plan for the water industry that addressed the need for demand management and careful long-term assessment of the balance between supply and demand. One outcome was that OFWAT imposed mandatory leakage targets on every water company. In general, these targets have resulted in an overall reduction of the level of leakage. However, with pressure on resources becoming ever greater it is important to continue to develop better and more efficient leakage detection and location methods to lower the economic level of leakage.

As leakage and unaccounted for water are both integral components of a mathematical model that is used to describe the behaviour of the hydraulic performance of a distribution network it is logical that modelling leakage should be part of the holistic approach to distribution network management.

The research described in the thesis identifies how the hydraulic model may be applied to monitor and minimise leakage in a network whilst simultaneously considering the effects on pressure management and water quality.

A number of water companies have indicated that a significant amount of their leakage is caused by bursts that were thought to have originated as a result of transient pressure effects in their distribution networks.

Such transients, or surge effects, occur when a sudden change takes place in the state of a pipe system, for example, a sudden change in flow associated with the stopping of a pump or the closing of a valve. When this happens the kinetic energy carried by the fluid is rapidly converted into strain energy in the pipe walls and fluid when the flow is halted. This results in a pulse wave (pressure wave with increased or reduced pressure) that travels along the pipes of the network, spreading out from the point of generation. As the pressure wave travels through the network, energy transformation losses such as frictional losses and expansion of the pipe walls act so as to cause the wave to gradually decay until normal steady state conditions are once again restored.

In networks and pipelines, the movement of the pressure waves is complicated by the waves being reflected by closed valves, dead ends, reservoirs, pumps and other network assets so that complex patterns of waves develop.

Such surge waves can cause pipes to fail by a number of methods; for example, if the transient pressure is sufficiently high it might cause the pipe to fracture. If the pressure is small, cavitation may result, and the pipe could buckle. In addition, repeated surge events can result in metal fatigue that ultimately results in a burst.

Acceptable surge pressures are outlined in the British Standard (BS EN 1295) for the installation of plastic pipes such as PVC, HPPE, and MDPE. Pipes should not be subjected to surge pressures with amplitude greater than $\frac{1}{2}$ of their upper pressure rating. For example, a pipe with a maximum pressure rating of 100 MWC should not be subjected to surge pressures with amplitude greater than 50 MWC.

Historically, little regard has been paid to the control of surge effects within networks and this has led to many systems operating with surge pressures regularly stressing the pipe work and causing damage. To understand the impact of these waves on system performance, it is necessary to have a simulation tool that describes the governing processes for transients.

Then, by the application of a mathematical model to predict the effects of surge it is feasible to implement simple changes to either the operation or design of the distribution network that could alleviate the problem. A model to predict the pressure changes due to surge has therefore been developed as part of research programme described in the thesis.

1.4 The Asset Management Process

The upkeep, control, and operation of drinking water distribution networks can account for up to 80% of the capital costs credited to supply, treatment, and distribution of potable water. (*Clark, 1993*). The Asset Management Process (AMP) controls the level of funding available to operate and maintain the necessary assets.

During the early stages of the privatisation process, each water company had to declare their assets and their estimated value. Further, they had to put forward proposals for outlining future capital investment programs, which had to be supported by written evidence of need and cost. These proposals were scrutinised by the office of the Director General, Ofwat, to establish the 'K' factor. K is a factor in a mathematical formula. The formula is used to determine the level of price increase above inflation which water companies are judged to need to finance the capital programme for necessary improvement work to the network in order to meet the newly established statutory standards of service.

Consequently, the water companies commissioned Asset Management Studies and, for drinking water distribution systems, four main areas of work were addressed. These work areas included:

- A list, and description, of the physical characteristics of all underground assets (water mains, fittings and fixtures)

- Associated proposals for hydraulic rehabilitation in order to meet current and future (year 2015/16) pressure and flow requirements

- Proposals for structural rehabilitation where burst frequencies per unit length of main put customers at risk of supply interruptions

- Water quality rehabilitation schemes designed to make all supplies meet drinking water quality standards at the customer's taps by the year 2003

Subsequently, capital schemes were implemented to address rehabilitation and maintenance needs of networks that were identified as deficient by these Asset Management studies.

It was realised that the development of appropriate mathematical models could play a vital role in the rehabilitation strategy adopted. Such modelling could be used to determine the balance between the hydraulic, structural and water quality requirements of a distribution network and deliver an integrated solution. Subsequent post project appraisal could then measure the effectiveness of any scheme against predicted benefits and costs, such that OFWAT could better determine if customers were getting value for money for a given capital investment.

1.5 Industrial drivers

Since privatisation, the water companies in England and Wales have seen significant changes in working conditions and practices. Staffing levels have been reduced significantly and different methods of working, supported by new technologies, are being introduced in an attempt to create more efficient (as required by OFWAT) and more profitable businesses (as necessary for shareholders).

Research was seen as having an essential role to play in assisting the industry to meet the new legislative and efficiency requirements of the privatised business. However, there was a clear need for *additional* R&D to identify best working practices, the most available and appropriate of the new and emerging technologies, and as to how to exploit them to maximum effect. Some of these research initiatives and strategies formed essential components of an overall industry strategy and it was from within this framework that this research project was initiated. The application of the model developed as part of this thesis showed that it was possible to better understand all aspects of the water supply process and to identify ways of meeting the regulatory requirements whilst, at the same time, introducing operational efficiencies.

1.6 Water Quality

It has been show that discoloured or unpalatable water in a distribution network might arise from any, or a combination, of the following factors:

- A breakdown of a water treatment process
- Internal corrosion of iron pipes
- A reversal of flow direction within a pipe
- A disturbance of the sediment deposits in pipes

The ingress of polluting material

Bacteriological activity

Stagnation

The impact of chemical dosing e.g. secondary chlorination

Machell, (1996), demonstrated a link between water quality complaints in a distribution network and the hydraulic operation of the network.

One of the key factors in determining the quality of water at a particular location within a network is the age of the water at that location. Age of water has been associated with loss of disinfectant residual, taste and odours, increased biological activity, corrosion, and discoloured water. (*Banks 1997, Zegerholm & Bergstrom, 1996, Boulos et al 1992, Haudidier et al, 1988, Mathieu et al, 1993, Block et al, 1995 and Mallevalle 1982*)

Chlorine, used as the disinfectant at many water treatment sites, is a non-conservative substance. It decays over time because of reactions within the bulk water flow and at the pipe walls. The longer the time of travel between the point of chlorine addition and the point of use, the lower the level of chlorine remaining in solution.

Chlorine is used as a disinfectant to kill bacteria and inhibit their future growth. Because long transit times decrease chlorine residuals, the likelihood of bacteriological re-growth is increased. Where chlorine residuals are constantly low, it is possible for bio-films to develop on the pipe walls. Material leaving the bio-film may increase the bacteriological activity in the bulk water flow leading to regulatory failures and, possibly, customer complaints. (*Keevil et al, 1992*)

Areas of a distribution network where age is shown to be the greatest should be considered during network zone design and appraisal. Where possible, new network zone configurations should be such that the age of water in the network is minimised. If the overall age profile of water within a network is reduced it follows that associated water quality issues will also be reduced.

When the overall age profile is minimised the remaining high age areas of the network (if any) may be targeted for water quality monitoring. These areas will be more susceptible to quality failures, will be indicative of the worst water quality within the zone under normal operating conditions, and so may be targeted for proactive maintenance such as flushing. Increasing the age of water because of refurbishment or rezoning should therefore be avoided where practicable and application of the model may be used to minimise age profiles across entire networks.

As well as financial penalties for delivering water of an unsatisfactory standard to consumers, there is a possibility of having an operators licence revoked. The cost associated with investigation of these incidents can also be relatively high so taking proactive action to minimise the risk of a failure event has multiple benefits.

Boxall et al (2002) described an approach to predict the occurrence of discoloration events in distribution networks. In this study, impact of a flush event may be simulated and a model has been developed to predict how such discoloration events occur. This represents a major advance in respect of discoloration, but there are many other factors that influence water quality.

1.7 Other Factors

Because network performance requirements change as housing and industry developments take place, it was clear that models need also to forecast the effect of growth in demand on performance. This approach would ensure that a minimum lifetime is engineered into network design, and that the design does not result in the deterioration of the quality of water contained within. Also, the effects of localised disruptions for repair or rehabilitation work could be investigated and a plan devised for any operational changes to ensure minimum customer disturbance and cost.

1.8 Summary to Introduction

In summary, the combined model may be used to determine the effect of every operational change to the network including operating regimes in order that the effects of any work on customers is minimal and that best value for the investment is realised.

Subsequently the software was applied in a different manner to that traditionally employed. Instead of analysing a single leakage control zone in order to address a specific issue, for example pressure reduction within the zone, all the components of the model were applied in a complimentary manner to simulate the hydraulic and water quality issues within entire water supply system simultaneously.

Finally, the software was adapted to provide automatic reporting of the hydraulic and water quality characteristics of the distribution network, in near real time. The timely use of measured data and

subsequent analysis allows the step operational change from reactive to proactive network management. Proactive management has several advantages, for example, it allows more efficient leakage detection and location and the protection of customers from standards of service failures.

In order to achieve this goal it was required to instrument the system. As part of the work presented in the thesis, specifications for appropriate instrumentation were drawn up such that measurements of the hydraulic and water quality parameters could be made within the harsh environment and high-pressured distribution system. The specification also included appropriate instruments to measure surge and indicative of events such as bursts or discoloured water. The instruments were subsequently manufactured and successfully installed on the full-scale distribution network.

The data collected by the instruments was validated and prepared in a format appropriate for use by the modelling software. The necessary software developments undertaken are described.

This introduction has highlighted the need for a holistic hydraulic and water quality model for potable water distribution systems. This thesis details the development and application of one such model.

1.9 Aims and Objectives

The specific aims of the thesis were:

To enhance a hydraulic model to accurately simulate all the dynamic elements in a distribution network, including the effects of transients.

To develop a water quality model that can accurately determine the age of water throughout a distribution network and to simulate the movement and concentration of conservative and non-conservative substances within the network.

To describe the concepts, processes and modelling approaches that may be applied to biological activity and sediments in distribution.

To combine the enhanced hydraulic model and new water quality model with existing transient modelling functionality.

To calibrate and verify the combined model

To validate the modelling software tools on a “virtual” distribution network

To apply the combined hydraulic, transient and water quality models to a real distribution network in a holistic manner

To enhance the combined model to provide online functionality

To specify (for manufacture) appropriate instrumentation to gather the necessary network data

Subsequently, a further objective was to apply the model with a view to:

Optimising the hydraulic performance of a water supply network by eliminating low pressures and reducing unnecessary high pressures and leakage.

Better understanding the impact of transient pressure waves on water quality and mains bursts

Improving the knowledge of age of water throughout the network

Enhancing the understanding of non-conservative substance behaviour

Predicting the propagation patterns of conservative substances throughout the network

Illustrating the potential operational benefits of proactive distribution network management.

1.10 Thesis content

Following the introductory preamble in Chapter 1 that provides an understanding of the background to the work, Chapter 2 of the thesis presents a review of the literature pertaining to the understanding of the performance of distribution networks. The review is broken down into several components including legislation, hydraulic modelling, water quality, water quality modelling, and asset management issues.

Chapter 3 provides a physical description of the distribution network used in the study, why this network was chosen for the work and the geographical features. It provides details of the network assets and how they were operated before this work being undertaken. There is a summary of the performance of the network under this operational regime and a statement about how and why this could be improved.

One of the key areas of the work involved gathering appropriate data from the distribution network. Chapter 4 discusses the instrumentation that was specified, designed, built and used to obtain the necessary data. The logical subject area breakdown describing hydraulic, transient and

water quality instruments have been designed to operate in separate sections. Design, performance and installation characteristics for each type of instrument are discussed.

Building the necessary network model(s) is described in Chapter 5. This includes a comprehensive account of how the mathematical models that were utilised in this study were built. The Chapter presents a comparison of the initial network hydraulic performance (managed by traditional modelling methods) and after the implementation of the new, holistic, integrated modelling approach.

Transient work is presented in Chapter 6, where the transient effects of switching on and off a pump are assessed, and repeated mains failures are addressed.

Chapter 7 details developments made to improve the knowledge of water quality in a network by modelling the age of water and conservative / non-conservative substance propagation. Reducing the overall age of water and improving disinfection residuals are considered as a demonstration of water quality improvements. As well as determining substance concentrations throughout the network, the propagation utilities also facilitate contingency planning. This is clearly highlighted by showing how poor water quality incidents may be efficiently managed using the water quality model. The chapter is concluded by reference to the development of a biological model and a sediment transport model where concepts and ideas as to how such models might be used are presented.

Chapter 8 portrays the details of an online modelling approach with case studies concerned with leakage and incident management. A summary of the findings of the work is presented in Chapter 9 together with conclusions and recommendations for further work.

Chapter 2 - Literature Review

2.1 Legislation

The ageing of water supply infrastructures, and concerns over the safety of drinking water in general, has resulted in the introduction of comprehensive water industry legislation.

The United States Environmental Protection Agency, (USEPA), the European Union (EU), and the UK Department of The Environment Transport and Roads (DETR) / Drinking Water Inspectorate (DWI) have introduced drinking water specific guidelines and regulations. In 1994, the USEPA proposed the Disinfectants / Disinfection By-products amendments to the Safe Drinking Water Act of 1986 (<http://www.epa.gov/>) that governs water quality standards in the US. The standards laid out in this legislation were applicable not only after treatment but at the point of use also.

In 1998, the European Union adopted a new Drinking Water Directive (98/83/EC). This introduced values for water quality parameters that were generally more stringent than the existing ones (<http://www.europa.eu.int/comm/environment/water/>) although a small number of requirements were actually eased. As in the US, the EU legislation also included stipulations that water has to meet regulatory guidelines *throughout* the network and at *the point of use*, not only immediately after treatment as was previously the case.

In the European Union, the legislation is designed to ensure a continuous supply of drinking water to all and, through the introduction of stringent drinking water standards, to safeguard water quality. Further, through economic regulation, it is designed to protect consumers from unnecessarily high costs being passed on by water companies because of inefficient production and delivery operations. Leakage from water supply systems is also under the legislative umbrella. At the time of the first mandatory reporting of leakage by UK water companies values of 30% of product were common, (*OFWAT*), and therefore mandatory leakage targets were imposed on the industry.

Because of the industry regulation and to some degree competition, the distribution of drinking water has become a challenge to the water industry. Not only from a quantitative and qualitative

viewpoint, but also through the need to reduce leakage and operational costs and to improve and / or maintain standards of service.

With regard to quantity, for England and Wales, the demand of consumers has to be continually met without interruption (*Water Act 1989*), (*Water Industry Act 1991*). In respect of quality, there is a need to comply with EU and UK drinking water regulations for aesthetic, bacteriological and chemical quality that are becoming ever more stringent (*Water Supply [Water Quality] Regulations 1989*). Leakage targets are set annually on a company-by-company basis by the regulator. Also, through associated articles (*The Water Supply (Water Fittings) Regulations 1999* and *The Water Supply (Water Fittings) (Amendment) Regulations 1999*) that replace the water bylaws, requirements for avoiding contamination and waste on the customer's side, and for enforcement, which involves water undertakers, are laid down.

The strengthened drinking water legislation and guidelines require an uninterrupted supply of high quality water at the point of use and include standards for aesthetic, chemical and microbiological parameters. This has led to investment in more advanced water treatment processes that have raised the quality of drinking water leaving the treatment site to the required standards. However, during the distribution process as much as 30% of the supply can be unaccounted for, (*OFWAT*), and the water quality often deteriorates becoming unacceptable by the time it reaches the point of use. As the regulations become more severe, water utilities will have to find solutions to help them manage this problem and the use of modelling tools is one technological approach that can provide benefits.

2.2 Hydraulic Simulation Models

Drinking water distribution networks are comprised of a complex layout of pipes of different ages and material types. The asset ageing process, storage capacity, action of corrosion cells, surge events, and the chemical and biological characteristics of the water within the network affect the quality of the water that emerges at point of use.

To enable effective hydraulic operation, a distribution network has to be supported by dynamic elements such as storage reservoirs, pumps and valves. Flow and pressure throughout the network are determined by the way these dynamic elements are utilised. For example, the level of water in a storage reservoir will determine the pressure in certain parts of the network. The pressure can be

modified by the use of pumps and valves. However, the operation of pumps and valves may result in transient pressure effects that can damage network assets and reduce water quality.

Flow is governed by pipe diameter, the condition of the internal walls of the pipe and the head of water available from storage and pumps. It can also be affected significantly by demand patterns, especially by large industrial users. In order to manage the hydraulics of a distribution network effectively it is necessary to create a mathematical model that can simulate pressure and flow in every element of the system.

Since the early 1980's, there have been a large number of such models available to assist with distribution network planning and operation. Some of the early tools were relatively simple, used only to design small extensions to a network to supply, for example, a new housing development. Others were designed to enable pressure reduction schemes or design pump operation characteristics on more complex networks (*Ginas, EPAnet, Stoner, LICwater, Piccolo*) but none were capable of simulating large complex networks with numerous dynamic elements. This was partly due to the technology not being available but also because of computer processing constraints. In many cases, models were difficult to use and the results required expert interpretation.

As computer power has increased over the years, so has the quality and complexity of the modelling tools available. It is now possible to simulate the operational characteristics of networks containing large numbers of dynamic elements and many thousands of pipes. However, models from different parts of the world have evolved differently because of local requirements. For example, early versions of Stoner had an engine that calculated fire-fighting flows as part of the general simulation. This was because fire flows have to be available by law in the United States. In the United Kingdom however, no specific account was taken for fire flows, as their provision was not a statutory obligation.

Machell, 1991, undertook a survey of over thirty modelling packages available worldwide and their capabilities. The survey determined the state of the art at that time. It endeavoured to determine if any benefits could be obtained by using the tools for distribution network management and / or a collaborator willing to assist with the development of appropriate modelling tools. The survey revealed that there were only four "off the shelf" modelling tools that were suitable for this type application. Of these, only one company was willing to enter a collaborative agreement to modify the existing software and provide support over the research project timescales. This model therefore became the basis of the models developed for this thesis.

Advances in modelling software capabilities and associated technologies between 1983 and 1993 were reviewed by *Elton & Green, 1996*. They highlighted that the industry was still a long way from reaching state of the art when it came to effective and efficient distribution network modelling. It was notable however, that modelling of dynamic network elements had improved greatly over that period. Network models had moved on from “best fit”, containing <100 pipes and nodes and only critical operational elements, to models containing >1000 pipes and nodes and numerous dynamic elements, that were calibrated to ± 1 mwc and $\pm 5\%$ flow.

Advances were also being made to model build techniques over this time in that they were becoming more automated, and hence faster and cheaper, through links to Graphical Information Systems and databases containing operational information. *Casey & Schindler, 1996*, presented an historical perspective that they then brought up to date by discussing modelling/GIS application developments in a major water company. *Mellor, 1996*, described concurrent technological advances in client-server architecture and telemetry systems that would enable much of the automation of model building and data capture to take place.

Model / user interfaces were also developing throughout this time period and some models were being made easier to interpret by using graphical presentation facilitated through other software such as Computer Aided Design (CAD) packages. The models developed in this thesis remove the need for third party add-ins such as CAD packages by integrating the graphical user interface into the modelling software. The design of the interface allows easy access for entry of all model data, and a variety of output types for ease of interpretation of simulation output.

Despite the many advances in hydraulic modelling, most of the available models are deficient in some way. In order to make them acceptable, work rounds have been developed by the software vendors or water companies. For example, every valve in the Stoner software had to have a 1-mm pipe bypass added in order that the network was not seen as “disconnected” at each valve. Work rounds such as this however, did allow companies to use the models to good effect for many aspects of hydraulic analysis and design. This thesis describes improvements that were made to a hydraulic model in order that it could be used to accurately simulate the operation of a large complex network without the need for work-rounds.

2.3 Water Quality

Following a number of pollution incidents in the UK and the US, modelling was looked at as a tool that might provide water quality as well as hydraulic information. The first development was to produce a model that could predict the movement of polluted water through a distribution network. *Clarke et al., (1993)*, demonstrated the relationship between the distribution network itself and water quality at point of use and highlighted that the regulatory agencies were beginning to promote the use of water quality models to predict the movement of contaminants. The work in this thesis was promoted as a result of several tonnes of Aluminium Sulphate being introduced into a storage reservoir in Southern England, and hence into the distribution network. Hundreds of people received contaminated water and started legal proceedings against the Water Company responsible. Had appropriate modelling tools been available to the company and suitable instrumentation installed in the network, this unfortunate event would have been detected early, possibly even before any customers were affected.

There are many other reasons for wishing to model water quality. Frequently, distribution networks are supplied from a number of different treatment plants each with different source water(s) and treatment process train. Blending of these disparate sources can lead to unwanted reactions occurring. For example, the mixing of chlorinated and chloraminated water leading to formation of strong taste and odour. Similarly, because a distribution network is, in effect, a large storage vessel, it acts as a further treatment stage. It is like a bio-chemical reactor, subject to continuous variations in flow and pressure that influence the physical, chemical and biological processes occurring within. For example, the growth and decay of bio-film and its subsequent sloughing from the pipe walls and transport through the system or the formation of Tri-halo-methanes (THM) or the decay of disinfectant residual.

Modern water treatment produces high quality water that meets all relevant standards and criteria. However, the quality of the water leaving the plants can deteriorate rapidly, and sometimes dramatically, within the distribution network due to the many complex and interrelated processes. *Besner et al., (2001)* reported that the factors influencing these processes were difficult to correlate. However, some of them are self-evident and are undoubtedly a result of the hydraulic operation of the network. *Machell, (1996)*, determined the cause of 72 water quality complaints in two distribution networks. It was clearly shown that 68 of the complaints were generated as a direct result of network operations involving mains repair and the associated valve operations.

These complaints could all have been avoided if propagation models had been available to predict the movement of discoloured water following each network operation.

Age of water is an important factor to take into account when trying to understand water quality issues in drinking water distribution networks. As water ages within a network, it may undergo a number of physical, chemical, and / or biological changes. The changes may thus render it unsatisfactory to the user, the Regulators, and the Water Company that owns or is responsible for the management and performance of the assets that comprises the network.

Changes in water quality may be brought about by contact with the different materials that may be found in the network, including the pipe material, sediments or biological matter within the pipes or adhered to the pipe wall. *Clarke et al., (1993)*, demonstrated the relationships between the distribution network materials and water quality at point of use. *Van der Kooij* undertook extensive work that related distribution network materials to the promotion of bacteriological re-growth, release of organic and inorganic substrates, and permeation of contaminants through plastic pipes, all of which had a retrograde effect on water quality. *Hopman et al., (1992)* showed that Polyethylene pipe was more susceptible to permeation than PVC.

Hulsmann et al., (1986) developed a methodology for understanding the causes of water quality deterioration within a distribution network. Much of this work relied on manual sampling and analysis of both the water and the pipes themselves, but they also developed a continuous monitor for water quality parameters that included oxygen, temperature, turbidity, pH, redox, conductivity and pressure. The equipment was large and inefficient and required a water stream to be diverted from the mains being monitored. However, in conjunction with the manual samples/analysis the approach provided good data that they used to promote the setting up of a research group to carry the investigations further. It would be of great benefit to water utilities and researchers alike to be able to gather network data via small instruments installed into the network itself and via remote methods. Failed standards frequently relate to taste, odour, discoloration and unsatisfactory bacteriological quality. *Mallevalle, (1987)*; *Burlingame and Anselme, (1995)*; determined these problems could be caused by chlorination, microbial intrusion during low-pressure and surge events, microbiological growth, by pipe corrosion or long contact time with the materials that comprise the network assets.

Maier (1998), showed that the occurrence of Polycyclic Aromatic Hydrocarbons was linked to the presence of coal tar lined pipes and chlorine disinfectants. The longer the contact times with the materials of the network the greater the risk of deterioration of water quality. Because it is not

certain where such pipes have been placed in distribution networks they cannot simply be replaced or rehabilitated. The standards for PAH in drinking water are very low, so it is clear that minimising the time drinking water is in contact with coal tar lined mains is crucial to ensure low PAH levels therein.

It is apparent that long residence times within a distribution network can lead to:

Loss of disinfectant residual – this leads to diminished oxidation-reduction potential thereby lowering the bactericidal properties of the water. *Olivieri, (1986)*, studied the stability and effectiveness of chlorine based disinfectants in water distribution networks and showed that combined chlorine, in the form of Chloramines, was the most persistent. Reduced disinfectant residual in turn increases the risk of bacteriological survival and re-growth. An increase in biological activity may generate tastes, odours, and promote corrosion, resulting in complaints and unsatisfactory regulatory samples. The ability to be able to model chlorine residuals would obviously be of great benefit for the management of some of the biological aspects of distribution networks.

Much work has been done to understand bacteriological survival and re-growth in distribution networks. *Prevost et al., (1992)* identified that the main substrate responsible for microbiological re-growth in distribution networks was the biodegradable fraction of the available carbon in the system. This finding was supported by *Piriou et al.,(1998)* who also noted that this factor was more important than the Heterotrophic Plate Counts at the inlet to the system. In 1992 *Lloyd, (DWI)*, presented evidence to an International group gathered in the UK that as many as 40% of all bacteriological samples that failed the standards were associated with bacteriological re-growth and biofilms. Other factors identified by contributors at the conference were high water temperatures, lowland supplies, treatment failure, and contamination of sample taps, inappropriate sample tap location, and service reservoir contamination. These factors can be controlled to some extent and *Le Chevallier* suggested that engineers should always use smooth surfaces, maintain circulation in the distribution networks and use materials that have no disinfectant demand and that are nutrient free. *All these factors are affected by the age of the water in the network.*

Characklis, (1980), identified that fluid velocity (as shear stress) strongly influenced biofilm formation in that the water velocity influences the mass transfer rates from bulk water to bio film and the detachment rate of material from the bio film. He also showed that the available organic carbon governs the extent of bio film growth.

Donlan, (1990), showed a negative relationship between flow velocity and heterotrophic plate counts, and a strong positive relationship between temperature and plate count supporting these findings. *Models can be used to calculate shear stress at the pipe wall* and determine which pipes provide optimal shear stress conditions for growth of bio film. Where practicable, the velocities in these pipes can then be altered by changing the flow regime of the network.

To minimise microbial contamination it is common practice to disinfect drinking water by adding chlorine or chloramines before supplying the distribution network. Doses are sufficient to meet the chlorine demand of the supply and to maintain a certain residual of chlorine throughout the distribution network. All forms of chlorine however, are strong oxidising agents and they react with organic material to create disinfection by-products, some of which are potential carcinogens (*Chang et al., 2001*). They also react with iron to form corrosion by-products (*Frateur et al., 1999*) that may result in discolouration of the supply.

If chloramination is used, and the process is not carefully controlled, this method of disinfection can lead to excessive growth of nitrifying bacteria (*Skadsen, 1993*) (*Holt et al., 1996*). Nitrification decreases the level of disinfectant residual, oxygen, alkalinity and pH, and increases nitrite, nitrate and heterotrophic bacteria numbers (*Wolfe et al., 1988; Cunliffe, 1991; Le Chevallier et al., 1991; Odell et al., 1996*).

The chemical and biological characteristics of the bulk water volume entering the network also contribute to the quality changes that occur because of residence time within the network. *Le Chevalier 1992*, identified three of the main causes of microbiological water quality problems as being breakthrough from the treatment process, cell growth within the network using available substrates, and disinfection for the control of biofilm.

Micro-organisms such as anaerobic bacteria, protozoa, copepods and nematodes can be commonly found in bio-film, (*Geldreich, 1996*). The specific characteristics of the water distribution system, such as surface pipe-roughness, pipe-material and the hydraulic flow regime, can also have a significant influence on water quality particularly in respect of microbial bio-film growth.

Uneven pipe surfaces support higher bio-film densities than smooth walled pipes by providing protection from detachment due to the effects of shear stress (*Chang and Rittman, 1988*)

Pipe materials can be a problem in their own right. Corrosion of iron pipes generates products that react and destroy disinfectant. The process create tubercles that increase pipe roughness, become points of precipitation of organic compounds and provide cracks for bacteria shelter and even

growth (*LeChevallier et al., 1987; Prevost et al., 1998*). Old corroded pipes provide excellent hospitable environments for micro-organisms. In fact, pipe corrosion processes release some constituents into the bulk water that actively promote microbial re-growth. Simultaneously, this re-growth enhances corrosion rates (*Emde et al., 1992; Korshin et al., 1996; LeChevallier et al., 1993; Pisigan and Singley, 1987*).

The literature on microbes in drinking water networks is very extensive. Much work is dedicated to predicting the numbers of micro-organisms present in a network. *Machell, 1994*, applied a different approach. He developed a model that took into account a number of operational factors including flow, changes in flow direction, chlorine residual, turbidity, age of water, and effects of pressure transients amongst others on each pipe within the network. He attempted to correlate hydraulic operation and its effects on water quality by determining which pipes in a network provided better conditions for bacteriological survival / proliferation and where the organisms would travel if they came into the planktonic phase. This simplified approach negates the need for in depth understanding of biological dynamics and, as most of the model input is produced automatically from the hydraulic model, simple chemical tests and the user, it is relatively straightforward to apply.

Formation of disinfection by-products - as well as being potential carcinogens (*Utsumi, 1992, Harren-Freund & Pereira*), disinfection by-products such as Trihalomethanes (THMs) can provide food for micro-organisms (*Block, 19*), thereby promoting bacteriological re-growth in water distribution networks. The drinking water quality standards for Trihalomethane(s) are low. Table 1 shows individual THMs and their permitted values.

Benzo 3,4 pyrene	ng/l	10
Tetrachloromethane	µg /l	3
Trichloroethene	µg /l	30
Tetrachloroethene	µg /l	10

Table 2.1 - Maximum concentrations for individual THM's (See Table 1)

It is therefore essential to minimise their production / development to avoid regulatory failures. Disinfection by-product formation of compounds such as Trihalomethane, (THM), is a relatively slow process. Because the formation process is slow, residence time (related to the age of water), and temperature, affects the formation of such species. In the presence of precursors, minimising the age of water therefore has a direct effect on reducing the level / risk of THM formation.

For bacteriological indicator organisms such as Coliforms and E-coli the quality standards are 0 per 100 ml.

High contact times with pipe materials and sediments - imparts taste and odours to the water. Anyone who drinks water that has stood for some time in a polythene cup will notice the difference in taste when compared to that from a China cup under the same conditions for example.

Reduced oxygen content - provides anaerobic conditions, tastes, and odours.

An increase in colour and / or turbidity may be brought about by the dissolution of metals such as Iron that can also impart taste and odour to the water. *Hussman et al., (1986)*, showed a negative linear relationship between oxygen content and turbidity and a significant linear relationship between iron and turbidity.

Precipitation reactions - for example, the oxidation of manganese, causing it to be precipitated as black manganese dioxide. This phenomenon gives rises to turbid or discoloured water, and damage to articles of clothing caused by washing machines using the water. The vigorous wash cycle of modern washing machines is easily capable of oxidising manganese (*Machell, 1989*). It is thought that turbulent flow conditions within a network will also promote oxidation of certain iron and manganese species.

Sedimentation - Areas of a distribution network where the age of water is high will by definition, be areas of low velocity. Consequently, particles entrained in the flow may be deposited and collect on the surface of the pipe. *Gauthier et al., (1996)* characterised the materials deposited in drinking water distribution networks. The work suggested that there were several different physical, chemical and biological mechanisms involved in the formation of the deposits, and that the relationship between particulate and dissolved phases was very dynamic and complex. Further, all the deposits studied were colonised with micro-organisms that were being sheltered and nourished by the deposits and were taking part in the particulate / dissolved state interchange mechanisms. *Rooke, (1996)* explained how the microbial fauna in a distribution network influences corrosion mechanisms and highlighted the most favourable conditions under which microbial corrosion would be most prolific.

Following a prolonged period of undisturbed operation a distribution network reaches a state of hydraulic equilibrium. If the network configuration is altered, deposited materials may be re-suspended leading to discoloured and turbid water that will, in turn, give rise to consumer complaints and unsatisfactory samples. It is required therefore to be able to establish where in the network re-suspended materials will travel. A conservative propagation model can be used for this

purpose. Moreover, the model could also be used to determine travel times for pollutants from any point of ingress into the network.

Some materials such as oxides of Iron and Manganese have a propensity to accumulate significant quantities of metal species when left undisturbed. *Machell, (1988)*, showed that manganese oxides sorb, and can accumulate, large amounts of different metals such as lead and cadmium. This process leads to a build up of toxic metals that could be re-mobilised by physical or chemical action thereby reaching the point of use and / or causing regulatory failures. *Hintelmann, (1993)*, found that biofilms also absorb toxic metals through the study of the accumulation of mercury compounds via a passive absorption mechanism.

The Hydraulic flow regime - Within the network may have an influence on water quality due to re-suspension of deposited solids, bio-film detachment, and the erosion of corrosion by products and sediments due to shear stress changes, particularly those induced by surge, (*Stewart, 1993; Skipworth et al, 2000*), and conversely sedimentation (which favours bacterial and metal accumulation) and build up of corrosion by products in pipes in which the flow is low, particularly dead ends.

Environmental factors - Such as temperature, and chemical factors such as pH and dissolved oxygen influence corrosion mechanisms, taste and odour, microbial growth and hence significant changes in water quality within the system, (*LeChevallier and Shaw, 1996*). *Machell & Banks, (1997)*, studied the temperature at fifty sites in a drinking water distribution network. It was shown that, in this particular network, the temperature was always higher within the network than the temperature of the incoming water thus providing enhanced conditions for microbial survival or growth. The ability to be able to monitor temperature at the inlet to and within distribution networks would provide a good indication of when this trigger is active.

The problem is further compounded in complex networks where water from different parts of the network (and often from different water sources / treatment works) mixes, resulting in the occurrence of chemical reactions.

It is clear from the above that a water distribution network can be considered to act as a further stage of water treatment. Many physical, chemical and biological processes influence changes in water quality during distribution and the distribution network is a complex ecosystem in its own right. Currently these processes are not fully understood and, to meet ever more stringent water

quality standards, the water industry needs tools and techniques to predict the quality of water as delivered to customer's taps.

This project aims to address some of these needs through the development and application of integrated mathematical modelling tools designed to provide a better understanding, and hence facilitate some control, of these processes.

2.4 Water Quality Simulation Models

A number of water quality models have already been developed. These can be categorised in three groups:

“*First-order*” models that describe water quality using simple first-order kinetic mass-balance equations, “*fundamental-process*” models that describe reactions using sets of inter-dependent mass-balances and “*operational management process models*” designed to assist water utilities in trying to, for example, reduce the number and consequences of discoloration events.

One of the most commonly used “*first-order*” models for water quality is EPANET (Rossman *et al.*, 1994). This is a hydraulic model with first-order kinetics incorporated to predict the level of chlorine within the network. It is based on the extended period simulation approach. This and other commercially available “*first-order*” models such as Water-CAD, H₂Onet, Stoner and Piccolo reasonably estimate disinfectant decay due to some biological and chemical reactions (Le Chevallier *et al.*, 1990, Powell *et al.*, 2000, Rossman *et al.*, 1994). However, they do not take into account some important variables, which may explain why models of this type cannot be calibrated unless consistently high chlorine residual is maintained throughout the network being modeled. The models are not flexible in that most of the input is hard coded and so applied similarly to every situation. It is important to be able to change all the variables in a model, as they will be specific to individual networks. The model developed for this thesis allows all the variables to be user defined or maximum flexibility.

Biswas, (1993), proposed a first order decay model that included axial and radial transport components to determine chlorine concentration in pipes. His model has been included into the model developed in this thesis and then improved by taking into account additional parameters including differential pressure and temperature changes. All the parameters in the new model

have user definable values to make the model more flexible than its predecessors and thus better calibration is achievable.

“*Fundamental-process*” models describe, for example, bacterial metabolism (bio-film processes) and disinfectant decay by using sets of interdependent, multi-species, mass-balance equations based on the fundamental reactions and their interaction with each other. Several studies have begun to reveal the complex structure of bio-films attached pipes surfaces. *Keevil & Walker, (1992)*, used Differential Interference Contrast Microscopy to show the true characteristics of biofilms. The results showed that biofilms were not a homogeneous medium and were, in fact, a mosaic of microenvironments that provided havens for microorganisms against biocide, extremes of oxygen content, eukaryotic grazing and other growth factors. Further work by this group demonstrated that biofilms grew “fronds” that reached out into the flow and were very resistant to shear pressure but susceptible to shock of the type generated by pressure transients.

Attempts to model the bacterial growth in water distribution networks have been made. A typical example is Piccobio, a deterministic model, developed not only to predict bacterial growth but also to locate the zones of high risk of biological proliferation (*Piriou et al., 1998*). Piccobio appears to be the basis for a useful model for biological water quality modeling even though it does not incorporate the complex dynamic processes of the bio-film development (*Loosdrecht et al., 1996; Picioreanu et al., 2000*) or detachment from pipe-walls. One difficulty of application is that it requires a knowledge of existing biofilm characteristics that is very difficult to obtain. However, *Okkerse et al., 2000*, showed a new method for quantification of bio-film thickness and variability, the Laser Triangulation Sensor that will assist in the Piccobio approach. The theory of bio-film kinetics requires a gradient in concentration for a substance to be transported in and out of the bio-film (*Arshad et al., 1998*). It is apparent that modeling biological systems is very difficult and complex and work will continue for many years. The water industry however, needs tools to help them achieve regulatory standards of service now.

Machell, (1994), applied a different approach and developed a model based on distribution network *operational* factors, and variables that supported or did not support survival of microorganisms. The model assumes that organisms are present in the network at locations best suited to their survival and are mobilised by hydraulic phenomenon. The factors considered included flow, changes in flow direction, chlorine residual, turbidity, age of water, BDOC levels, and effects of pressure transients amongst others. Rather than attempt to model complex biological systems, he has attempted to correlate hydraulic operation and its effects on microbiological water

quality. By determining which pipes in a network provide better physical and chemical conditions for bacteriological survival / proliferation and where the organisms would travel if they became entrained in the planktonic phase. This simplified approach negates the need for in depth understanding of biological dynamics and, as most of the model input is produced automatically from the hydraulic model, simple chemical tests, and the user, it is relatively straightforward to apply.

Drinking water distribution networks have been shown to be ecosystems in their own right. They are complex and poorly understood due to difficulty of access and measurement of appropriate performance parameters, system-to-system variation, and a high sensitivity to physio-chemical changes within the network. Each water distribution network has its own flow, pressure, leakage and bio-chemical features and develops unique chemical and microbiological characteristics.

Given onerous water quality legislation, the inevitable short-term water quality deterioration due to bursts and network operations, the continuous ageing of network assets, and the failure of current water quality models to deal with all the interactions between physical, chemical and biological processes, *a new generation of integrated water quality models is required*. The way in which modelling is used within water companies also needs re-evaluating and moving from a reactive analysis to a proactive operational management tool through the timely acquisition and use of measured network data.

Lu et al., (1995) developed a simple integrated model accounting for simultaneous transport of substrates, disinfectants and micro-organisms as well as for the major biological processes. This model applies to steady state conditions only. Modification of this model to an unsteady (dynamic) state, so that the accumulation of fixed bacteria vs. time, at a specified cross-section (i.e., at a specified distance from the inlet), can be monitored will be very important, or it will not be applicable to real distribution networks.

2.5 Asset Management

Asset management has now become the keyword within the water industry. As water utilities adopt operational methodologies based more and more on these techniques, and as competition within the industry drives companies to reduce their operating costs it is imperative that integrated modelling tools are developed that allow a more proactive measurement and management of flow,

pressure and water quality simultaneously. The level of “pro-activeness” will be determined by the timing of the availability of measured data from the network.

One of the major failings of current modelling techniques is that simulations rely on repeating a fixed set of hydraulic conditions over a given time period, the norm being a 24 hour cycle. The hydraulic data used to produce this repeating pattern of hydraulic conditions is “normalized” by the way it is collected and cleaned to remove pressure spikes and abnormal events. The resultant hydraulic basis is a continually repeating diurnal pattern of flow and pressure representing average performance characteristics over a single day. This approach is fine for design of network alterations such as pressure reduction schemes as, in general, the “average day” conditions reflected in the hydraulic basis have been proven accurate enough for this type of work.

This approach is not valid however for proactive use because of, for example, changes in service reservoir levels or unusual demands on the network, the hydraulic characteristics will not be exactly repeated over any time period particularly with regard to flow. It is required therefore to have a model that can have real time data imposed as boundary conditions in order to produce accurate hydraulic simulation results over long periods. This is especially important for age of water and propagation calculations that rely for their accuracy on the flow information supplied by the hydraulic engine.

Much of the work cited in this literature review has demonstrated or commented on the difficulties of maintaining chlorine residuals throughout a distribution network. Work by *Carlson, (1991)* suggests that measurement of the oxidation-reduction potential may be used as an indicator of chlorine disinfection efficiency. *Hussman et al., (1986)* demonstrated the usefulness of collecting real time water quality data to better understand the hydraulic / water quality dynamics with a network. It follows then, that if suitable instrumentation were installed at key locations within a network, it would be possible to monitor the effectiveness of the chlorination throughout the network and collect water quality data for modelling and proactive management.

2.6 Basis of Thesis

The literature review has highlighted that the understanding of water quality changes in distribution networks is not complete.

This thesis describes the development of this new generation of integrated water quality models simultaneously accounts for all the processes related to water quality deterioration in a single model. These processes include age of water, biological potential, conservative and non-conservative propagation, disinfection by-product formation, and sedimentation characteristics as well as hydraulic parameters flow, leakage, pressure and transient pressure.

It outlines the specification of the instrumentation used to gather the network data required for understanding network behaviour and calibrating the models, and describes how the instruments were installed and the data collected, analysed, and used in the model.

The thesis then explains how the model was applied to a real drinking water distribution network to demonstrate the methodology and the benefits gained by its application. The benefits to the water industry through real time acquisition and analysis of network data to

Finally, the application of the model in real time has been shown to provide significant benefits with regard to leakage monitoring and detection. It has also been used to detect and determine the travel path of discoloured water.

Chapter 3 - The Study Distribution Network

3.1 Distribution Network Zone Hierarchy

A large water company may have thousands of kilometres of water supply mains. In order to aid regulatory performance reporting these mains are divided into a series of “zones”. The largest of these zones is the Asset Management Planning Zone (AMP Zone) that contains supply, treatment and distribution assets for a single water supply where possible. The Director General, Ofwat, is informed of all performance-related issues associated with the assets within each AMP Zone.

An AMP zone may comprise of one or more Water Supply Zones where a Water Supply Zone serves a population of 50,000 or less. Each Water Supply Zone is further split into what are termed 8B Zones. An 8B Zone serves a population of 5000 consumers or less. Her Majesty’s Drinking Water Inspectorate monitors the quality of the water supplied into the 8B zones and has powers to prosecute water companies who fail to supply water to the prescribed quality.

AMP zones are further sub-divided into Leakage Control Zones. This type of zone is used to determine the leakage within that part of the system. The total leakage for the company is calculated from the accumulated figures from the individual leakage control zones. Where possible, the boundaries of 8B and Leakage Control Zones are coincident.

Figure 3.1 shows the inter-zone relationships within the study area.

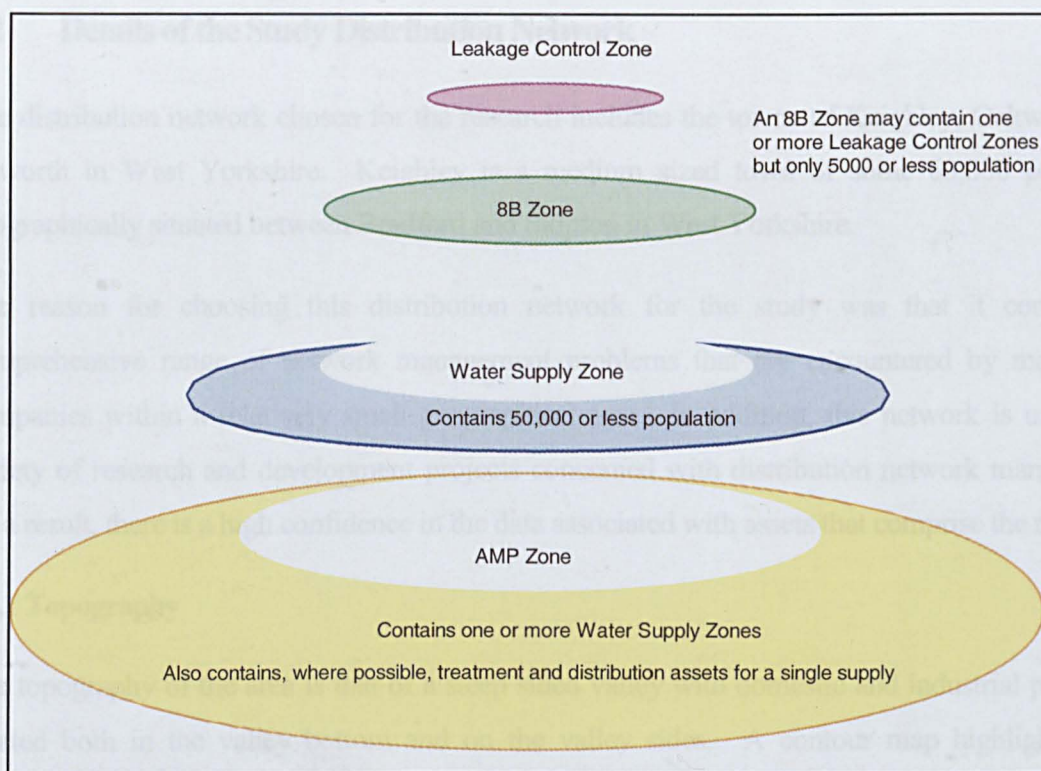


Figure 3.1 Inter zone relationships

The flow from each water treatment plant and service reservoir in any water supply system is measured and recorded by water meters. The flow into and out of each zone down to Leakage Control Zone level is also monitored and recorded to facilitate a water balance and hence calculate losses through leakage and unaccounted for water use.

Each Leakage Control Zone also has a pressure logger that is generally located at a point indicative of the lowest pressure in that part of the network. The network chosen for this study is described in detail in the following sections.

3.2 Details of the Study Distribution Network

The distribution network chosen for the research includes the towns of Keighley, Oakworth, and Haworth in West Yorkshire. Keighley is a medium sized town of some 60,000 population geographically situated between Bradford and Skipton in West Yorkshire.

The reason for choosing this distribution network for the study was that it contained a comprehensive range of network management problems that are encountered by most water companies within a relatively small geographical area. In addition, this network is used for a variety of research and development projects concerned with distribution network management. As a result, there is a high confidence in the data associated with assets that comprise the network.

3.2.1 Topography

The topography of the area is that of a steep sided valley with domestic and industrial properties located both in the valley bottom and on the valley sides. A contour map highlighting the geography of the area is shown in Figure 3.2.



Figure 3.2 Contour map of the study network area

There are significant variations in the ground level over the area of the study network. Elevation changes as much as 200 m in distances of 1 km. As a result, a number of pressure reduction schemes have been implemented to limit the occurrence of high mains pressures.

3.2.2 Zone Layout and Interconnectivity

As described previously in section 3.1, the network has been divided into a number of discrete Leakage Control Zones (LCZ). Figure 3.3 shows a diagrammatic layout of the 11 leakage control zones that comprise the study network that are listed in Table 3.1.

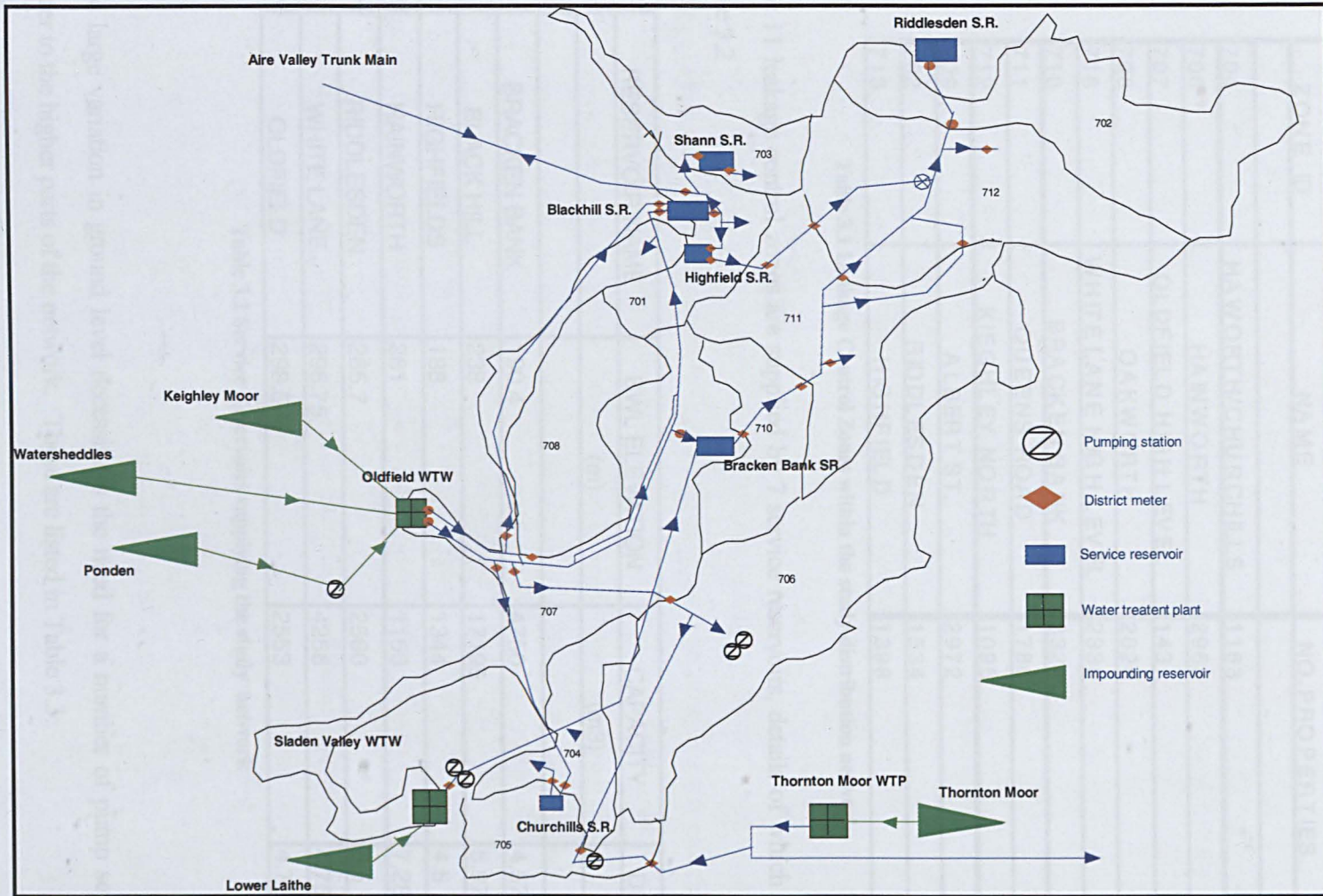


Figure 3.3 Layout and connectivity of the Leakage control zone

ZONE ID	NAME	NO. PROPERTIES
704	HAWORTH/CHURCHILLS	1163
706	HAINWORTH	2968
707	OLDFIELD HIGH LEVEL	143
708	OAKWORTH	2026
716	WHITE LANE HIGH LEVEL	2338
710	BRACKEN BANK	3324
711	QUEENS ROAD	1786
712	KIEGHLEY NORTH	1095
709	ALBERT ST.	2972
702	RIDDLES DEN	1534
713	HIGHFIELD	1398

Table 3.1 Leakage Control Zones within the study distribution network.

The 11 leakage control zones are supplied by 7 service reservoirs, details of which are listed in table 3.2

RESERVOIR NAME	BWL ELEVATION	CAPACITY	DEPTH
	(m)	(m3)	(m)
BRACKEN BANK	190.4	4750	4.57
BLACK HILL	238	17195	5.97
HIGHFIELDS	188	1344	4.5
HAINWORTH	281	1150	7.28
RIDDLES DEN	265.7	2560	3.5
WHITE LANE	296.75	4256	4.75
OLDFIELD	298.5	2553	4.78

Table 3.2 Service Reservoirs supplying the study network

The large variation in ground level necessitates the need for a number of pump sets to deliver water to the higher parts of the network. These are listed in Table 3.3.

PUMP LOCATION	METHOD OF CONTROL	AVERAGE PUMP FLOW	COMMENTS
Sladen Valley WTW	Level Control	60 l/sec	Pump to White Lane Reservoir
Sladen Valley WTW	Not In Use	N/A	Pump to Bracken Bank Reservoir NOTE: Not operated at present
Hill Top Booster	Level Control	12 l/sec	Controlled by level from Hainworth Tanks
Hainworth Booster	Pressure Control	0.1 l/sec	Very small booster

Table 3.3 Pump location and flow details

The service reservoirs supply water to the customers through 150 km of pipe work. The water mains vary in size from 50 mm to 400 mm and span a range of ages from circa 1900 to new mains. The majority of the mains infrastructure is comprised of Iron pipes (circa 70%). The rest is a mixture of plastic including PVC and MDPE and a small number of steel pipes. Service pipes include copper, galvanised, and plastic. The internal condition of a pipe or its roughness is described by a “C” value. The higher the C value the smoother the internal surface of the pipe. C values within the iron pipes in the network range from 30 to 120 depending on age and location. The plastic materials are relatively new and therefore have high C values.

In order to visualise the connectivity of the various operational elements within the study distribution network, Figure 3.4 illustrates the layout of the water mains and the position of the key assets.

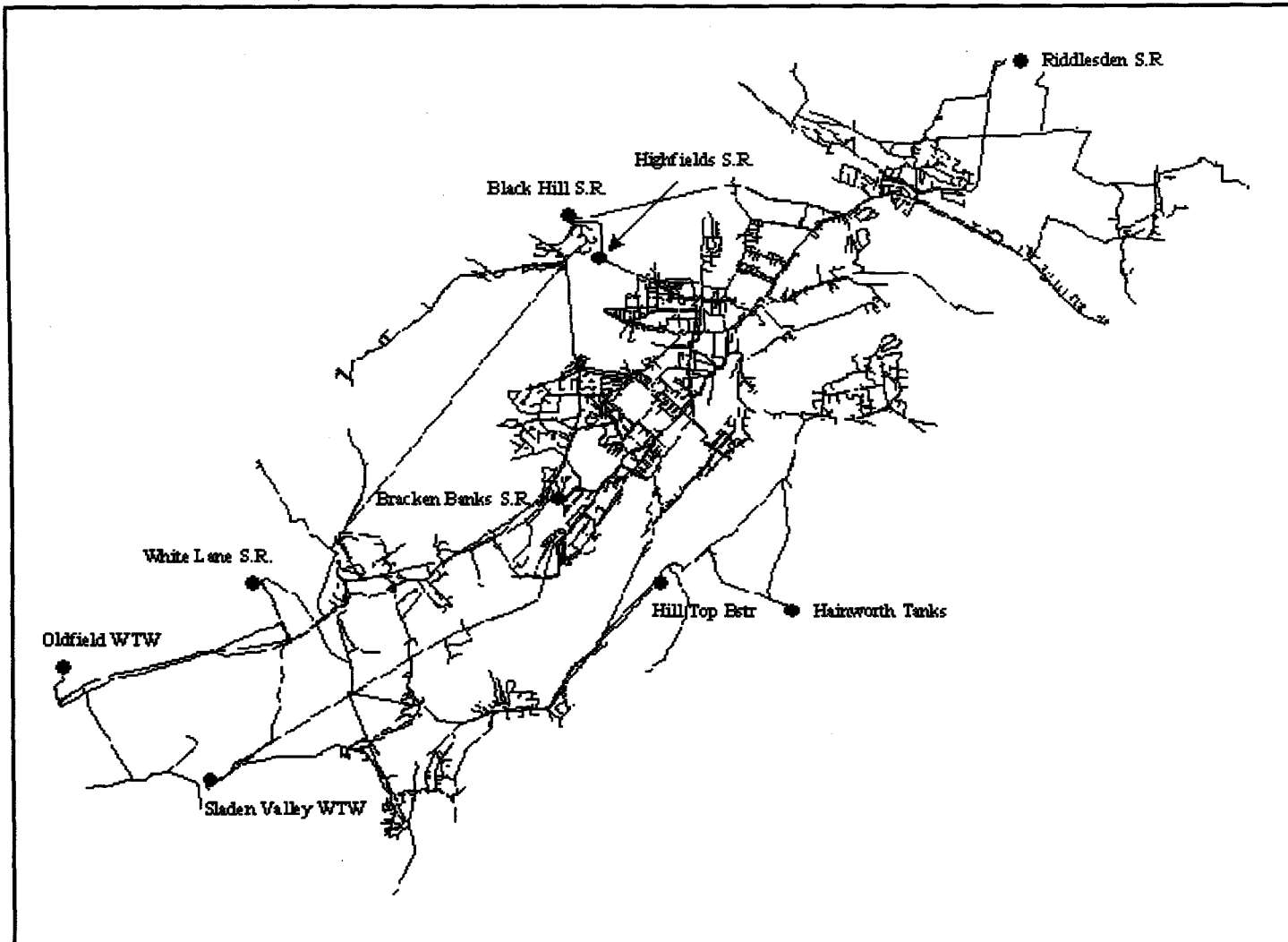


Figure 3.4 Layout of the water mains and position of key assets in the study network

3.3 Operational Features of the Study Network

The majority of the network is supplied by two water treatment plants, Oldfield and Lower Laithe (WTW), which are located in the South Western corner of the geographic area.

Water from Oldfield is gravity fed eastwards to White Lane service reservoir supplying a small number of properties in zone 707 on the way. From White Lane service reservoir the supply continues feeding zones 716 and 708 finally terminating as one of the inlets to Black Hill service reservoir, as shown in Figure 3.3.

Zone 704 and Zone 706 are also fed from White Lane service reservoir. Most of Zone 704 is pressure reduced while the majority of Zone 706 is fed via a small booster pump, Hill Top Booster, which is operated by level control from the relatively small Hainworth service reservoir. Hainworth service reservoir actually comprises two circular above ground tanks.

The water from Sladen Valley WTW can be pumped via two pump sets. One pump set pumps via a direct main through a metered inlet to White Lane Service reservoir. The other pump set is currently not in regular use, but can supply water to Bracken Bank Service reservoir. This main between Sladen Valley and Bracken Bank is currently operated under gravity feed, the pumps only being used when necessary.

From Black Hill Service reservoir there are three supply outlets, one into Highfield Service reservoir, one into Shann Service reservoir and a bi-directional connection into the Aire Valley Trunk Main, all of which are individually metered.

Highfield Service reservoir and Bracken Bank Service reservoir are the main supply reservoirs for the town of Keighley itself. Highfield supplies Zone 713 and part of 709, and Bracken Bank supplies Zones 710, 711, 712 and the remainder of Zone 709. These latter four zones are arranged in a cascading manner with one Zone feeding the next, the order of the cascading system being 710, 711, 709 and 712.

Zone 702, situated at the eastern edge of the network, is fed by Riddlesden Service reservoir that takes its supply from Graincliff water treatment plant. Figure 3.5 shows the extent of supply of individual service reservoirs at the beginning of the study.

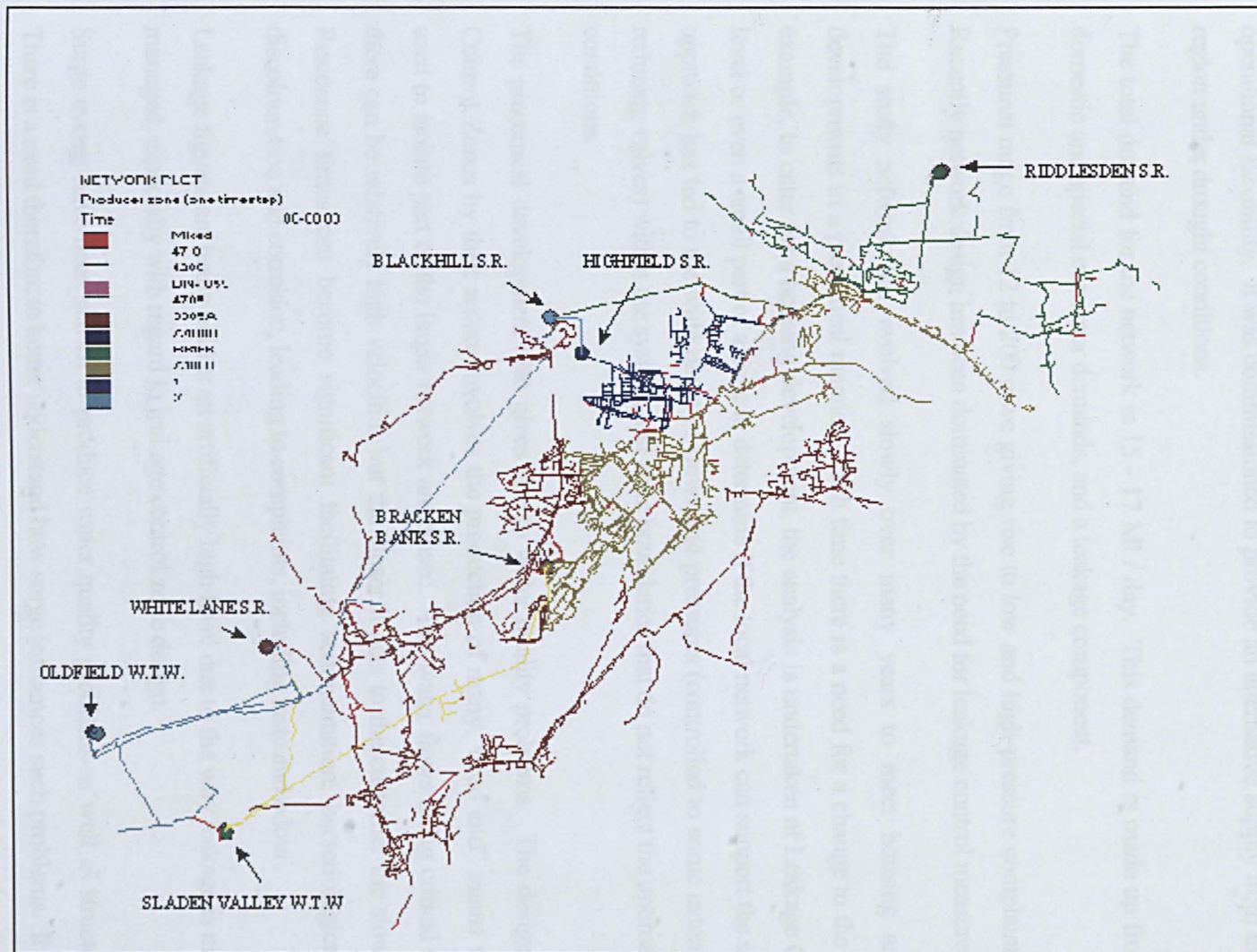


Figure 3.5 The extent of supply of individual service reservoirs

A recently installed 400-mm main connects Riddlesden Service Reservoir with Black Hill Service Reservoir. This main, at present, is only used to supplement the level in Black Hill Service Reservoir as necessary to meet variations in the daily demands. Its primary purpose is one of operational flexibility. It was commissioned to provide an alternative supply from the East of the region under drought conditions.

The total demand for the network is 15 - 17 Ml / day. This demand is made up from industrial, domestic and special customer demands, and a leakage component.

Pressures range from 12 to 200 mwc giving rise to low and high-pressure complaints and bursts. Recently network design has been dominated by the need for leakage control measures.

The study network has evolved slowly over many years to meet housing and industrial developments in a piecemeal manner. Each time there is a need for a change to the network, for example, to cater for a housing development, the analysis is undertaken at Leakage Control Zone level or even a small part of a zone to determine if the local network can support the scheme. This approach has led to the wide range of flows and pressures (controlled to some extent by pressure reducing valves) within the system, and zone boundaries that do not reflect the optimal operational conditions.

The piecemeal development also gives rise to water quality problems. The design of Leakage Control Zones by their nature involves the production of many "dead end" mains where valves used to isolate part of the larger network are closed. The water flows along critical routes where there can be relatively high velocities but the closer it gets to the customer the slower it travels. Residence times can become significant facilitating sedimentation, bacteriological re-growth, discolouration and corrosion, leading to complaints, including taste and odour.

Leakage figures are being held at an artificially high level due to the way leakage is monitored and managed, especially with regard to leakage control zone design.

Surge events have been shown to produce water quality problems as well as structural damage. There is a need therefore to better understand how surge influences such problems. It is clear from the above operational summary that the study network provides the ideal location for research to further explore an integrated modelling approach that includes transient analysis, improved understanding of water quality and improved operational practice.

The way this has been attempted is described in Chapters five through seven whilst Chapter four details the necessary instrumentation that was required to accompany the model development.

Chapter 4 - Instrumentation

4.1 Background

A model relies on good quality data for the purpose of model calibration and verification, so the instrumentation used to collect such data is one of the vital components of model development. Instruments provide raw data from which to derive information about the performance of a number of parameters of the system being measured. Analysis of the information allows informed decision-making resulting in more effective operation and control.

For hydraulic design and monitoring purposes, and more recently the need to accurately determine leakage levels in distribution networks, manufacturers have developed flow and pressure instruments to a very high level of sophistication. Off the shelf instruments are capable of measuring flow and pressure to an accuracy of 0.1% and even 0.01% if the instruments are rated correctly for purpose and manufacturers installation procedures are strictly followed.

Over the last 10 years, the water companies have invested heavily in improvements to water treatment works as a result of legislative pressures and the needs to meet water quality standards of service and efficiency targets set by OFWAT. Water treatment processes rely heavily on instrumentation technology and continuous development of monitoring and control instrumentation has taken place. Allied to programmable logic, closed loop control technologies and SCADA systems much of the water treatment process is now fully automated and water leaving the treatment plants is generally of a very high quality.

However, the good quality water leaving water treatment plants was found to be deteriorating as it was transported through the pipe networks to the customers. Therefore recent focus for improving water quality standards of service has shifted to the distribution network. In the past there has been little research and development for this instrument application compared to that for other water industry assets. The reason for this has been that the focus of water supply effort since the start of the century has been on providing quantity of water rather than quality. Also, distribution assets are mostly underground and access is difficult and expensive. There has been an "if it is running satisfactorily, leave it alone" philosophy and work is generally only undertaken as a reaction to some specific network event such as a burst, a customer complaint, or planned rehabilitation.

Although there are many water quality sensors available for laboratory and open channel applications, none could be installed directly into a water main. Collection of time series water quality data from a pressurised environment was never considered as an industry need. This attitude, the perceived development costs for instruments for such a hostile environment, and the lack of demand from the industry resulted in almost no development taking place for this application.

For the purpose of this study and to understand the interactions between network hydraulic and water quality performance it was required to take frequent measurements of both hydraulic and water quality parameters. The data was used for validation and calibration of the models. Traditional manual sampling techniques were considered impractical due to the high cost and logistics of physically taking the number of samples required, and of the transport to the laboratory and cost of analysis. In order to meet the data requirement therefore, commercially available hydraulic instrumentation was utilised but it was necessary, for the reasons outlined above, to develop suitable water quality instrumentation specifically for purpose.

Pressure transients may affect an entire distribution network but usually large main laying or pumping schemes where surge may be a potential problem are not the subject of detailed surge analysis. The analysis completed often consists of first principal theory based hand calculations that, by their nature, have to be limited to the small number of pipes that are directly linked to the scheme. The calculations do not extend to the distribution mains either side of the scheme mainly due to time constraints in the completion of this type of calculation. Because of this approach and the general assumption that surge is not a global network issue, instrumentation is not well developed for this application. With the advent of computer based transient pressure mathematical modelling packages however, it is now possible to consider entire networks in a single calculation.

Instrumentation to provide the necessary data are available but transient pressure loggers are expensive and their capability for extended time data capture is limited due to the sampling frequencies that are required to observe the shape and amplitude of transient events. Because of this, only 2 were deployed for this study and only a limited section of the study network was monitored and modelled.

4.2 Hydraulic Parameters

“Spectralog” instruments were used to record flow and pressure at a number of key network locations, such as inlets to leakage control zones, zone interconnections or exports, as shown in Figure 4.1. All these points had flow meters installed as part of a leakage management project and data was recorded at 15-minute time intervals.

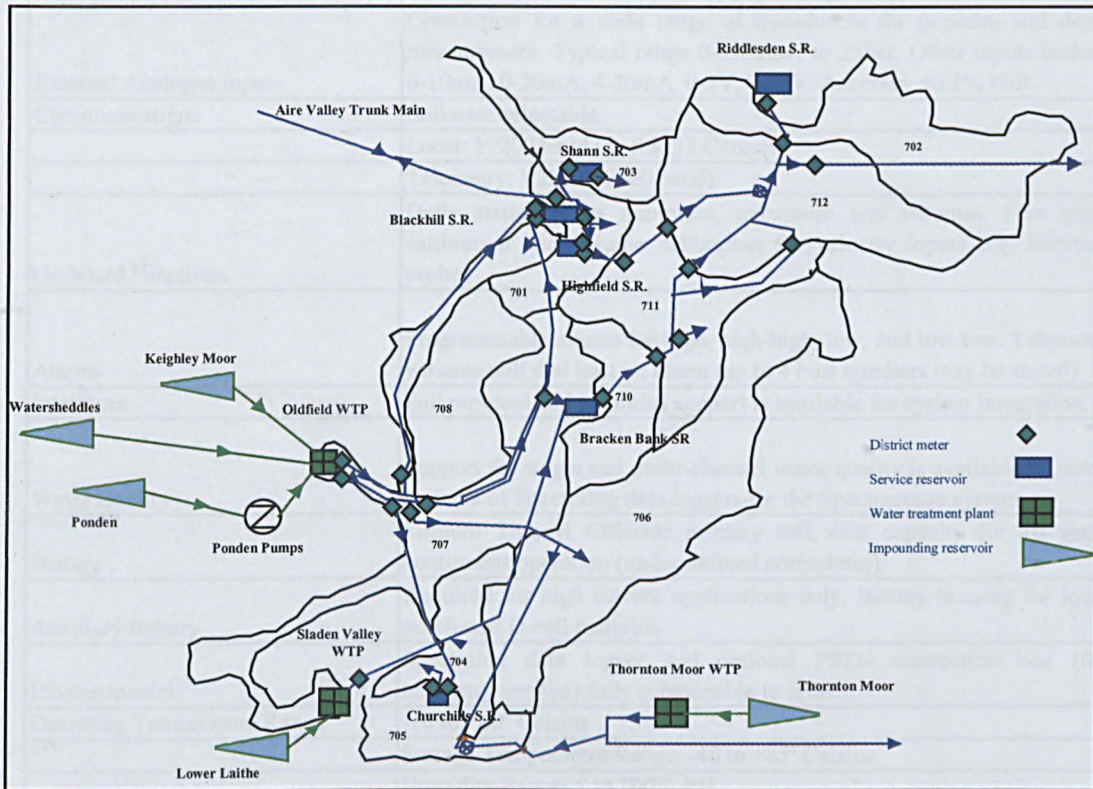


Figure 4.1 Location of the hydraulic measurement locations

The Spectralog instruments are robust and compact, built to IP69 requirements and interface to many flow meter types. This makes them easy to install and capable of withstanding the harsh environment associated with distribution networks. Table 4.1 details the specification of the instrument.

Memory	Cyclic, block or stop when full
	A minimum of 128 Kbytes (completely non-volatile)
	Up to 380 days (at 15-minute intervals)
Logging Interval	1 second to 24 hours.
Digital Input	All contact closure and opto pulse units supported with maximum input pulse frequency 1000Hz.
Internal Pressure Transducer	Ranges 0-10bar or 0-20bar. Accuracy $\pm 0.5\%$ FSR standard. Over pressure is 3 times the full range. Auto-zero calibration function.
External Analogue inputs	Connection for a wide range of transducers for pressure and depth measurement. Typical range 0-350mbar to 25bar. Other inputs include 0-10mA, 0-20mA, 4-20mA, 0-5V, 0-10V. Accuracy $\pm 0.1\%$ FSR.
Communications	Software selectable.
	Local: 19200 baud via RS232 Comms.
	Telemetry: V22bis (2400 baud)
On-board Functions	Daily statistics for minimum, maximum and volumes. Five point calibration and volume calibration for velocity inputs e.g. insertion probes
Alarms	Programmable alarms for high, high-high, low, and low-low. Telemetry variants will dial host on alarm (up to 4 host numbers may be stored)
Interfaces	Full protocol and technical support is available for system integration.
Water Quality	Support for single and multi-channel water quality is available on some variants of Spectralog data loggers for the Spectracense system.
Battery	Lithium Thionyl Chloride primary cell with capacity for 10 years continuous operation (under defined normal use).
Auxiliary Battery	Required for high current applications only. Battery housing for local purchased D-cell batteries.
Environmental	Spectralog data logger and optional PSTN connection box (for telemetry version) fully submersible to IP68.
Operating Temperature Range	-10 to +60° Celsius
	Storage Temperature Range: -40 to +85° Celsius
	Humidity Range: 5 to 100% RH
Dimensions	110mm x 60mm x 275mm Telemetry Logger.
Weights	1.5kg Telemetry Logger.
Altitude	The maximum operational altitude is 5000 metres un-pressurised.
Shock and Vibration	The shock is in accordance with a drop under gravity on to any flat surface from a height of 1m. The vibration withstand is in accordance with BS2011 part 2.1 Fc - Sinusoidal Vibration with the following severity: 10-57 Hz-0.075mmDA, 57-150 Hz-1g pk.

Table 4.1 SpectraLog instrument specification

Figure 4.2 shows a picture of the instrument and its installation at a flow meter site.

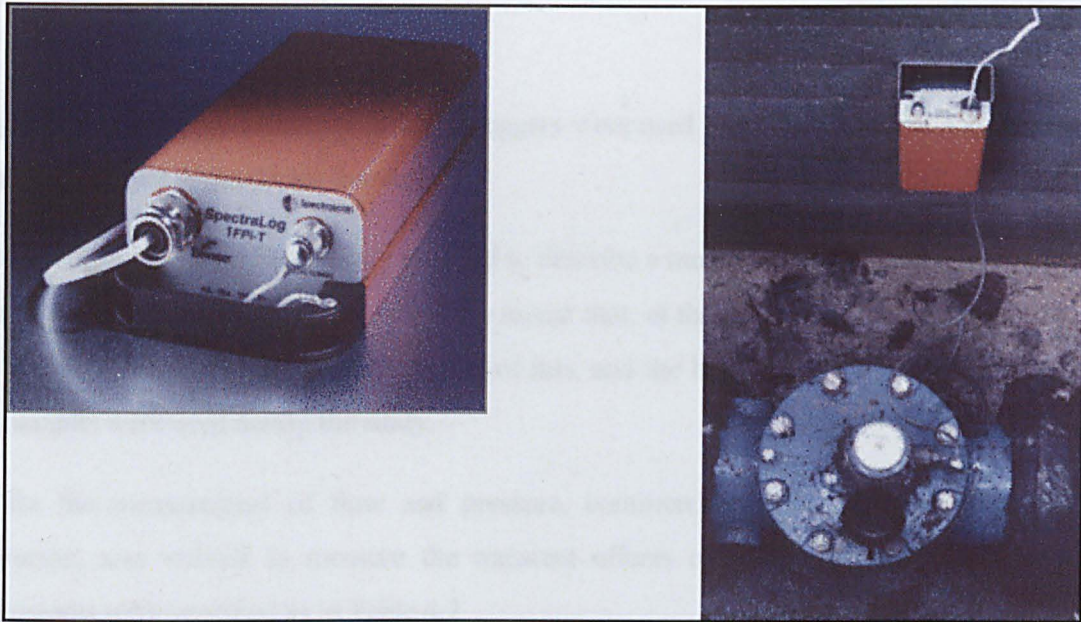


Figure 4.2 The Spectralog instrument on bench top and installed in field

Calibration certificates issued by the manufacturer certified that the instruments had been thoroughly tested and calibrated over their operational range. However, as a quality check and to avoid recording erroneous data, each instrument was subjected to laboratory “dead weight testing” of the internal pressure transducer before being installed in the field. This was thought to be important because if any one instrument (or more) was wrongly calibrated or damaged, this would pose extreme difficulties with the validation of the mathematical models that used the data as boundary conditions.

Each instrument was equipped with onboard data logging and the analysis functionality was embedded in the firmware. Flow and pressure were recorded on separate programmable data channels. The programs allow the instruments to be matched to a variety of flow meter types to ensure the correct data was recorded as each meter has a different flow calibration factor / pulse unit. It also permitted calibration offsets to be applied to the pressure readings where the instrument had to be located at a different level to the water main.

4.3 Transient Pressure

High-speed pressure instruments and data loggers were used to capture duration and amplitude of transient pressure surges in the network.

Because of the large amount of data required to describe a transient event, samples had to be taken at between 10 and 20 times a second. This meant that, at the higher sample rate, the memory of the logger was filled in 24 hours. Because of this, and the high cost of the equipment, only two instruments were used during the study.

As for the measurement of flow and pressure, commercially available equipment, Radcom Centurion, was utilised to measure the transient effects of surge events. These high-speed instruments were specified as in Table 4.2

Logger type	Single channel pressure
Range	0 to 200 mwc
Sensitivity	0.1 to 0.4 mwc (user defined)
Operating conditions	Operating temperature -10 to $+50$ °C
	1, 5, 10 and 20 samples a second
Memory	2 million readings + data compression (2 days data at 10 Hz sampling frequency)
Interface	RS232c PC interface

Table 4.2 Transient pressure logger specification

In order to effect calibration, the pressure transducers were dead weight tested in laboratory conditions. The instruments were then installed above and below a booster pump, and starting and stopping the booster pump generated pressure surges to field test the instruments. Figure 4.3 shows the layout of the pump house and the instrument locations.

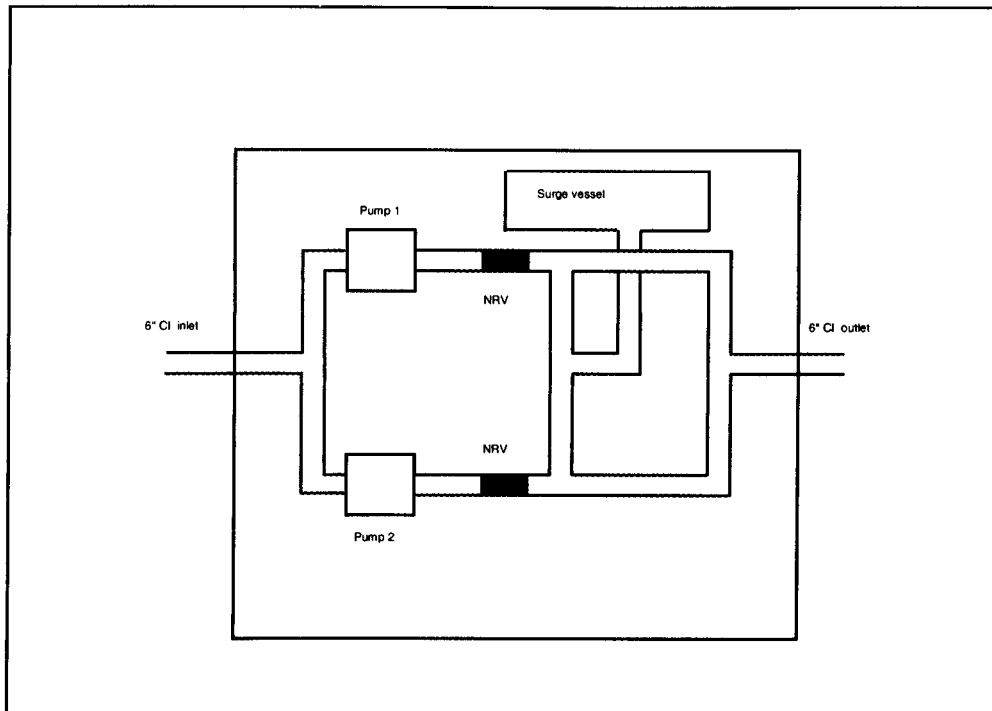


Figure 4.3 The pump house and instrument location

The pumps undergo a three stage stepped change when started or stopped, to reduce the magnitude of the surge generated. The instrument tolerance band was set to the minimum value of 0.1 mwc in order to obtain maximum resolution over the recording time. A period of approximately five minutes was allowed to ensure the pressure had equilibrated before starting and stopping the pump.

4.4 Water Quality Parameters

The environmental conditions associated with distribution networks are extremely hostile. When inserted into a water main the water quality instruments had to be capable of withstanding continuous external pressures of up to 180 mwc with additional surge pressures reaching over 220 mwc. The electronics had to be housed in wet conditions and be able to tolerate significant temperature changes (a 25⁰C shift is not uncommon). They had to be tolerant of residual chlorine and be protected from any solid material travelling in the bulk flow. As no such instruments existed at the start of the project, they were developed specifically for the task. The sensors were required to operate in a closed pressurised pipe system comprised of ferrous metals, non-ferrous metals, and plastics and they had to be able to withstand disinfection with a 100mg.l-1 sodium

hypochlorite solution prior to installation. The measurement units and operational conditions are shown in Table 4.3

Channel	Determinant	Measurement Units	Operating conditions
1	Temperature	°Centigrade	0 > 25 °C
2	pH	pH scale	5.5 > 9.5
3	Dissolved Oxygen	% saturation (as mg/l)	0 > 12
4	Conductivity	µS/cm	40 > 1100
5	Turbidity	NTU	0.01 > 4
6	Pressure	MWC	<= 180 m
7	Redox Potential	mV	0 > 800 mv
8	Cabinet temperature	°Centigrade	0 > 25 °C

Table 4.3 Water Quality determinants and operational conditions

The performance characteristics of the instruments are shown in Table 4.4.

Determinant	Accuracy	Repeatability	Resolution	Response time
Temperature	± 0.1	± 0.1 %	0.1 °C	< 5 seconds
pH	± 0.1	± 0.1 %	0.1	< 60 seconds
Dissolved Oxygen	± 0.1	± 0.1 %	0.1 mg/l	< 60 seconds
Conductivity	± 5	± 0.1 %	5 µS/cm	< 5 seconds
Turbidity	± 0.1	± 0.1 %	0.01 NTU	< 5 seconds
Pressure	± 0.1	± 0.1 %	0.1 m	< 5 seconds
Redox Potential	± 0.1	± 0.1 %	Best achievable	< 5 seconds
Cabinet temperature	± 0.1	± 0.1 %	0.1 °C	< 5 seconds

Table 4.4 Performance characteristics of the Water Quality instruments

Figure 4.4 shows the component parts of the water quality instrumentation. The design was modular in order that the user could combine any number of determinants and hardware functionality providing a very flexible tool.

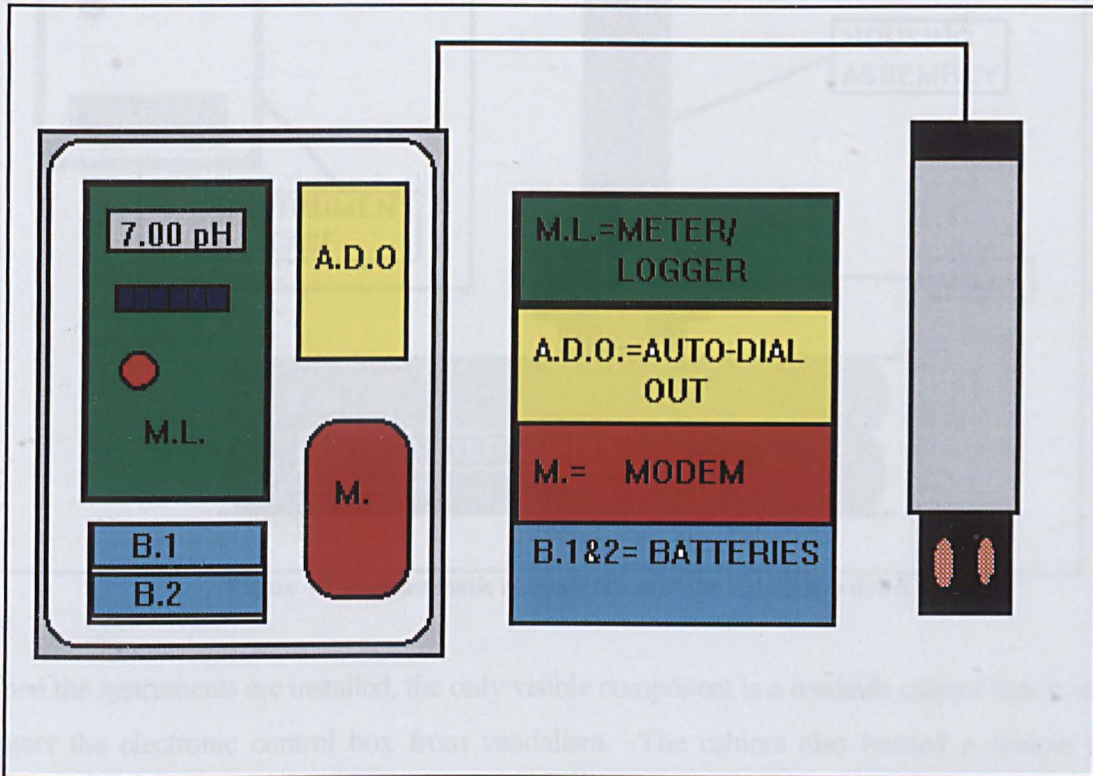


Figure 4.4 The component parts of the water quality instrumentation.

Each instrument could be used to log all or any individual channel at user defined time intervals down to 1 minute. Each channel incorporated high and low alarms that were set to trigger when a parameter exceeded normal operating boundaries.

Forty eight water quality monitors were installed at strategic locations the Keighley Distribution Network, as detailed in Figure 4.5 shows the physical installation details of the instrument housings within the pipe-work.

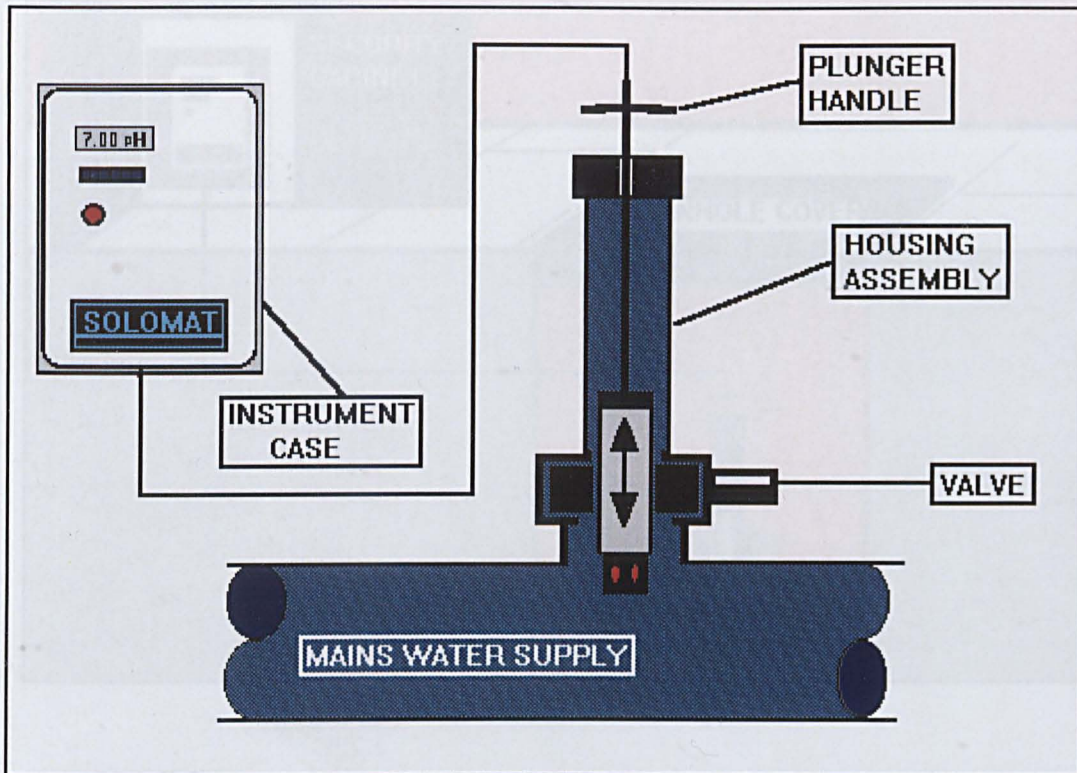


Figure 4.5 Diagrammatic representation of the installation detail

When the instruments are installed, the only visible component is a roadside cabinet that is used to protect the electronic control box from vandalism. The cabinet also housed a sample tap to facilitate manual measurement of any of the determinants for secondary calibration checks.

Figure 4.6 provides an overview of the complete instrument / logger / telecommunications site installation.

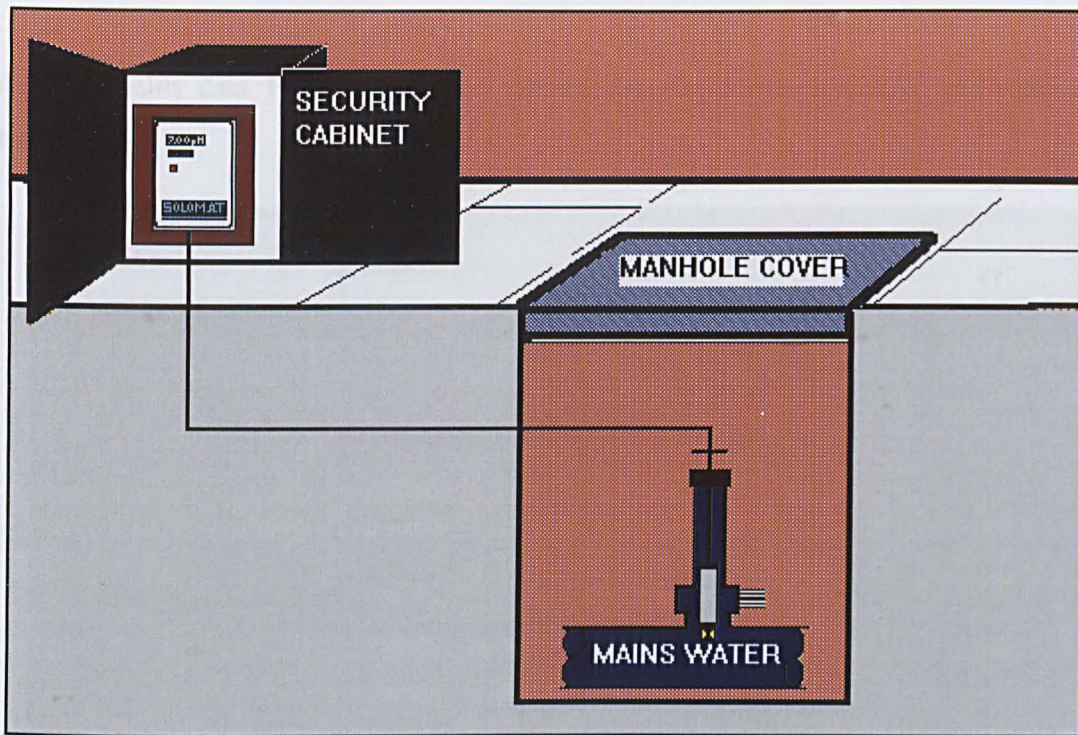


Figure 4.6 Complete measurement site layout

4.5 Data Collection and Transfer

The distribution network used in this project benefited from having a basic telemetry system. An operator at a central location could therefore remotely download network data from any number of different measurement sites.

This facility was exploited to the fullest extent by developing communications software that could remotely access both the hydraulic and the water quality instruments. Data was then downloaded automatically at user-defined frequencies. The software contacts the measurement sites by PSTN lines using a modem. It can download the entire logged data set or capture the latest single measurement.

This software made it possible to collect as much data as was required without the need for site visits or manual download to laptops and secondary transfer of data to PC for processing. Further, it provided a link between measured field data and the online functionality of the mathematical model facilitating a real time view of the hydraulic and water quality performance of the network at any user defined time and / or time interval. The communications software included functionality that permits the user to look at the data as it is transferred and to view data history from any of the measurement sites. Figure 4.7 shows the main screen highlighting both hydraulic

and water quality data. Figure 4.7 Screen capture showing current hydraulic and water quality values.

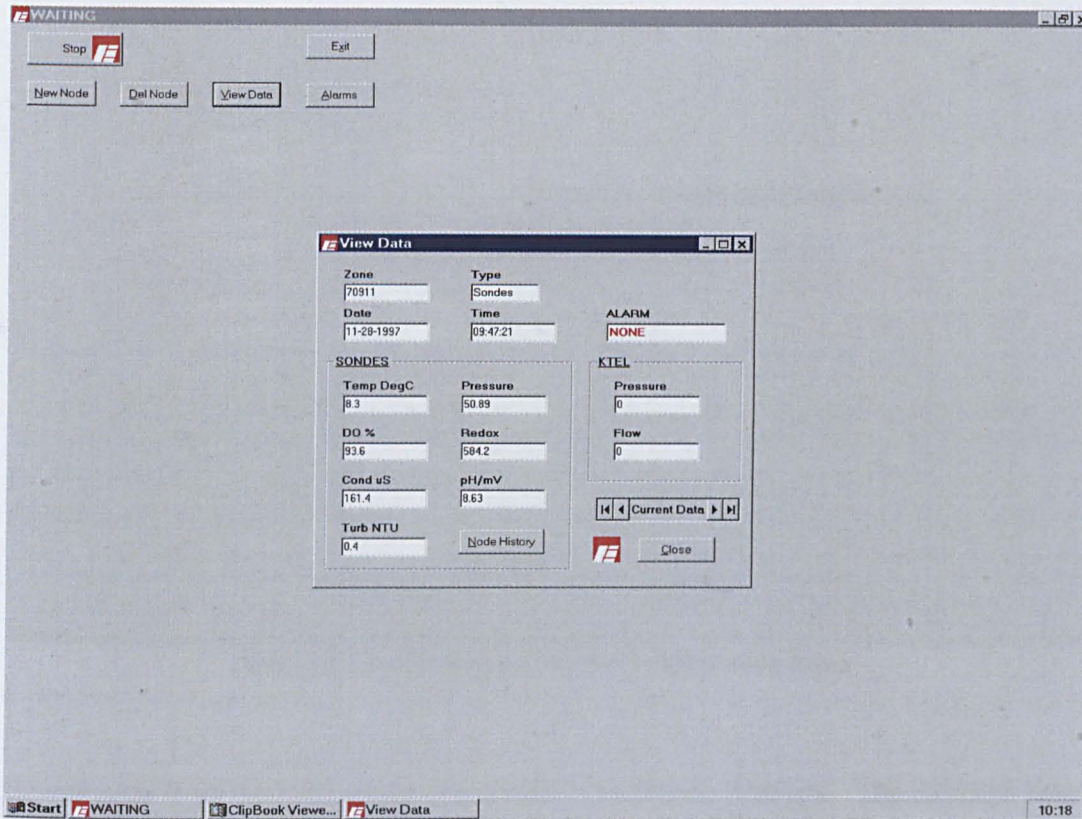


Figure 4.7 Current hydraulic and water quality values

As detailed earlier, the communications software was used to generate alarms on any of the determinants. This facility can then be used to generate on screen alarms for the user. The alarm status is shown in Figure 4.7 above.

The software is programmable to allow the user to access only the data required at any particular time. Channels may be turned on or off as is required. Figure 4.8 shows the dialogue box where the individual parameters may be selected.

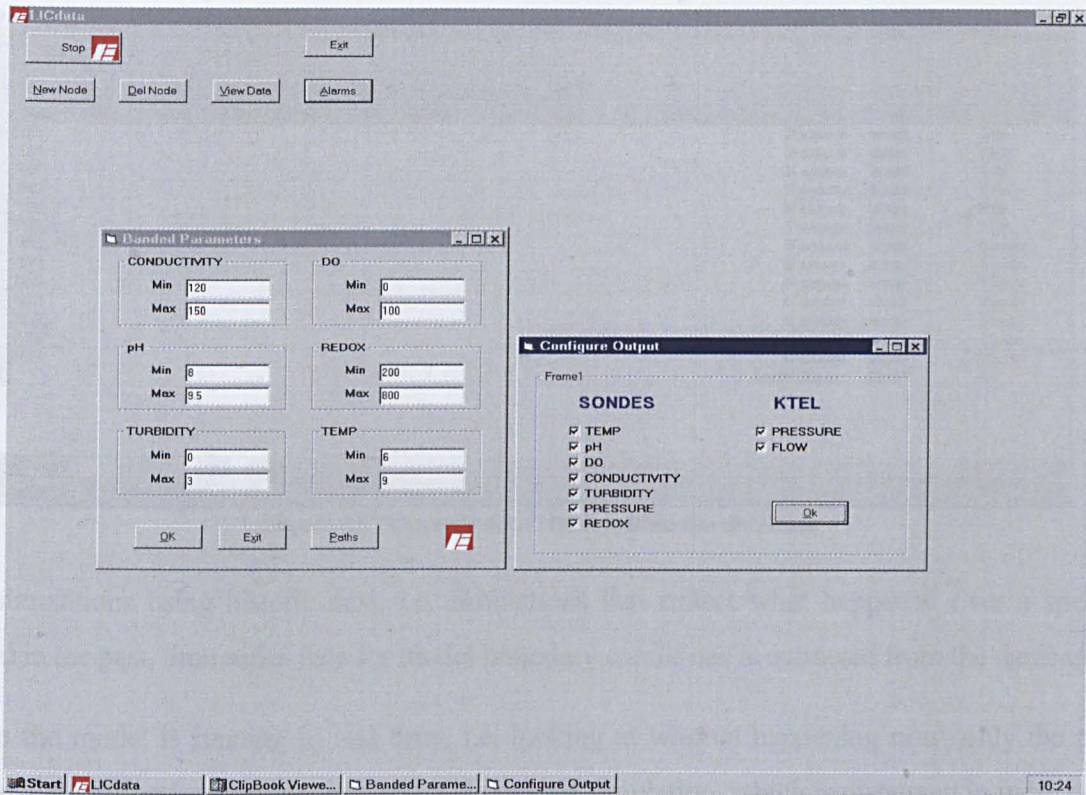


Figure 4.8 Communications software configuration screen

Data downloaded from the instruments was stored in an Access database. The database held both historic and current network data. The modelling software used this data as boundary conditions for simulation. Table 4.5 shows the “current data” table within the database.

Zone	Flow	pressure	Pt100	pH/mV	Salin	conduct	turbid	depth	ISE	4000temp	index	Date	time	oldata	type
70201											32	xxxxxxxxxx	xxxxxxxxxx	0	Ktel
71201											33	xxxxxxxxxx	xxxxxxxxxx	0	Ktel
70901											34	xxxxxxxxxx	xxxxxxxxxx	0	Ktel
70902											35	xxxxxxxxxx	xxxxxxxxxx	0	Ktel
71101											36	xxxxxxxxxx	xxxxxxxxxx	0	Ktel
71301											37	xxxxxxxxxx	xxxxxxxxxx	0	Ktel
70911											38	xxxxxxxxxx	xxxxxxxxxx	0	Sondes
70913											39	xxxxxxxxxx	xxxxxxxxxx	0	Sondes
70929											40	xxxxxxxxxx	xxxxxxxxxx	0	Sondes
70920											41	xxxxxxxxxx	xxxxxxxxxx	0	Sondes
70912											42	xxxxxxxxxx	xxxxxxxxxx	0	Sondes
71326											43	xxxxxxxxxx	xxxxxxxxxx	0	Sondes
71327											44	xxxxxxxxxx	xxxxxxxxxx	0	Sondes
*											[Counter]	#Name?	#Name?	0	

Figure 4.9 “Current data” table within the database

For simulations using historic data, i.e. simulations that reflect what happened over a specific period in the past, time series data for model boundary conditions is extracted from the database.

When the model is running in real time, i.e. looking at what is happening now, only the most current data for each parameter is used. This type of simulation, which is discussed in more detail in Chapter 8 uses stored network states as the starting point of the simulation and automatically updates these to reflect the latest boundary conditions. In this way the model output shows both the current network state and any previous network state at a given time.

The model stores results in order that the user can produce a variety of different output types to assist understanding and decision-making. The next chapter of the thesis describes the way in which the hydraulic performance of the system was established.

Chapter 5 - Hydraulic Analysis

5.1 Building the Hydraulic Model

5.1.1 Background

Hydraulic models of the individual Leakage Control Zones comprising the study network had been constructed a number of years before this project using the Ginas software. However, changes to the distribution network assets and the network demands during the time between the construction of the models and the start of this project made them potentially no longer representative of the current network operation. It was necessary therefore to amend the models to incorporate the current network details and convert them into the Aquis framework.

Amendments to the model included the addition of new housing developments, some water main rehabilitation and replacement, inclusion of pressure reduction schemes and network re-zoning. Initial preparation therefore began by rebuilding the Leakage Control Zone models to reflect current network assets, their configuration and associated demands. The individual leakage control zone models were re-built and then merged to produce a single model of the entire distribution network using the Aquis software.

This “system” or Water Supply Zone model was then used to analyse the study distribution network using a traditional modelling approach whereby local problems in the network were solved without due regard to the impact on or from the complete network. This work consisted of an analysis phase and development of improvement schemes designed essentially to manage pressure locally whilst taking into account the need for adequate flow. The first section of this chapter presents the details of this analysis.

5.1.2 The Hydraulic Model Build Process

Hydraulic model building is a complex task and the main activities undertaken in the building of the hydraulic model are highlighted in the flowchart shown in Figure 5.1.

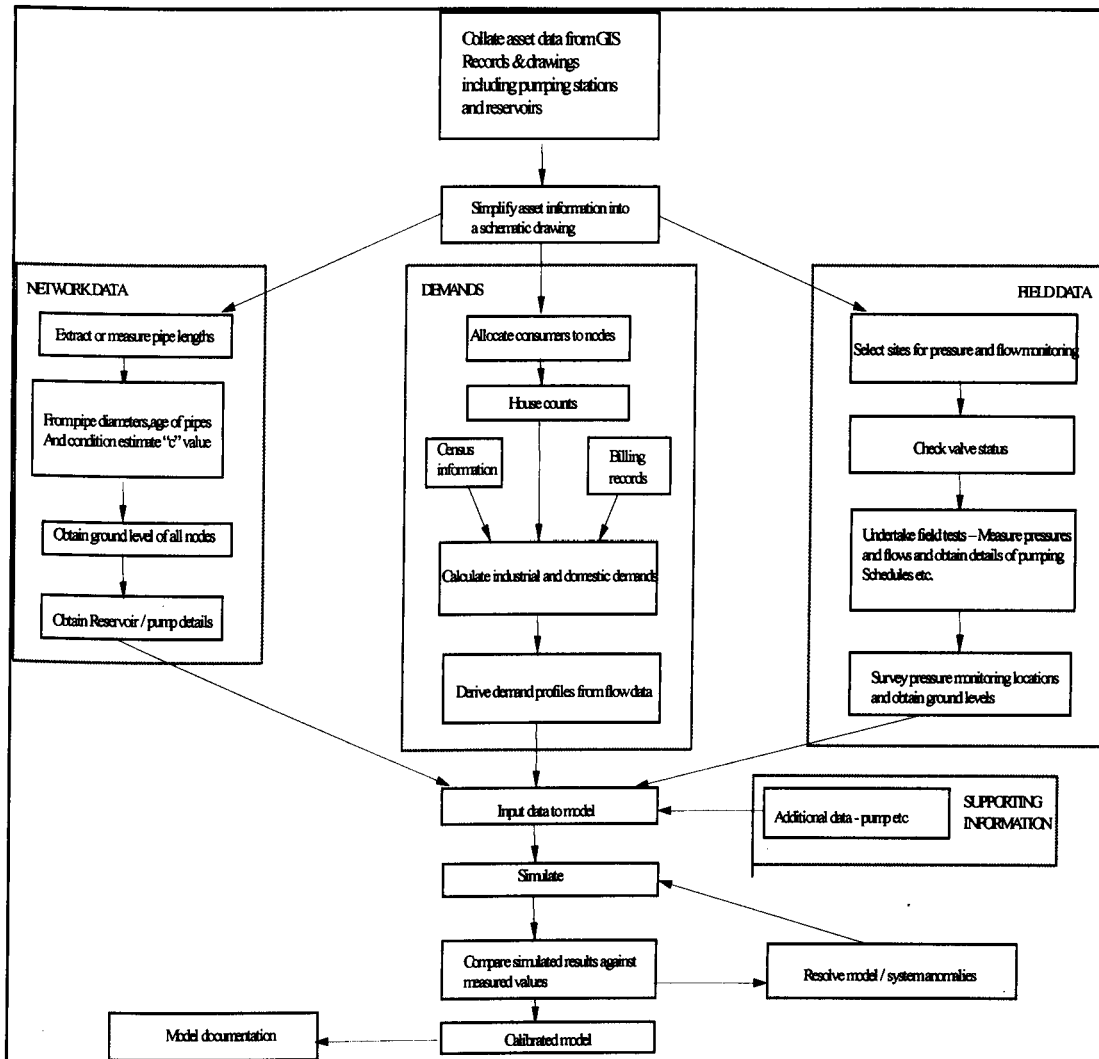


Figure 5.1 The model building process flow chart

The following sections describe how the process was used.

5.1.2.1.1 Asset Data

The asset data, which is the static data representing the individual component parts of the network was acquired from a Graphical Information System (GIS). This information was supplemented by expert knowledge and physical measurement and observation where necessary.

5.1.2.1.2 GIS Data

The majority of the necessary water main records were stored on the GIS system. This system contained comprehensive details of the underground assets and their attributes. It was possible to retrieve single pieces of information, for example, the detail of the water mains or to have digitised background maps with any amount of asset information superimposed as an overlay. Figures 5.2 and 5.3 show digitised background map and digitised background map with network details superimposed respectively.

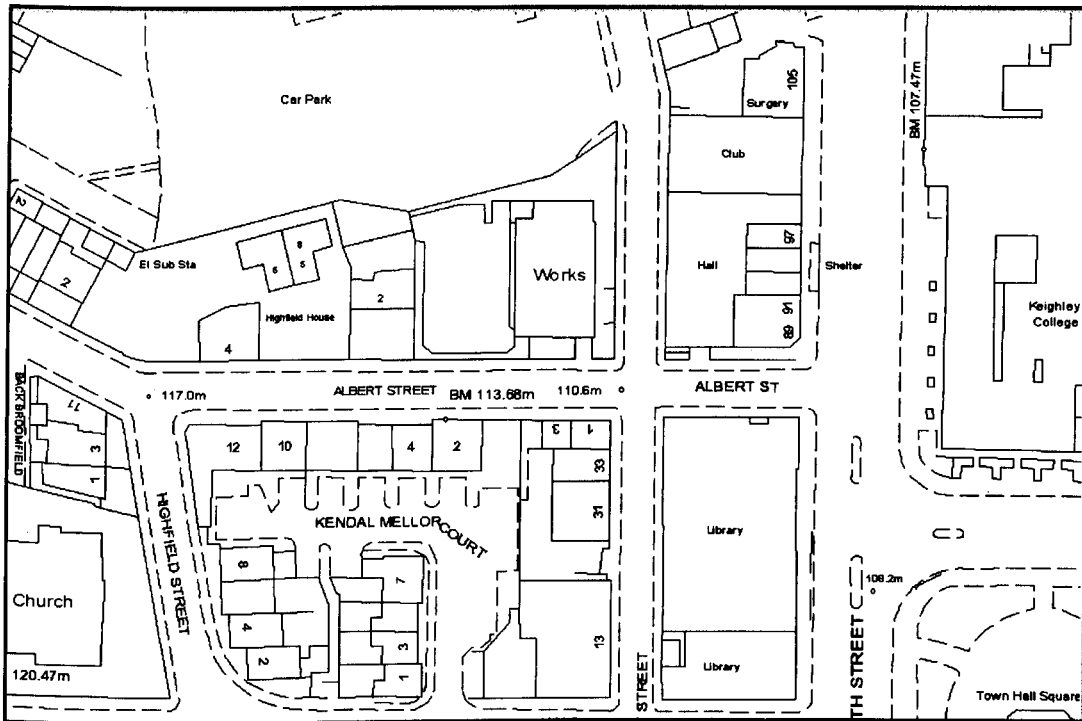


Figure 5.2 GIS plot of digitised background map details

The digitised background map provides essential information about element connectivity and location of properties within the Leakage Control Zone boundary

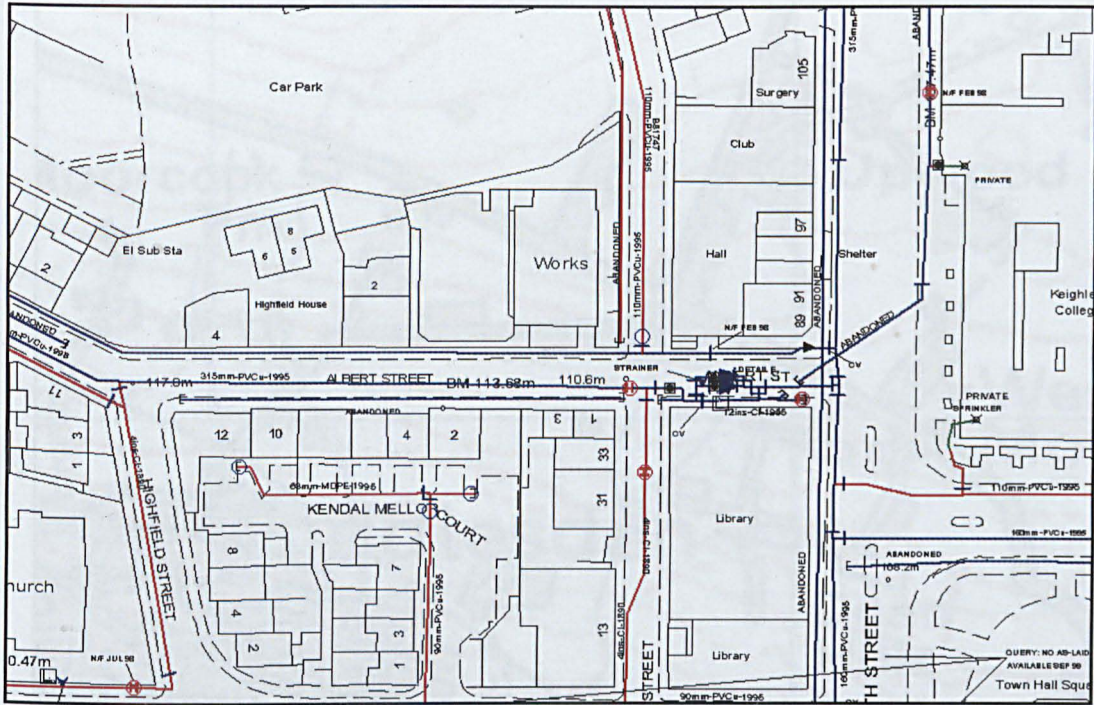


Figure 5.3 GIS plot of digitised background map with network overlay

The water network overlay highlights water mains, fire hydrants, valves, water storage reservoirs and any other asset information contained on the GIS system and each element has its geographical co-ordinates and element specific details attached.

The GIS system was updated regularly with new element information. For example, information about new building developments and any rehabilitation and renewal work that had been carried out on the networks. This enabled properties to be allocated to the correct nodes in the model.

The system also contained Ordnance Survey data providing an aspect of the topography of the zone that can be used as a backdrop for the water network information. Figure 5.4 shows the geographical detail of the Ordnance Survey data including contour lines.



Figure 5.4 GIS plot with Ordnance Survey background showing contour lines

The GIS system was interrogated to obtain and export the model asset data. The data was extracted by “querying” the GIS dataset. The complete GIS data set included large amounts of information not required for the hydraulic model such as, for example, many nodes that are superfluous to requirements, abandoned pipes and text and graphics which are not used in the model. The query was designed therefore to extract only the minimum detail essential for building the hydraulic model. This included:

- Pipes - Co-ordinates, length, diameter, material type, age.

- Valves - Boundary, pressure reducing, pressure sustaining, non-return and their co-ordinates

- Pumps – Co-ordinates

- Service Reservoirs – Co-ordinates

and any other element that had an impact on the network model. The query also determined where nodes were placed in the model. Not all nodes in the GIS were transferred to the model. The higher the number of nodes the longer the simulation takes.

Figure 5.5 shows how the query chose only nodes that were essential for the model, for example, where a pipe diameter changes or where two or more mains connect to each other. The green spots are the nodes chosen for the model. The diagram shows there are elements such as hydrants (red circles) that have not been turned into model nodes. This process allowed the production of a simple model schematic of the network that was then used for the planning of the rest of the model build and calibration process.

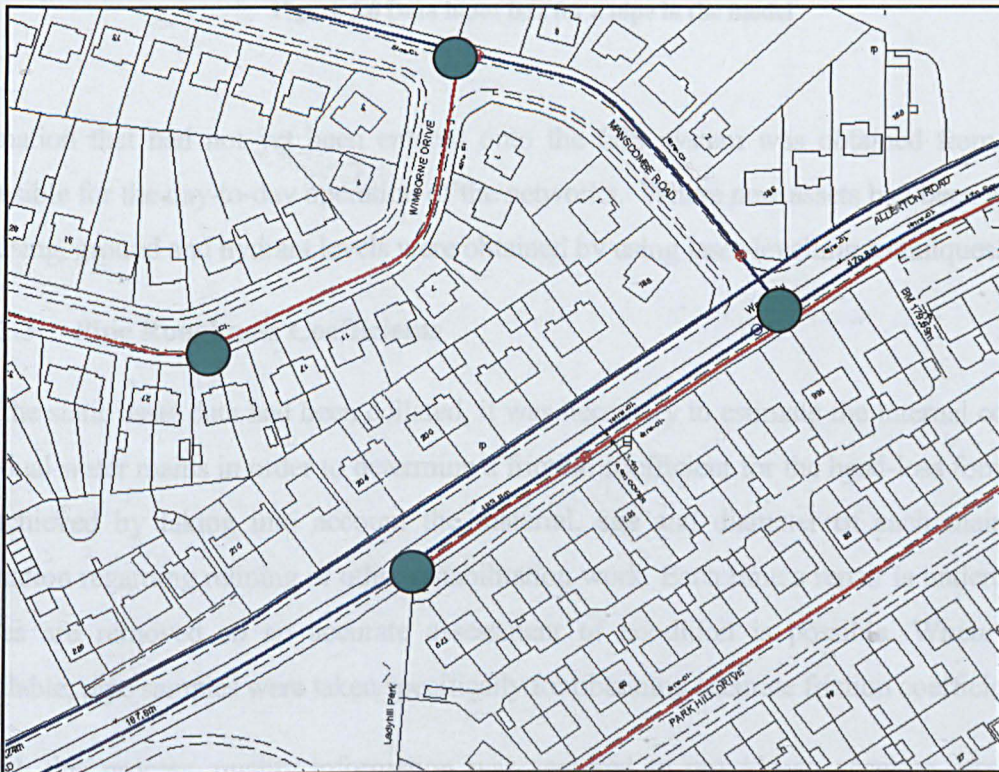


Figure 5.5 Example nodes chosen for hydraulic model

The model schematic was used to allocate properties to appropriate model nodes. Level information for the nodes was acquired partly from the GIS system and partly by using the OS contour information to extrapolate the node levels. Data for elements such as pumps and service reservoirs was obtained from a diversity of data sources including manufacturers information, physical measurement and technical drawings. All this data was manually entered into the model via the graphical user interface. Figure 5.6 shows the data entry box for a pipe.

Field	Value	Out
Pipe name:	L-0179	<input type="checkbox"/>
Comment:	Riddlesden East	<input type="checkbox"/>
Node 1:	6203	<input type="checkbox"/>
Node 2:	6062	<input type="checkbox"/>
Length [m]:	575.00	<input type="checkbox"/>
Installation year:	1948	<input type="checkbox"/>
Demand zone:	K712	<input type="checkbox"/>
Pipe type:	Modified	<input type="checkbox"/>
Int. diameter [mm]:	150.00	<input type="checkbox"/>
Friction factor [mm]/[-]:	140.0000	<input type="checkbox"/>
Local pressure drop coef. [-]:	0.00	<input type="checkbox"/>
Material:	Cast Iron	<input type="checkbox"/>
Lining:	Epoxy resin	<input type="checkbox"/>
Nominal diameter [mm]:	*	<input type="checkbox"/>

Buttons: OK, Cancel, Help, Reverse Nodes

Figure 5.6 Data input box for a pipe in the model

Information that had not yet been entered onto the GIS system was obtained from the teams responsible for the day-to-day operation of the networks. Where new assets had been added, such as housing, ground and hydrant levels were obtained by using laser-levelling techniques.

5.1.2.1.3 Pipe Roughness Coefficients

Once the static asset data had been collated, it was necessary to estimate the internal condition of individual water mains in order to determine a friction coefficient for the head-loss formula. This was achieved by taking into account the material, age and diameter of each main and any information regarding relining or other rehabilitation work. Each time a repair is undertaken, pipe samples are removed so an accurate assessment of condition is possible. Where data was unavailable, pipe samples were taken specifically for obtaining accurate friction coefficients.

Through this process, quality information was acquired to provide an accurate assessment of friction coefficient for the majority of pipes in the system. Groups of pipes in a particular area of the network with a given size and material were allocated the same friction coefficient. However, coefficients were varied for pipes in different parts of the network to allow for different rates of degradation due to water chemistry or other factors. The mains condition can be represented by one of two factors depending on which head-loss formulas are utilised by the hydraulic engine. If the Colebrook-White relationship is used then a "k" factor is required. If Hazen-Williams equation is used then the coefficient is called a "C" value. For this study Hazen-Williams and "C"

values were used, as it has been shown to be the more appropriate for the sizes of mains modelled and their internal condition. Figure 5.7 depicts the Hazen Williams formula.

$$G = \frac{3.58821 \times 10^{-6} \cdot C \cdot d^{2.63}}{L^{0.54}}$$

$$h_j - h_i = (Q/G)^{1.85}$$

Where:-

- d = diameter in mm
- L = length in m
- C = Hazen-Williams Coefficient
- G = Conductance
- Q = Flow ls⁻¹
- h_j = head upstream in m
- h_i = head downstream in m

Figure 5.7 Hazen Williams Formula

Other heights were obtained from plans of, for example, service reservoirs and the pipe work associated with them. The rest of the levels were taken from Ordnance Survey maps using contour lines, extrapolation and knowledge of the depth of the mains in the ground.

Once the static model data had been compiled it was required to input known performance data that have a significant impact on the hydraulic performance of the network. This included, for example, the dynamic data such as zone inflows and exports, large industrial users and domestic demands. The dynamic data was collected as part of an extensive field test.

5.1.3 The Field Test

Field test data included measurement of flow and pressure at specific locations across the network (Chapter 4 - Figure 4.1). Flows were measured to provide data with which to undertake a demand analysis and pressures were measured for model calibration purposes. Both flow and pressure measurements were used for model calibration purposes. (Section 5.1.4).

The exercise produced a data set that reflected the performance of the network over a snapshot of time, i.e. the data was only relevant to the network configuration prevailing at the time the field test was undertaken.

5.1.3.1.3 Flow Data

Flow data for the model was collected from a variety of sources. These are detailed below.

5.1.3.1.1 Zone Inflows and Exports

All Leakage Control Zone inflows and exports were measured by flow meters or insertion probe flow meters and recorded by data loggers. Flow was also recorded at inlets to sub-zones (parts of the network isolated from the rest by a single device such as a pressure-reducing valve) where possible. A typical days data from all metered locations was then used to determine the industrial and domestic demands. This was achieved by allocating all the supply inputs onto the model, making an allowance for all measured demands and applying a demand analysis that used an estimated domestic demand along with nodal property allocations to calculate the domestic consumption profile and hence the overall nodal demands. The remaining “unaccounted for” water was assumed to be leakage. The leakage volume was then allocated to all demand nodes proportionally to the number of properties at the nodes.

Four basic demand profiles were allocated to nodes. Large industrial, major industrial, domestic and unaccounted for water. Unaccounted for water is assumed to be leakage but undoubtedly contains some legitimate demand. These standard demand profiles were “normalised” and factored by the average node flow in the model. The profile shapes were derived by the Water Research centre and are used by most (British) water companies.

5.1.3.1.2 Industrial Demands

All customers taking flows in excess of 400 m³ per annum, or those who have an unpredictable demand are metered as they can have significant impact on the hydraulic operation and performance of the network. This type of consumption, normally industrial, tends to have the same usage profile every day, based on type of business, and so can be imposed on the model as a standard demand curve at the appropriate model node. There are 5 different shapes of profile for this type of normalised demand. As an example, Figure 5.8 shows a typical industrial “10-hour” flow profile.

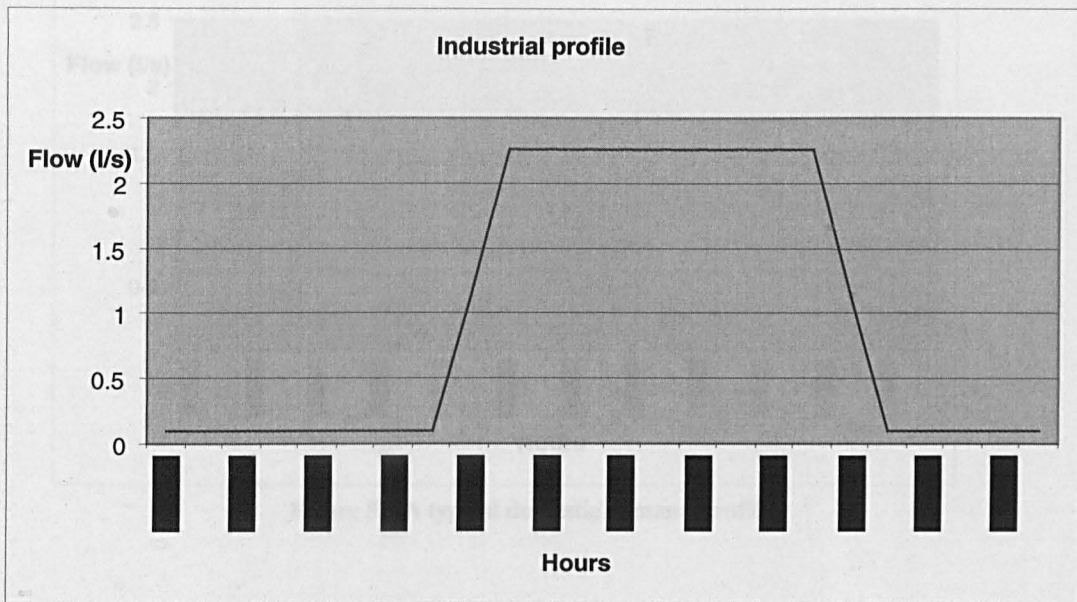


Figure 5.8 A typical industrial demand profile (10 hr)

All the metered customers were allocated a normalised profile of this kind in the hydraulic model. The overall demand allocated to this user type was adjusted by factoring this profile up or down as indicated necessary by the previous 12 months data.

However, where the user was unpredictable or had very high flows, for example a major industrial user (usually $>10,000 \text{ m}^3/\text{annum}$), actual measured data was used to generate daily demand profiles.

5.1.3.1.3 Domestic Demands

Domestic demand was based on number of properties and occupancy rate using 132 litres per person per day. For example, if a node had 10 properties allocated to it and each property has 3 residents the demand allocation was $10 \times 3 \times 132 = 3960 \text{ l/day}$. This value is sometimes adjusted for socio-economic groupings, although the primary information is now taken from domestic consumption monitors. As with the industrial demand a normalized profile was assumed and was factored up or down with the factor determined from the demand analysis. Figure 5.9 depicts a typical domestic demand profile.

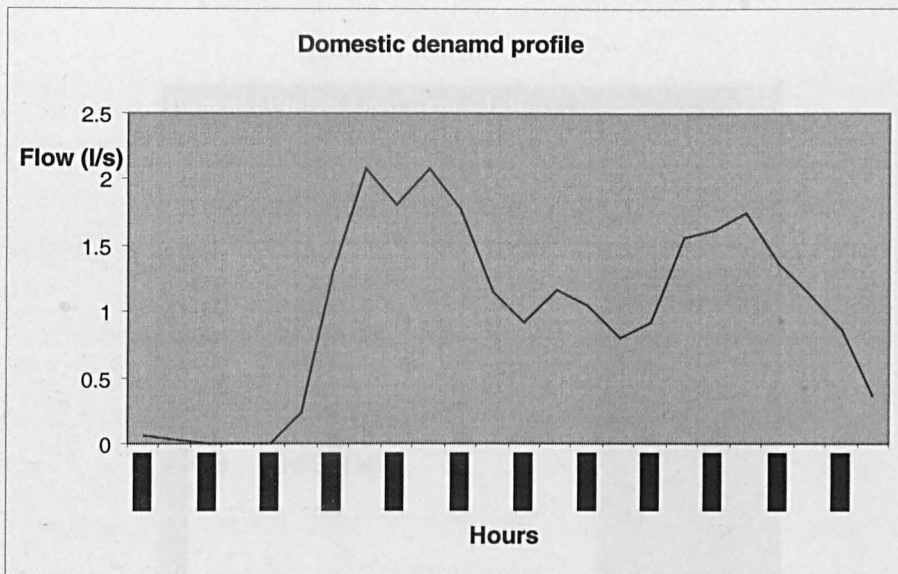


Figure 5.9 A typical domestic demand profile

5.1.3.1.4 Unaccounted for Water

Unaccounted for water was allocated at a flat rate across all demand nodes. The amount allocated at each node was determined by the demand analysis.

These normalised demands and measured data sets were allocated to appropriate nodes in the model and were used to “drive” the hydraulic simulation. Figure 5.10 shows a list of typical demand profiles for a model.

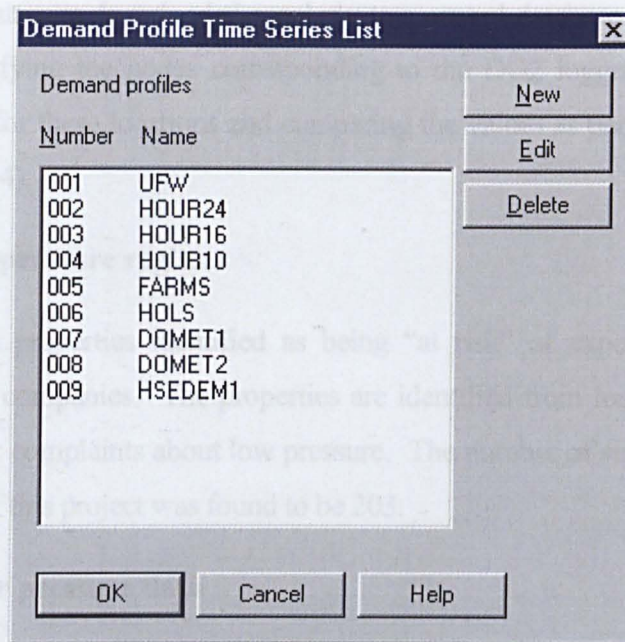


Figure 5.10 A typical domestic demand profile

5.1.3.1.4 Pressure Data

Pressure data was collected from a variety of sources and these are detailed below.

5.1.3.2.1 DG2 logged data

There is a statutory requirement placed upon water companies to maintain minimum water pressure standards of service for their users and to report failures to comply. This is called DG2 reporting because water companies have to report on a series of performance issues to the Director General, Water, and each performance indicator is prefixed with DG. DG3, for example, is unplanned interruptions to supply.

Low-pressure areas were present within the study network at the onset of this project. One example was the streets around Bracken Bank Service Reservoir where high flows caused by bulk water transport combined with the effect of corroded mains generated high head loss in the water mains.

As part of the standards of service monitoring exercise, DG2 data loggers recording pressure measurements were placed at critical nodes within each LCZ. DG2 loggers are usually located at the highest elevation points in a leakage control zone although, in some cases, properties are placed at risk of low pressure due to poor main conditions rather than because of their elevation.

The DG2 pressure data was downloaded regularly to a central database and this data was used in the project by identifying the nodes corresponding to the DG2 logger locations in the model, predicting pressures for these locations and comparing the values as part of the model calibration process. (Section 5.1.4).

5.1.3.2.2 Low pressure register

A database of those properties identified as being “at risk” of experiencing low pressure is maintained by water companies. The properties are identified from local knowledge, modelling studies, and customer complaints about low pressure. The number of such properties in the study network at the start of this project was found to be 203.

5.1.3.2.3 Other pressure data

As well as the above two methods of data collection an average of 20 pressure loggers were located in each Leakage Control Zone in order to obtain sufficient data to undertake the model calibration procedure. The pressure loggers were distributed evenly across a zone and usually located on fire hydrants. An accurate elevation for each node where pressure was measured was obtained using laser-levelling techniques.

5.1.3.3 Data Smoothing

The data used for model calibration was pre-processed using a bespoke software package called LACE. LACE imported the measured data files and was used to smooth the data. Smoothing is required because the hydraulic simulation engine cannot deal with sporadic changes that cause “spikes” in the data. *Skipworth, (1997)*, showed that these spikes could therefore prevent the validation process.

Figure 5.11 shows a data set before and after 2 levels of smoothing of pressure data.

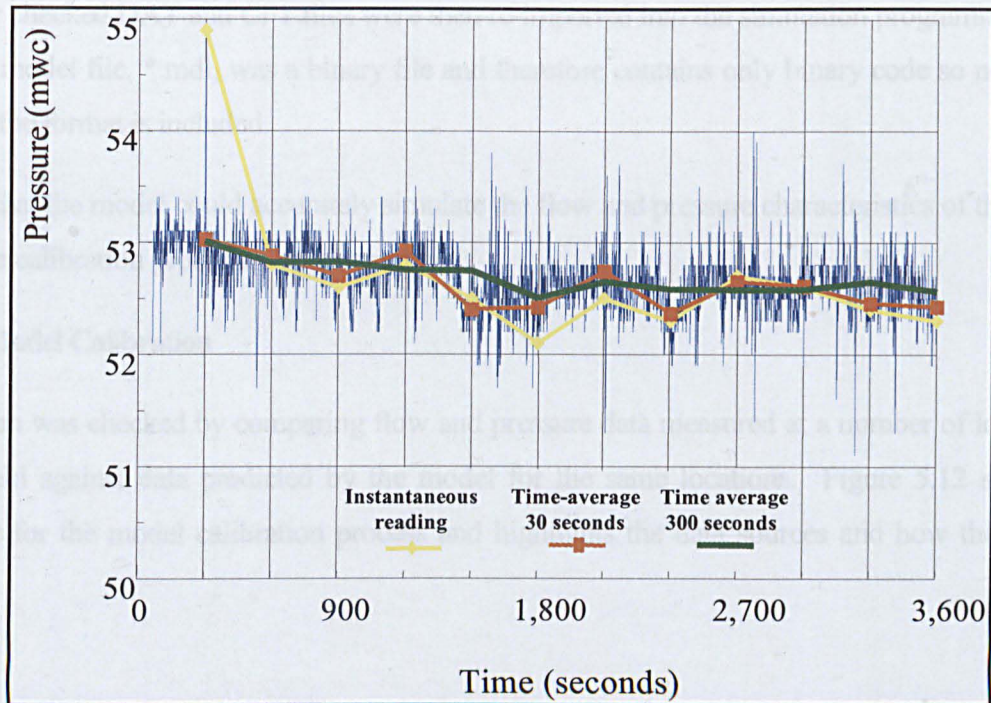


Figure 5.11 Effect of data smoothing

Figure 5.11 clearly illustrates how the instantaneous measurements containing “noise” can be smoothed to minimise the noise whilst retaining the same overall shape time-series profile. Once smoothed, the data was stored in hourly intervals (represented by squares in Figure 5.11) representing a 24-hr period.

Following simulation of the network during the model calibration process, these time-series were available for comparison by plotting measured (reference) values against simulated (predicted) values. Once all the necessary data had been merged within the model file, an initial simulation was undertaken to test the integrity of the model. This process automatically created 2 ASCII files.

A *.DAT file that contained all pipe, node and dynamic element data (pumps, reservoirs, valves etc.) and demand allocations

A *.CF1 file – Containing diurnal profiles associated with nodal demands and pressure/flow reference data

Once generated the new model files were subjected to a number of validation checks for missing or incorrect data. Any errors were corrected manually.

The error checked DAT and CF1 files were then re-imported into the simulation programme. The resultant model file, *.mdl, was a binary file and therefore contains only binary code so no figure showing the format is included.

In order that the model could accurately simulate the flow and pressure characteristics of the study network a calibration procedure was completed.

5.1.4 Model Calibration

Calibration was checked by comparing flow and pressure data measured at a number of locations in the field against data predicted by the model for the same locations. Figure 5.12 shows a flowchart for the model calibration process and highlights the data sources and how they were used.

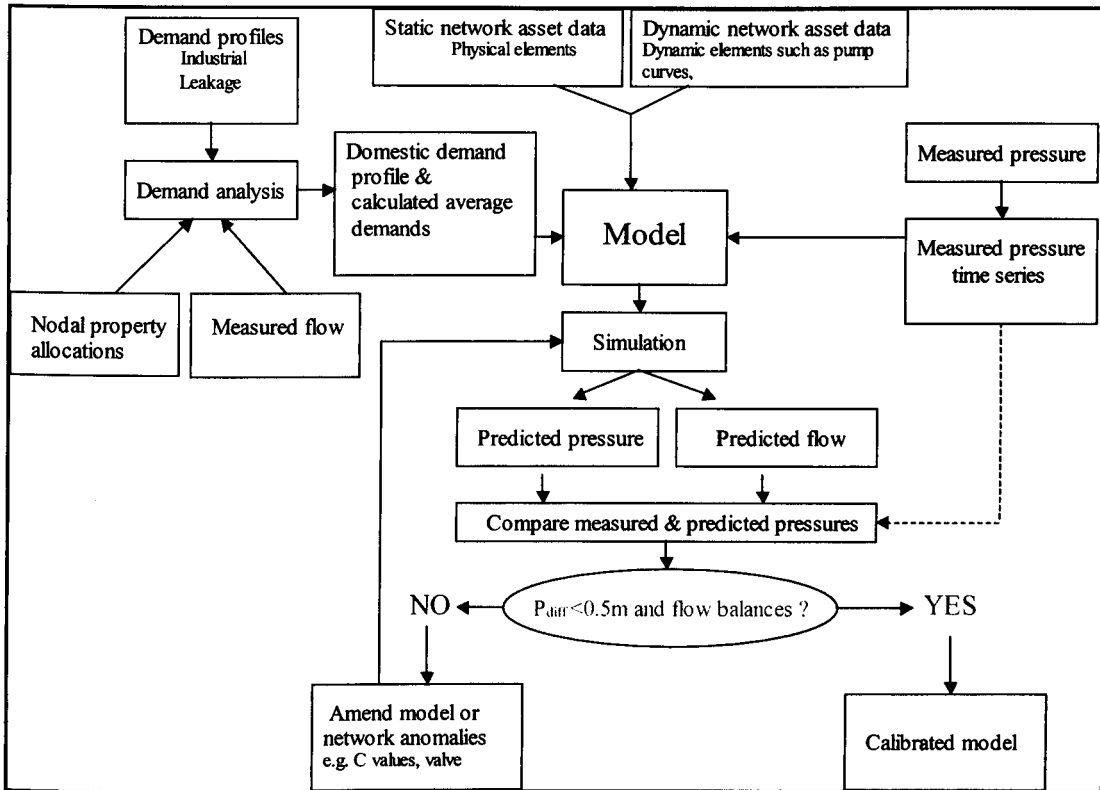


Figure 5.12 Model calibration flow chart

Knowing which nodes had properties allocated, the measured industrial flows, the leakage demand profile and the shape of the domestic profile, the demand analysis determined the domestic demand profile and calculated the average demands from the measured zone flow.

The model uses this information to predict flow and pressure in every node and pipe in the network. The calibration procedure is then dependent upon agreement of predicted and measured pressures at nodes and in pipes.

5.1.4.1 Use of flow data

Measured flow data was used in the following ways:

A flow *into* the system can be regarded as a negative demand and therefore allocated to the network inlet node(s) in the model. It was possible therefore to impose the measured network inlet flow(s) into the model at the inlet node(s). (This was also taken account of during the demand analysis). Flows along pipes were used for calibration of for example, reservoir or pump flows.

Where data was not already available, flows were also used, for example, to empirically derive pump curves, or to determine when pumps switched on and off. Figure 5.13 shows how measured flow data for a pump can be used to determine the switching times. It is clear from the plot that the pump turns on at 11 am and off at 2 am.

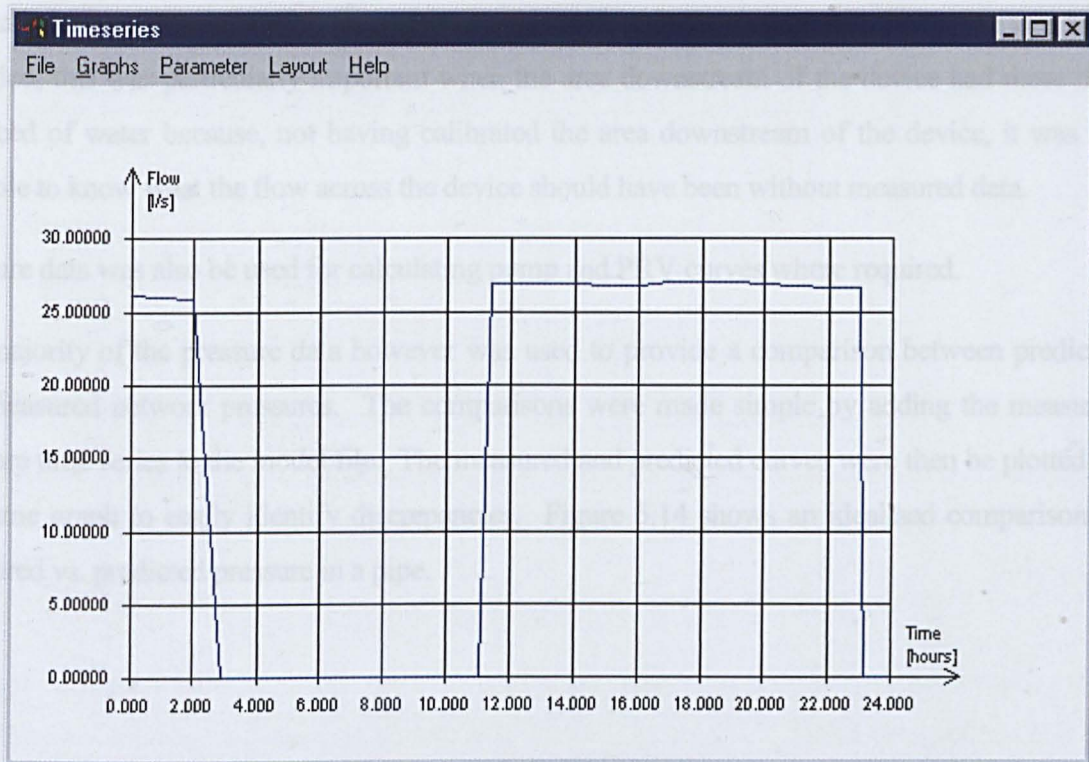


Figure 5.13 Measured pump flow

5.1.4.2. Use of Pressure Data

Pressure data from the loggers used in the field test was utilised in a number of ways. Like flow data, pressure time-series were imposed at sources, for example the service reservoirs, or inlet nodes. This ensured the driving force in the model reflected real network characteristics. For the parts of the model that were difficult to calibrate, pressures were imposed at nodes immediately downstream from pumps, pressure reducing valves and (where necessary) service reservoirs. In this way, it was possible to calibrate several small, discreet areas without having to worry about the knock on effects of these devices. When the small areas had been calibrated, the devices were re-introduced to the model. Any discrepancy between the predicted and measured results was then assumed to be due to the devices and the device characteristics amended accordingly.

re-introduced to the model. Any discrepancy between the predicted and measured results was then assumed to be due to the devices and the device characteristics amended accordingly.

In order for this process to work it was necessary to impose a recorded flow across the device from the node on the upstream side of the device. Otherwise, it would not have been possible to calibrate the area upstream of the device. In order to do this the flow across the device was to be recorded, this was particularly important when the area downstream of the device had more than one feed of water because, not having calibrated the area downstream of the device, it was not possible to know what the flow across the device should have been without measured data.

Pressure data was also be used for calculating pump and PRV curves where required.

The majority of the pressure data however was used to provide a comparison between predicted and measured network pressures. The comparisons were made simple by adding the measured pressure time series to the model file. The measured and predicted curves were then be plotted on the same graph to easily identify discrepancies. Figure 5.14 shows an idealised comparison of measured vs. predicted pressure in a pipe.



Figure 5.14 Comparison of measured pressure vs. predicted pressure in a pipe

There are a large number of graphs associated with the calibration process and, for brevity, only a small number are reported here, as shown in Figures 5.15 to 5.17. These demonstrate that the model was accurately calibrated

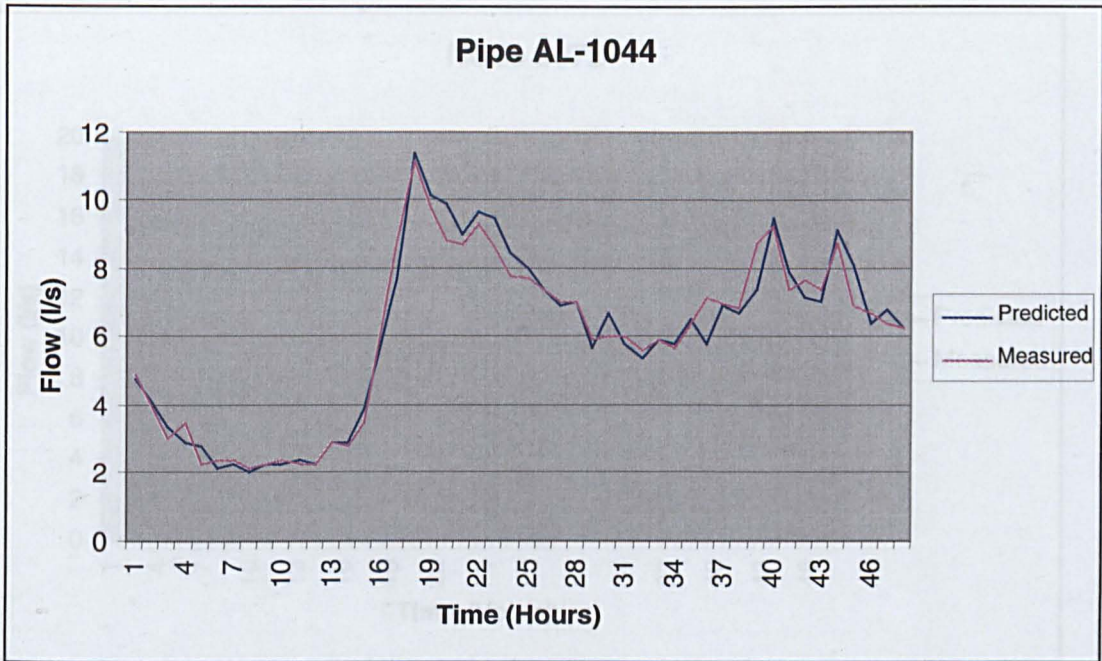


Figure 5.15 flow calibration in Pipe AL-1044

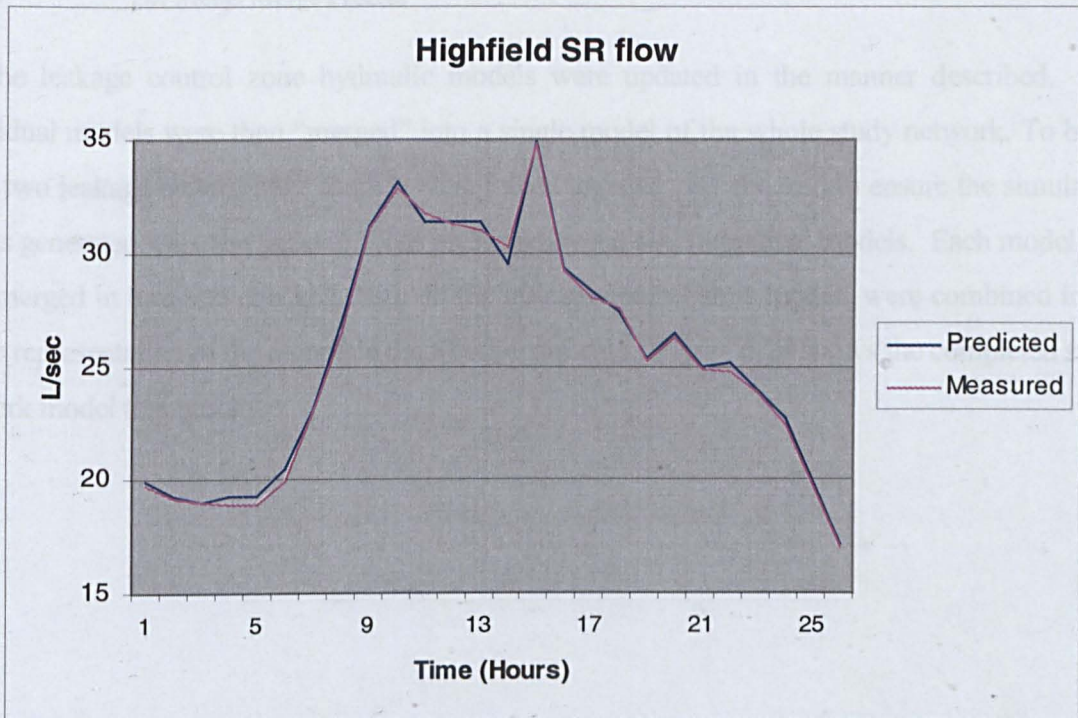


Figure 5.16 flow calibration at Highfield Service Reservoir

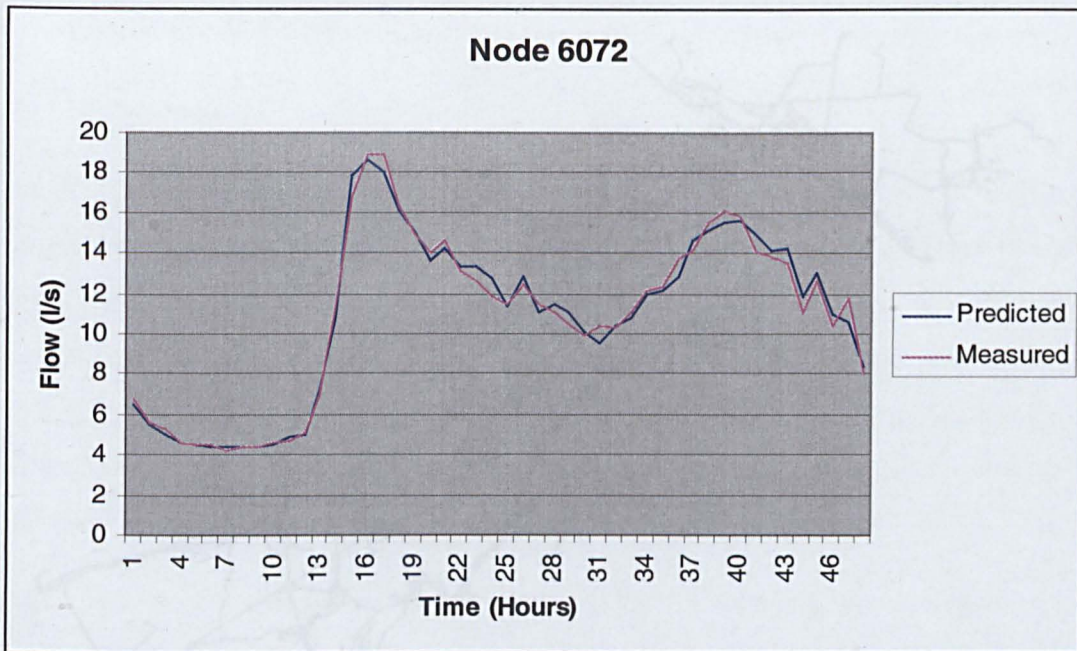


Figure 5.17 flow calibration at Node 6072

5.1.4.3 The Model Merge Process

All the leakage control zone hydraulic models were updated in the manner described. The individual models were then “merged” into a single model of the whole study network. To begin with, two leakage control zone models were joined together and checked to ensure the simulation results generated were the same to those predicted by the two individual models. Each model was then merged in turn and checked, until all the leakage control zone models were combined into a single representation of the complete distribution network. Figure 5.18 shows the completed study network model topography.

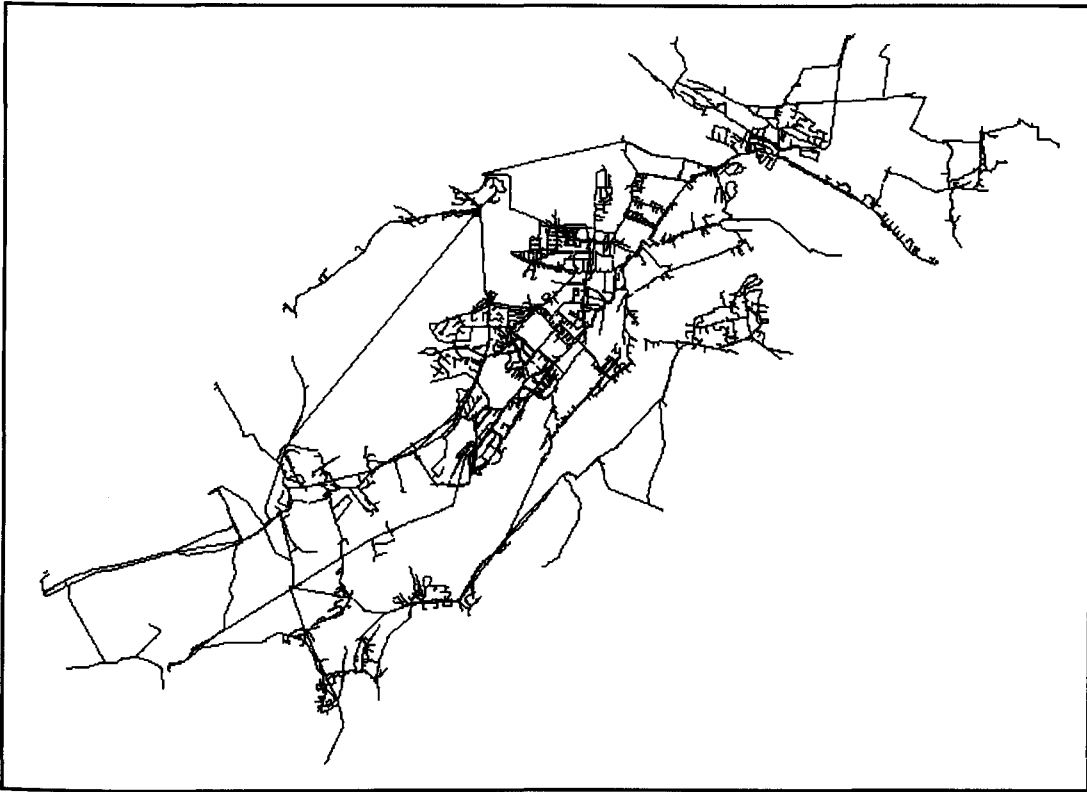


Figure 5.18 The study distribution network model

The merged network model was then used to undertake hydraulic analysis of the study distribution network by a traditional approach whereby local hydraulic problems are resolved on an individual basis and without regard to water quality or surge analysis.

It was also used as the basis for the new integrated approach promoted by this thesis. In the new approach the hydraulic problems are considered in an holistic and integrated way as that included simultaneous hydraulic, transient and water quality modelling.

The manner in which the hydraulic component of the traditional model was applied is now described.

5.2 Hydraulic Analysis

5.2.1 Background

Hydraulic analysis of the study network was undertaken to identify standards of service failures relating to flow and pressure. Areas of low pressure and areas of unnecessarily high pressure were identified, and solutions to the problems were designed using a traditional modelling approach. The traditional approach consisted of hydraulic analysis that identified areas suffering standards of service failure with respect to flow and / or pressure. Then, to reflect current practice, localised schemes were designed to correct or reduce the effect of the problems within a leakage control zone, one at a time, without consideration being given to the network as a whole.

However, all significant current network constraints, for example, service reservoir storage, pumps, and pressure reducing valves remained in place, and new schemes had to retain these assets wherever possible. The configuration of the assets could be changed however, for example, the set point on a pressure-reducing valve could be manipulated if required. Connections to other mains in close proximity, installation of new pressure reducing valves and partial re-zoning by changing the location of zone boundary sluice valves were also considered where appropriate.

Ordinarily, no modelling of leakage, surge or water quality was undertaken as the traditional approach was based on ensuring quantity and continuity of supply rather than quality. However, in this case, leakage *was* modelled to demonstrate the importance of understanding the relationship between flow / pressure management and leakage performance. The reference point for the leakage analysis was the official leakage figures reported for the zones determined by direct flow measurement. The reduction in leakage brought about by the traditional approach to the pressure problems was determined.

The analysis and solution design was then repeated using the new integrated modelling approach. The integrated approach was applied differently to the traditional in that, where required, all current network constraints were removed other than service reservoirs, the majority of the pipe network itself, and the available supply from the water treatment plants. Parameters considered included:

Pipe C value

Pipe diameter

Installation of new mains (minimal)

Removal of and installation of pressure reduction valves

Pressure reduction valve settings

Installation, or change of position, of sluice and boundary valves

Installation of new pumps

Objectives also included the addition of the minimal amount of new mains, the use of the minimum number of control assets, and to have the lowest possible amount of pumping to minimise scheme cost and overall network complexity.

5.2.2 Hydraulic Analysis – Traditional Method

5.2.2.1 Low Pressure

Under normal operating conditions, areas of distribution networks may suffer from low-pressure problems because of their elevation, or a combination of poor mains condition and sudden rises in demand resulting in high friction losses. Areas where the pressure falls below 17 mwc at any time during a 24-hour period are deemed to be failing the minimum standards of service defined by the industry regulators.

Areas “at risk” of suffering standards of service failure related to pressure are those locations where the pressure falls in a band between 17 and 23 mwc. These areas may suffer failures with the required standards, for example, if new customer developments are added to the network or when demand is higher than normal creating higher flows and head losses within the network.

5.2.2.1.1 Identification of Low Pressure Areas

The hydraulic model was used to undertake a 24-hour quasi-dynamic simulation to predict the hydraulic characteristics of the study distribution network under the current operational regime and constraints.

The modelling software was used to automatically generate a list of all nodes that fall below a user-defined minimum pressure at any time during the period of the simulation. This functionality was used to produce a list of failing nodes at the time of peak flow conditions.

The nodes on the list were then investigated and, where possible, solutions to the low-pressure problems were devised using a traditional modelling approach.

5.2.2.1.2 Low Pressure: Results and solutions

The Individual low-pressure pipes / areas were identified and a solution for each was proposed. The solutions were designed using a traditional modelling approach and engineering judgement. Figure 5.19 highlights the location of each low-pressure pipe.

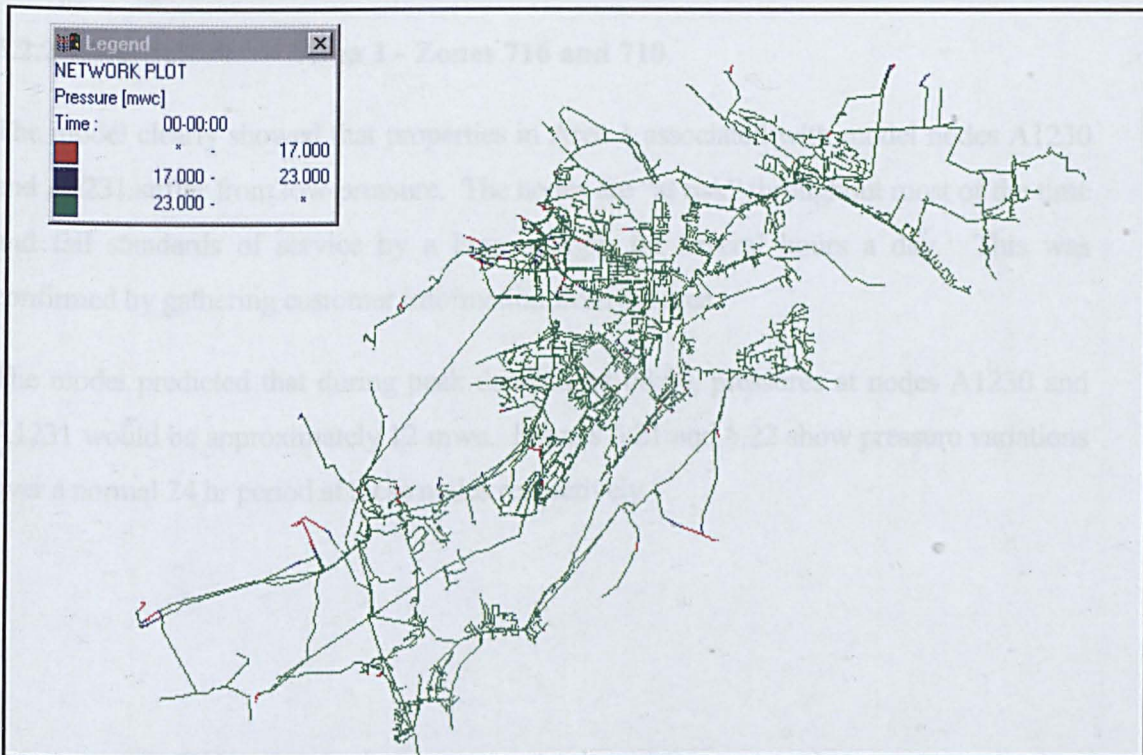


Figure 5.19 Low-pressure areas within the network

The plot highlighted a number of areas within the network that were below the standards of service levels all or some of the time throughout a normal 24 hour period. Each area is dealt with in turn by identifying the nodes where pressure is lowest, determining the magnitude of the problem and then designing a solution for each case. Figure 5.20 shows the areas where pressure schemes were undertaken.

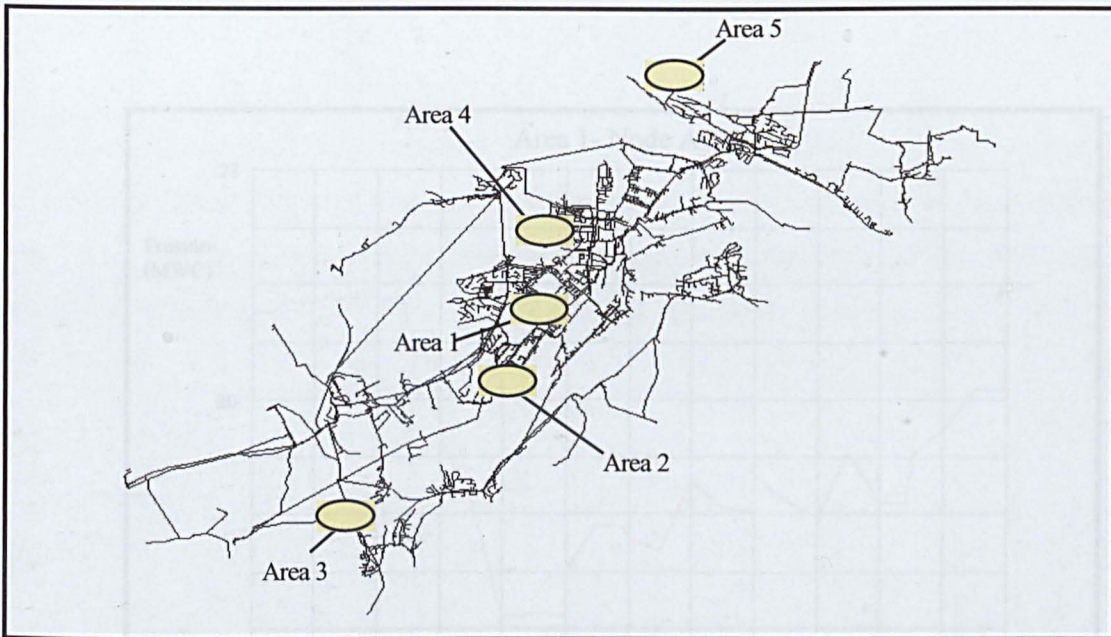


Figure 5.20 Areas where pressure schemes were undertaken

5.2.2.1.2.1 Area 1 - Zones 716 and 710

The model clearly showed that properties in Area 1 associated with model nodes A1230 and A1231 suffer from low-pressure. The nodes are “at risk” throughout most of the time and fail standards of service by a large margin for several hours a day. This was confirmed by gathering customer information from the area.

The model predicted that during peak demand, at 08:00, pressures at nodes A1230 and A1231 would be approximately 12 mwc. Figures 5.21 and 5.22 show pressure variations over a normal 24 hr period at these nodes respectively.

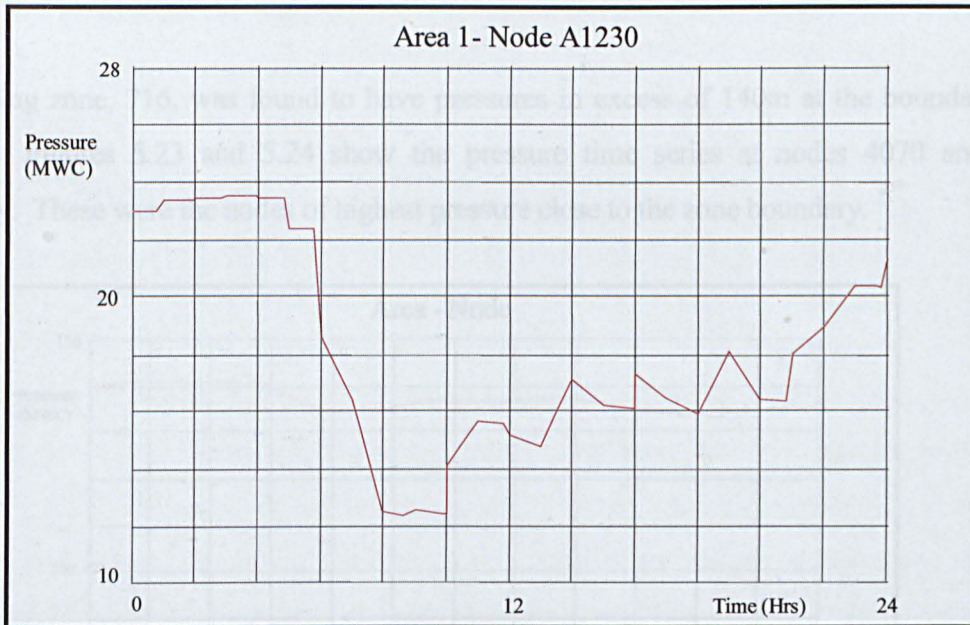


Figure 5.21 Pressure profile Area 1 node A1230

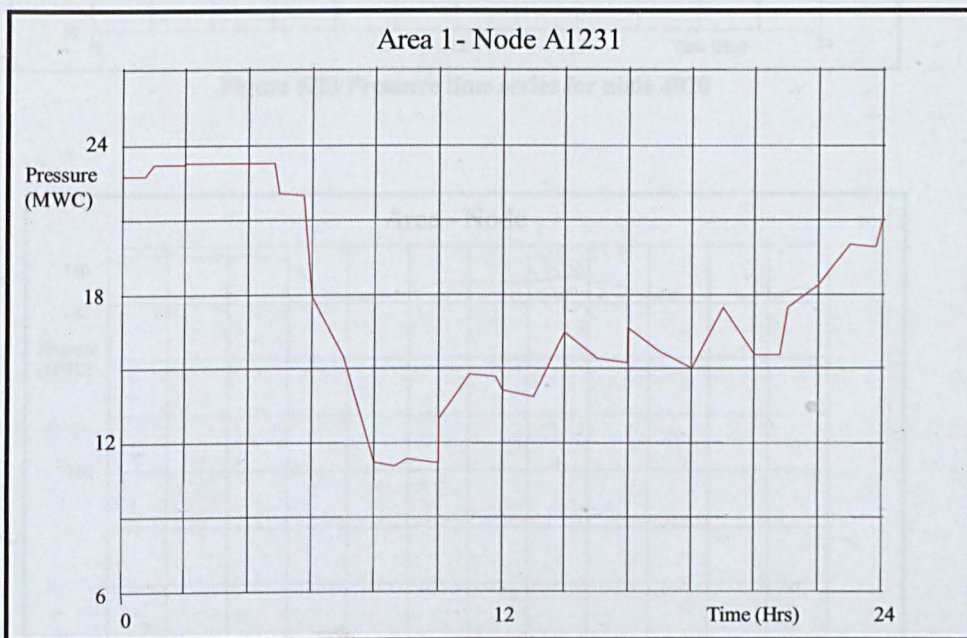


Figure 5.22 Pressure profile Area 1 node A1231

This problem will be exacerbated further at times of abnormally high or peak annual consumption as head loss will increase with increased flow.

Solution

The adjoining zone, 716, was found to have pressures in excess of 140m at the boundary with zone 710. Figures 5.23 and 5.24 show the pressure time series at nodes 4070 and 4090 respectively. These were the nodes of highest pressure close to the zone boundary.

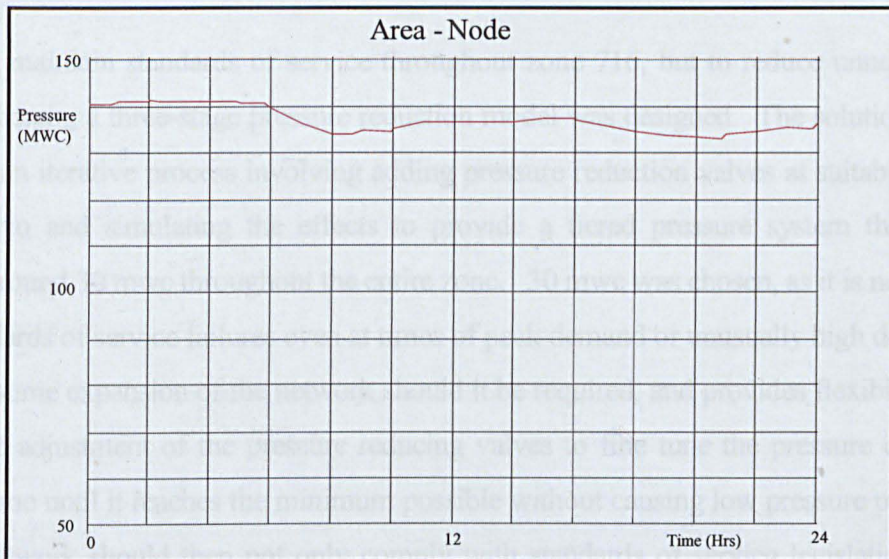


Figure 5.23 Pressure time series for node 4070

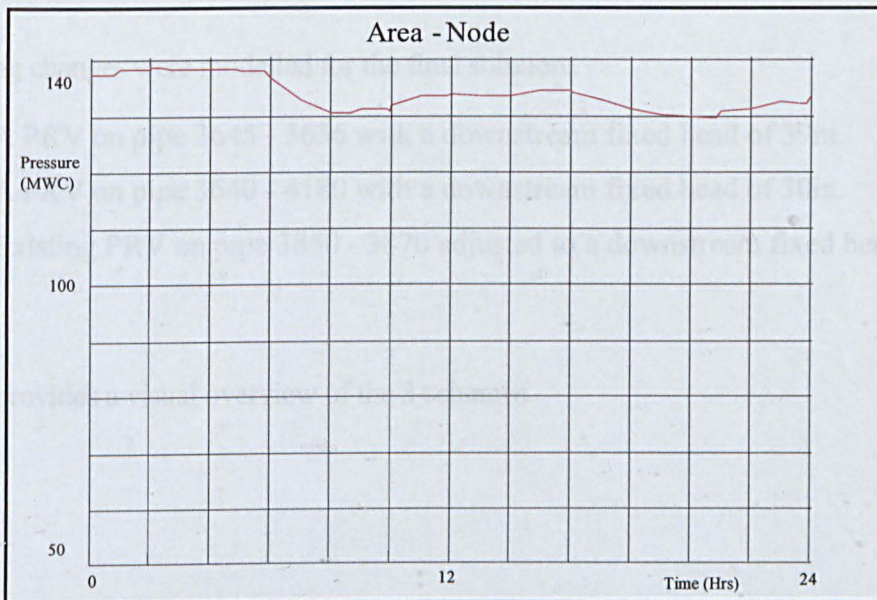


Figure 5.24 Pressure time series for node 4090

Because the pipes in zone 710 suffering low pressure were immediately adjacent to a zone with very high pressure, 716, it was logical to move the failing nodes into the high-pressure zone. The zone was then analysed to determine how best to manage the *high* pressures.

The problem was resolved by moving the zone boundary valves in order that the properties associated with nodes A1230 and A1231 in zone 710 were transferred into zone 716.

In order to maintain standards of service throughout zone 716, but to reduce unnecessarily high pressures therein, a three-stage pressure reduction model was designed. The solution was arrived at through an iterative process involving adding pressure reduction valves at suitable locations in the model to and simulating the effects to provide a tiered pressure system that maintained pressures around 30 mwc throughout the entire zone. 30 mwc was chosen, as it is not so low as to cause standards of service failures even at times of peak demand or unusually high demand. It also allows for some expansion of the network should it be required, and provides flexibility of control through the adjustment of the pressure reducing valves to fine tune the pressure control over a period of time until it reaches the minimum possible without causing low pressure problems. The resultant network should then not only comply with standards of service legislation but should have a reduced propensity for burst mains because of the lower overall pressure across the entire zone.

The following changes were modelled for the final solution:

A PRV on pipe 3645 - 3636 with a downstream fixed head of 39m.

A PRV on pipe 3640 - 4180 with a downstream fixed head of 30m.

Existing PRV on pipe 3860 - 3870 adjusted to a downstream fixed head of 30m.

Figure 5.25 provides a visual overview of the 3 schemes.

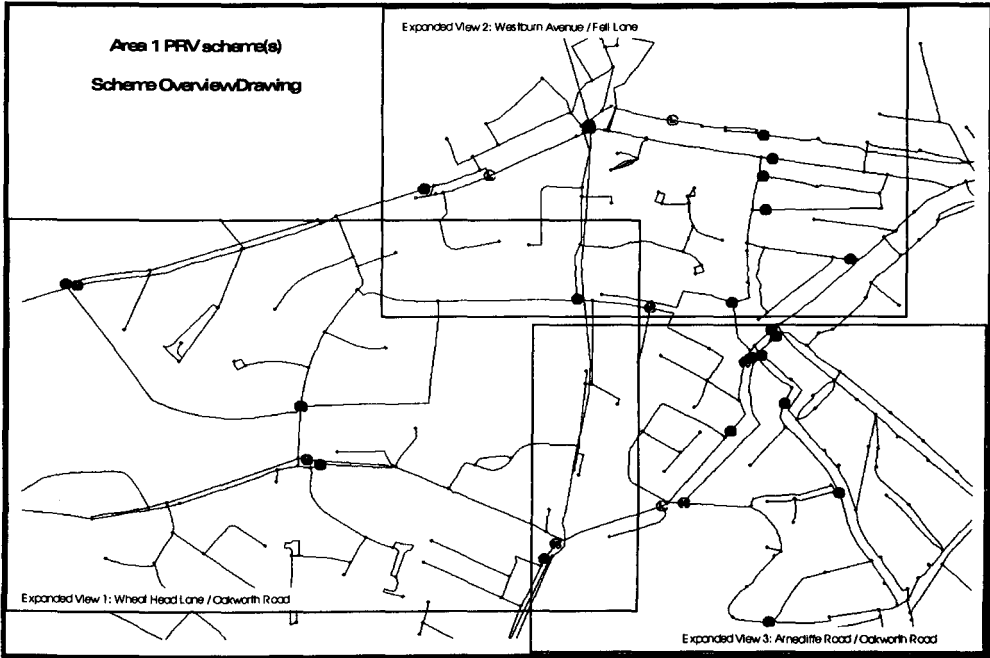


Figure 5.25 provides an overview of the whole scheme

Figures 5.26 to 5.28 highlight the detail of each component part scheme.

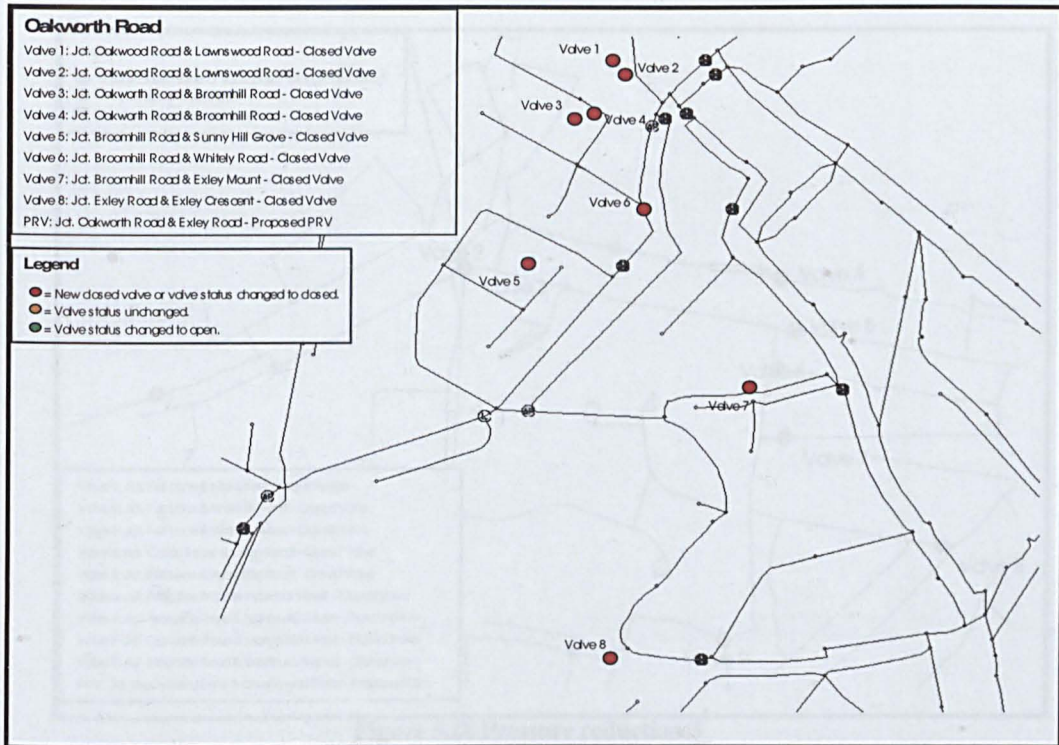


Figure 5.26 Pressure reduction 1

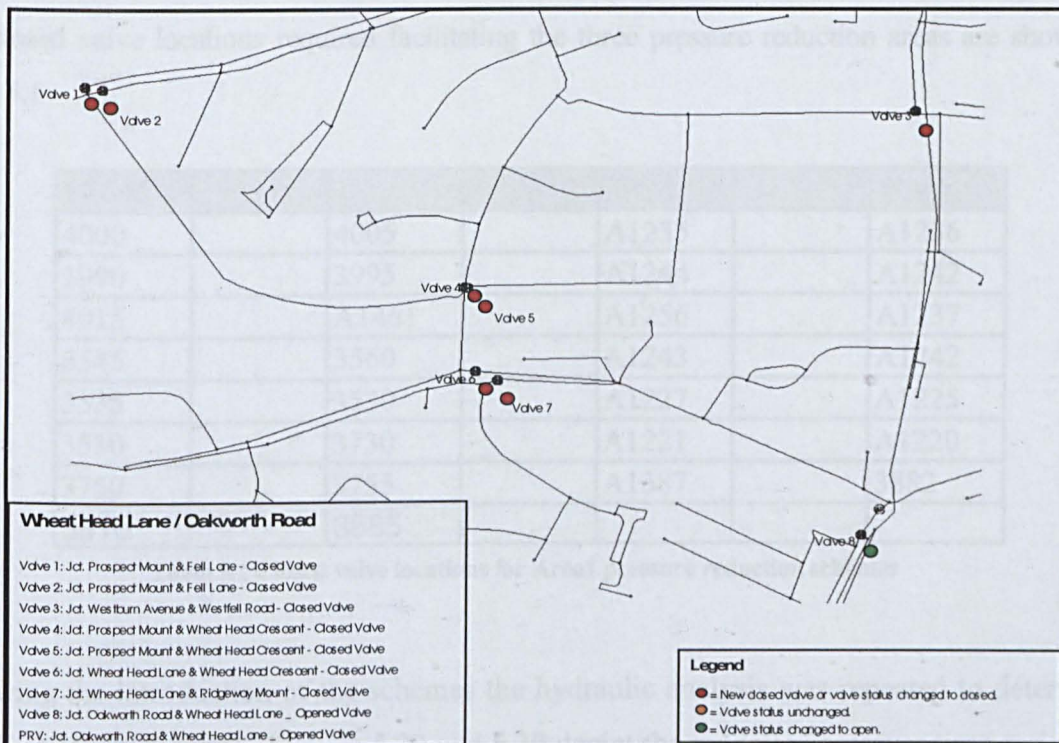


Figure 5.27 Pressure reduction 2

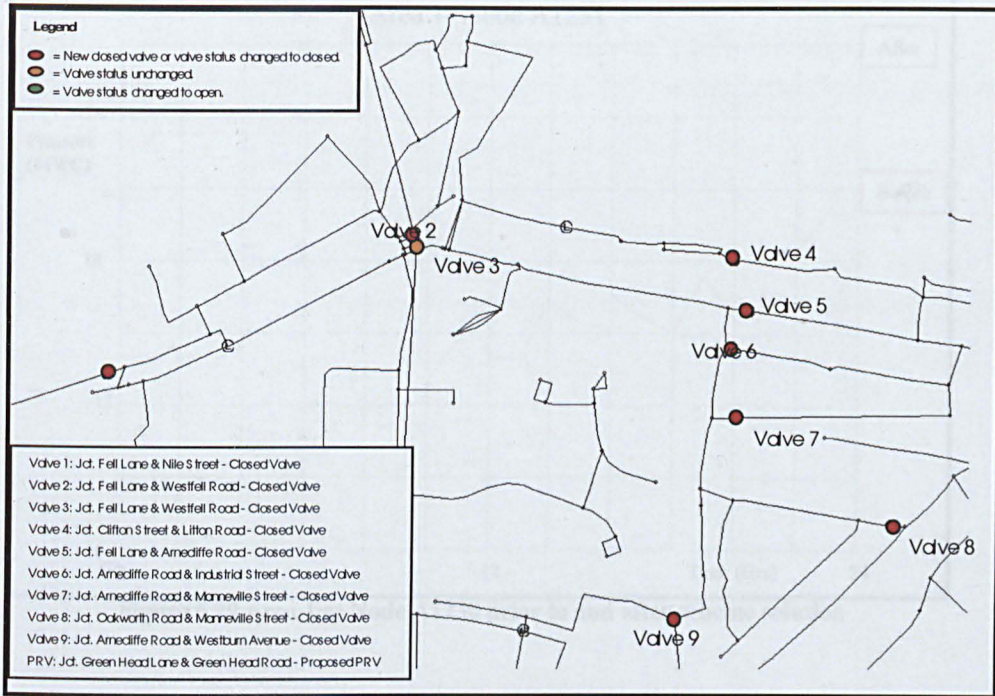


Figure 5.28 Pressure reduction 3

The closed valve locations required facilitating the three pressure reduction areas are shown in Table 5.1

FROM		TO		FROM		TO
4000		4005		A1235		A1236
3990		3995		A1244		A1242
4015		A1461		A1256		A1237
3545		3560		A1243		A1242
3535		3530		A1227		A1225
3530		3730		A1221		A1220
3750		3755		A1387		3882
3810		3885				

Table 5.1 Closed valve locations for Area1 pressure reduction schemes

Following the introduction of the schemes the hydraulic analysis was repeated to determine the effect of the changes. Figures 5.29 and 5.30 depict the modelled pressure time series for the failing nodes before and after implementation of the solution.

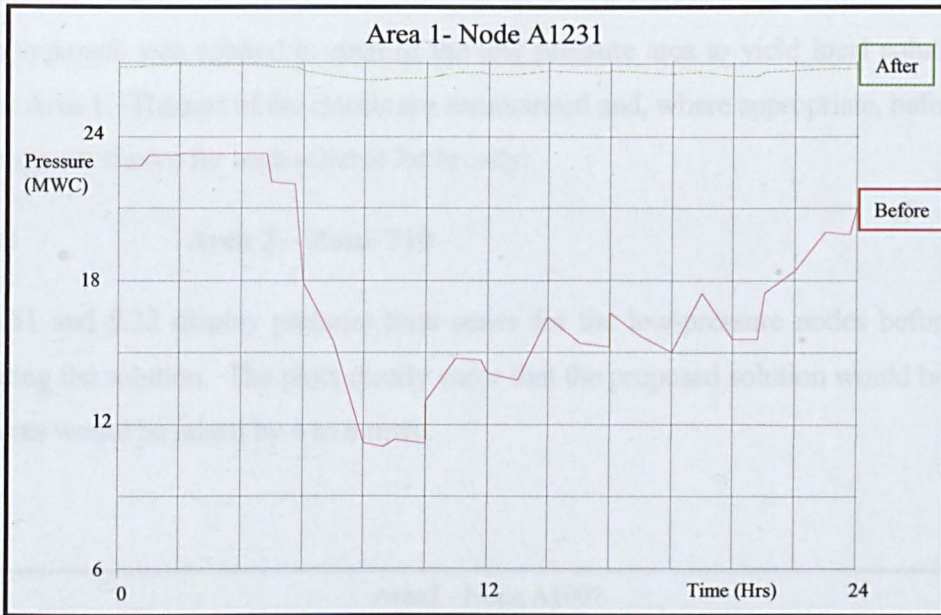


Figure 5.29 Area 1 at Node A1230 prior to and after scheme solution



Figure 5.30 Area 1 at Node A1231 prior to and after scheme solution

The plots clearly demonstrate that the proposed solution would be successful and that the resultant pressure profile would be much more stable than was previously the case.

The same approach was applied to each of the low pressure area to yield local solutions as that derived for Area 1. The rest of the results are summarised and, where appropriate, before and after pressure plots are shown for each scheme for brevity.

5.2.2.1.2.2 Area 2 – Zone 710

Figures 5.31 and 5.32 display pressure time series for the low-pressure nodes before and after implementing the solution. The plots clearly show that the proposed solution would be successful and pressures would be raised by 4 to 6 mwc.

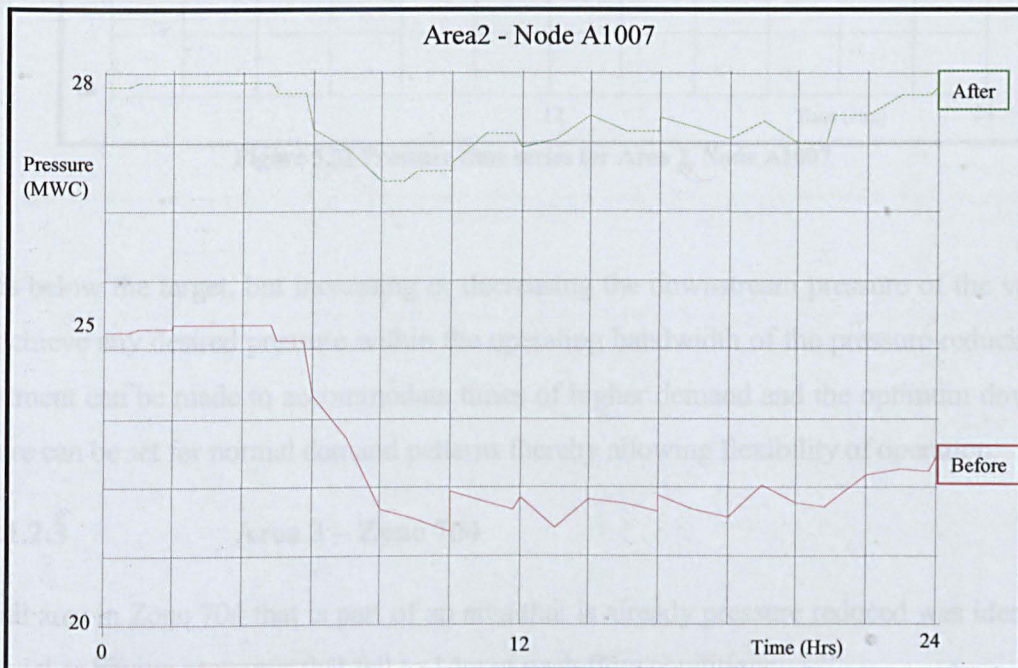


Figure 5.31 Pressure time series for Area 2, Node A1049

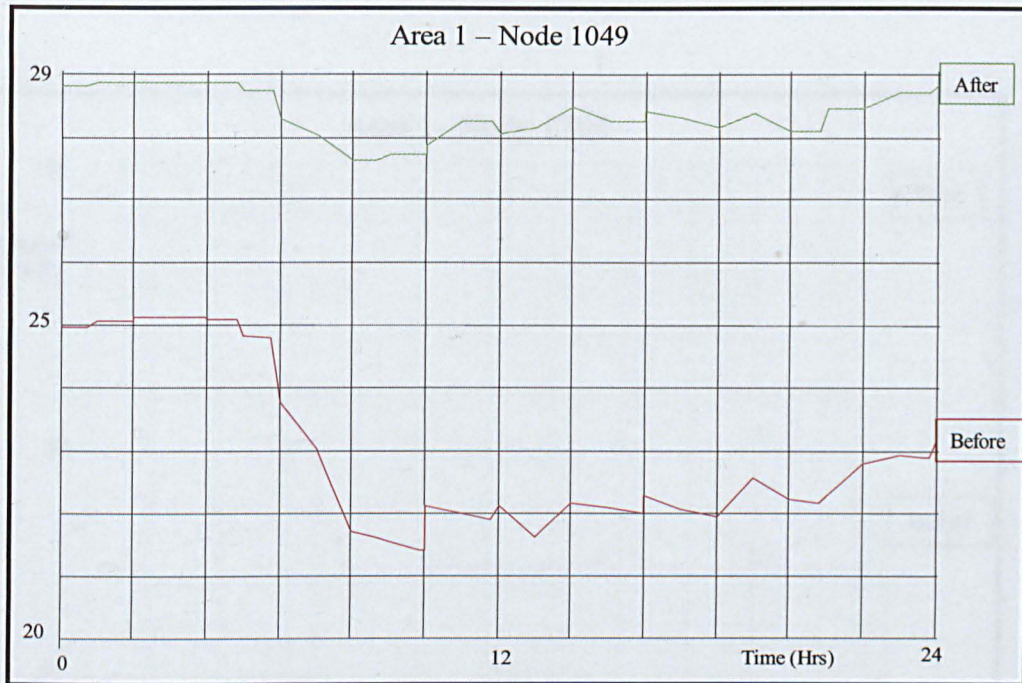


Figure 5.32 Pressure time series for Area 2, Node A1007

This is below the target, but increasing or decreasing the downstream pressure of the valve may now achieve any desired pressure within the operating bandwidth of the pressure-reducing valve. Adjustment can be made to accommodate times of higher demand and the optimum downstream pressure can be set for normal demand patterns thereby allowing flexibility of operation.

5.2.2.1.2.3 Area 3 – Zone 704

A small area in Zone 704 that is part of an area that is already pressure reduced was identified by the model as having pressures that fell to 14m at peak flow conditions.

Figures 5.33 and 5.34 show the pressure time series plots both before and after implementation of the proposed solution. It can clearly be seen that the solution lifts the pressure significantly.

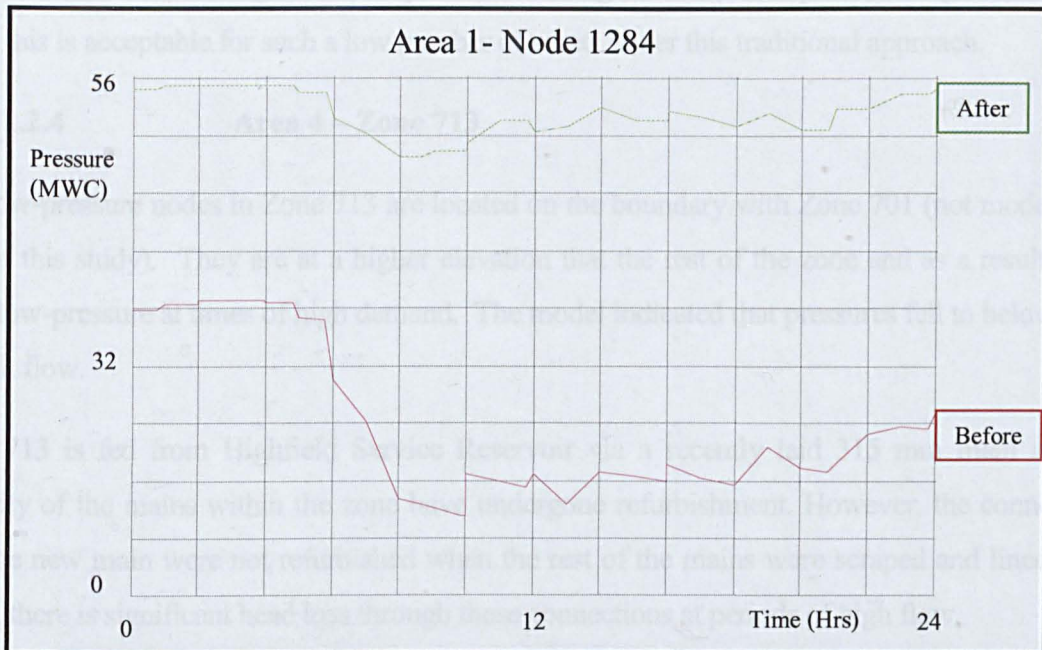


Figure 5.33 Pressure time series for Node 1284

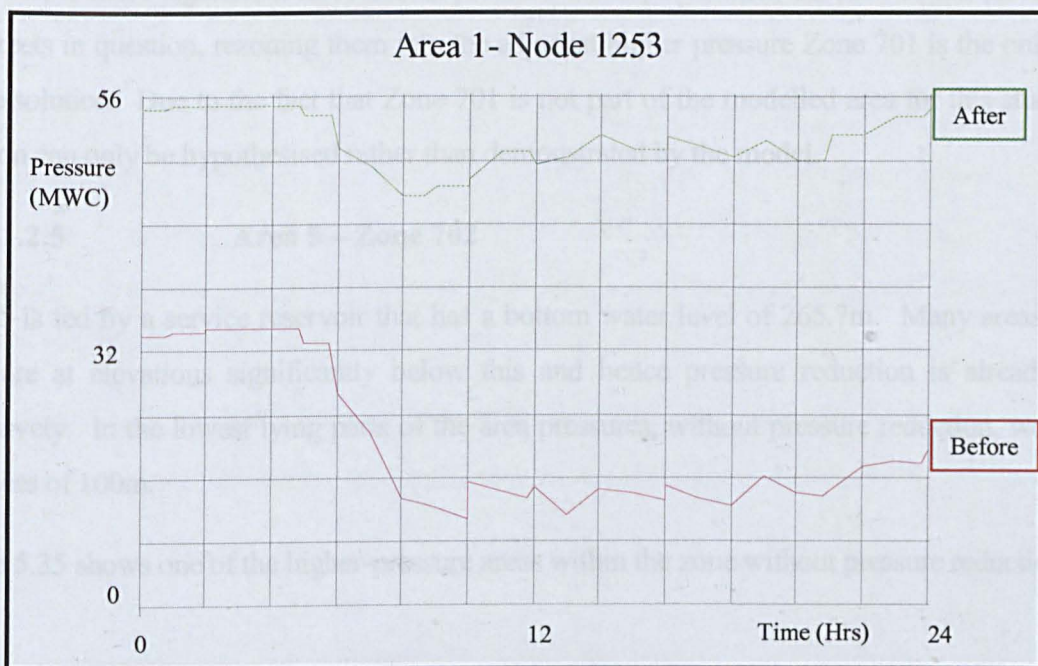


Figure 5.34 Pressure time series for Node 1253

It is clear from the plots that the solution removes the low-pressure problem although the resultant profile is a little on the high side of optimal. Although the resultant pressure is greater than 50 mwc, this is acceptable for such a low number of assets under this traditional approach.

5.2.2.1.2.4 Area 4 – Zone 713

The low-pressure nodes in Zone 713 are located on the boundary with Zone 701 (not modelled as part of this study). They are at a higher elevation than the rest of the zone and as a result suffer from low-pressure at times of high demand. The model indicated that pressures fell to below 20 m at peak flow.

Zone 713 is fed from Highfield Service Reservoir via a recently laid 315 mm main and the majority of the mains within the zone have undergone refurbishment. However, the connections into the new main were not refurbished when the rest of the mains were scraped and lined. As a result, there is significant head loss through these connections at periods of high flow.

Refurbishing these poor condition connections would undoubtedly improve the situation. However, if more significant increases in pressure are required then, because of the elevation of the streets in question, rezoning them into the adjacent higher pressure Zone 701 is the only other viable solution. Due to the fact that Zone 701 is not part of the modelled area for this study, this solution can only be hypothesised rather than demonstrated by the model.

5.2.2.1.2.5 Area 5 – Zone 702

Area 5 is fed by a service reservoir that has a bottom water level of 265.7m. Many areas of the zone are at elevations significantly below this and hence pressure reduction is already used extensively. In the lowest lying parts of the area pressures, without pressure reduction, would be in excess of 100m.

Figure 5.35 shows one of the higher-pressure areas within the zone without pressure reduction.

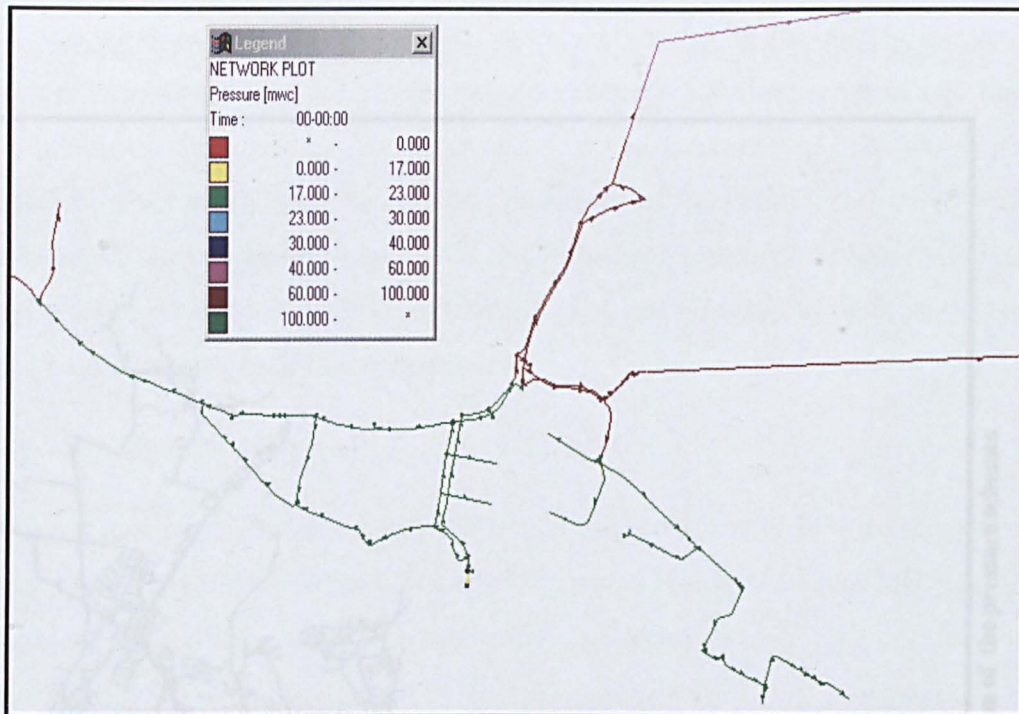


Figure 5.35 The higher pressure area within Area 4 without pressure reduction

With pressure reduction, these pressures are reduced to approximately 50m. However, at the end of the main leading up to node 6038 there is a demand that is at an elevation such that, in when the pressure reducing valve is in operation, the pressure drops to 8m at peak flow at this point. It is not possible to re-adjust the PRV setting to increase the pressure to this highest property, without significantly increasing pressures throughout the pressure reduced parts of the zone thereby losing the benefits of the scheme. In order to resolve this, it was necessary to install a small booster pump to feed the demand as well as the pressure reduction scheme.

Following implementation of all the schemes, the model was used to identify the results. Figure 5.36 shows the remaining areas suffering low pressure highlighted in red, and 17 to 23 mwc in green.

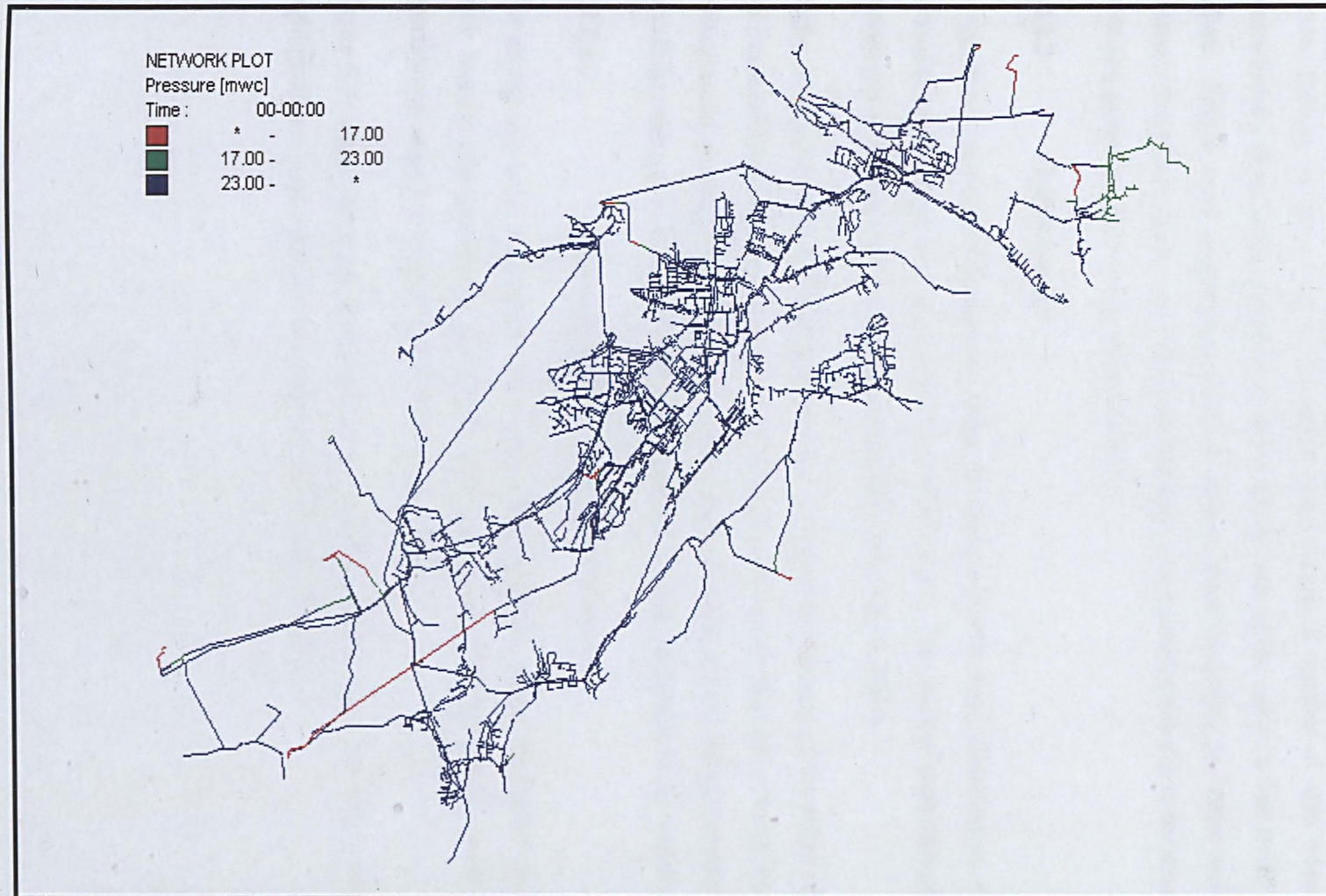


Figure 5.36 Areas with low pressure following implementation of the pressure schemes

Further investigations of these sites show that some are the mains directly beneath service reservoirs or in trunk mains where no customers are connected so there are in fact no standards of service failures on these mains. However, there remain a number of sites where pressure is unsatisfactory all or some of the time and a whole area of the network that could be at risk of failure. Further work would be required to resolve these remaining problems and the resultant schemes could be complex and would not be likely to be cost effective due to the small benefit that would be gained for the capital outlay necessary

5.2.2.2 High Pressure

A significant number of properties within the study network were identified as suffering from pressures over 60 mwc and, in some cases, over 140 mwc. The cause of such high pressures is the topography of the area with ground levels ranging from 90m to 280m.

High mains pressure in a distribution network is undesirable because of the effect on leakage and the increased likelihood of causing bursts. Once a leak occurs, the rate of water loss increases in relation to the pressure in the mains. If water escapes through a burst at high pressure there is high probability damage to the surrounding geographical area and properties in the vicinity.

5.2.2.4.1 Identification of high pressure areas

The model was used to highlight those areas of the network where the highest mains pressures were located and discussions were held with operational staff to identify areas of particular concern with regard to excessive mains pressures.

Figure 5.37 shows the areas of the distribution network broken down into three pressure bands to highlight those areas with pressure in excess of 50 and 100 mwc.

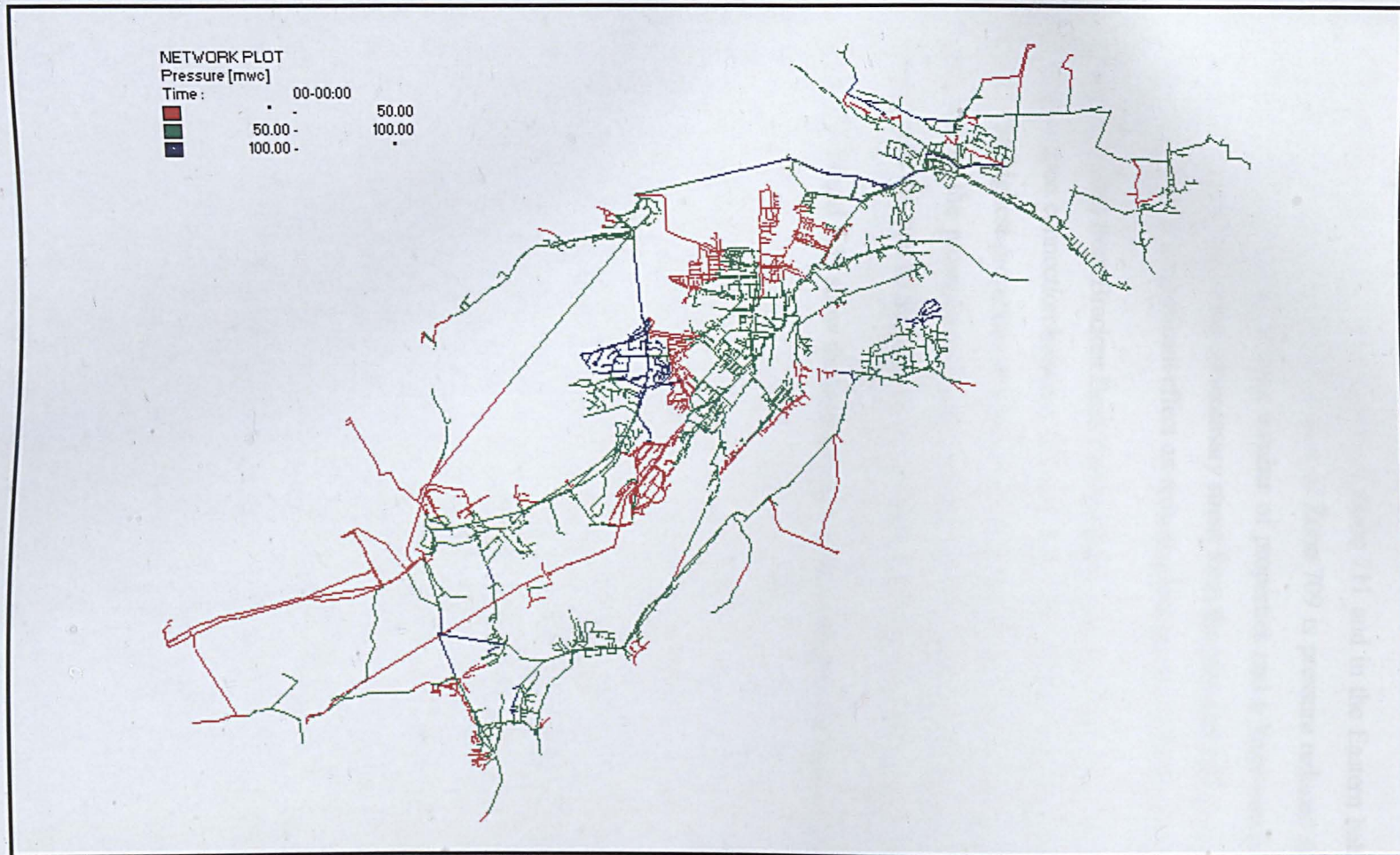


Figure 5.37 High pressure areas within the study distribution network

5.2.2.4.2 High Pressure - Results and solutions

5.2.2.4.2.1 Area 2 - Zones 711 and 709

The mains pressures in the majority of Zone 711 and in the Eastern half of Zone 709 were, on average, greater than 80m (The West of Zone 709 is pressure reduced and fed from Zone 713). These two areas contain a large number of properties and a large number / length of pipes and hence, as well as removing unnecessary stress from the pipes and fittings, lowering pressures within them would have a significant effect on reducing leakage levels in the study network.

Zone 711 is fed from Bracken Bank Service Reservoir via Zone 710. The zone inflow meter is on the only open connection between 711 and 710. The Eastern half of zone 709 is fed from zone 711. The highest properties in Zone 711 have pressures of approximately 50 mwc at peak flow and, due to the presence of blocks of flats at this location, it is necessary to maintain these pressures in this part of the zone to ensure demand is met at the highest property.

Figures 5.38 and 5.39 show the high-pressure time series for two typical nodes.

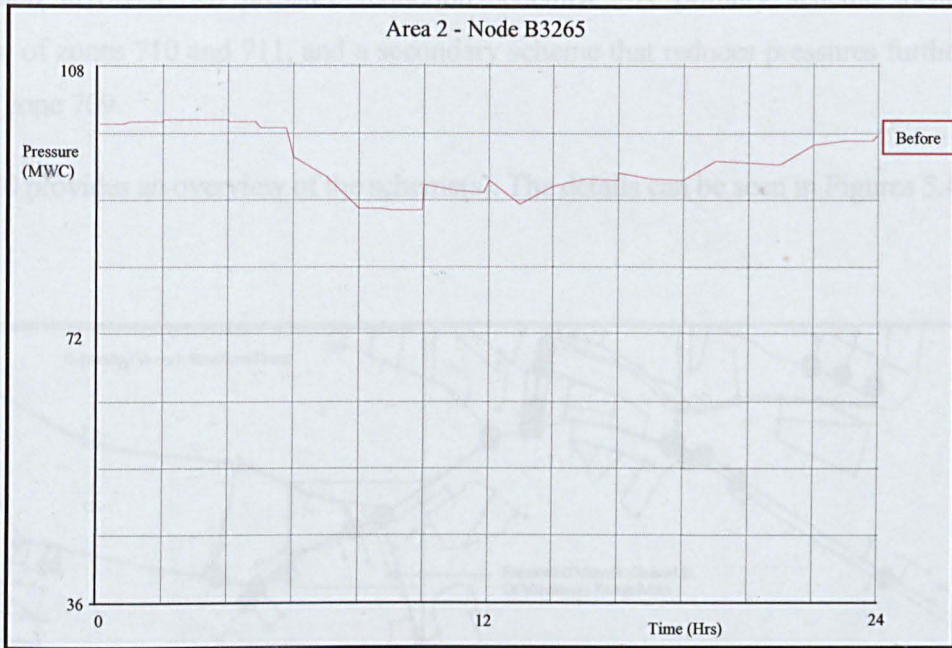


Figure 5.38 High pressure time series for node B3265

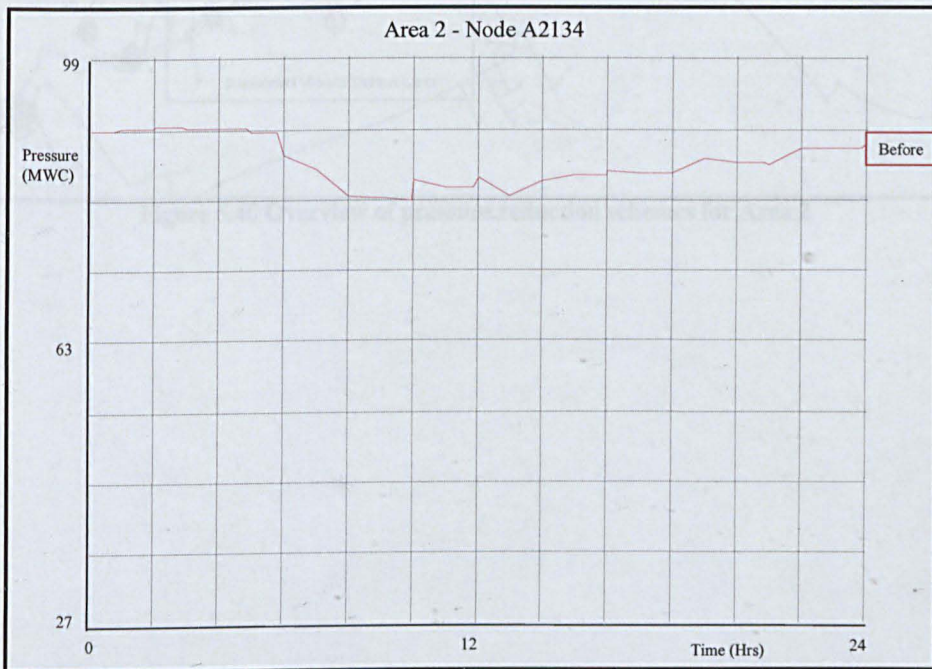


Figure 5.39 High Pressure time series at Node A2134

Solution

The solution involves two pressure reduction schemes. A primary scheme located at the connection of zones 710 and 711, and a secondary scheme that reduces pressures further to a subsection of zone 709.

Figure 5.40 provides an overview of the scheme(s). The details can be seen in Figures 5.41 to 5.43.

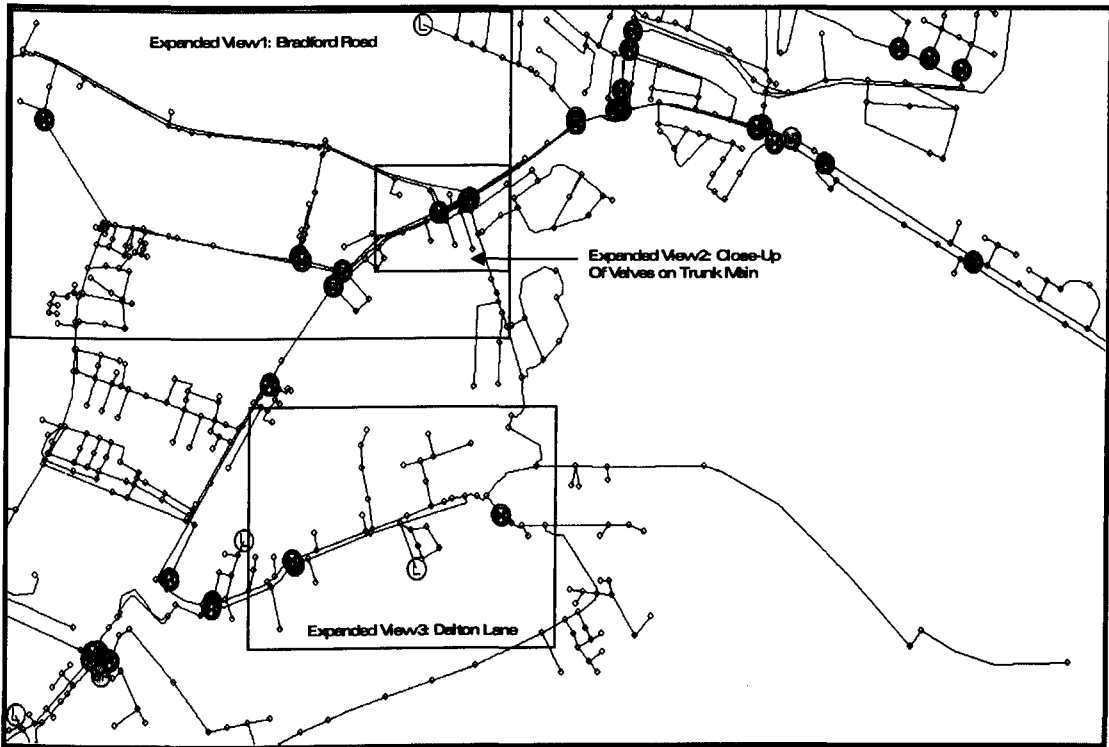


Figure 5.40 Overview of pressure reduction schemes for Area 2

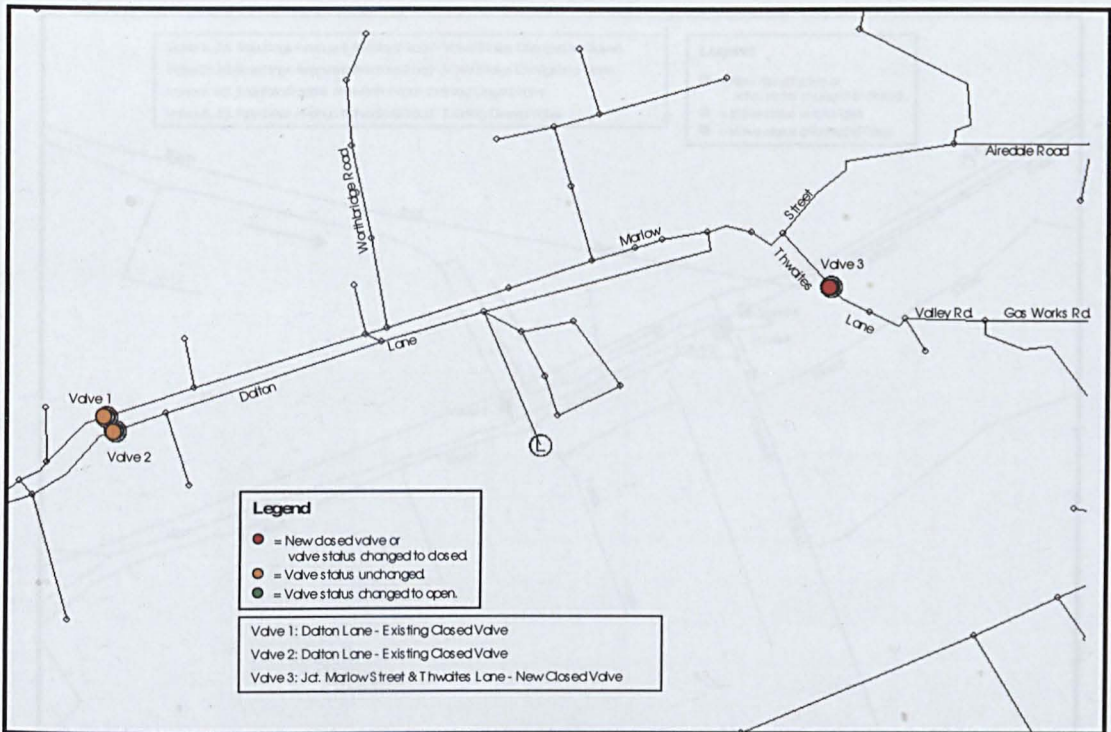


Figure 5.41 Detail 1 Area 2 for pressure reduction schemes

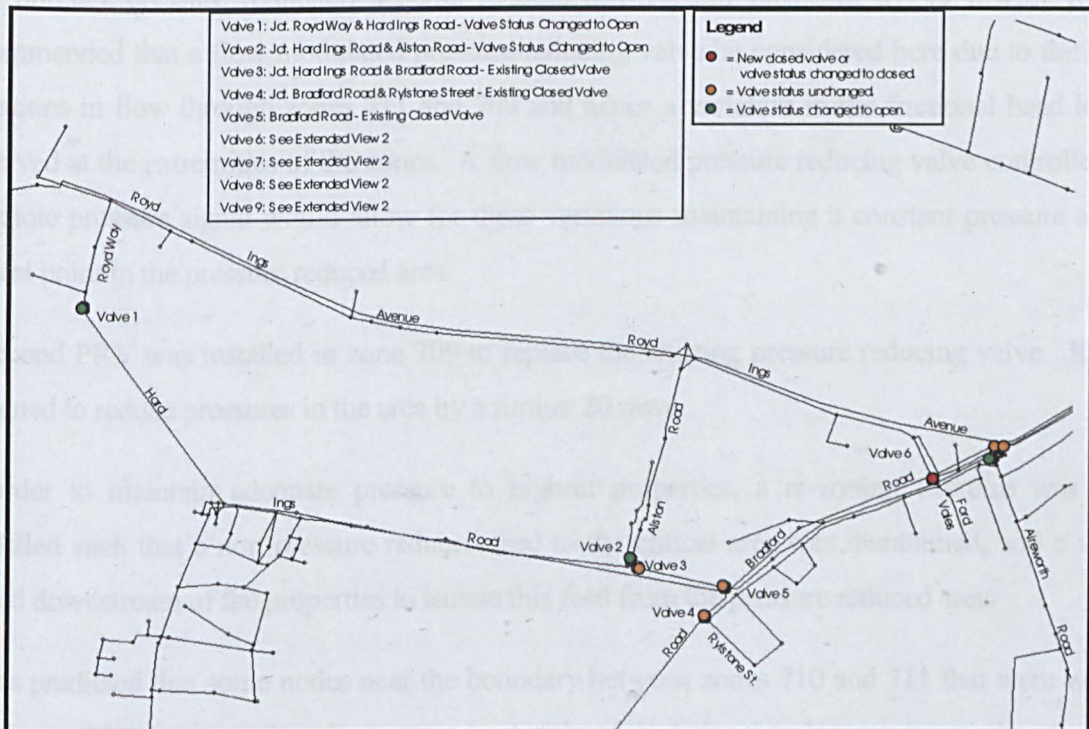


Figure 5.42 Detail 2 Area 2 for pressure reduction schemes

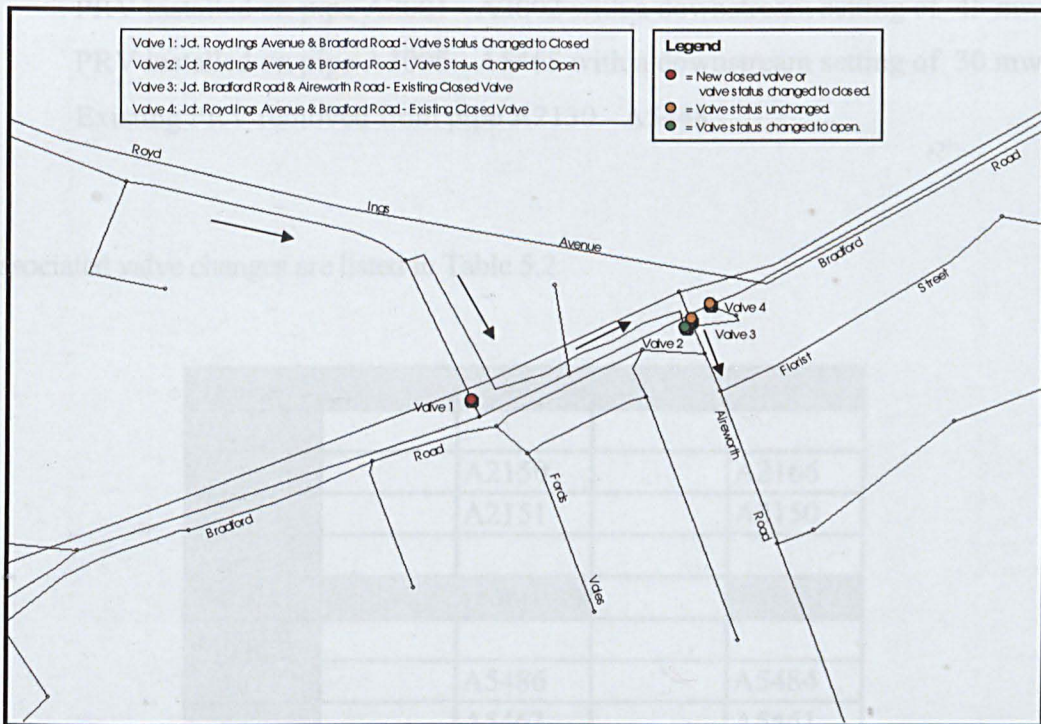


Figure 5.43 Detail 3 for Area 2 pressure reduction scheme

The primary pressure reduction scheme reduces downstream pressures by 45 mwc. It was recommended that a flow modulated pressure-reducing valve be considered here due to the large variations in flow through zones 711 and 709 and hence a variation in the frictional head losses observed at the extremities of the zones. A flow modulated pressure reducing valve controlled by a remote pressure signal would allow for these variations maintaining a constant pressure at the critical point in the pressure reduced area.

A second PRV was installed in zone 709 to replace the existing pressure reducing valve. It was designed to reduce pressures in the area by a further 20 mwc.

In order to maintain adequate pressure to highest properties, a re-zoning exercise was also modelled such that a non pressure reduced feed to the critical area was maintained, and a valve closed downstream of the properties to isolate this feed from the pressure reduced area.

It was predicted that some nodes near the boundary between zones 710 and 711 that were within the proposed pressure reduced area, were at risk of experiencing low-pressures due to their elevation. It was therefore necessary to move the boundary valve locations between 710 and 711 such that these higher elevation nodes were incorporated into zone 710.

The details of the changes made in the model to simulate the changes were:

PRV installed on pipe A2001 - A2002 with a downstream setting of 45 mwc.

PRV installed on pipe A3005 - A5465 with a downstream setting of 30 mwc

Existing PRV removed from pipe A2130 - A5448

The associated valve changes are listed in Table 5.2

Valves		From		To
closed				
		A2150		A2166
		A2151		A2150
Valves		From		To
opened				
		A5486		A5484
		A5463		A5461
Boundary		From		To
valves				
moved		A2033		A2045
		A2055		A2057

Table 5.2 Valve changes for pressure reduction Area 2

The simulation predicted that by routing the water to supply the flats by rezoning them, large head losses of over 10 mwc would occur at peak flows. These head losses are due to a section of main in the new flow route being in poor condition. To prevent these low-pressure problems, it would therefore be necessary to replace or refurbish this main when the proposed pressure reduction of 711 and 709 was implemented. Figures 5.44 and 5.45 show the 24-hour plots of the pressure variations at typical nodes before and after the implementation of the pressure reduction scheme. A reduction in mains pressure of between 45m and 35m is clearly apparent.

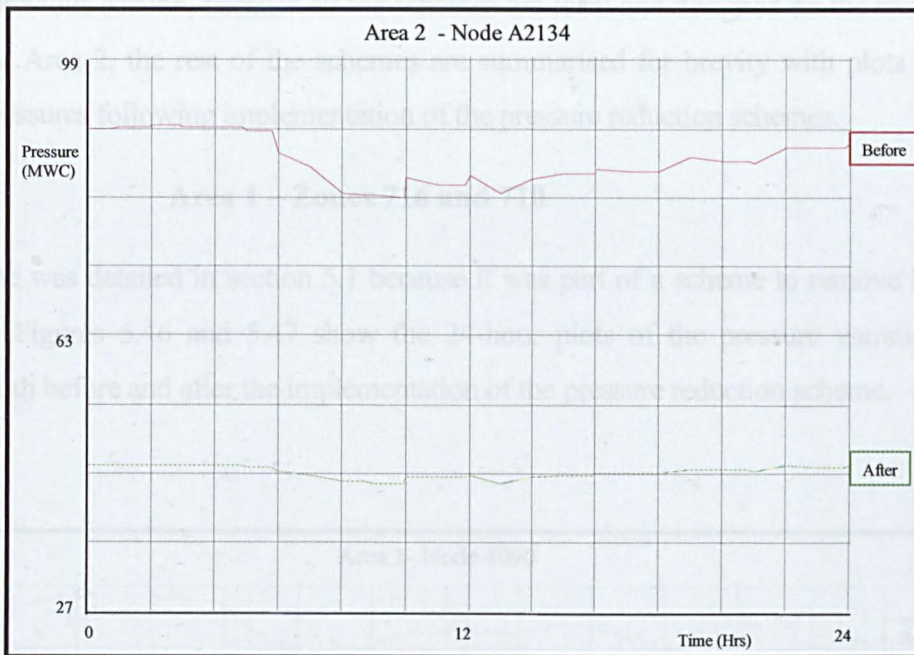


Figure 5.44 Pressure time series Node A2134 before and after pressure reduction.

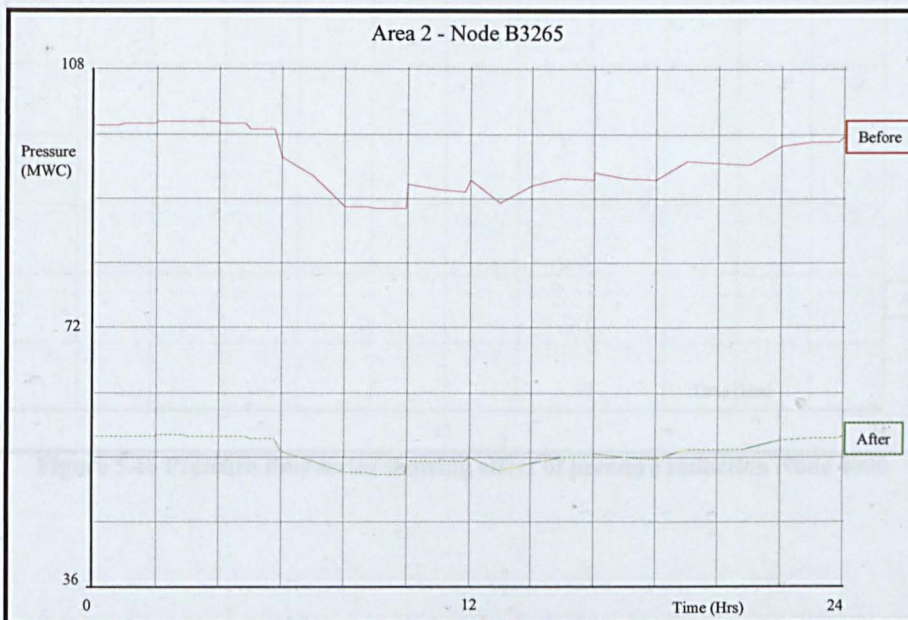


Figure 5.45 Pressure time series Node B3265 before and after pressure reduction.

The average mains pressure in Area 2 was reduced from 84m to 47m.

As in the previous section, because all the schemes are local and designed on the same principals as those for Area 2, the rest of the schemes are summarised for brevity with plots showing the resultant pressures following implementation of the pressure reduction schemes.

5.2.2.4.2.2 Area 1 – Zones 716 and 710

This scheme was detailed in section 5.1 because it was part of a scheme to remove low-pressure problems. Figures 5.46 and 5.47 show the 24-hour plots of the pressure variations at these locations both before and after the implementation of the pressure reduction scheme.

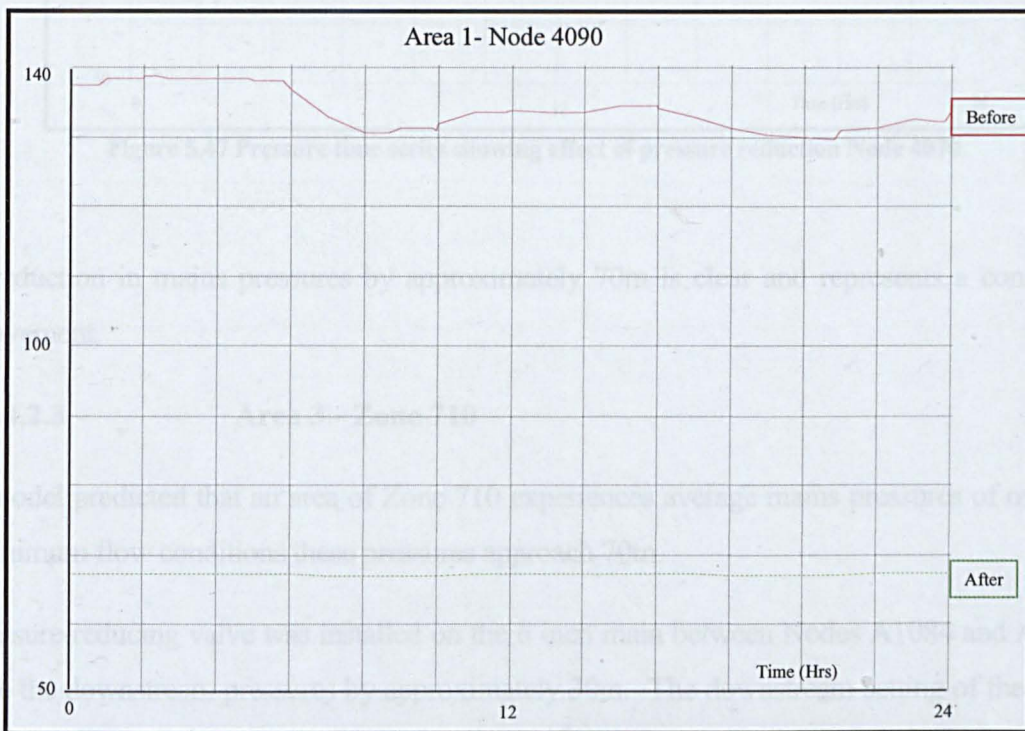


Figure 5.46 Pressure time series showing effect of pressure reduction Node 4090

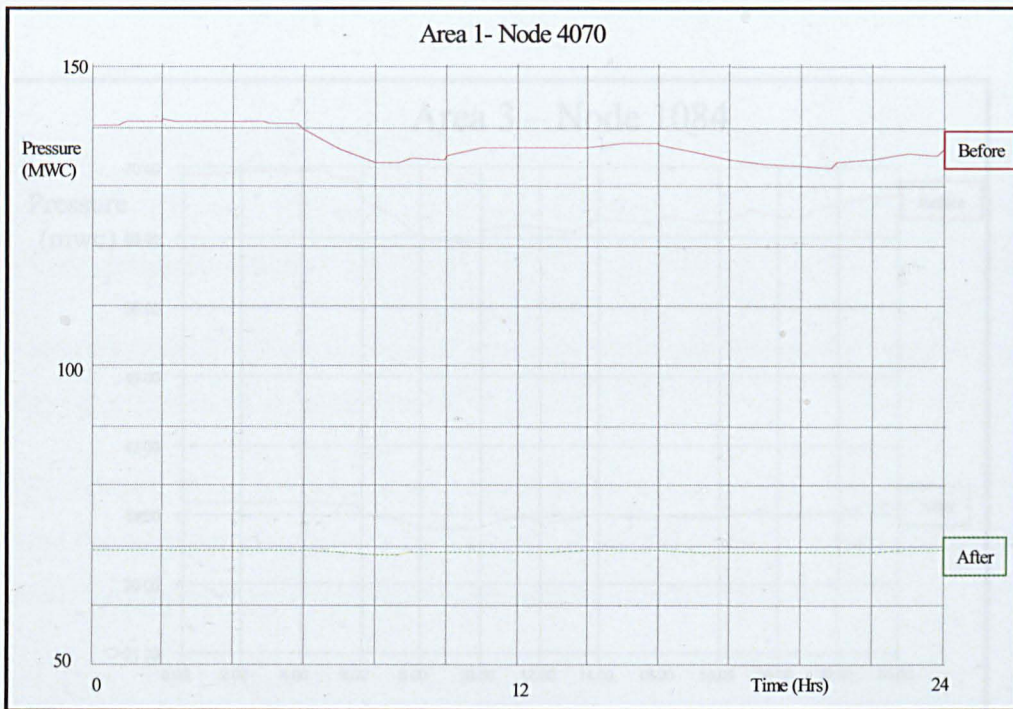


Figure 5.47 Pressure time series showing effect of pressure reduction Node 4070

The reduction in mains pressures by approximately 70m is clear and represents a considerable improvement.

5.2.2.4.2.3 Area 3 - Zone 710

The model predicted that an area of Zone 710 experiences average mains pressures of over 60m. At minimum flow conditions these pressures approach 70m.

A pressure-reducing valve was installed on the 6 inch main between Nodes A1084 and A1080 to reduce the downstream pressures by approximately 30m. The downstream setting of the valve to achieve this was 35 mwc. In addition, a number of valves were closed to isolate the area from the rest of zone 710. Figures 5.48 and 5.49 show the 24-hour plots of the pressure variations at these locations before and after the implementation of the pressure reduction scheme. The reduction in mains pressures of approximately 35m is readily apparent.

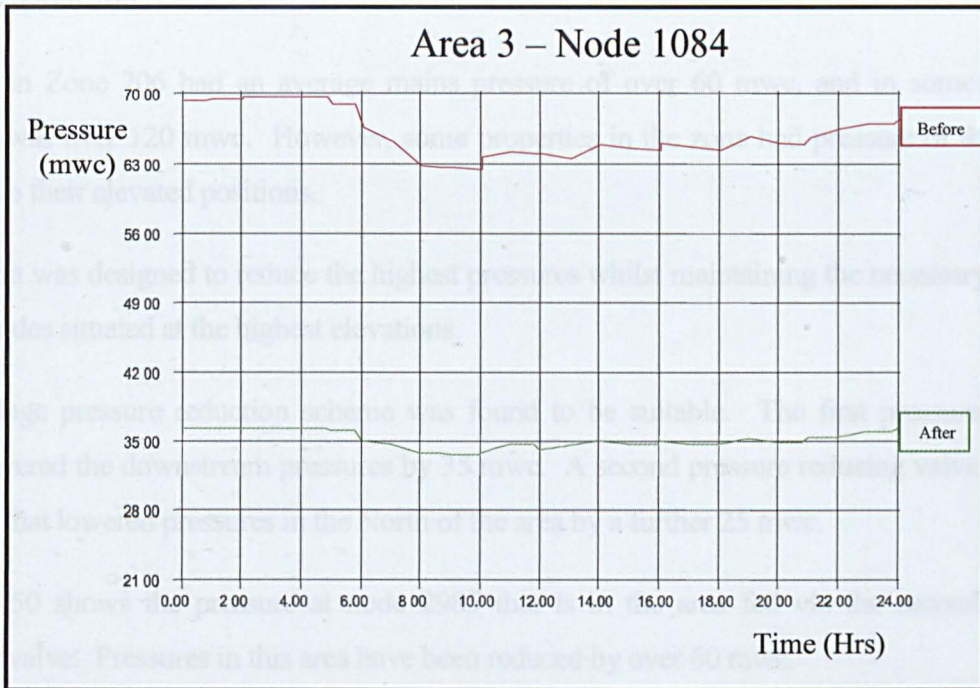


Figure 5.48 Pressure time series showing effect of pressure reduction Node 1084

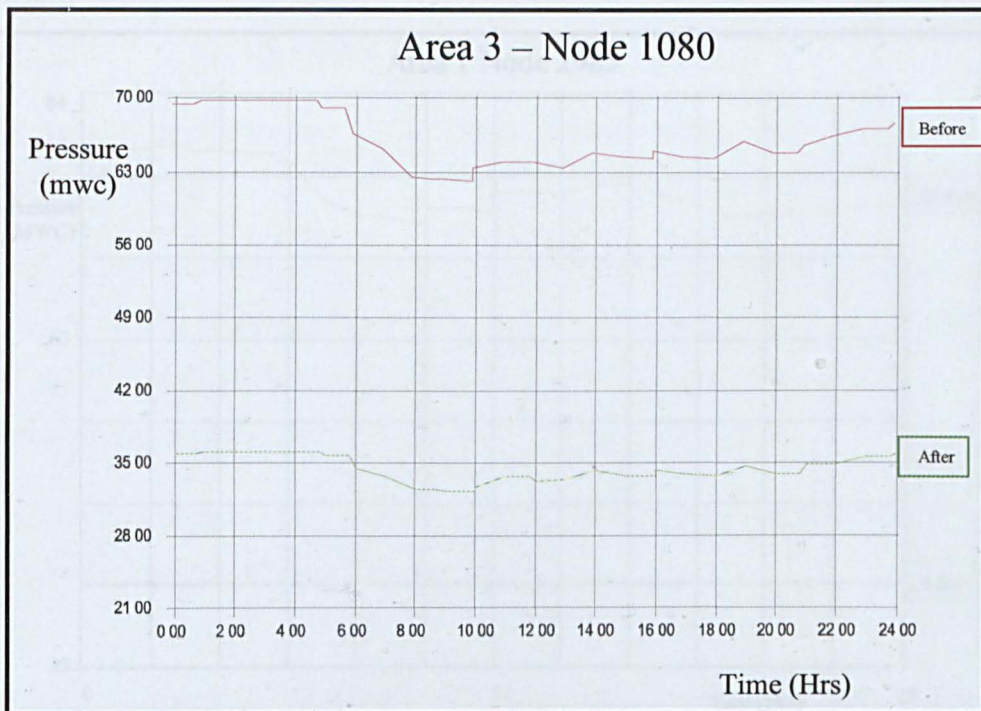


Figure 5.49 Pressure time series showing effect of pressure reduction Node 1080

Area 4 - Zone 706

An area in Zone 706 had an average mains pressure of over 60 mwc, and in some locations pressure was over 120 mwc. However, some properties in the zone had pressure of 40 mwc or less due to their elevated positions.

A solution was designed to reduce the highest pressures whilst maintaining the necessary pressure for the nodes situated at the highest elevations.

A two-stage pressure reduction scheme was found to be suitable. The first pressure-reducing valve lowered the downstream pressures by 35 mwc. A second pressure reducing valve was then installed that lowered pressures in the North of the area by a further 25 mwc.

Figure 5.50 shows the pressure at node 2985 that is in the area fed via the second pressure reducing valve. Pressures in this area have been reduced by over 60 mwc.

Figure 5.51 shows the pressure variations at node 2925 where pressures have been reduced by 35 mwc.

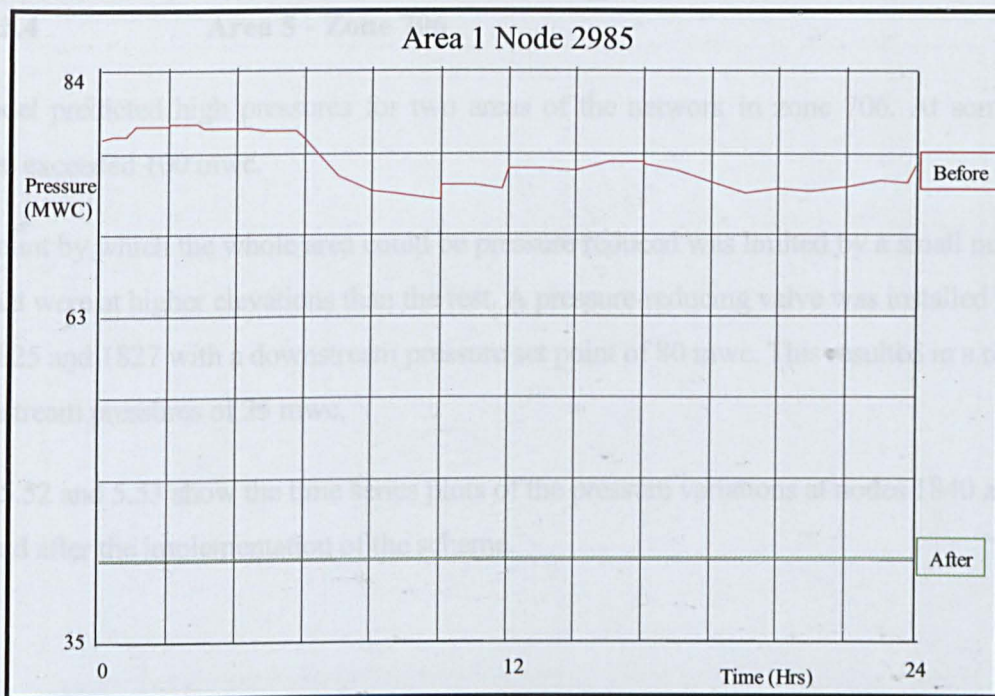


Figure 5.50 Pressure time series for node 2985 before and after pressure reduction

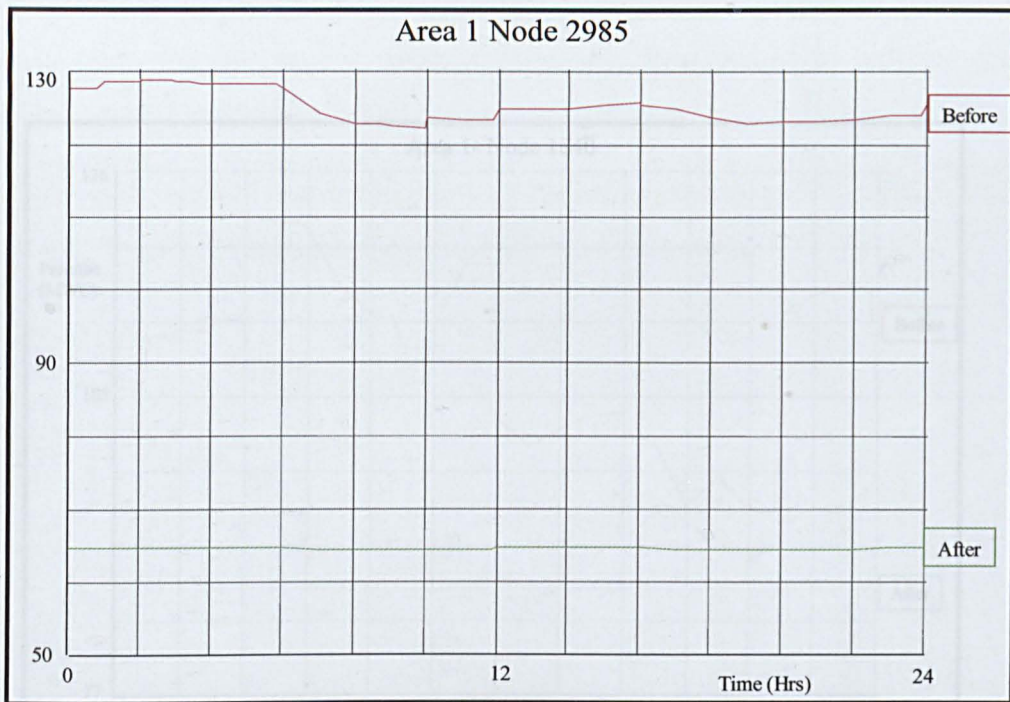


Figure 5.51 Pressure time series for node 2925 before and after pressure reduction

The plots clearly demonstrate the effectiveness of the solution.

5.2.2.4.2.4 Area 5 - Zone 706

The model predicted high pressures for two areas of the network in zone 706. At some nodes pressures exceeded 100 mwc.

The amount by which the whole area could be pressure reduced was limited by a small number of nodes that were at higher elevations than the rest. A pressure-reducing valve was installed between nodes 1825 and 1827 with a downstream pressure set point of 80 mwc. This resulted in a reduction of downstream pressures of 25 mwc.

Figures 5.52 and 5.53 show the time series plots of the pressure variations at nodes 1840 and 1865 before and after the implementation of the scheme.

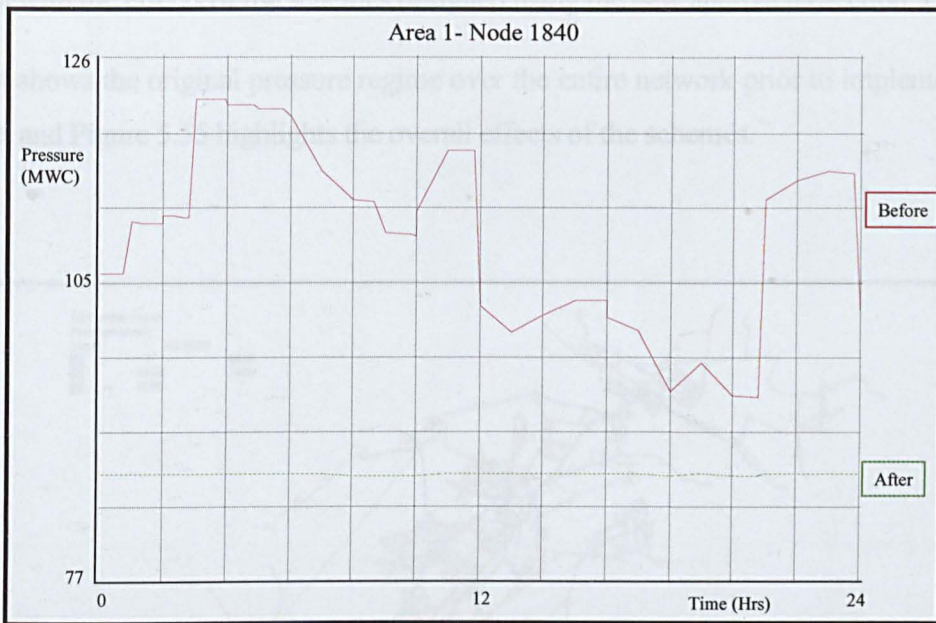


Figure 5.52 Pressure time series for node 1840 before and after pressure reduction

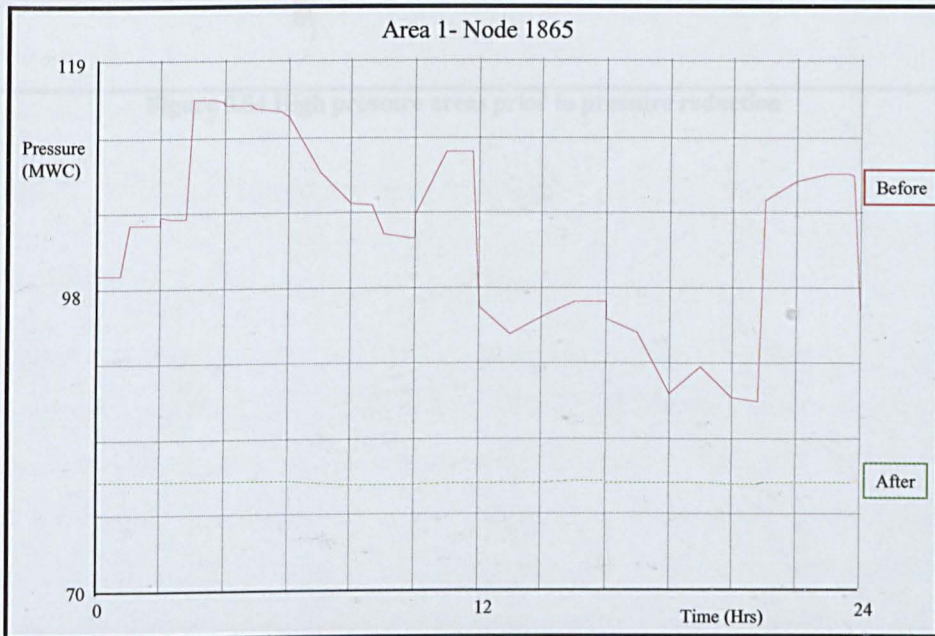


Figure 5.53 Pressure time series for node 1865 before and after pressure reduction

A reduction in pressure of approximately 15m is readily apparent, as is the smoothing effect of the pressure-reducing valve on the downstream pressure.

The effect of these schemes on the whole network was then plotted in order to enable a comparison with the effects of the schemes designed using the new approach (Section 5.3).

Figure 5.54 shows the original pressure regime over the entire network prior to implementation of the scheme, and Figure 5.55 highlights the overall effects of the schemes.

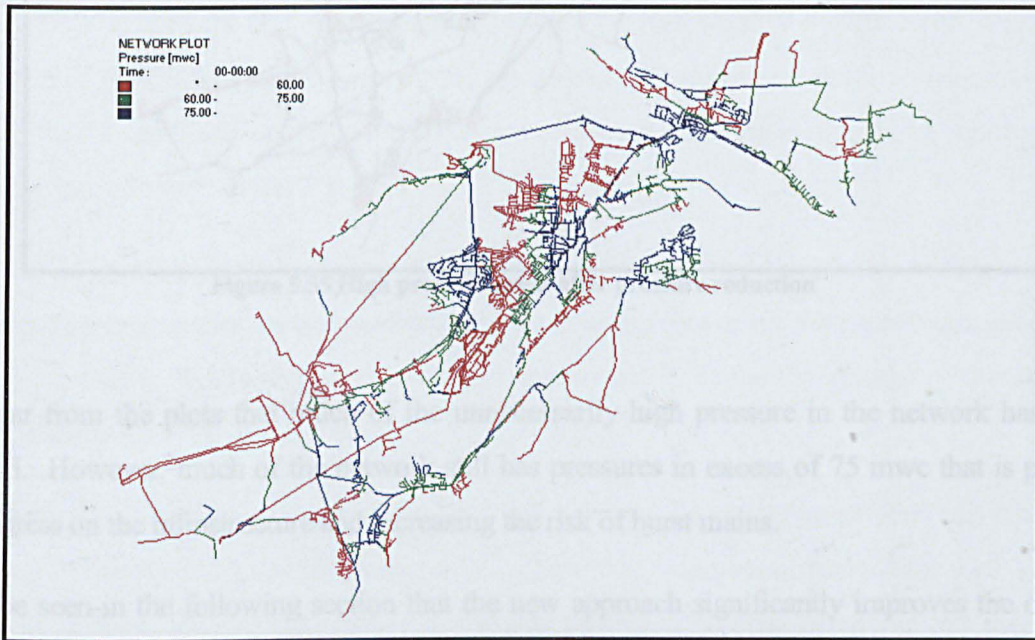


Figure 5.54 High pressure areas prior to pressure reduction

5.2.3 Hydraulic Analysis - New (Integrated) Approach

5.2.3.1 Background

Most distribution networks have evolved over long periods of time in a piecemeal manner and, as shown in the previous section, this leads to complex and difficult operational issues with, for example, pressure management in individual network zones. The new approach involved investigating a complete reorganisation of the way that the whole study network was designed and operated. The nature of the method means that pressure schemes are designed for the entire network as part of a holistic approach and actually become the zones. Unlike the previous section, therefore, low and high pressure are dealt with simultaneously.

The existing zone structure was removed from the model of the original network, and the operational philosophy re-examined without reference to the existing network configuration other than to retain the major assets such as service reservoirs and the majority of the mains infrastructure. The reason for this was to develop a network configuration that was based only

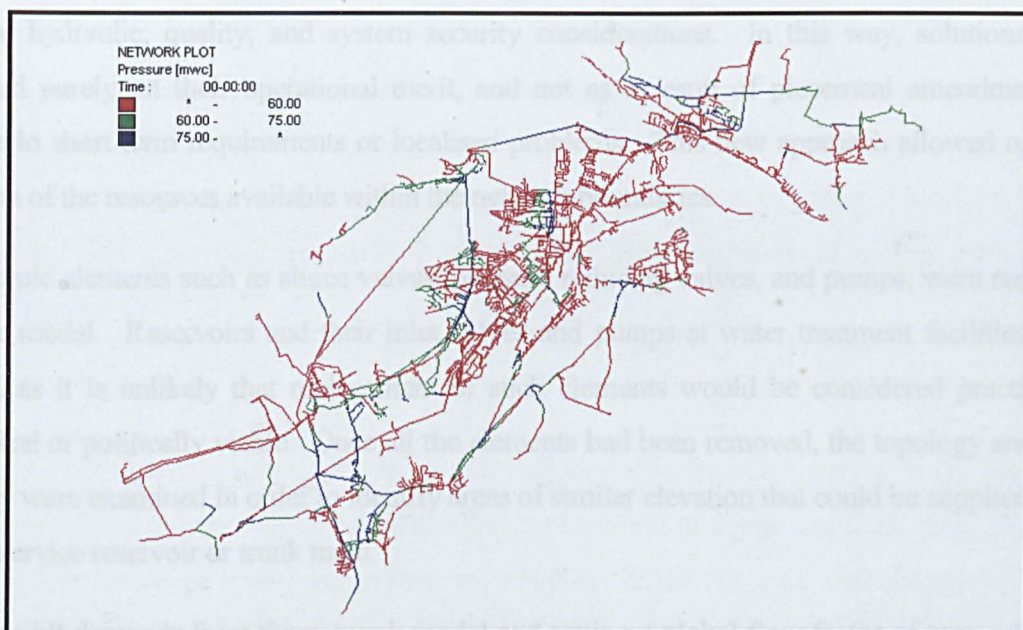


Figure 5.55 High pressure areas after pressure reduction

It is clear from the plots that much of the unnecessarily high pressure in the network has been removed. However, much of the network still has pressures in excess of 75 mwc that is putting undue stress on the infrastructure and increasing the risk of burst mains.

It will be seen in the following section that the new approach significantly improves the overall pressure regime across the study network.

5.2.3 Hydraulic Analysis - New (Integrated) Approach

5.2.3.1 Background

Most distribution networks have evolved over long periods of time in a piecemeal manner and, as shown in the previous section, this leads to complex and difficult operational issues with, for example, pressure management in individual network zones. The new approach involved investigating a complete reconfiguration of the way that the whole study network was designed and operated. The nature of the method means that pressure schemes are designed for the entire network as part of a holistic approach and actually become the zones. Unlike the previous section therefore, low and high pressures are dealt with simultaneously.

The existing zone structure was removed from the model of the original network, and the operational philosophy re-examined without reference to the existing network configuration other than to retain the major assets such as service reservoirs and the majority of the mains infrastructure. The reason for this was to develop a network configuration that was based only

upon the hydraulic, quality, and system security considerations. In this way, solutions were developed purely on their operational merit, and not as a result of piecemeal amendments in response to short term requirements or localised problems. This new approach allowed optimal utilisation of the resources available within the network boundaries.

All dynamic elements such as sluice valves, pressure reducing valves, and pumps, were removed from the model. Reservoirs and their inlet valves and pumps at water treatment facilities were retained, as it is unlikely that replacement of such elements would be considered practicable, economical or politically viable. Once all the elements had been removed, the topology and pipe geometry were examined in order to identify areas of similar elevation that could be supplied from a single service reservoir or trunk main.

Removing all demands from the network model and setting a global flow factor of zero achieved this. Then, when a simulation was performed and a pressure plot of the whole network generated, the resulting output highlighted pressure variations due solely to the topography of the network and identified areas of similar elevation.

A principle design factor was the need to avoid the creation of a cascading zone arrangement that is a feature of the existing network configuration. There are a number of reasons for this. For any operational change, for example adding a pressure reduction scheme to one of the zones, the impact of the change has to be taken into account in each of the cascaded zones downstream. This can severely limit the operational flexibility and make the design and implementation of effective pressure management measures significantly more problematic.

In addition, for incidents such as pollution ingress, because a cascading system has one zone feeding another, it is more difficult to contain an area affected by the pollutant whilst maintaining supplies to the other zones.

Finally, when water has to pass through a number of zones, there is a potential for age related water quality problems, especially for those properties located furthest from the source. Because of their location towards the end of the supply system velocities will be low and sediments will settle in the pipes supplying these properties. The presence of the particulate matter and potentially increased biological activity may exacerbate corrosion, generate taste and odour, and discoloured or turbid water complaints. In such a situation, the time of travel for the water to reach the consumer is likely to be significant. Chlorine levels will therefore be low, resulting in a higher potential for biological re-growth.

An important consideration when modelling the zone reconfiguration was the role of trunk mains. For example, there was a 400 mm main connecting two major service reservoirs that was transporting relatively small amounts of water, around 12 l.sec⁻¹. As the main was not used to supply any of the leakage control zones it was considered to be an under utilisation of this resource, and that there was considerable potential for using this main to supply parts of the network directly.

5.2.3.2 Methodology

Using the model with only service reservoirs and pipe work present as the starting point, each area within the model was considered in terms of the best method of supplying water to it using the current system resources whilst optimising the pressure within the area and minimising water quality effects by analysing the age of water throughout the entire network.. A plot of the configuration of the revised network is provided in Figure 5.56.

5.3.3 Results and Solutions

5.3.3.1 Overview

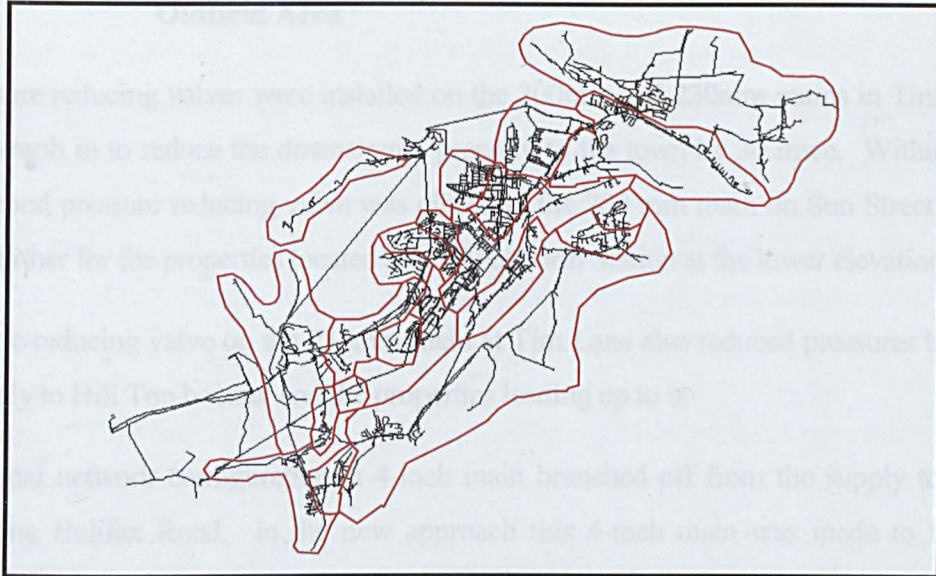


Figure 5.56 The study network redesigned using the new approach

The salient features of the network configuration compared to the existing configuration are described in the following section. The original network had areas that suffered both high and low pressure. The network was built up around a 'spine' of leakage control zones that cascaded into one another making pressure management and supply security difficult and in some cases impossible. The new approach that led to the solutions in 5.3.3 took a holistic view of the network and resolved all pressure problems simultaneously as opposed to the local solutions put forward by a traditional approach.

5.2.3.3 Results and Solutions

5.2.3.3.1 Oldfield Area

Twin pressure reducing valves were installed on the 200mm and 230mm mains in Tim Lane that supply Howarth in to reduce the downstream pressure to the town by 30 mwc. Within Haworth itself, a second pressure reducing valve was placed in the 200 mm main on Sun Street, to reduce pressures further for the properties located around Haworth Station at the lower elevations.

The pressure-reducing valve on the 230mm main at Tim Lane also reduced pressures by 35 mwc for the supply to Hill Top booster and the properties leading up to it.

In the original network configuration, a 4-inch main branched off from the supply to Hill Top Booster along Halifax Road. In the new approach this 4-inch main was made to feed along Hainworth Wood Road, as far as Parkwood Rise, supplying properties that were formerly part of Zone 711. To achieve this it was necessary to insert a 6 inch main with a length of 200m between nodes 2485 and A2261, Halifax Road and Hainworth Lane.

An existing pressure reducing valve in the 4-inch main on Halifax Road was retained to reduce pressures by 60 mwc, this pressure being dictated by the highest elevation properties downstream in the Woodhouse Way area.

As there was no alternative means of supplying Thwaites Brow other than using the original regime of Hill Top booster and Hainworth service reservoir, pressure reduction was implemented in order to minimise the occurrence of high pressures in this area.

The 12inch main (twin 12 inch mains for some sections) that runs between White Lane service reservoir and Black Hill service reservoir along Keighley Road, was used to supply a number of areas along its path. Each of these areas has been individually pressure reduced to deliver the optimum pressure dictated by its elevation. The model was amended to reflect installation of pressure reducing valves at the following locations:

On Providence Lane, Oakworth, to reduce pressures by 70 mwc to the properties downstream close to the River Worth.

On the 6 inch main on Colne Road, Oakworth, to reduce pressures by 50 mwc to properties in the Station Road area.

On the 6 inch main on Goose Cote Lane, to reduce pressures by 50 mwc to a relatively large area including Harewood Road, Greystones Drive, Valley View and Oakbank Broadway.

At the junction of Oakworth Road and Wheat Head Lane, two pressure reducing valves were added, one to supply the Occupation Lane / Cambourne Way area, and the second to reduce pressures by 80 mwc to the area to the North of Wheat Head Lane as far as Fell Lane.

At the junction of Fell Lane and Westfell Road a pressure reducing valve was added to reduce pressures by 100 mwc to an area that includes properties along Fell Lane towards Lund Park, and on the Eastern edge of Lund Park that was previously part of Zone 710.

At the northern end of the 12-inch trunk main near Black Hill Service Reservoir, there is a branch that leads down Laycock Lane. In order for this area to be effectively pressure reduced, the main feeding the high elevation properties on Braithwaite Edge had to be valved so that it was not included in the pressure reduced area. This allowed a pressure-reducing valve to be added on the 6 inch main on Braithwaite Road that reduced pressures downstream by 60 mwc.

5.2.3.3.2 Keighley Area

Significant changes were implemented within the Keighley area of the model.

Zones 711 and 712 are no longer fed from Bracken Bank service reservoir in a cascading arrangement as they were before, but have pressure reduced connections into the trunk main between Riddlesden service reservoir and Black Hill service reservoir. One of the advantages of the construction of a single model covering the entire study network is that multiple sources are included in the model and hence all supply possibilities were investigated.

In the reconfigured model, Bracken Bank service reservoir only supplies part of the area of the existing zone 710 and the properties off Worth Way and along Parkwood Street. The highest properties of the existing Zone 710 have been valved such that they are included within the area supplied by the pressure-reducing valve off the White Lane to Black Hill 12 inch trunk main located at the Junction of Fell Lane and Westfell Road.

This allowed pressure reduction of the Northern half of the existing Zone 710 by the addition of a pressure-reducing valve on the 15-inch trunk main at the junction of Ingrow Lane and Ashbourne Road. This pressure-reducing valve reduces pressures by approximately 22 mwc. A second pressure-reducing valve was added on the Queens Road 15 inch main to further reduce pressures

by 30 mwc to the Worth Way and Parkwood Street area. The 15-inch main was valved off at the junction of Bradford Road and Dalton Lane.

Highfield service reservoir now supplies a modified area of the existing Zone 713 and a significant proportion of what was originally Zone 711. Previously, the pressure-reducing valve on Albert Street supplied an area of Zone 709. This has been altered so that now the pressure-reducing valve supplies water to the northern half of the existing Zone 711 and also feeds back to supply properties that were formerly in Zone 713. As a result of these changes, the higher elevation properties in former Zone 713 are maintained on direct supply from Highfield service reservoir, while those properties of lower elevation have been re-valved so that they are within the area served by the Albert Street pressure-reducing valve.

The area of Zone 709 known as the Albert Street area has been altered so that it now takes its supply from the new cross town trunk main that runs between Riddlesden and Black Hill service reservoirs. A connection into this new main has been made near the junction of Hard Ings Road and Skipton Road. This connection is pressure reduced to maintain the optimum pressure in the area it supplies and pressures have been reduced by 130 mwc.

A second connection into the new cross-town main, already in existence close to the junction of Grange Road and Bradford Road has been modelled as a pressure reduced supply that feeds the remainder of the existing Zone 709 and all of Zone 712. This includes properties on Aire Valley Road, Dalton Lane, Marlow Street and Thwaites Lane, as well as the properties on Bradford Road between the River Aire and the Leeds Liverpool Canal. In the reconfigured model, a pressure-reducing valve on the connection to the cross-town main regulates pressures such that they are kept below 50mwc over the whole area.

Figures 5.50 to 5.52 are network plots of the whole of the study distribution network model displaying the pressure variations over the network. In each case the pressure bands are chosen to highlight the properties experiencing high pressures within the over 50 mwc and over 100 mwc bands.

Figure 5.57 represents the current network configuration.

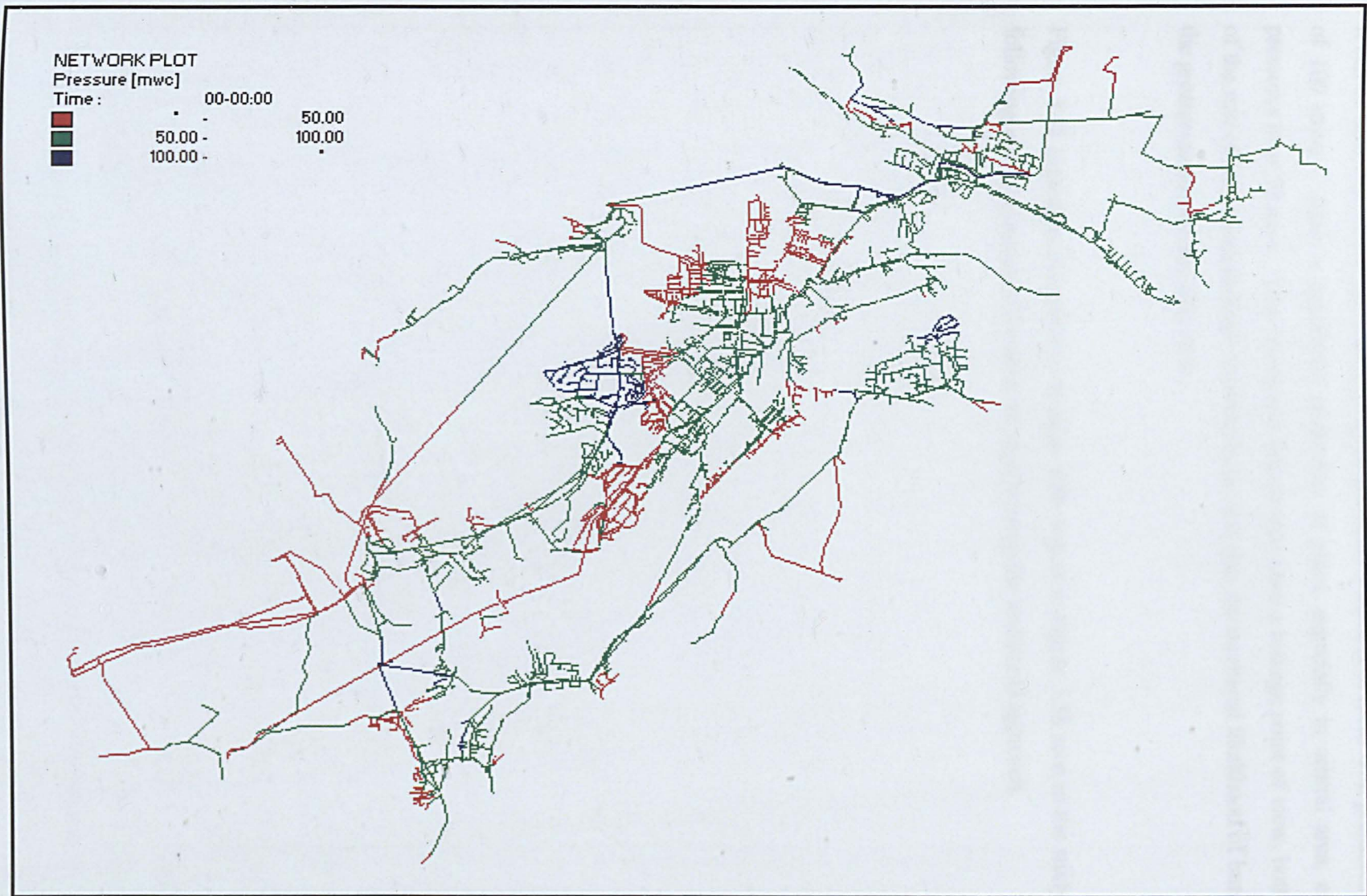


Figure 5.57 Pressure regimes with original network configuration

It can be seen that the current network regime produces a number of areas with pressures in excess of 100 mwc. Also, a significant proportion of pipes, especially in central area, experience pressures over 50 mwc. This situation is undesirable from a leakage point of view, both in terms of the rate of water loss through existing leaks, and also the increased likelihood of bursts due to the greater stresses on the pipe work.

Figure 5.51 highlights the pressure changes with respect to Figure 5.58 seen in the study network following reconfiguration of the network regime using the traditional approach.

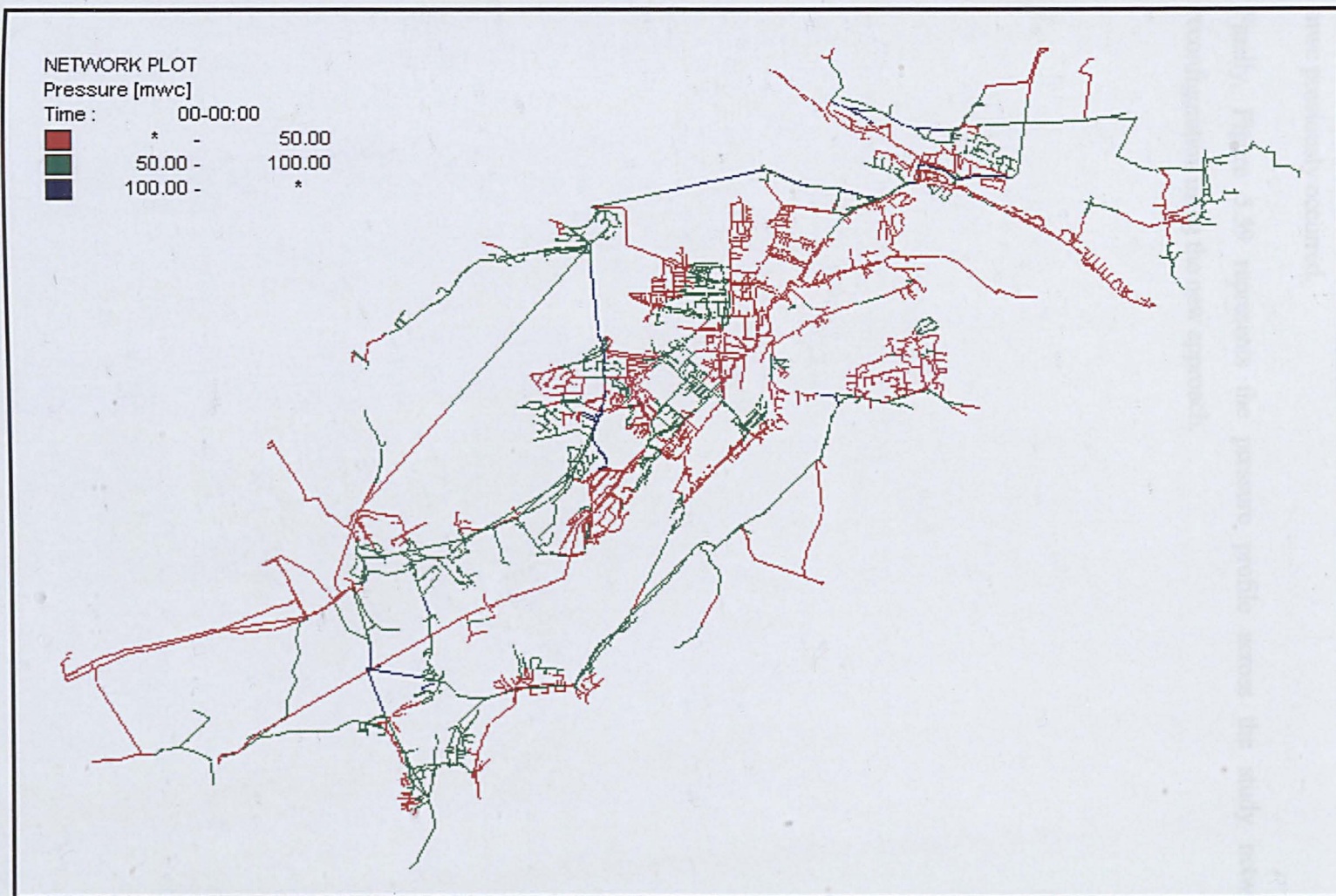


Figure 5.58 Study network pressures following reconfiguration via the traditional method

It is clear that the pressure management schemes significantly reduce the number of pipes experiencing pressures over 50 mwc and almost eliminates any pipes where pressures of over 100 mwc previously occurred.

Finally, Figure 5.59 represents the pressure profile across the study network following reconfiguration using the new approach.

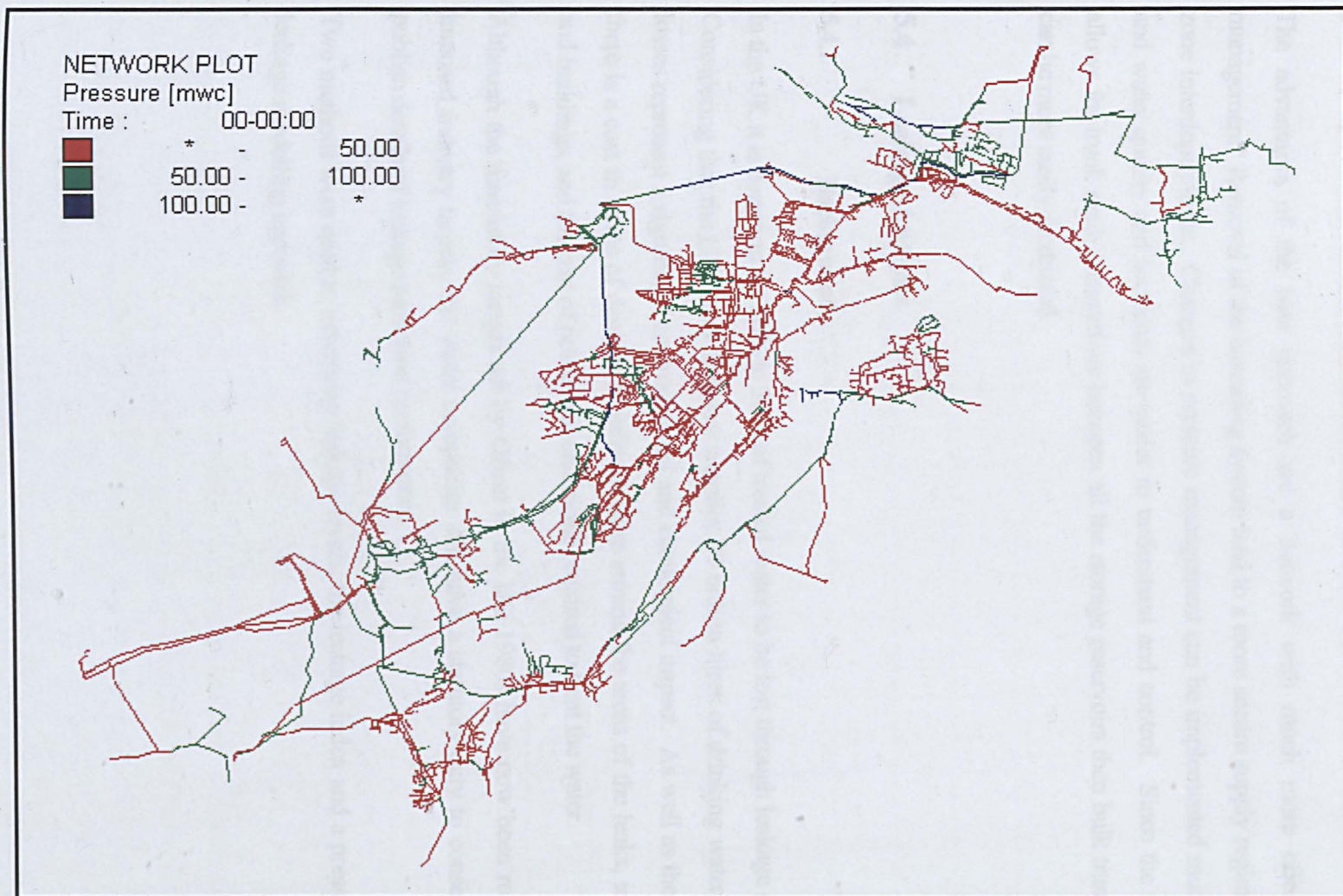


Figure 5.59 Study network pressures following reconfiguration by the new method

The reconfiguration of the network using the new method can be seen to have been similarly effective in eliminating the areas with pressures of over 100 mwc but has resulted in a much greater proportion of the network experiencing pressures of less than 50 mwc.

The advantages of the new approach are a network with much more effective pressure management. Removal of the cascading system led to a more secure supply regime and reduced zone interdependence. Changes in pressure management can be implemented much more easily and water quality and incidents are easier to understand and control. Since the new approach allows for trunk main connections between all the storage reservoirs then bulk transport of water can be more easily facilitated.

5.4 Leakage Analysis

5.4.1 Background

In the UK it is common for 20% to 30% of treated water to be lost through leakage (*Ofwat, 2001*). Considering that the UK water industry supplies 20 billion litres of drinking water per day these losses represent a significant environmental and economical impact. As well as the loss of water, there is a cost in terms of damage to infrastructure around the areas of the leaks, including roads and buildings, and the cost of power and chemicals required to treat the water.

Although the mandatory targets set by Ofwat in the late 1980s have now been replaced by self-imposed industry targets, the water companies still have a statutory duty to conserve water and publish details of leakage reduction performance.

Two methods were used to determine leakage levels, the leakage index and a pressure dependent leakage modelling approach.

5.4.2 The Leakage Index

The leakage index provided a comparison of relative leakage rates due to changes in the average zone or network pressure brought about by the proposed schemes. Figure 5.60 shows the relationship between leakage index and average zone pressure. (*Technical Working Group on Waste of Water, Leakage Control Policy and Practice. National Water Council Standing Technical Committee Report No. 26, July 1980*). It shows an almost exponential relationship between increasing pressure and loss of water.

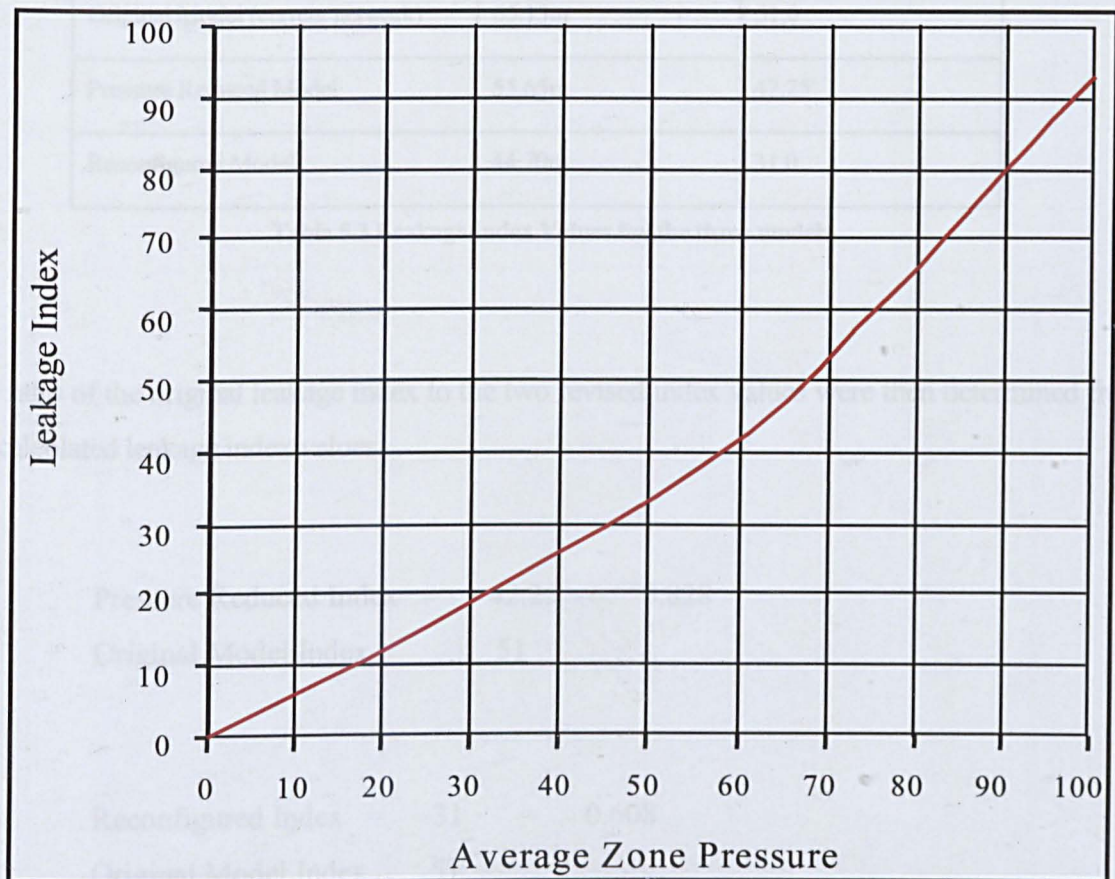


Figure 5.60 Relationship between Leakage index and Average Zone Pressure

Average network pressures were calculated for each of the models of the study network i.e. the model of the network in its original configuration, the model of the network after schemes designed by traditional methods, and the model reflecting the zone reconfiguration designed by the new integrated approach.

This was done by importing simulation output data from the models into a spreadsheet and averaging the pressure, at time of minimum flow, i.e. when pressure is highest, at each demand node. The results of these calculations are shown in Table 5.3

MODEL	AVERAGE NETWORK PRESSURE	LEAKAGE INDEX VALUE
Original Model (current network)	65.15m	51.0
Pressure Reduced Model	55.65m	42.25
Reconfigured Model	44.70m	31.0

Table 5.3 Leakage Index Values for the three models

The ratio of the original leakage index to the two revised index values were then determined from the calculated leakage index values.

$$\begin{aligned} \text{Pressure Reduced Index} &= \frac{42.25}{51} = 0.828 \\ \text{Original Model Index} &= 51 \end{aligned}$$

$$\begin{aligned} \text{Reconfigured Index} &= \frac{31}{51} = 0.608 \\ \text{Original Model Index} &= 51 \end{aligned}$$

The figures indicate that in the case of the traditional approach model, the expected leakage rate will be 0.706 of its original value and for the new approach model the value would be 0.608 of the original leakage rate.

The leakage rate in the original study network was estimated to be 20% of the total consumption. The total daily consumption is 16,000 m³ per day, therefore 3,200 m³ of water are lost each day due to leakage. The traditional approach would reduce this to 2650 m³, and the new approach to 1946 m³.

5.4.3 Pressure Dependant Leakage

For a leak of fixed size, the higher the pressure at the location of the leak, the greater the rate of leak flow will be. It is therefore useful to compare relative leak flow rates, for leaks of constant diameter at a number of different points within the network before and after network changes to observe the effects of pressure reduction.

The rate of pressure dependant leakage through an orifice of specified diameter at a number of different nodes within the network was therefore determined using the three models i.e. current network configuration, the network modified by traditional approach, and the model of the network reconfigured by the new approach. Three leaks were introduced into each of the pressure-reduced zones (9) within each model. Their locations are shown in Figure 5.61.

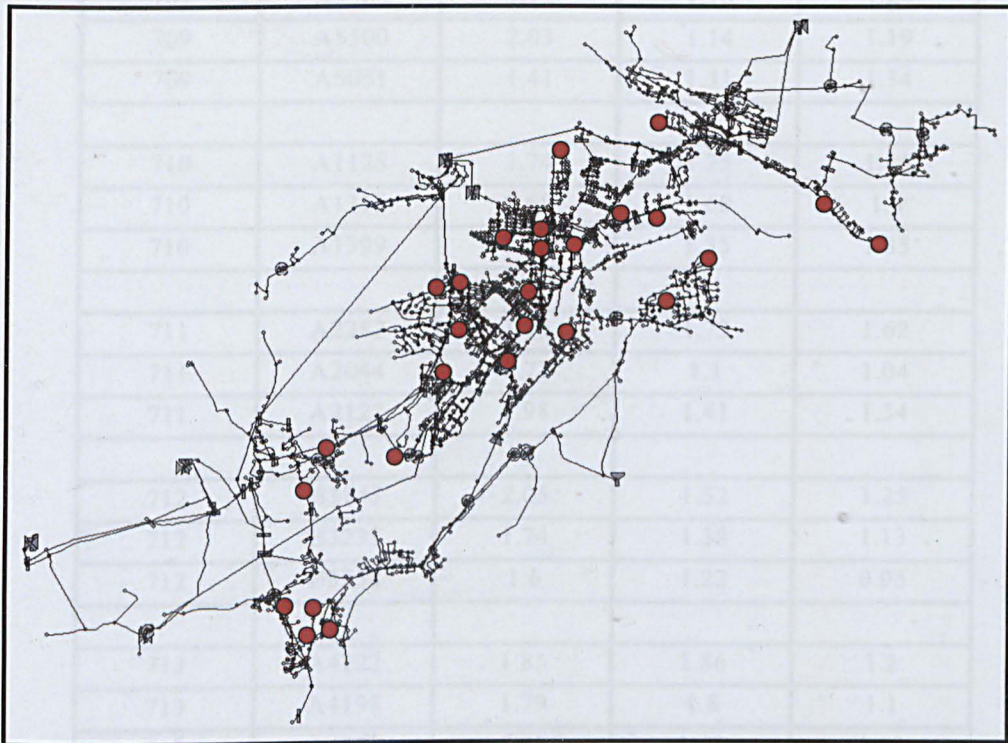


Figure 5.61 Leak locations for pressure dependent leakage

The leaks were all given the same nominal diameter (10mm). For each model, a 24-hour simulation was run and a plot of the predicted leak flow for selected nodes was generated.

A summary of the results can be seen in table 5.4.

Leakage Control ZONE	Node	Maximum Leakage flow rate lsec ⁻¹		
		Original network	Traditional approach	New approach
704	1415	1.79	1.73	1.23
704	1638	1.16	1.16	1.15
704	1445	1.73	1.31	1.18
706	1940	2.01	1.72	1.23
706	3165	2.29	1.57	1.57
706	2885	1.76	1.22	1.22
708	565	2.06	2.05	1.44
708	845	1.83	1.8	1.15
708	615	1.81	1.8	1.12
709	A5460	1.73	1.18	1.24
709	A5500	2.03	1.14	1.19
709	A5051	1.41	1.31	1.34
710	A1125	1.74	1.25	1.46
710	A1315	1.68	1.68	1.4
710	A1399	1.38	1.35	1.35
711	A2282	1.78	1.73	1.62
711	A2044	1.77	1.1	1.04
711	A2122	1.98	1.41	1.34
712	B3147	2.05	1.52	1.25
712	B3231	1.74	1.38	1.13
712	B3258	1.6	1.22	0.95
713	A4222	1.85	1.86	1.2
713	A4198	1.79	1.8	1.1
713	A4058	1.53	1.53	1.53
716	1085	1.91	1.89	1.32
716	4090	2.46	1.71	1.57
716	3730	2.22	1.31	1.18

Table 5.4 Pressure Dependant Leakage Rates

The table shows a significant reduction in leakage flow rate. Figures 5.62 to 5.73 show time series pressure dependant leakage flows at specific nodes in the model.



Figure 5.62 Time series of leak flow at node 1940 – Original network

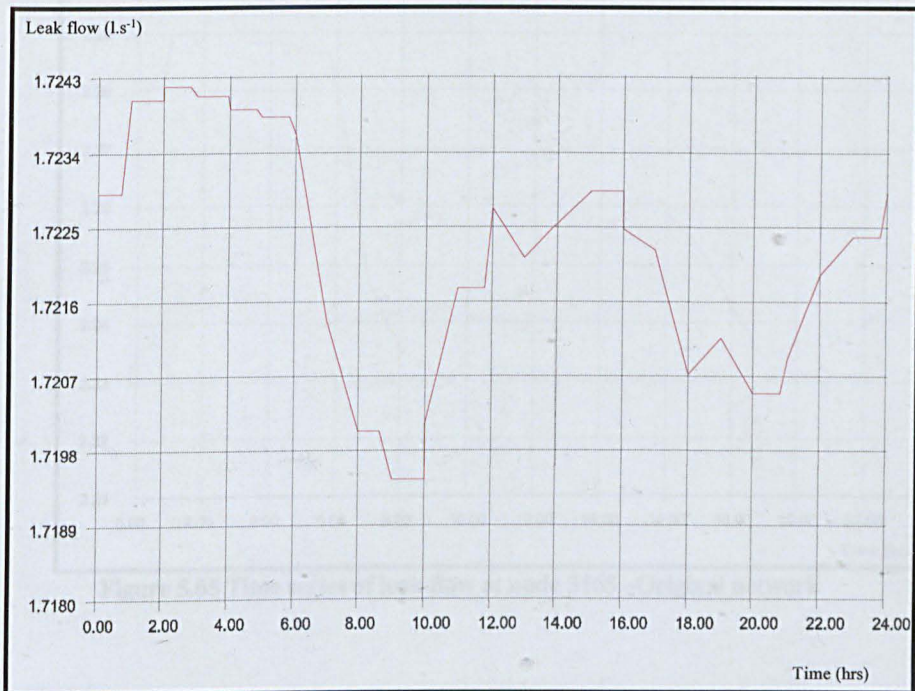


Figure 5.63 Time series of leak flow at node 1940 – Traditional approach



Figure 5.64 Time series of leak flow at node 1940 – New approach



Figure 5.65 Time series of leak flow at node 3165 –Original network



Figure 5.66 Time series of leak flow at node 3165 –Traditional approach



Figure 5.67 Time series of leak flow at node 3165 –New approach



Figure 5.68 Time series of leak flow at node 3147 - Original network



Figure 5.69 Time series of leak flow at node 3147 - Traditional approach



Figure 5.70 Time series of leak flow at node 3147 –New approach



Figure 5.71 Time series of leak flow at node 4090 –Original network



Figure 5.72 Time series of leak flow at node 4090 –Traditional approach



Figure 5.73 Time series of leak flow at node 4090 –New approach

An accumulated total of the volume of water lost due to the leaks placed on the nodes in each model, in m³, were obtained from the simulation output files. For the three models described in Table 5.4, the leakage volumes for a typical 24-hour period were:

4097 m ³	for the original network model
3139 m ³	for the traditional approach model
2405 m ³	for the new approach model

A significant reduction in leakage, particularly in Zones 712 and 709 is apparent for the traditional approach model. However, the figures for the new approach model demonstrate that even greater improvements were achieved when the study network was considered as whole rather than individual zones. These figure are higher than for the Leakage Index method but in good agreement with regard to percentage of volume lost.

5.4.4 Relating Leaks to High Mains Pressure

High mains pressure puts unnecessary stress upon elements of the network especially pipe work and pipe joints thereby increasing the probability of a structural failure or seepage.

? mains bursts were repaired in the study network between June 1996 and June 1997. These bursts represent a significant cost in terms of lost water, the manpower required to find the bursts and repair them, and possible compensation payments for damage caused by the escaping water.

Figure 5.74 shows the pressure levels with the original network configuration. In this configuration, there is a significant proportion of unnecessarily high pressure. Superimposed upon this plot are the locations of mains bursts, taken from maps of burst occurrences plotted on a GIS system.

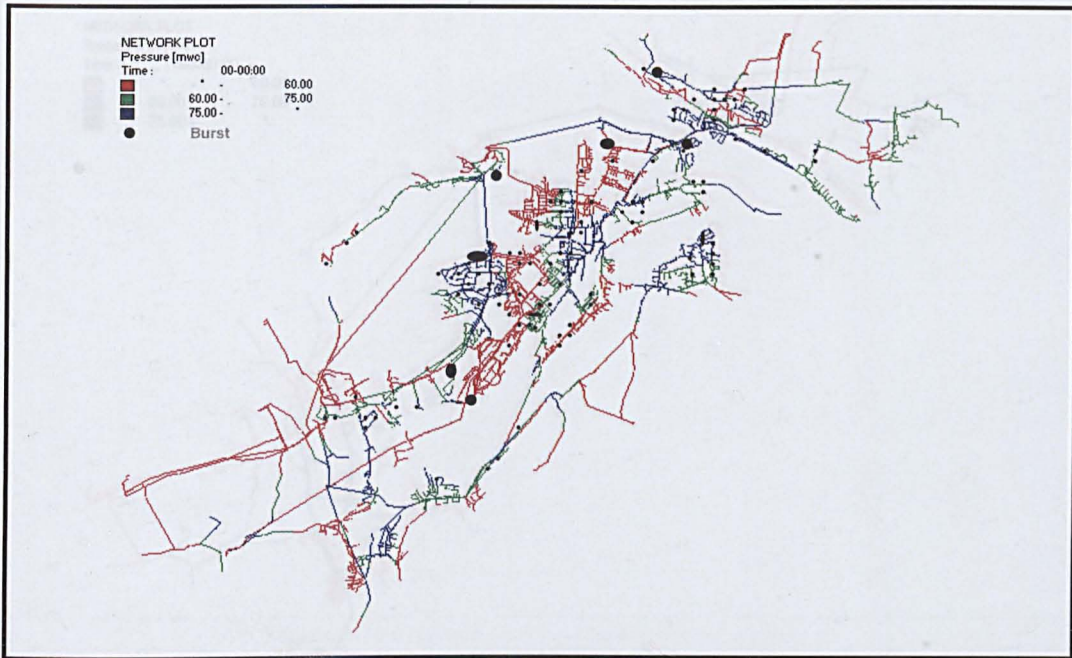


Figure 5.74 Burst data overlaid on original network pressure plot

There is an obvious correlation between the occurrence of bursts and the location of high-pressure mains. Figure 5.75 shows a similar network plot (the pressure band divisions are the same), of predicted values following implementation of pressure reduction designed by the traditional approach.

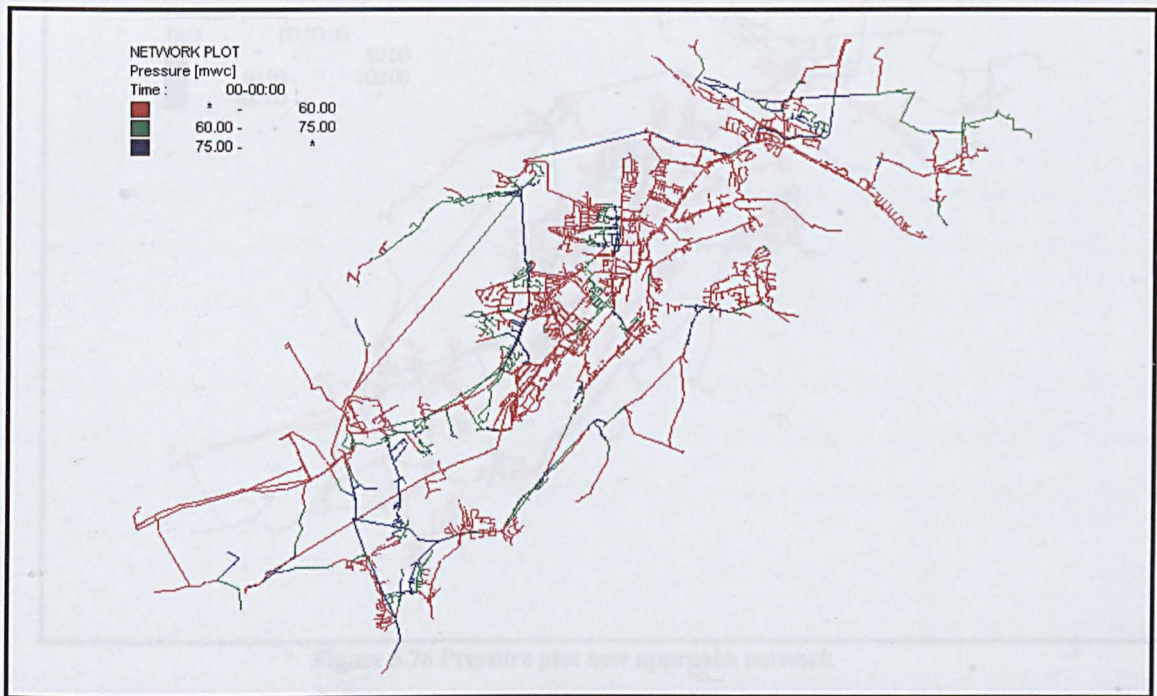


Figure 5.75 Pressure plot of the traditional approach network

It is clear that the extent of the areas of high pressures is significantly reduced. Figure 5.76 is the same plot for the network reconfigured by the new approach.

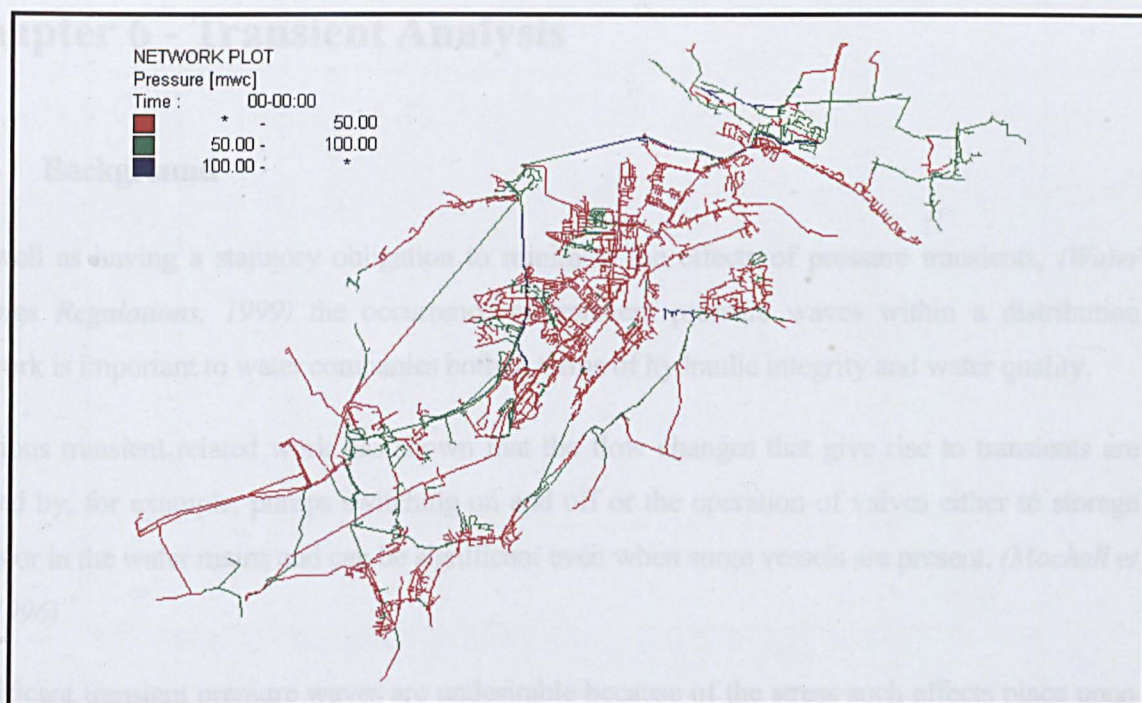


Figure 5.76 Pressure plot new approach network

A further reduction in the extent of the high-pressure areas is clear.

5.5 Summary Remarks

It has been shown that the new approach produces significantly better results than the traditional approach with regard to design of pressure control for leakage management - 23% saving using the traditional approach, and 41 % via the new approach.

It is a logical conclusion that if the network were reconfigured using the new approach it would be less prone to mains bursting because of excessive pressure than would the original or that reconfigured by a traditional approach.

Chapter 6 - Transient Analysis

6.1 Background

As well as having a statutory obligation to minimise the effects of pressure transients, (*Water Fittings Regulations, 1999*) the occurrence of transient pressure waves within a distribution network is important to water companies both in terms of hydraulic integrity and water quality.

Previous transient related work has shown that the flow changes that give rise to transients are caused by, for example, pumps switching on and off or the operation of valves either to storage tanks or in the water mains and can be significant even when surge vessels are present. (*Machell et al., 1996*)

Significant transient pressure waves are undesirable because of the stress such effects place upon the pipe work and other assets, and the resultant increased probability of structural failure (*Woodward, 1964*). Pipe sections have a maximum pressure rating, and a maximum excursion pressure, which may be exceeded by the temporary pressure variations created during a surge event. Although there is a margin for safety built into the excursion pressure rating, repeated infringements impart a “toffee hammer” effect and can lead to structural failure. (*Jaeger, 1963*)

Recent work has shown that transient pressure waves can be responsible for sudden increases in turbidity. This may be caused by disturbance of sediments and / or biofilm in the network. *Keevil & Walker, (1995)* showed biological material that grows on the internal wall of the pipe being disrupted by pressure waves and hence becoming suspended within the bulk flow. This effect could be one of the reasons why most water companies have unexplained sporadic bacteriological failures. The shock of a pressure transient may also disturb material deposited within a main leading to discolouration and unpalatable water. It is therefore desirable to be able to model the effects of the operation of dynamic network elements that are likely to generate surge events leading to such problems. Even sudden changes in pressure caused by normal network demands can lead to increased turbidity from the re-suspension of sediment in the pipe network. *Machell, 1996*, recorded this effect. Figure 6.1 shows a turbidity response to the morning peak demand in the study network.

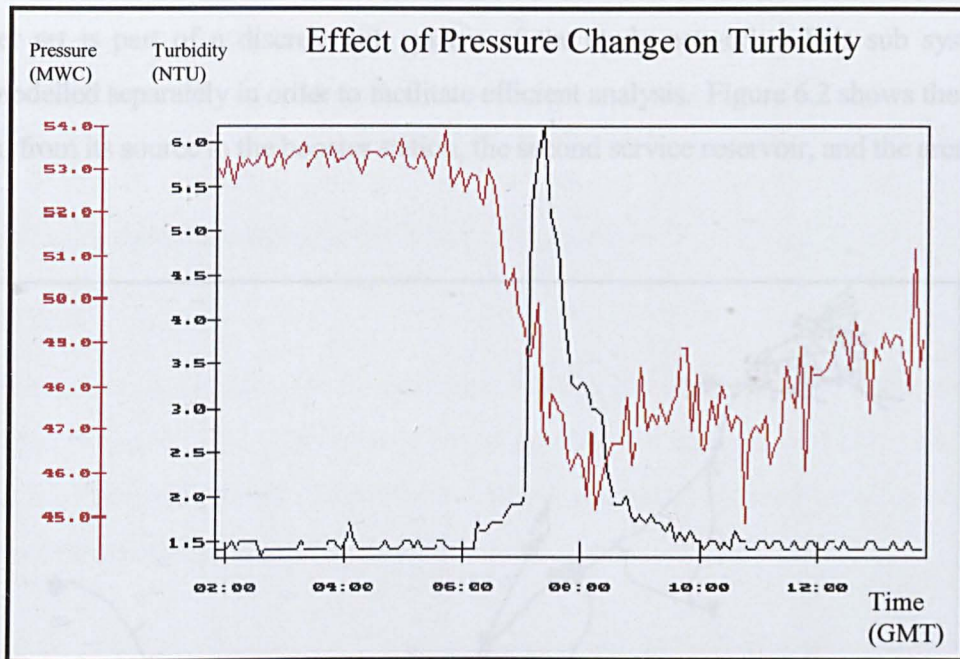


Figure 6.1 Turbidity response due to an increase in domestic demand

The modelling software used in this study included routines for transient analysis. These existing routines were used to examine the impact of transients across a whole leakage control zone within the study network. It is stressed that the author did not undertake any research to improve or modify the existing routines but the functionality was used to demonstrate the effects of the operation of a dynamic element, a pump, within the study distribution network. The author made measurements of pressure at locations upstream and downstream of a pump to compare observed and model predicted pressure effects for calibration purposes (6.2.4).

The software simulated the magnitude and distribution of transient pressure waves generated by the operation of the pump.

The operation of a booster pump was chosen as the event to model. The selection of this booster was dictated by the availability of accurate and up to date pump curve data. In addition, examination of burst information for the sub system containing the pump, had indicated that there were locations where repeated pipe / service bursts had been occurring.

Burst frequency data was plotted against the areas predicted by the model to be worst affected by the surge event in order to identify any correlation.

The pumping station transfers water, which originates from a service reservoir on one side of the network to another service reservoir on the opposite side. A level indicator in the second service reservoir controls pump operation.

The booster set is part of a discrete sub system of the study network. This sub system was therefore modelled separately in order to facilitate efficient analysis. Figure 6.2 shows the network sub system, from its source to the booster station, the second service reservoir, and the areas fed.

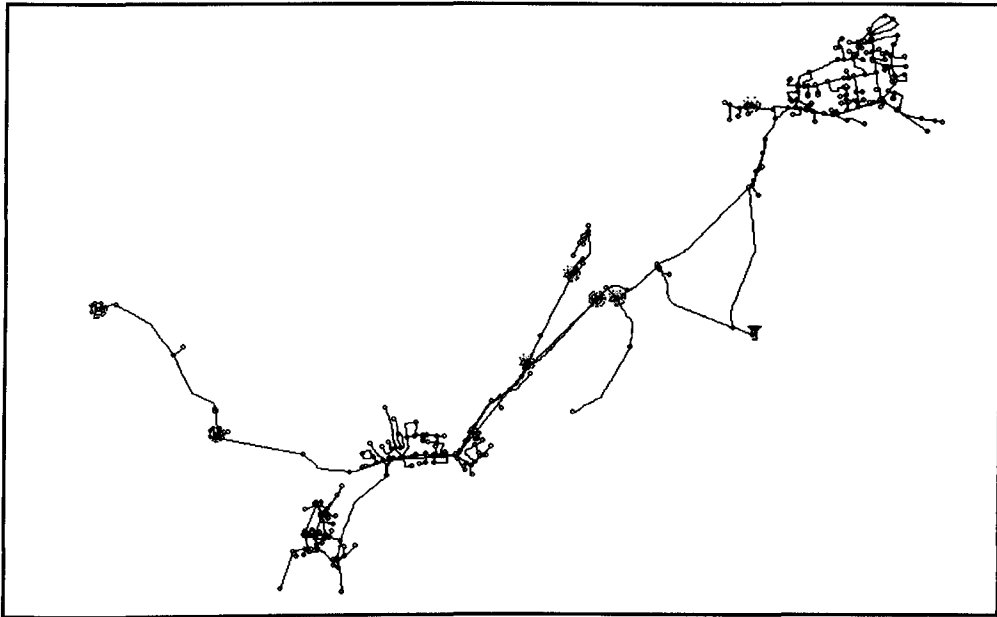


Figure 6.2 The sub network used for transient analysis

Having created the sub network model, a quasi-dynamic hydraulic simulation was run in order to provide the appropriate hydraulic description from which to begin the surge calculations. This initial set of hydraulic characteristics, in ASCII format, described a snapshot of the calculated pressures and flows for all pipes and nodes for the time the pump was switched.

The booster pump was switched on and off two minutes into the simulation, and the pressure waves generated by the event were plotted for a number of selected locations within the network where transient effects were manifest.

6.2 The Transient Model Build Process

The hydraulic model architecture was designed such that it also provides the basis for transient modelling. However, additional data beyond that required for normal hydraulic analysis is required to emulate the transient pressure effects of a surge event.

6.2.1 Pipe Data

The wall thickness and Young's Modulus or pipe celerity (speed of pressure wave travel) had to be allocated for each pipe. This data is based on the pipe material and class and may be found from literature (*Pickford, 1969*). In the model the variables may be allocated in any one (or combination of) the following ways:

- 1 Globally - a single wall thickness and Young's Modulus or celerity value to every pipe in the model. This method is efficient, but is very inaccurate because it takes no account of material, pipe diameter or pressure class.
- 2 Apply a wall thickness and Young's Modulus or a celerity value to pipes based on the diameter of the pipe. Using this method a file (a *.DPD file) containing a table of pipe diameters, their wall thickness and associated Young's Modulus is imported into the model. This again is a simple, efficient process and is more accurate than a global value, but it does not fully take material and pressure class into consideration. Additionally, where a model contains a pipe(s) with diameters not included in the *.DPD file, a wall thickness' and Young's Modulus value will have to be applied individually or using default values. Figure 6.3 shows the basic *.DPD file format.

```

DPD FILE - Created at: 10/24/01 10:40:08

=====
= Additional Property Data =
=====

-----
Pipe ID      Thickness  E-modulus  Pipe Celerity
      [mm]      [N/mm2]    [m/s]
-----
*DAPD  "L-0001"    10.00      2.9000e+005  *
*DAPD  "L-0002"    10.00      2.9000e+005  *
*DAPD  "L-0003"    10.00      2.9000e+005  *
*DAPD  "L-0004"    10.00      2.9000e+005  *

-----

*STOP

```

Figure 6.3 A *.DPD file

3 Individually - Apply specific wall thickness and Young's Modulus or celerity value to each pipe. This is by far the most appropriate method but is also very time consuming. Every modelled pipe must be cross-referenced against the base data in the GIS system (TRAMS) to determine the pipe characteristics, with the characteristics required for transient analysis being manually entered into the model at pipe level.

For this study it was decided to apply all the data manually to every pipe. Tables of Young's Modulus and graphs of celerity values that were collated for the purposes of the study are shown in Table 6.1 and Figure 6.4 respectively.

Pipe Material	Young's Modulus (N/mm ²)
Mild Steel	210,000
"Default" Pipe	205,000
Wrought Iron	197,000
Cast Iron	110,000
Concrete	14,000
PVC (rigid)	2,800
MDPE	1,100
HDPE	1,200

Table 6.1 Young's Modulus for a variety of materials

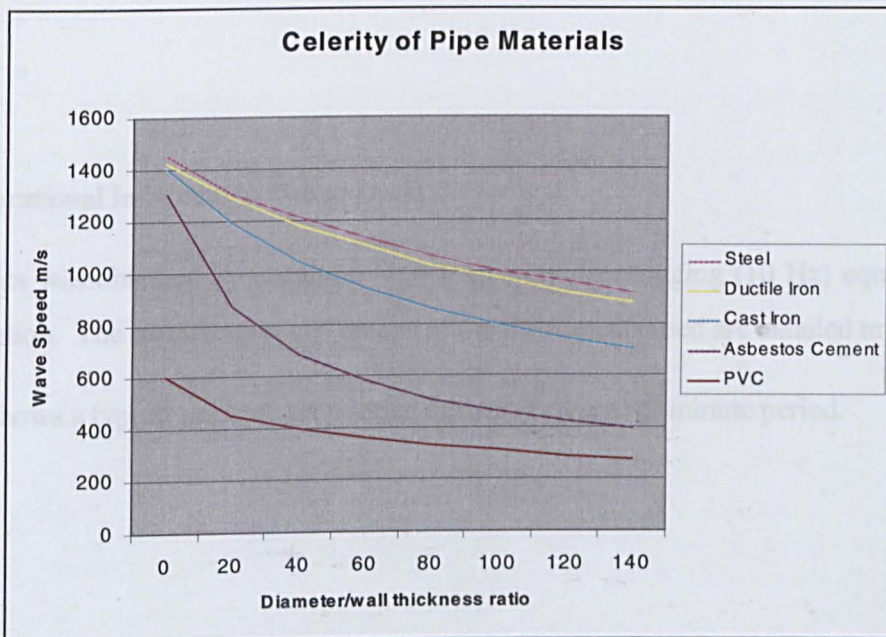


Figure 6.4 Celerity of pipe materials

Because of the large amount of time required to complete this process, and the expense of data collection the transient analysis was restricted to a leakage control zone within the study network which contained a pump. The pump was switched on and off a number of times to create the

transient effects at a time when the data loggers were recording. Figure 6.5 shows the area of the network modelled for the transient analysis.

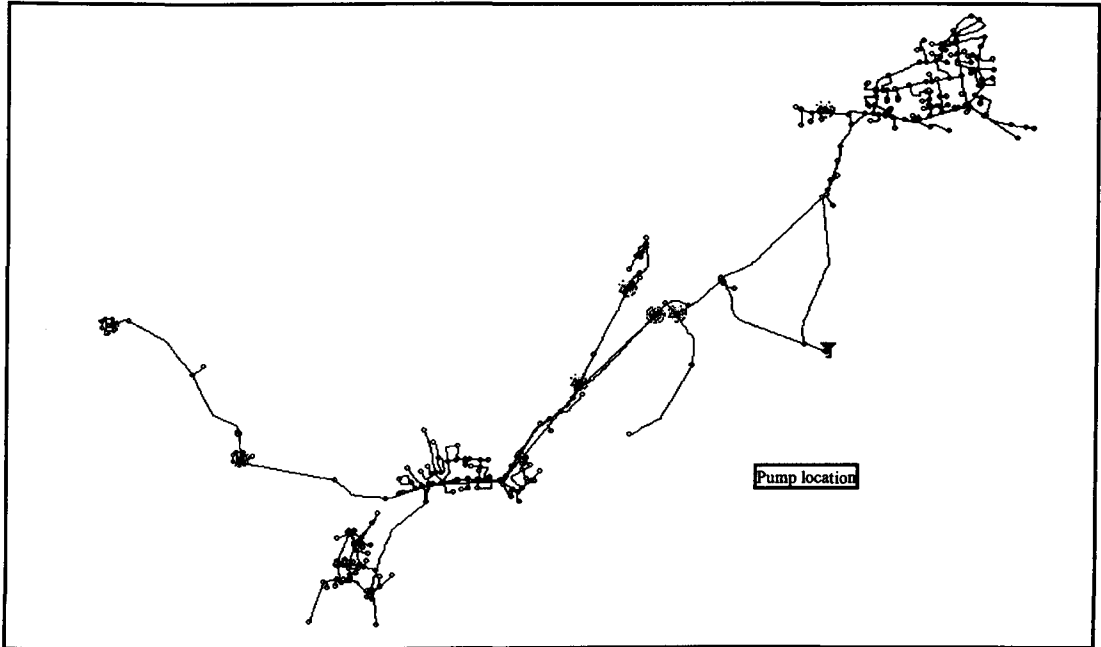


Figure 6.5 The area of the network used for the transient model

6.2.2 Operational Information (Surge Data)

Accurate data was obtained by installing high frequency data logging (10 Hz) equipment at the pumping station. The specification and details of the instruments used are detailed in Section 4.3

Figure 6.6 shows a typical pressure time series measured over a 40-minute period.

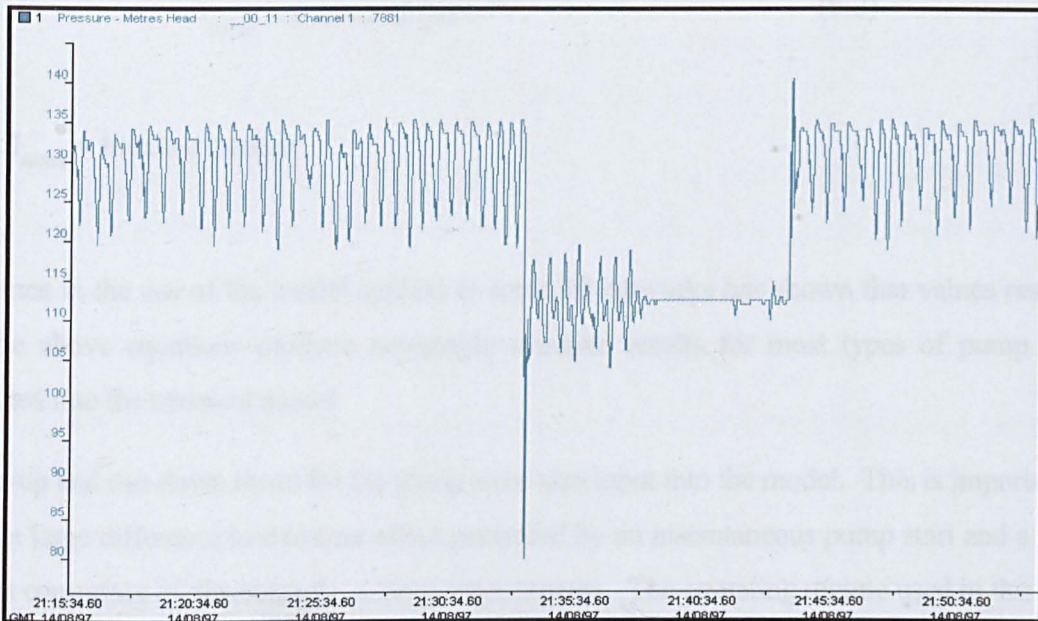


Figure 6.6 Time series flow data at 10 Hz for a typical pump trip

6.2.3 Pump Data

The moment of inertia and impeller diameter were allocated as pump characteristics in the model. Inertia data is usually obtained from the pump suppliers but in this study this data was not available. The inertia of the pump was derived from information for approximately 300 pumps from five different manufacturers used to produce equation 6.1.

$$I = 0.03768 (P/N^3)^{0.9556} \quad (6.1)$$

Where:

I = Inertia of pump

N= pump speed in 1000's of RPM

P = shaft power which is given by:

$$P = (\text{Density} * g * \text{Rated Flow in m}^3/\text{s} * \text{Rated Head in m}) / (\text{Efficiency} * 1000)$$

Equation 6.1 is for the pump assembly only and the inertia associated with the motor has to be added where:

$$I_{\text{motor}} = 0.0043 (P/N)^{1.48} \quad (6.2)$$

Where: I_{motor} = Inertia of motor

Experience in the use of the model applied to some 40 networks has shown that values resulting from the above equations produce acceptably accurate results for most types of pump when substituted into the transient model.

The run-up and run-down times for the pump were also input into the model. This is important, as there is a large difference in transient effect generated by an instantaneous pump start and a pump start that comprises of, for example, a three-stage process. The operating regime used in this study was an instantaneous switch on / switch off of the pump.

6.2.4 Model Calibration

The transient model build on the calibrated steady state hydraulic model that provides a steady state representation of the flow and pressure characteristics of the network at any given time. The transient model uses this data as the starting point for a dynamic simulation, i.e. the steady state hydraulic model provides the flow and pressure at every point in the model at the moment when the surge event is initiated.

To calibrate the model it was required to match the shape and magnitude of the observed and predicted pressure plots from each of the logger locations over the simulation period. However, unlike steady state hydraulic modelling where the calibration process is well understood and relies predominantly on adjusting the pipe roughness coefficients, calibration of a dynamic model can be affected by a wide range of factors. These include pipe connectivity, wall thickness, diameter, and class, Young's Modulus, pump behaviour (and pump data accuracy), valve definition and operation, and network demand variations. Relatively small changes in any of these variables can have a major effect on the way in which surge waves are generated and propagated throughout the network by the model.

The approach to calibration of a dynamic model therefore needs to be different to that normally employed on a steady state model. There are additional considerations when attempting to

generate a match between modelled and measured results for a transient model; for example, the shape of the pressure curve needs to be equivalent. Also, the pipe topography, material, size and physical condition and the character and behaviour of each of the dynamic assets such as pumps and valves affect the rate of damping of the initial pressure change and the superimposition of reflected pressure waves on the original wave, amongst other factors.

As a pipe's celerity value is a function of a number of factors including material, wall thickness, bedding, joint flexibility and proportion of entrained air, then adjusting the celerity is a useful method for matching simulated model results to measured values when one or more of the above factors is not accurately known.

In order to make result presentation simpler, the sub network containing the booster pump was divided into several areas. Figure 6.7 shows the location of each of the areas.

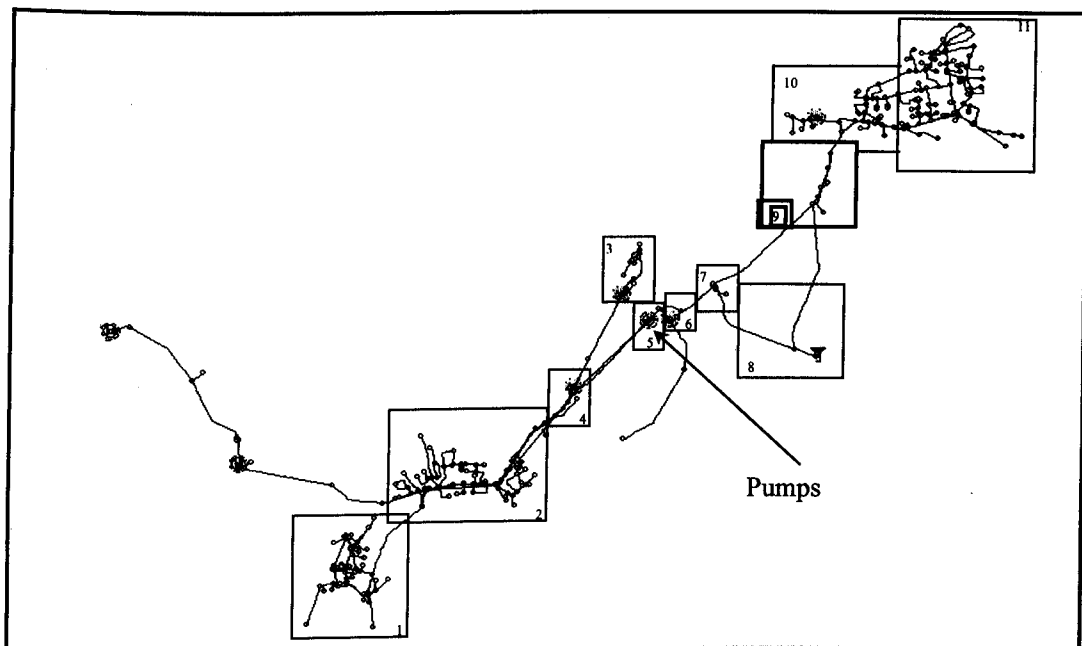


Figure 6.7 Location of the areas for surge analysis

A question to be answered concerns 'what is an acceptable level of calibration for a transient model?' Experience gleaned from the use of steady state hydraulic models has led to a typical standard of $\pm 1\text{m}$ for pressure and $\pm 10\%$ for flow.

However, for transient models there are no such precedents and hence the calibration procedure proceeded by changing variables on a trial and error basis.

6.3 Results

Figures 6.8 to 6.14 show the predicted variations in pressure, following the pump switching off, for a selection of the areas listed above. Each plot shows the pressure profiles for a number of pipes within an area.

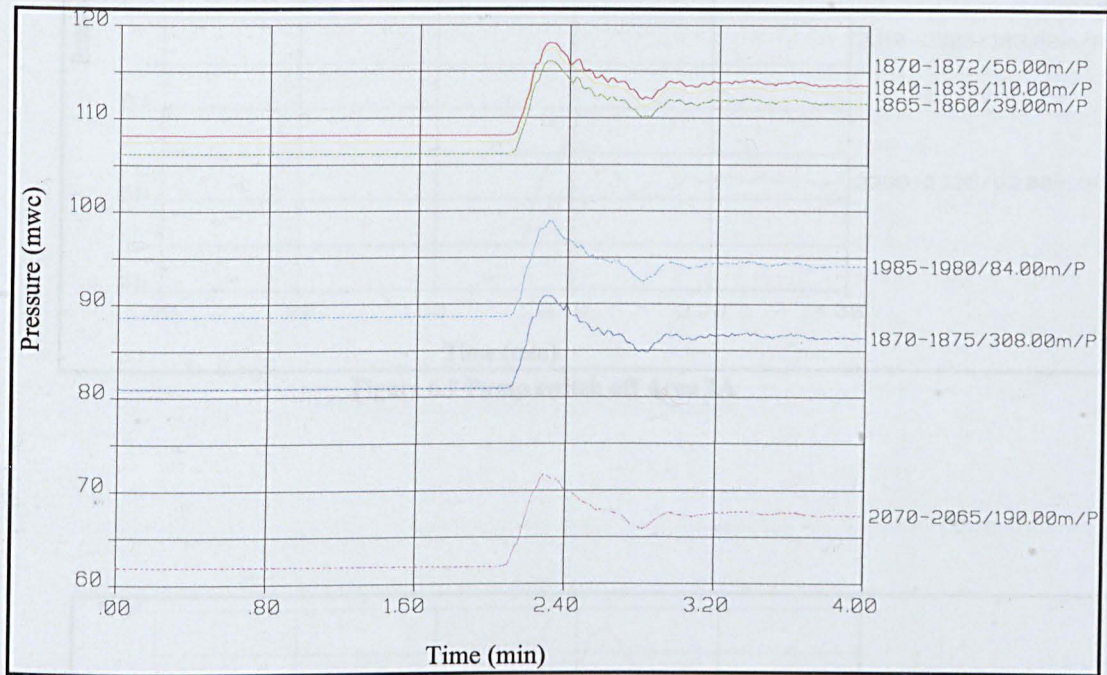


Figure 6.8 Pump switch off Area 1

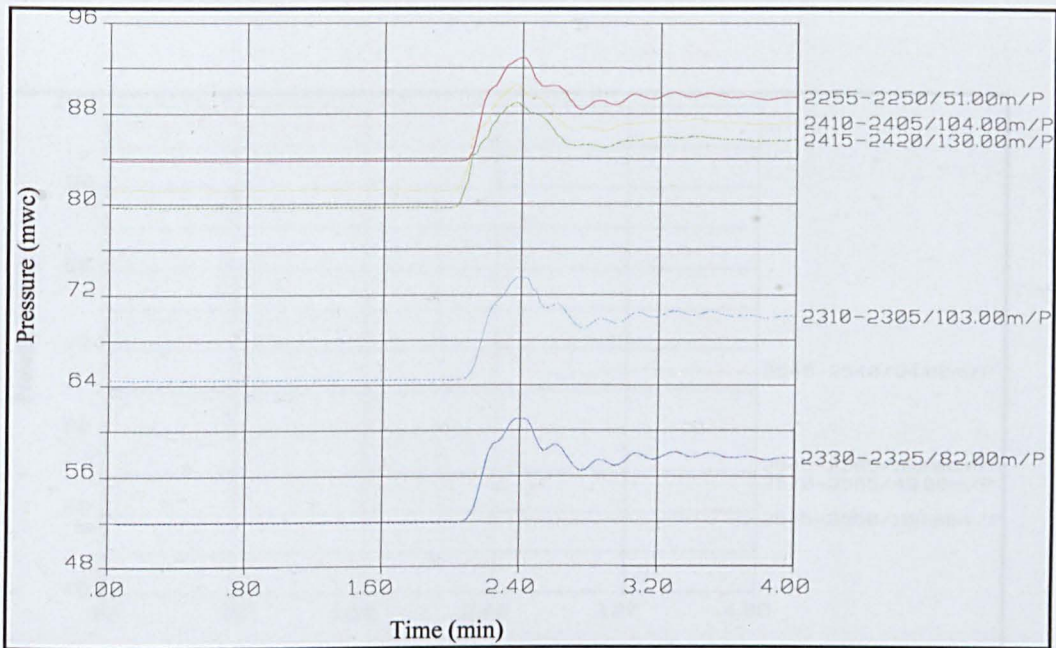


Figure 6.9 Pump switch off Area 2A

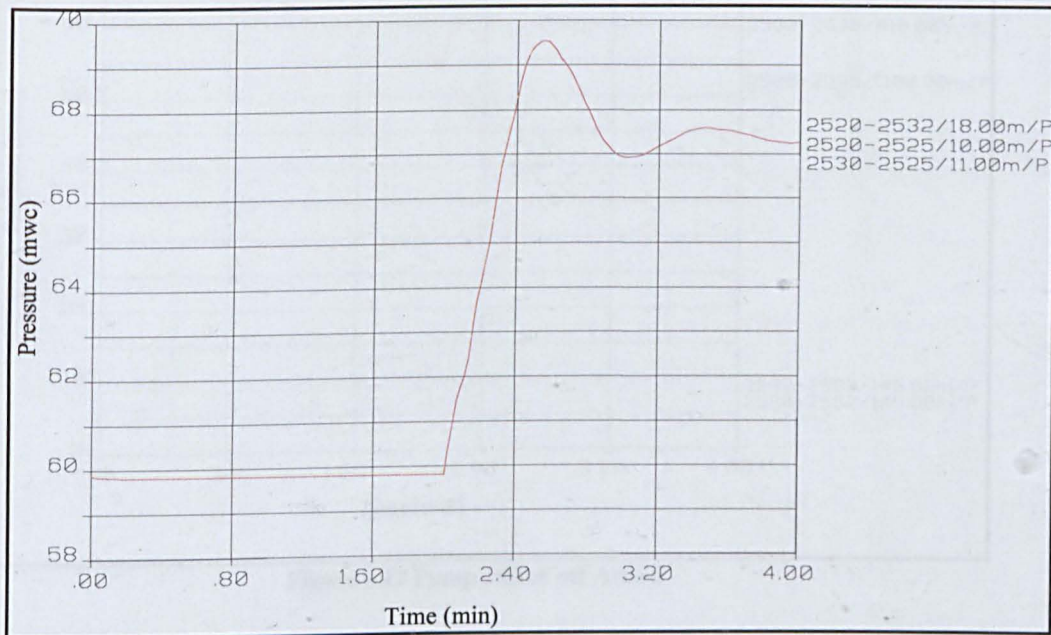


Figure 6.10 Pump switch off Area 5

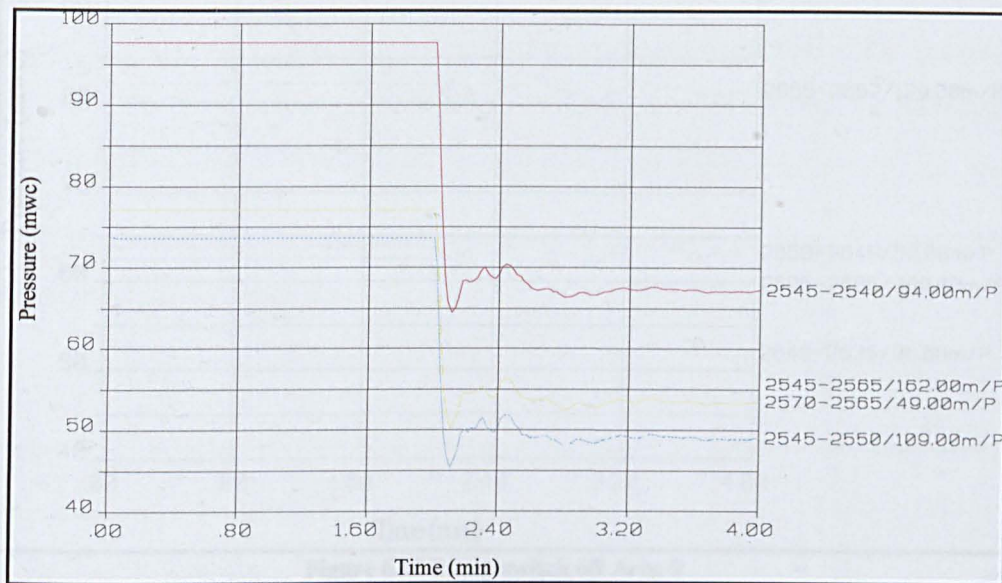


Figure 6.11 Pump switch off Area 6



Figure 6.12 Pump switch off Area 8

The largest magnitude pressure changes were predicted to occur in area 6, immediately downstream of the booster. Here, the pressure dropped from 97 mwc to 65 mwc in approximately 10 seconds. Further downstream from the booster in areas 8, 9, and 11, the magnitude of the



Figure 6.13 Pump switch off Area 9

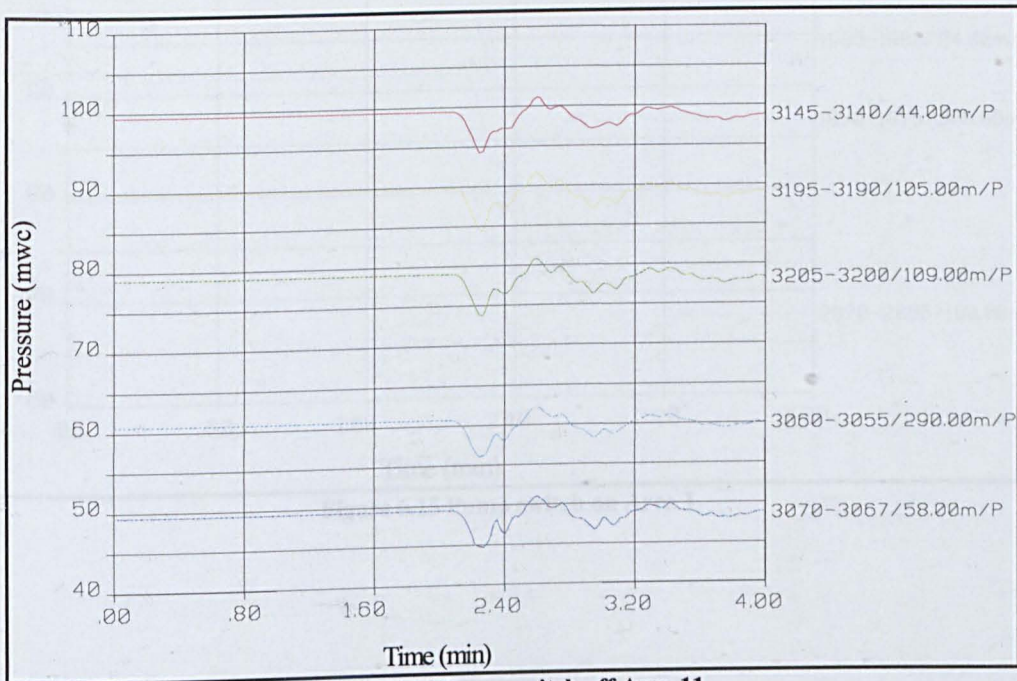


Figure 6.14 Pump switch off Area 11

The largest magnitude pressure changes were predicted to occur in area 6, immediately downstream of the booster. Here, the pressure dropped from 97 mwc to 65 mwc in approximately 10 seconds. Further downstream from the booster in areas 8, 9, and 11, the magnitude of the

pressure variation was smaller, 10 mwc or less, due to the pressure wave being dissipated by the pipe work and the damping effect of the service reservoir.

Upstream of the booster, a pressure variation of between 10 mwc and 12 mwc was observed in areas 5, 1 and 2. The pressure was approximately 5 mwc higher at the end of the simulation than at the start, this is due to there being less head loss in the trunk main between the service reservoir and the booster station when the pump is off than when it is operating.

Similar plots were generated to demonstrate the effect of the pump starting. Figures 6.15 to 6.21 show the predicted pressure time series.

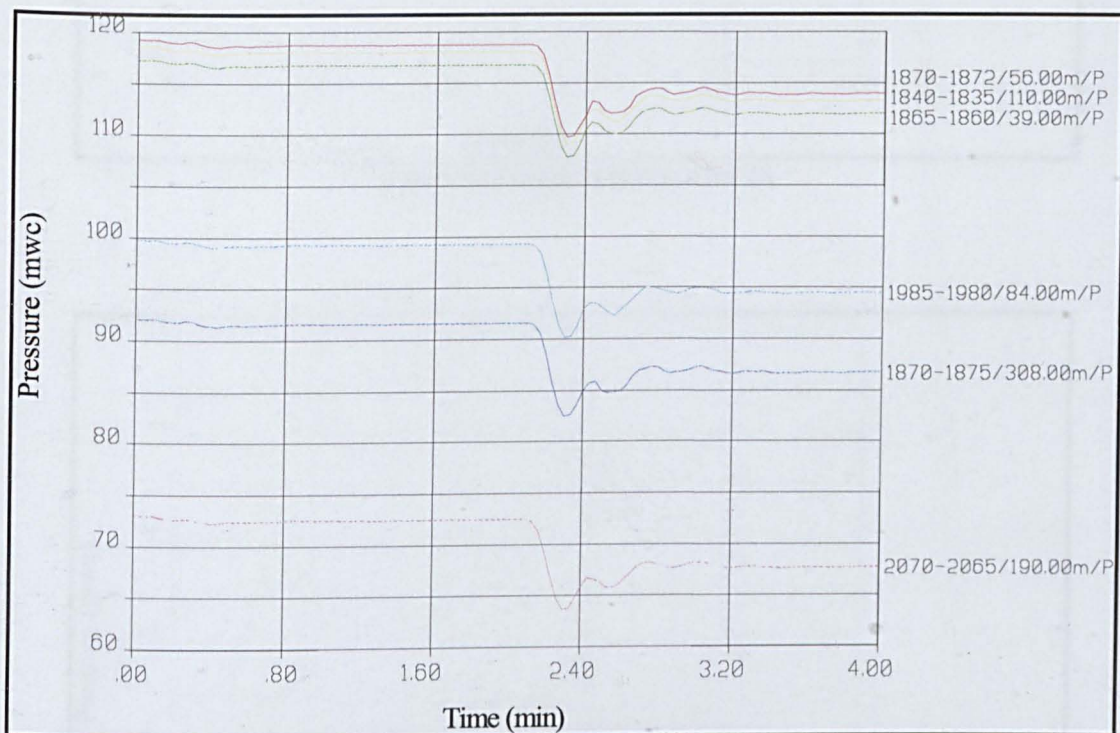


Figure 6.15 Pump switch on Area 1

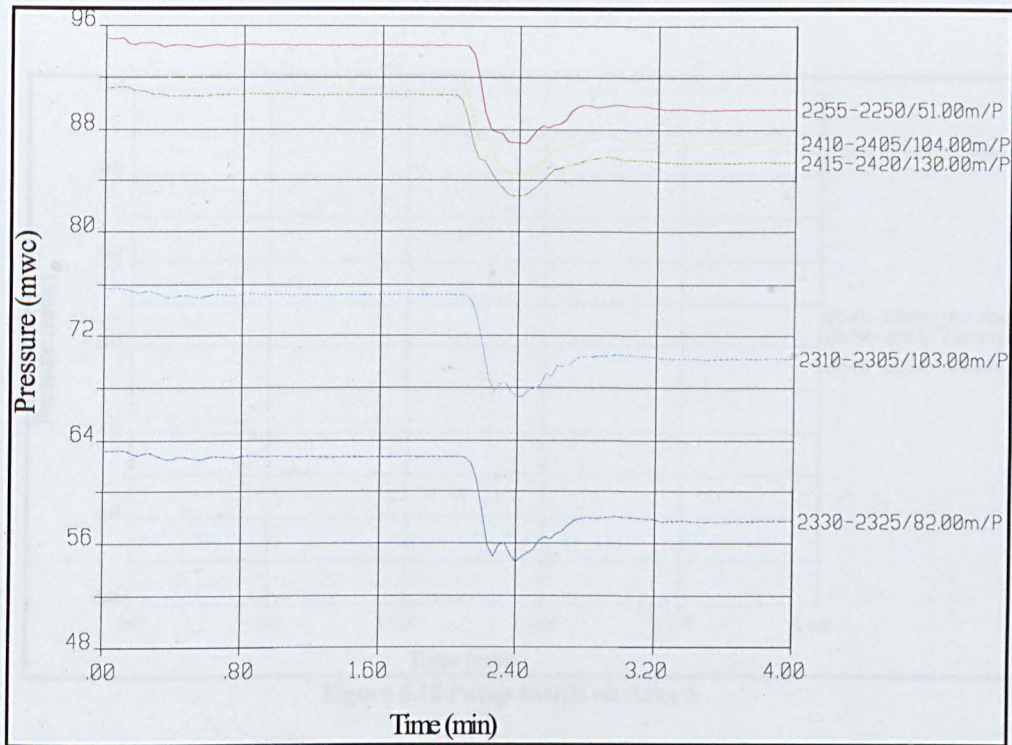


Figure 6.16 Pump switch on Area 2A

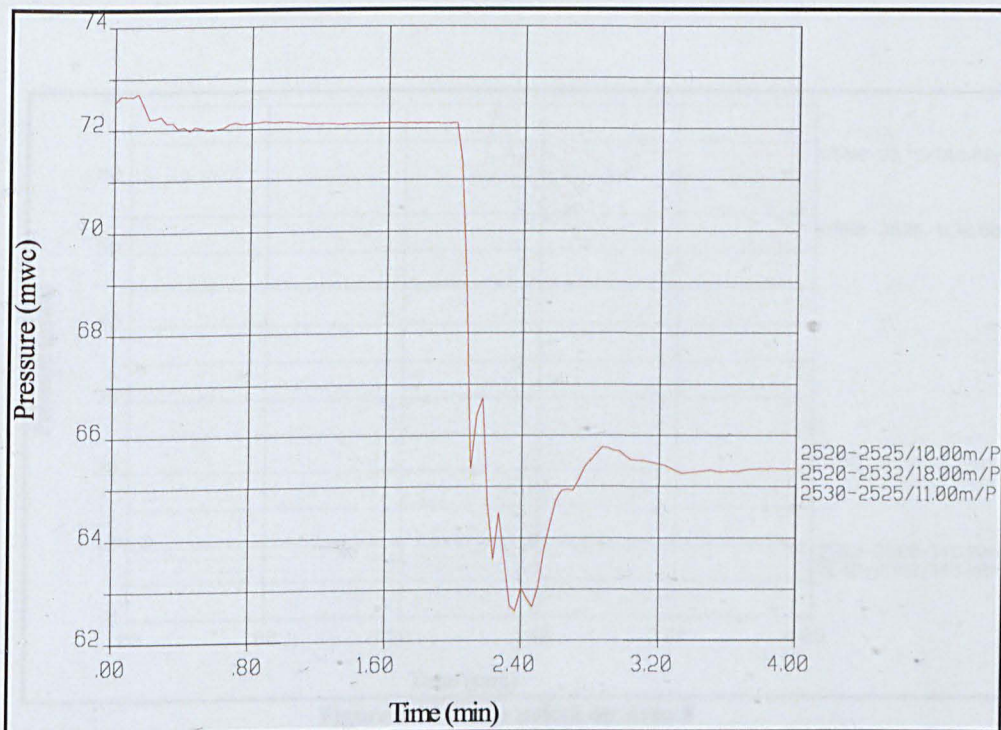


Figure 6.17 Pump switch on Area 5

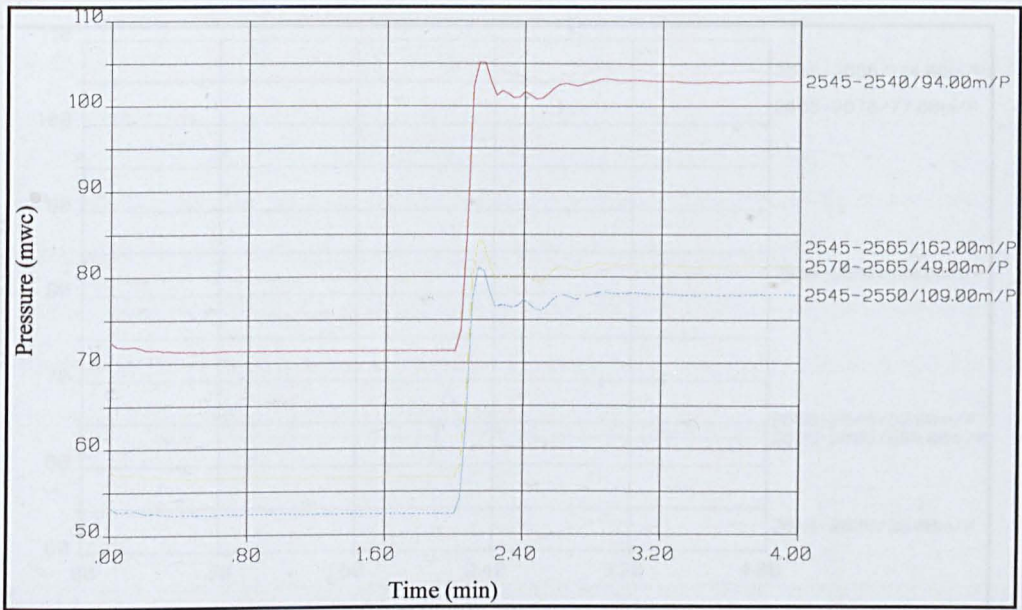


Figure 6.18 Pump switch on Area 6

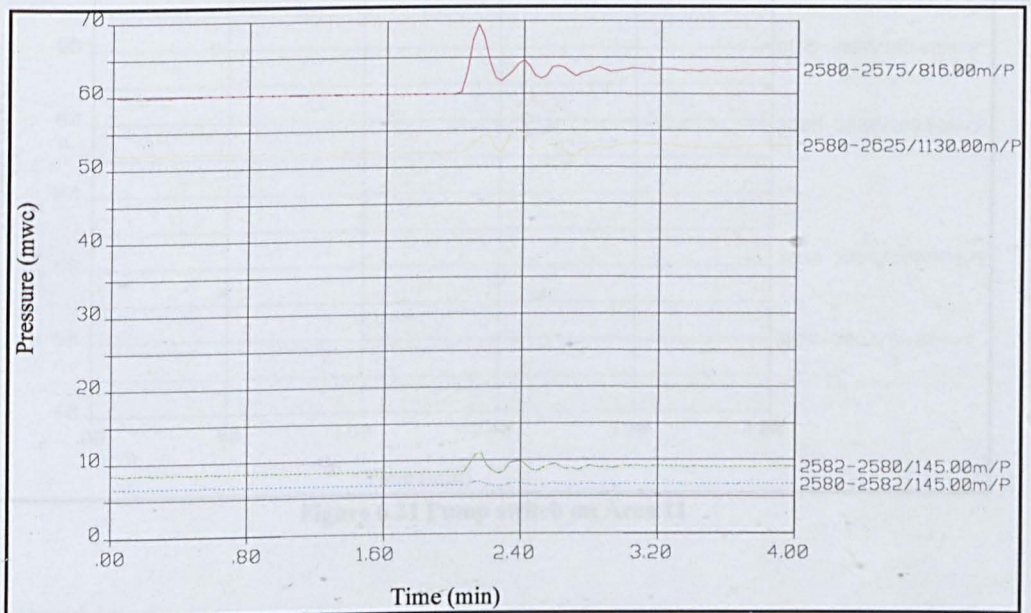


Figure 6.19 Pump switch on Area 8

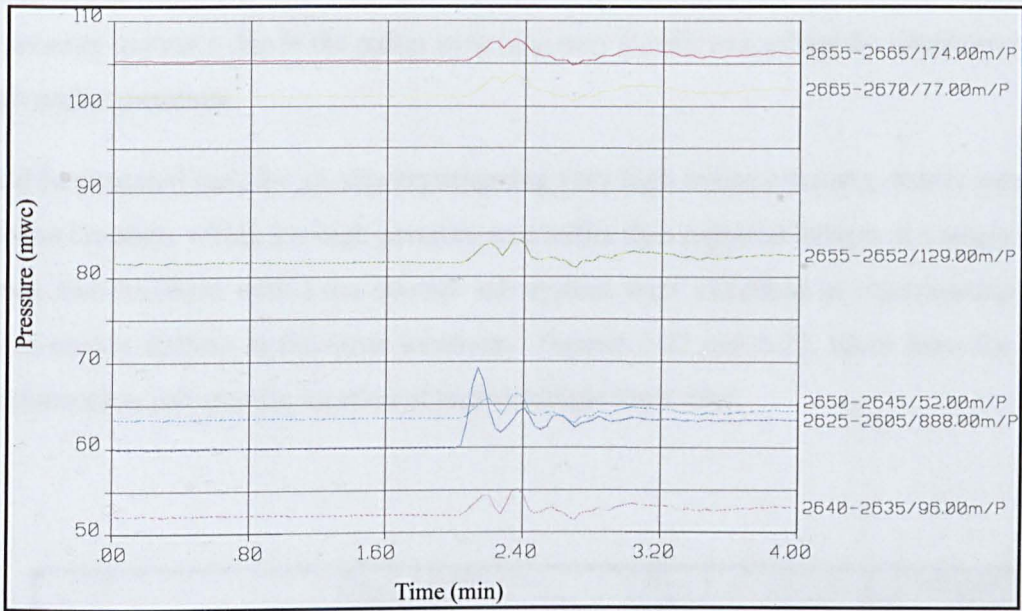


Figure 6.20 Pump switch on Area 9

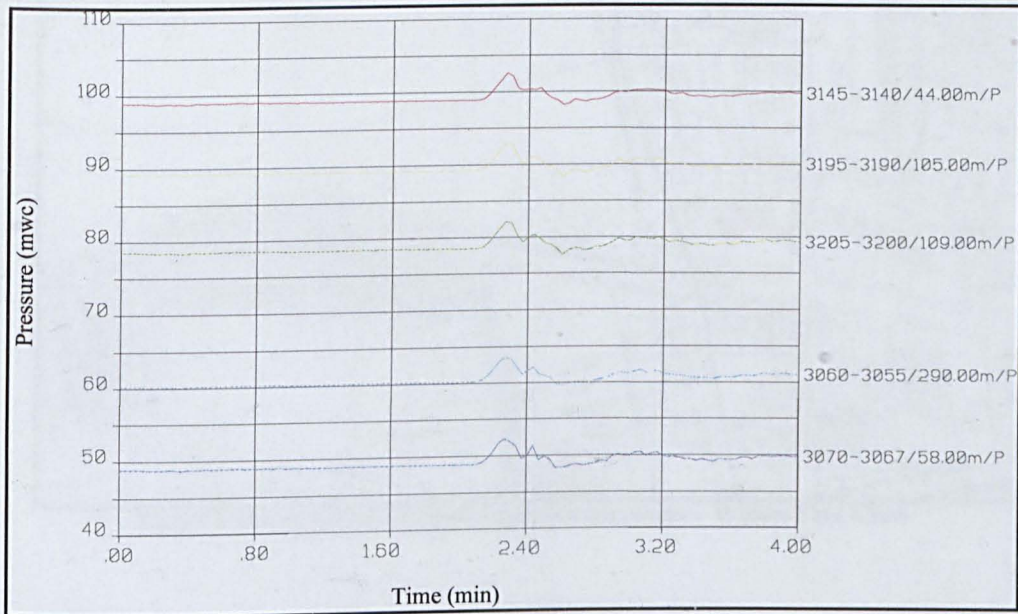


Figure 6.21 Pump switch on Area 11

From this plot it is clear that the surge effects are minimal and it would appear that the bursts might be due purely to the very high mains pressures occurring at these locations. The relatively small pressure increases due to the pump switching may simply exacerbate the situation created by the high mains pressures.

It would be expected that, for an area experiencing very high mains pressures, bursts would occur at different locations within the high-pressure area rather than repeated failures at a single location. However, two locations within the booster sub system were identified as experiencing multiple mains or service failures in the same locations. Figures 6.22 and 6.23, taken from the maps of burst information, indicate the location of these multiple burst sites.

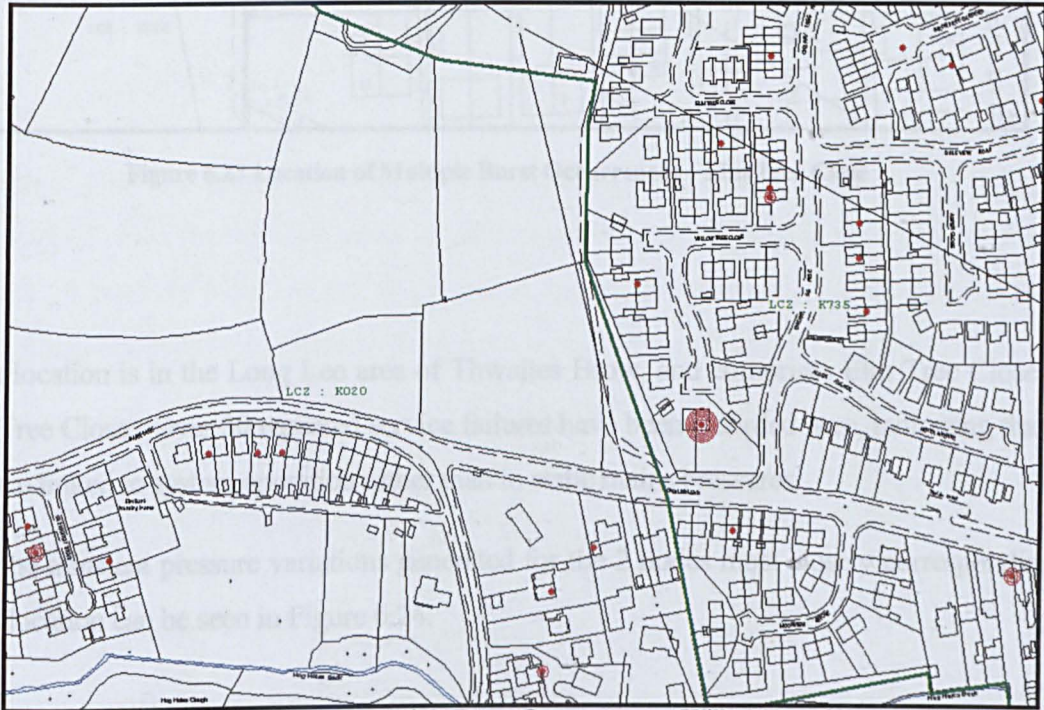


Figure 6.22 Location of Multiple Burst Occurrences – Willow Tree Close

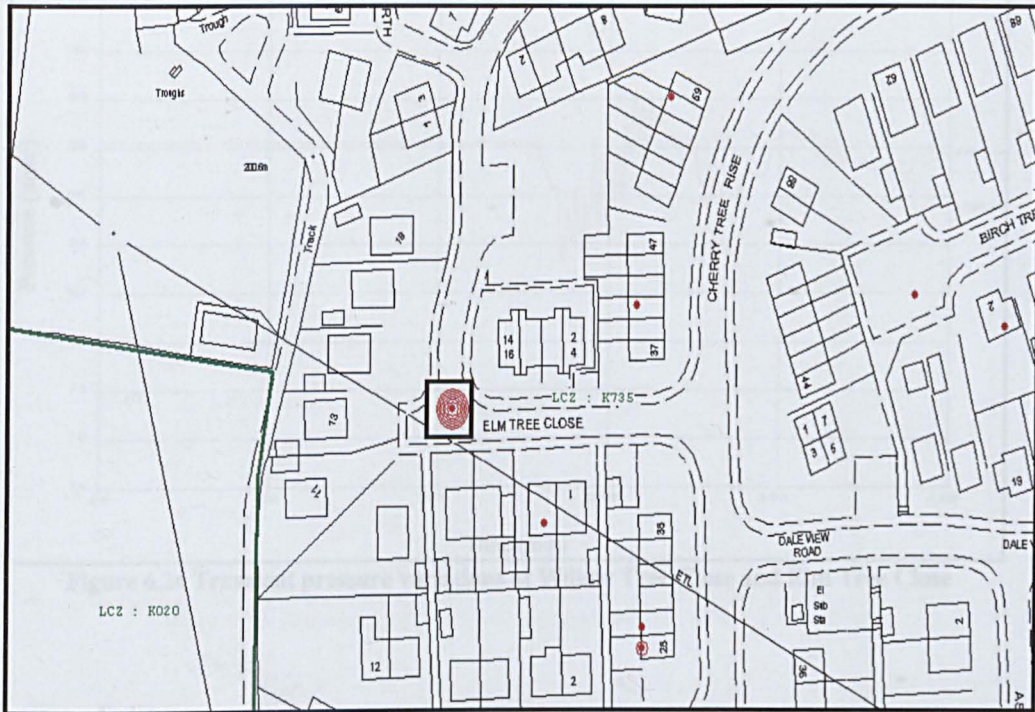


Figure 6.23 Location of Multiple Burst Occurrences – Elm Tree Close

The first location is in the Long Lee area of Thwaites Brow, and comprises Elm Tree Close and Willow Tree Close. Over 30 repeated service failures have been recorded here, indicating that the failures are linked to system operation rather than to static mains pressures.

Plots of the transient pressure variations generated for the 2 nodes most closely corresponding to the burst location can be seen in Figure 6.24.

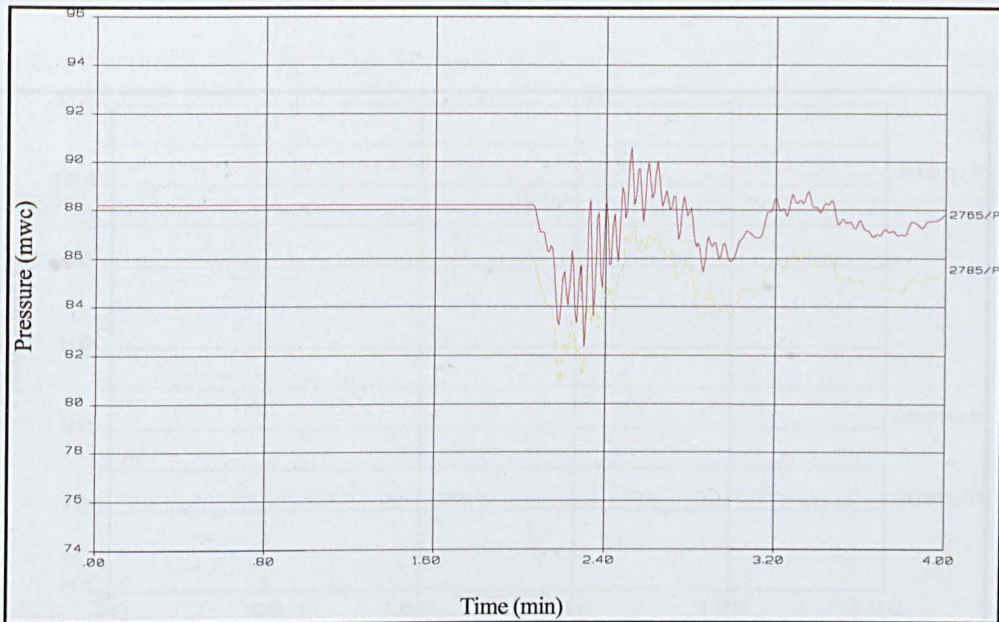


Figure 6.24 Transient pressure variations at Willow Tree Close and Elm Tree Close

It is apparent from the plot that, although the amplitude of the pressure change is not great, approximately 8 mwc, the pressure traces display a significant amount of high frequency pressure fluctuation. It is likely therefore that the repeated sudden pressure changes at these nodes are causing unusual and repeated stresses in the service pipes thereby contributing to their failure.

In contrast, plots for other nodes located a short distance away display a significantly reduced amount of high frequency pressure variation. Plots for three of the nodes can be seen in Figure 6.25.

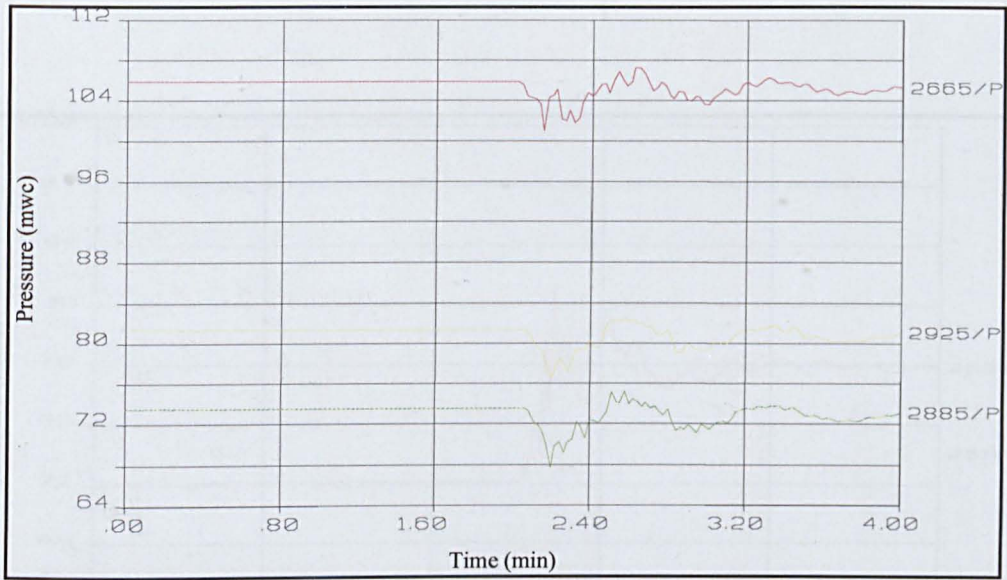


Figure 6.25 Transient pressure variations nearby Willow Tree Close

The second location was upstream of the booster near its junction with Lingfield Drive. Pressure variations at this location were generated and can be seen in Figure 6.26.



Figure 6.26 Transient pressure variations at Lingfield Drive

These plots show that significant, high frequency, pressure changes are occurring at these nodes. However, plots of the pressure variations at nodes 2390 and 2510, located relatively short

distances upstream and downstream of the location of the repeated mains failure, demonstrate a far lower frequency of variation, as shown in Figure 6.27.

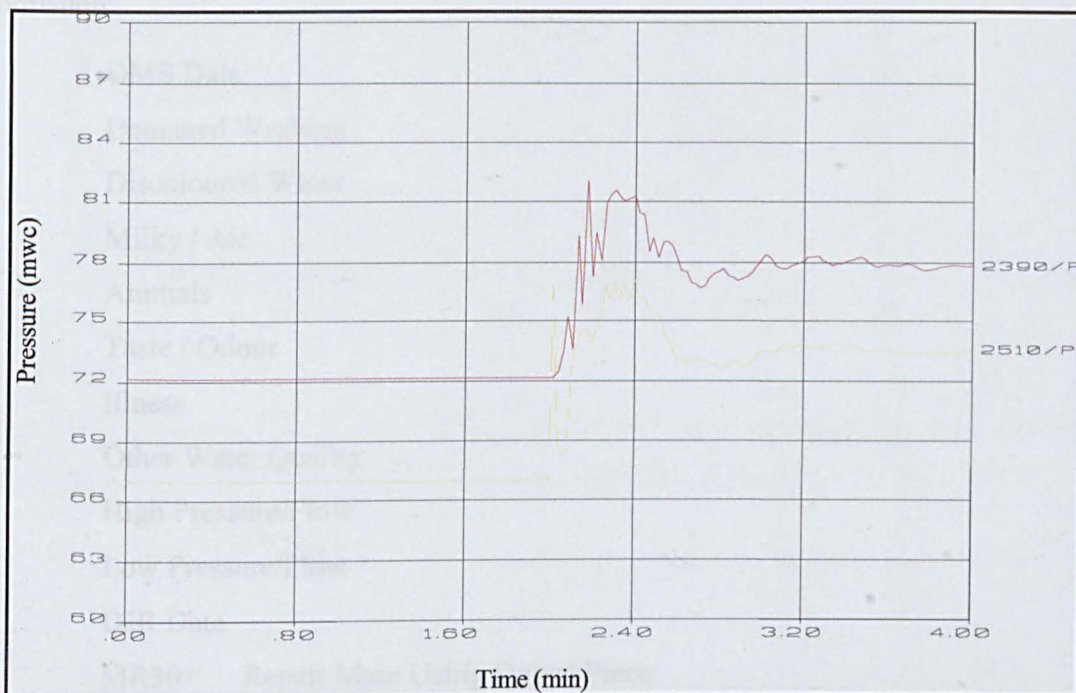


Figure 6.27 Plots of the pressure variations nearby Lingfield Drive

It appears that localised pressure wave reflection is taking place and in some locations these reflections are constructively interfering to produce the high frequency fluctuations observed at both multiple burst locations.

6.4 Correlation with bursts and water quality events

Once the dynamic model had been calibrated, the modelled surge pressure variations and frequencies at specific locations were compared against pipe burst and water quality complaint information. This was done in order to determine if there was any observable relationship between the presence of transients, burst mains and water quality complaints.

The graphical representation of the burst and water quality data was only located at the street centroid, and this proved to be a problem where the street was long or if there were more than one main in the street. Where a zone boundary crossed a street, there was no way of determining

which of the recorded events applied to which zone. This was further complicated as zone boundaries may have changed in the thirty-six month period.

TRAMS overlays were created that display, in addition to the water network, the following information:

OMS Data

Damaged Washing

Discoloured Water

Milky / Air

Animals

Taste / Odour

Illness

Other Water Quality

High Pressure/Flow

Low Pressure/Flow

DJR Data

MR30 Repair Main Using Dowel Piece

MR35 Repair Main – Other

MR39 Repair Major Burst

SE30 Repair Existing Service Pipe

SE35 Repair Leak in Chamber

The above data were displayed as points on a map background of the study network (at best to the street centroid). Figure 6.28 is an example of burst data plotted over a background map highlighting “clusters” of bursts

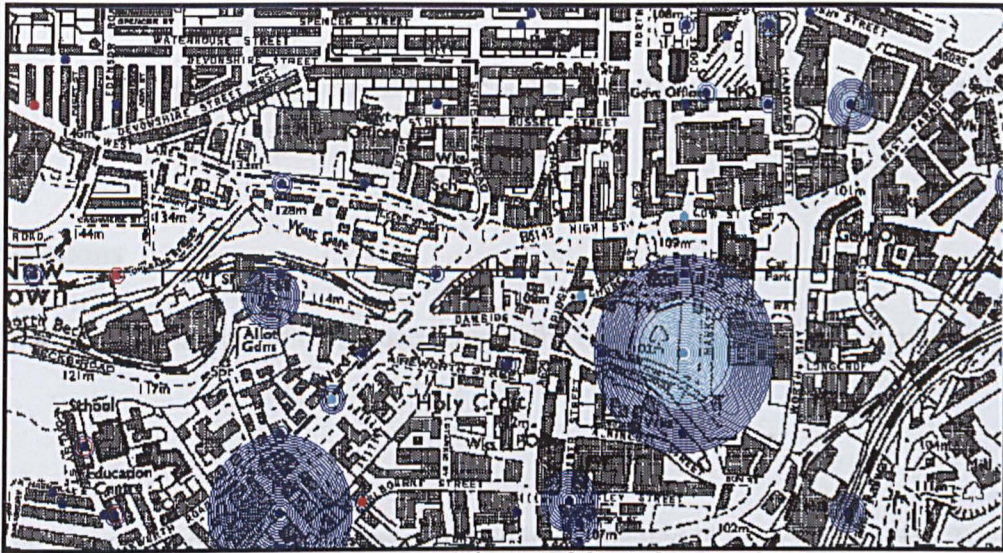


Figure 6.28 Burst events in part of the study network

Data was obtained as an MS-Access text data file created using an application that lists, based on Water Supply Zone, Post Code and LCZ, the following information:

- OMS Discolouration Complaints
- OMS Low Pressure Complaints
- OMS Taste/Odour Complaints
- OMS Burst Incidents
- DJR Mains Repairs

This data contained the actual address where the complaint(s) / incident(s) occurred, and it was this data that was used for correlating the recorded events to surge pressures and frequencies.

Associating individual burst or WQ records to specific surge events proved to be a problem because there was no way of knowing exactly when a burst or WQ incident actually occurred. The recorded incident time is dependent upon the time that the event was noticed, and the time the problem was recorded.

Given the above, in order to correlate surge effects with bursts and water quality events, the total number of burst and water quality incidents within the modelled area were identified. Then, where surge effects had been confirmed by logging and or modelling, and where a higher than expected number of burst / dirty water incidents had occurred, the surge effects were assumed to be the cause.

Figure 6.29 shows the pressure variations for the nodes corresponding to these burst locations.

6.5 Solutions

In order to reduce the number of bursts occurring in the booster sub network, two measures can be taken.

- 1 The pressure reduction schemes could be implemented (Chapter 5, Section ?). By implementing the pressure reduction schemes, the stress placed upon the mains and service pipes will be considerably reduced and hence they will be less likely to fail under the added pressure fluctuations created by the pump switching.
- 2 In order to reduce the surge waves resulting from the pump switching, it is suggested that a 'soft start / stop' mechanism is fitted to the booster pump. This will have the effect of slowing the rate of change of the pump speed as it starts up or stops and in turn this will reduce the high frequency pressure transients.

A simulation was carried out with the pump defined to start up from zero revolutions to normal operating revolutions over a period of 30 seconds. The results of the pressure variations caused by such a soft start have been plotted for the locations where previously the high frequency pressure variations were observed. As can be seen from Figures 6.29 and 6.30 there is no longer any evidence of the high frequency variations.

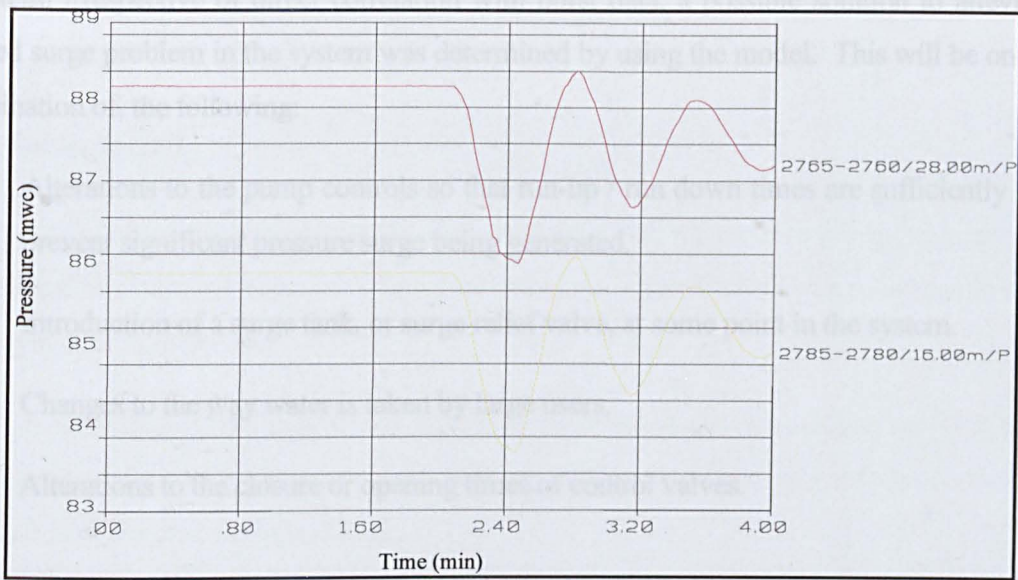


Figure 6.29 Pressure variation at multiple burst site after introduction of soft start pump

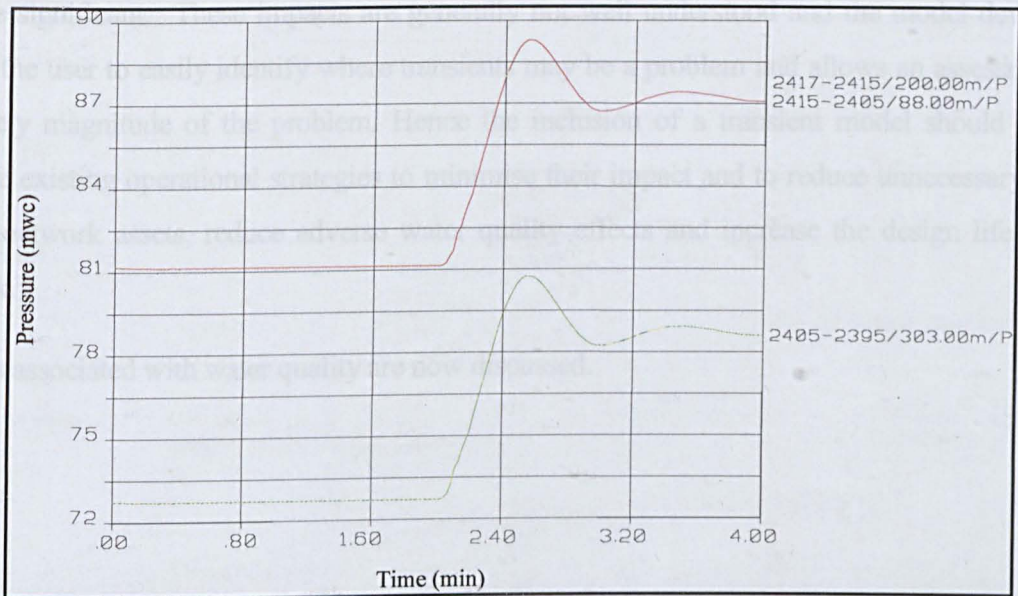


Figure 6.30 Pressure variation at multiple service pipe burst site after introduction of soft start pump

For those zones where the study has indicated that significant transient effects are present within the system, irrespective of direct correlation with burst data, a possible solution to alleviate the observed surge problem in the system was determined by using the model. This will be one of, or a combination of, the following:

Alterations to the pump controls so that run-up / run down times are sufficiently long to prevent significant pressure surge being generated.

Introduction of a surge tank, or surge relief valve, at some point in the system.

Changes to the way water is taken by large users.

Alterations to the closure or opening times of control valves.

6.6 Summary Remarks

It is clear that the application of the transient model has demonstrated that the impact of transients may be significant. These impacts are generally not well understood and the model developed allows the user to easily identify where transients may be a problem and allows an assessment of the likely magnitude of the problem. Hence the inclusion of a transient model should greatly enhance existing operational strategies to minimise their impact and to reduce unnecessary stress on the network assets, reduce adverse water quality effects and increase the design life of the network.

Aspects associated with water quality are now discussed.

Chapter 7 - Water Quality Analysis

7.1 Background

This section of the thesis describes the developments that were made in order to produce a mathematical model to determine the spatial and temporal concentration of a substance in any part of a water distribution network

Figure 7.1 again highlights the complex nature of the some of the interactions between network asset characteristics, water chemistry and biology and some of the physical properties of the materials within the distribution network.

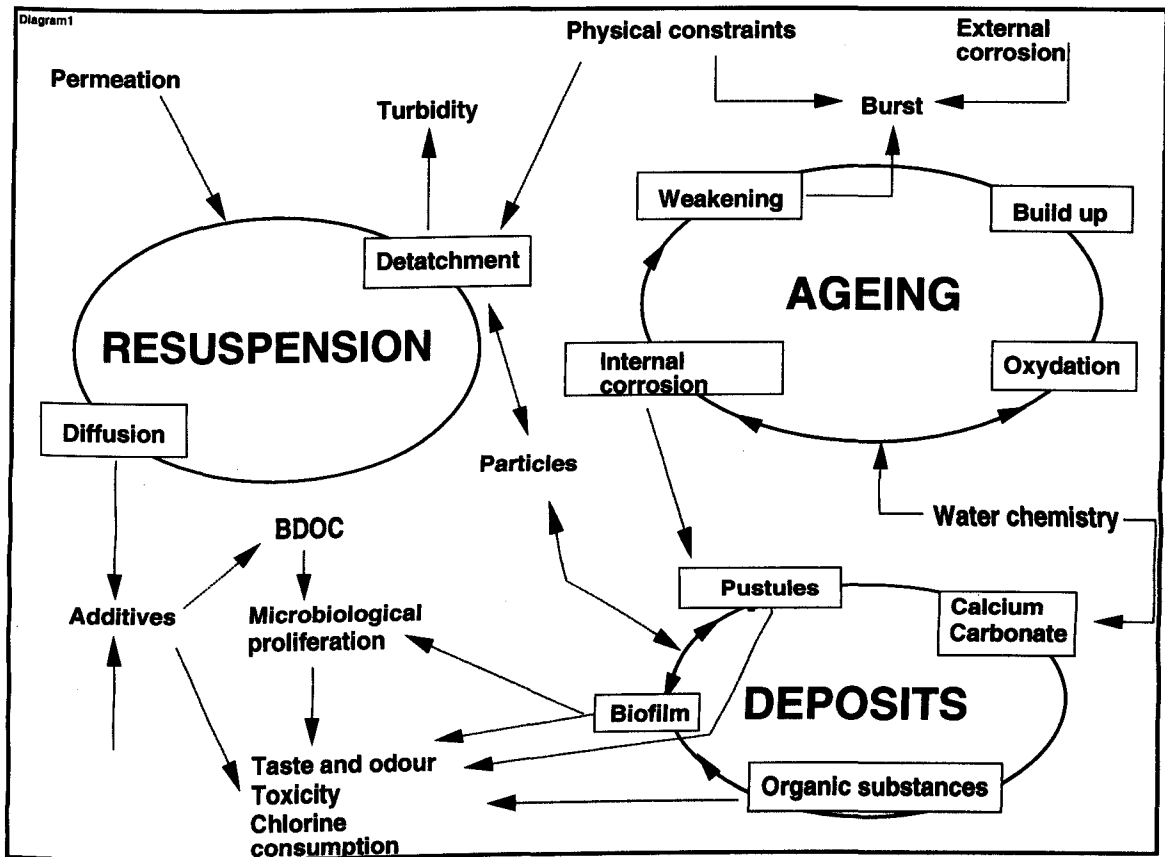


Figure 7.1. Some physical, chemical and biological interactions within a pipe

A better understanding of these processes may result in better operational practice. For example, should a particular water treatment process fail and allow unsatisfactory water to enter the distribution network, it is then possible to predict which consumers would receive the unacceptable water and when. Action may then be taken to prevent the customers being subject to

the poor quality water by the implementation of an appropriate control, or operational strategy, to maintain adequate quality at the consumers' property(s).

The same philosophy would apply to discoloured or turbid water generated as a result of operational changes that might, for example, have disturbed sediment accumulated in the pipes over many months or even years. Similarly, discoloured water generated because of corrosion mechanisms or any other phenomenon may also be traced as it travels through a network. Also, substances utilised to stimulate a chemical or physical process, e.g. the use of phosphate for the sequestering of iron, it is possible to determine where and at what concentration the phosphate should be introduced into the network to achieve the desired dose at all locations. By modelling propagation in this manner, it is possible to determine the optimal location for the introduction of remedial chemicals. This alleviates the problem of dosing large amounts of chemical in order to achieve a given minimum concentration in one part of the network while customers in other areas are overdosed. (In the case of fluoride, the concentration would be limited to 1.0 mg.l^{-1} by regulation). Dependant on the topography of the network, it may be possible to introduce an optimum dose at two or three key locations within the network to achieve a homogeneous concentration throughout rather than rely on a single source such as a water treatment plant. This approach is particularly important if the network has more than one source of supply because of the resulting dilution effects, or if there are exports from the network that result in changes in the boundary of mixing between different sources.

If a substance such as nitrate has a source concentration in excess of that recommended by the current legislation, propagation modelling becomes a tool for blending calculations. Work of this kind has resulted in resources that had previously been condemned being re-instated by blending with other, low nitrate supplies. As well as promoting re-commissioning of abandoned resources, this approach can save millions of pounds that would have been spent on engineering schemes designed to bring alternative supplies to the areas affected.

The benefits of such a model are clear. This chapter describes the development of a model to predict water quantity, and to demonstrate its applicability it has been applied to a study network in which a hypothetical incident where polluting material enters the Service Reservoirs feeding the network was simulated and in order to compare contingency plans.

To model the concentration and transport of a substance, it was necessary to fully understand the hydraulic characteristics of the network. This information was obtained from the output file of a

hydraulic simulation of the network, as this provided the essential details of network connectivity and the velocity and direction of flow in each pipe at all simulation time-steps.

7.2 Basic Water Quality Equations

7.2.1 Background

The basis of the 'substance propagation' model has followed the conventional approach as reported in the literature. A number of refinements and additional functions have been developed to improve model performance, results presentation and usability.

7.2.2 The Basic Water Quality Equation

The concentration of a substance $C(x,t)$ may be given by equation 7.1:

$$dC = \frac{\partial C}{\partial t} dt + \frac{\partial C}{\partial x} dx \quad (7.1)$$

Dividing (1) by dt gives:

$$\frac{dC}{dt} = \frac{\partial C}{\partial t} + \frac{\partial C}{\partial x} \frac{dx}{dt} \quad (7.2)$$

For a water particle flowing in the pipe the term dx / dt is V , the velocity, hence

$$\frac{dC}{dt} = \frac{\partial C}{\partial t} + V \frac{\partial C}{\partial x} \quad (7.3)$$

Where:

V is the velocity of water.

C is the concentration

x is position

t is time

k is the decay rate constant

If the change of concentration is a function of the concentration itself then:

$$\frac{dC}{dt} = -k C^n \quad (7.4)$$

Equation (4) is solved by integration. For exponent n equal to 0 and 1 respectively:

$$n = 0: C(t) = C(t_0) - k(t - t_0) \quad (7.5)$$

$$n = 1: C(t) = C(t_0)e^{-k \cdot (t - t_0)} \quad (7.6)$$

Where:

t is the actual time (s)

t₀ is the latest reference time (s), corresponding to a known / calculated concentration.

These solutions are only valid for particles / substances flowing with the speed of water, i.e. when:

$$dx / dt = V \quad (7.7)$$

7.2.3 Numerical Solution

The numerical solution is based on equation (7.5) and (7.6) ensuring equation (7.7) is fulfilled.

The solution takes place on a position / time grid. Figure 7.2 shows a representation of the position-time grid.

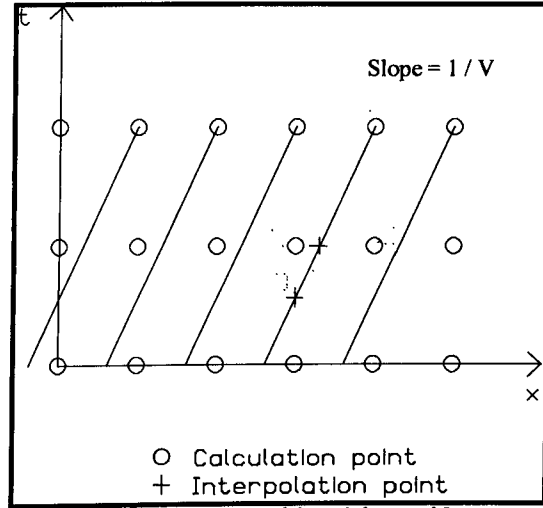


Figure 7.2 Position / time grid

Each pipe is subdivided in a number of calculation points, each of which is described by the position x (chainage [length of main] in meters).

A pipe in the simulation model is aligned with the x -axis, the upstream calculation point being located at $x = 0$, where:

Δx is user defined spacing between calculation points along the x -axis (m). The distance from point K to point J.

Δt is the time-steps between successive simulations (simulation time-step).

Distance between point J and point I.

V is numerical value of flow velocity of water (m/s).

The solution technique assumes that all concentrations are known at time t_0 (at point K and J). For a first order equation, the solution is defined by equation 7.6 along the slope line, therefore the concentration at time $t + dt$ can be calculated for point I.

The concentration in point A is calculated via interpolation between concentrations at points K and J. The concentration at $x = 0$, and $t = 0$, must be defined as a boundary condition in a node or calculated via a mixing formula from upstream pipes.

The model must have defined starting values from which to work. In this case, these are concentrations at a location at a time zero, or they can be calculated from information upstream of the zone inlet nodes if available, or from sub-net nodes. The user defines the time step between each simulation.

Depending on the actual conditions and selected values of Δt and Δx , the relation $\Delta t / \Delta x$ can either be greater or less than the velocity. If $\Delta x / \Delta t < V$, Figure 7.3, interpolation is made at point A.

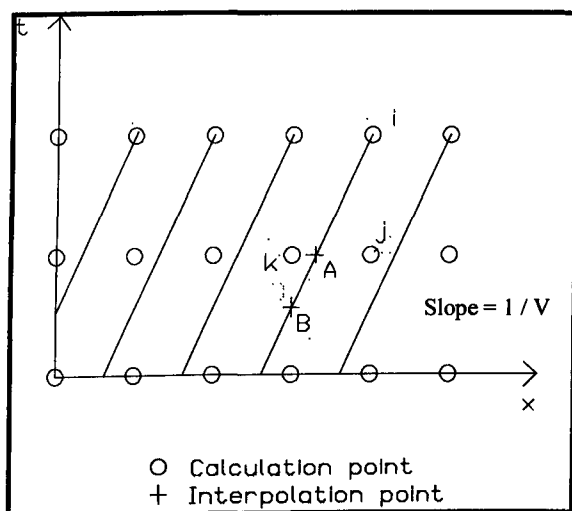


Figure 7.3 Position-time grid for $\Delta x / \Delta t < V$

If $\Delta x / \Delta t > V$, Figure 7.4, interpolation is made either in space, interpolation point A, or in time, interpolation point B.

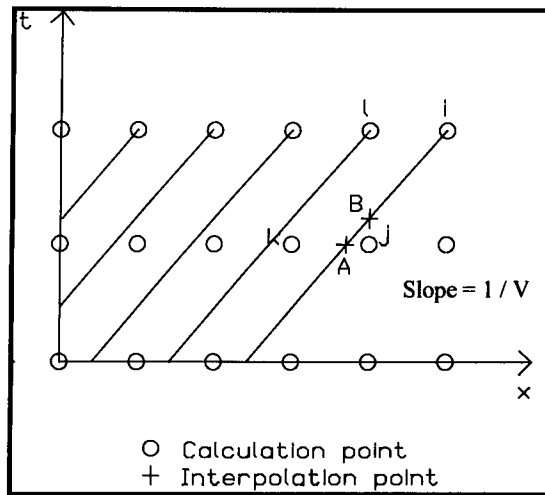


Figure 7.4 Position-time grid for $\Delta x / \Delta t > V$

In the latter case, the optimum choice is made automatically by the programme at each time step considers maximum adaption, i.e. a qualitative measure for the relative amount of interpolation, where adaption is given by:

$$ADAPTION = \left(1 - \frac{\sigma}{(N - N_p)^{0.5}}\right) 100\% \quad (7.8)$$

Where:

N is the total number of calculation points at a time.

N_p is the number of pipes.

σ is a function of the relative amount of interpolation made:

$$\sigma = (\sum r_i^2)^{0.5} \quad (7.9)$$

Where:

r_i is the distance between the actual interpolation point and the nearest (i 'th) calculation point relative to spacing dx .

It follows by definition that the adaption is within the interval from 50% to 100%.

7.3 Substance Propagation

7.3.1 Background

For conservative substances, for example Nitrate or Fluoride, it is possible, knowing the concentration and load of the substance and its point of entry into the distribution network, to predict where and at what concentration the substance will be in relation to individual nodes and pipes with time. The basic equations were coded into the model such that the conservative substance may be propagated through the network. For example, Figures 7.5 and 7.6 shows how Nitrate, that enters the network at the supply service reservoir for a period of 2 hours between 8 and 10 am.

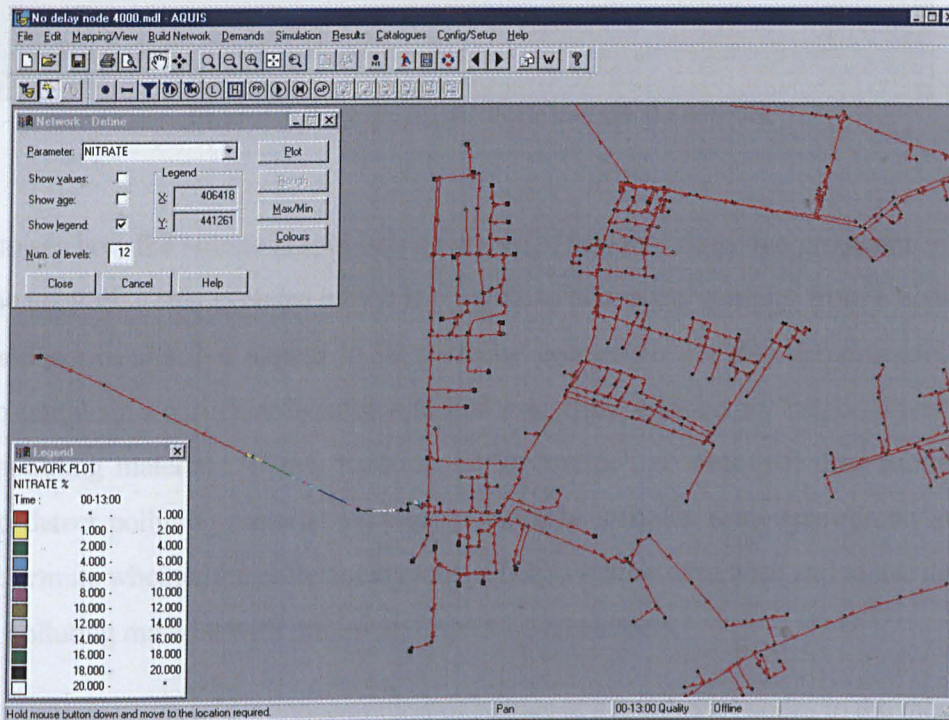


Figure 7.5 A slug of nitrate rich water entering the network

Figure 7.5 indicates how the slug has propagated after 16 hours.

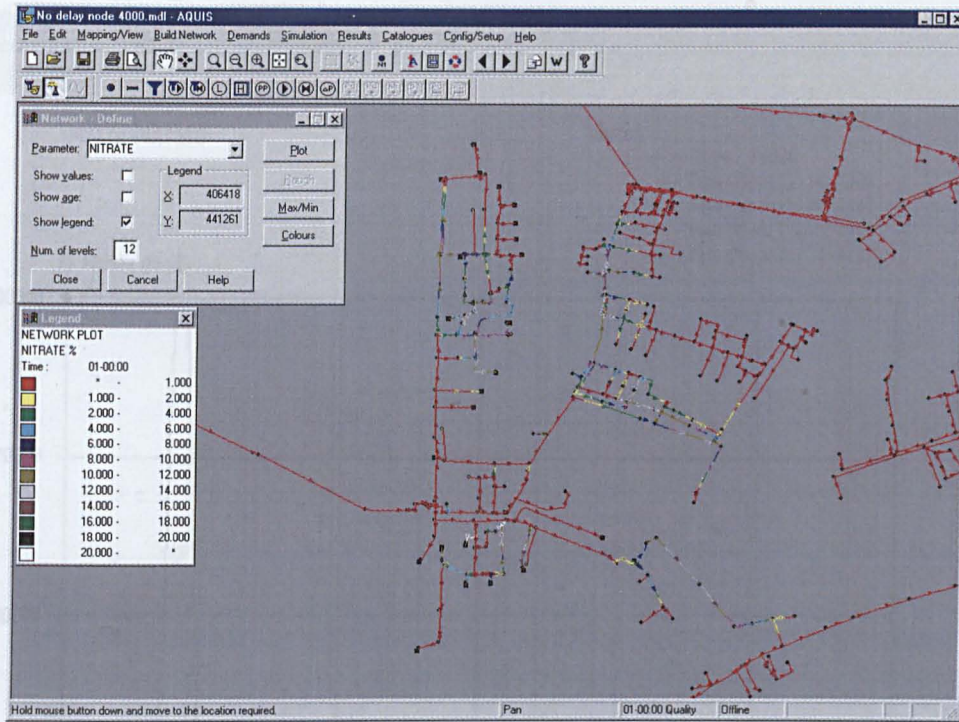


Figure 7.6 Propagation of Nitrate through the network

It is clear to see how the Nitrate is split into a number of separate slugs that propagate into separate areas of the network. This explains why it is possible to take water samples from a network that is polluted and get results that appear to be perfectly acceptable. It also demonstrates how it is possible to sample in a particular location and find everything satisfactory but, on repeat sampling, discover polluting material. If this functionality is 'online', or near real time modelling, it is possible to detect polluting material very early and take samples from appropriate locations in order to determine whether the pollutant presents a health risk or otherwise and to use the model to isolate the polluting material with minimum impact on consumers.

Figure 7.7 is a time series of the concentration of Nitrate at a number of nodes in the model.

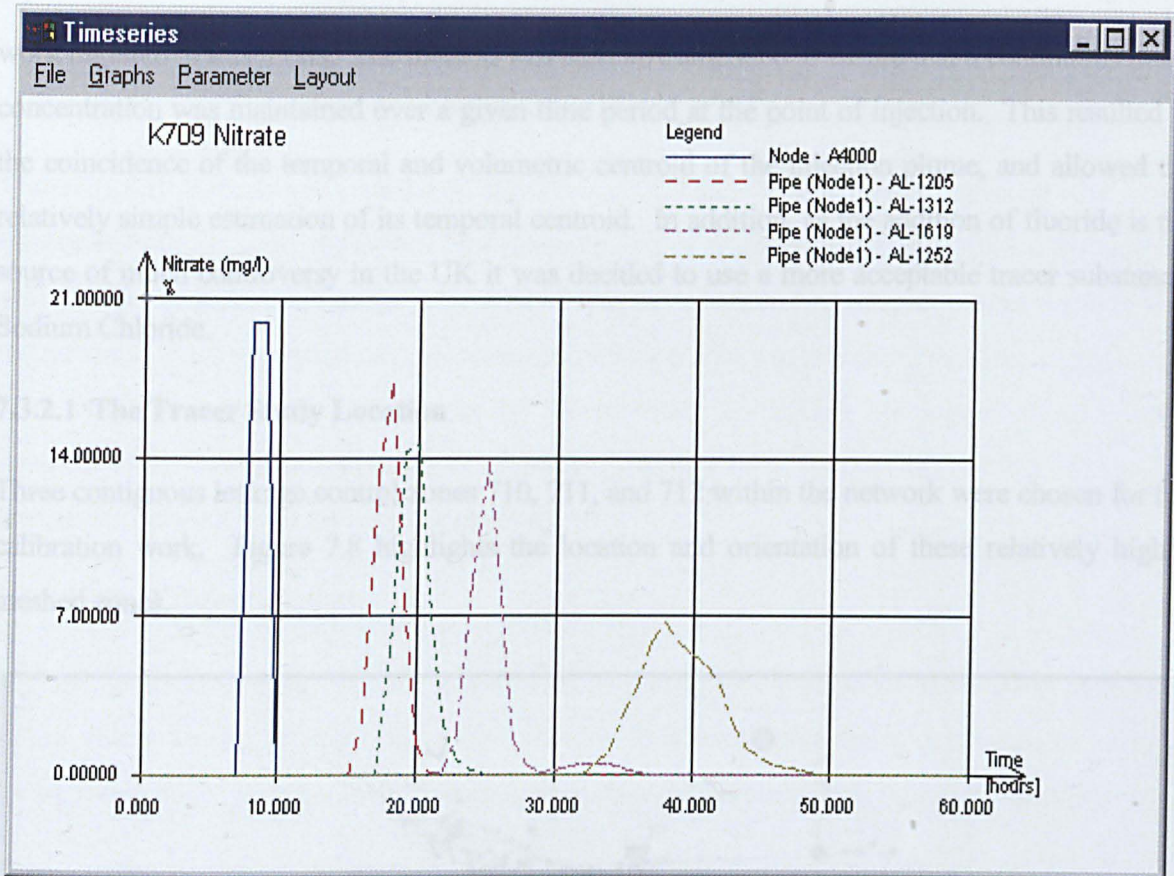


Figure 7.7 Time series of nitrate concentration at a number of nodes

It is clear from the diagram that 20 mg/l of Nitrate entered the network between 8 and 10 am (blue trace). The other traces are the resulting concentrations of Nitrate at nodes across the network moving away from the source. Dilution effects reduce the concentration and the profile is flattened due to dispersion effects with distance from source. This functionality was used to calibrate the age and the propagation models by introducing a tracer material (Sodium Chloride) into the network and measuring its time of arrival at a number of nodes across the network. In this way, it was possible to compare modelled against actual travel times and thereby obtain a calibrated model.

7.3.2. Model Calibration

The calibration methodology chosen was an adapted / enhanced version of a method tried in the USA. *Clark et al., and Skov et al., (1993)*, used fluoride as a tracer substance to measure travel times through a network to demonstrate the effects of storage in the network on water quality with a view to better design of storage, its location and management. The method provided data fit for purpose but because of data collection and tracer input methods was not accurate enough for the

work required in this thesis. The method was therefore amended to ensure that a continuous tracer concentration was maintained over a given time period at the point of injection. This resulted in the coincidence of the temporal and volumetric centroid of the injection plume, and allowed the relatively simple estimation of its temporal centroid. In addition, as the addition of fluoride is the source of much controversy in the UK it was decided to use a more acceptable tracer substance, Sodium Chloride.

7.3.2.1 The Tracer Study Location

Three contiguous leakage control zones 710, 711, and 712 within the network were chosen for the calibration work. Figure 7.8 highlights the location and orientation of these relatively highly meshed zones.

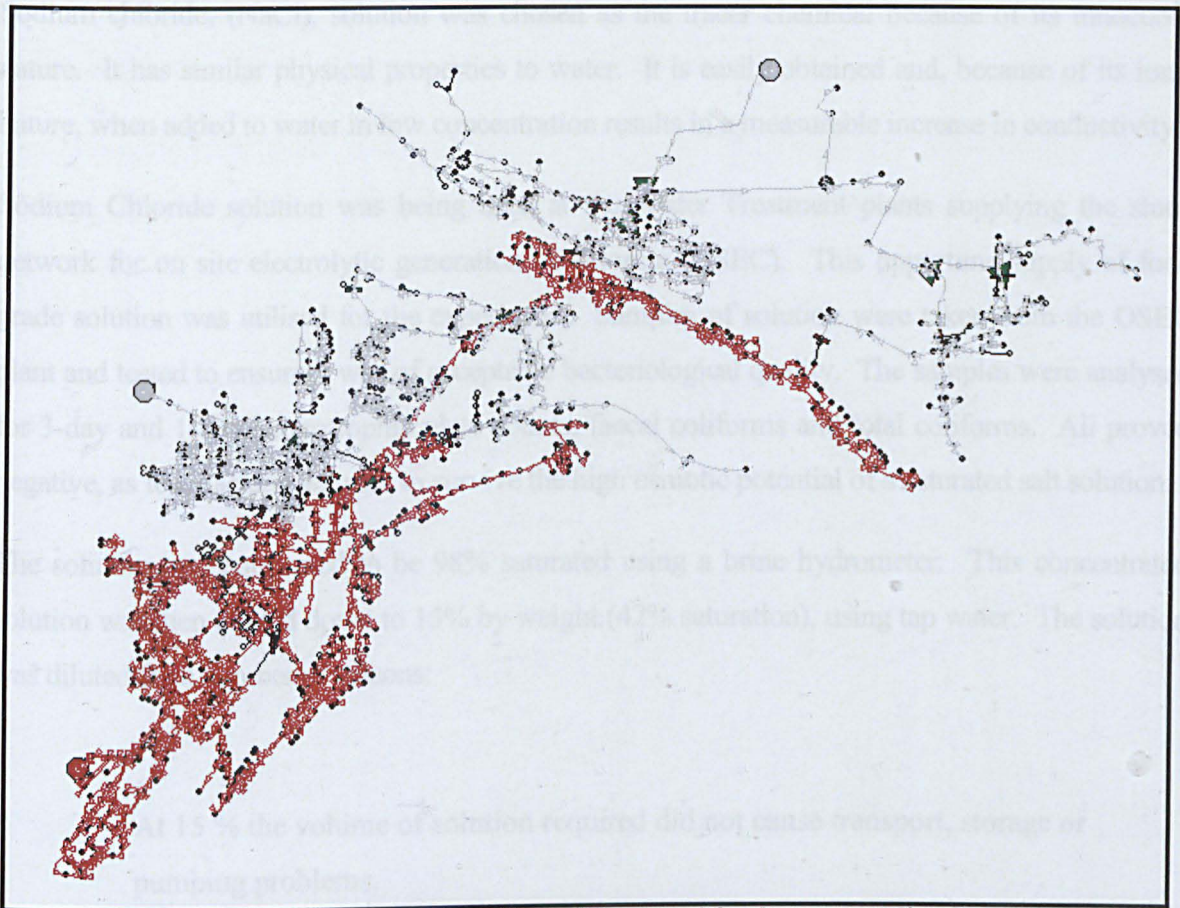


Figure 7.8 The Leakage Control Zones used for the tracer studies

The zones were fed exclusively from a single Service Reservoir for the period of the study. The network was therefore operated as a closed system with a single source of supply thereby

eliminating complexity caused by, for example, mixing with water from other parts of the network.

Water quality instruments installed in these zones (Chapter 4) were set to measure and record conductivity at 5-minute intervals. The concentration of tracer entering, and within, the network was therefore measured as a true time series using the conductivity channel on the water quality instruments. Each measurement was time stamped by a data logger and all the data logger clocks were synchronised to the system time on the computer controlling the logger set up criteria. Analysis of the data made it possible to obtain accurate time of travel data to a number of locations within the network from the point of injection.

7.3.2.2 The Tracer Solution

Sodium chloride, (NaCl), solution was chosen as the tracer chemical because of its innocuous nature. It has similar physical properties to water. It is easily obtained and, because of its ionic nature, when added to water in low concentration results in a measurable increase in conductivity.

Sodium Chloride solution was being used at the Water Treatment plants supplying the study network for on site electrolytic generation of chlorine (OSEC). This opportune supply of food grade solution was utilised for the experiment. Samples of solution were taken from the OSEC plant and tested to ensure it was of acceptable bacteriological quality. The samples were analysed for 3-day and 1-day heterotrophic plate counts, faecal coliforms and total coliforms. All proved negative, as few bacteria are able to survive the high osmotic potential of a saturated salt solution.

The solution was confirmed to be 98% saturated using a brine hydrometer. This concentrated solution was then diluted down to 15% by weight (42% saturation), using tap water. The solution was diluted for a number of reasons:

At 15 % the volume of solution required did not cause transport, storage or pumping problems.

A 15% solution has a freezing point of $-10\text{ }^{\circ}\text{C}$, which would allow the solution to stay liquid throughout the coldest temperatures likely to be experienced at the Service Reservoir site. (At concentrations above 15 %, very low temperatures could cause re-crystallisation).

Machell 1994, demonstrated that an increase in conductivity of 30 μS was required in order to show an obvious rise against background variance in the water supplying this network. The concentration of Sodium Chloride producing a 30 μS rise was calculated as 20 mg.l^{-1} as NaCl.

7.3.2.3 Tracer Solution Injection

The flow out of Bracken Bank Service Reservoir is monitored routinely for leakage monitoring purposes in the pipe immediately below the reservoir outlet. The flow data is captured as 15-minute time series, downloaded, and stored in the modelling system (Chapter 4). Because the pump that was used to inject the Sodium Chloride did not have flow proportional control, the flow patterns from the Service Reservoir were studied over a number of days to determine if there were periods when there was a steady flow in order to maintain a constant Sodium Chloride concentration throughout the period of tracer injection.

The flow data highlighted that a period from 12.10 to 13.10 each day provided the required window of stability of flow. Figure 7.9 shows 3 days flow data over this time.

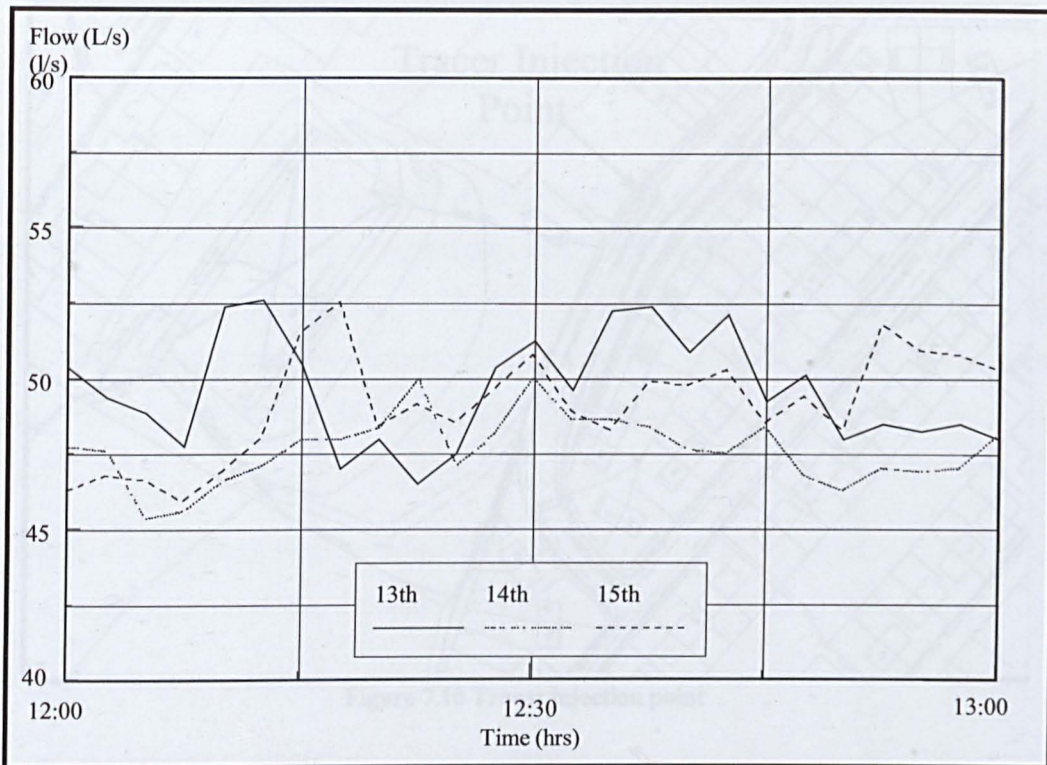


Figure 7.9 Flow from Bracken Bank Service Reservoir between 12:00hrs and 13:00hrs

The average flow for this time of day over a four-day period was calculated to be 50.5 l/sec^{-1} . This figure was therefore the assumed flow leaving the Service Reservoir for calculating the tracer solution pump rate. From the assumed flow rate and the required Sodium Chloride concentration, it was possible to calculate the required pump delivery rate of 7.33 ml.sec^{-1} .

The tracer solution was injected into the Service Reservoir outlet main via a Watson Marlow 505 Du/RL peristaltic pump operating at constant flow rate of 7.33 ml.sec^{-1} against a pressure of 20 mwc head. Figure 7.10 shows the location relative to the service reservoir and start of the network.

7.3.1.4.1 Results of Tracer Study

The conductivity was measured downstream of the injection point at a number of key measurement locations distributed throughout the network. This data was recorded and transferred to a spreadsheet for subsequent analysis.

Although there was a water quality monitor at the Service Reservoir three metres downstream of the injection point, this was found not to give a representative profile of the Sodium Chloride loading entering the system, due to insufficient mixing time being to produce a homogeneous Sodium Chloride concentration. An alternative site, Orange Road (site ID 71007) was therefore

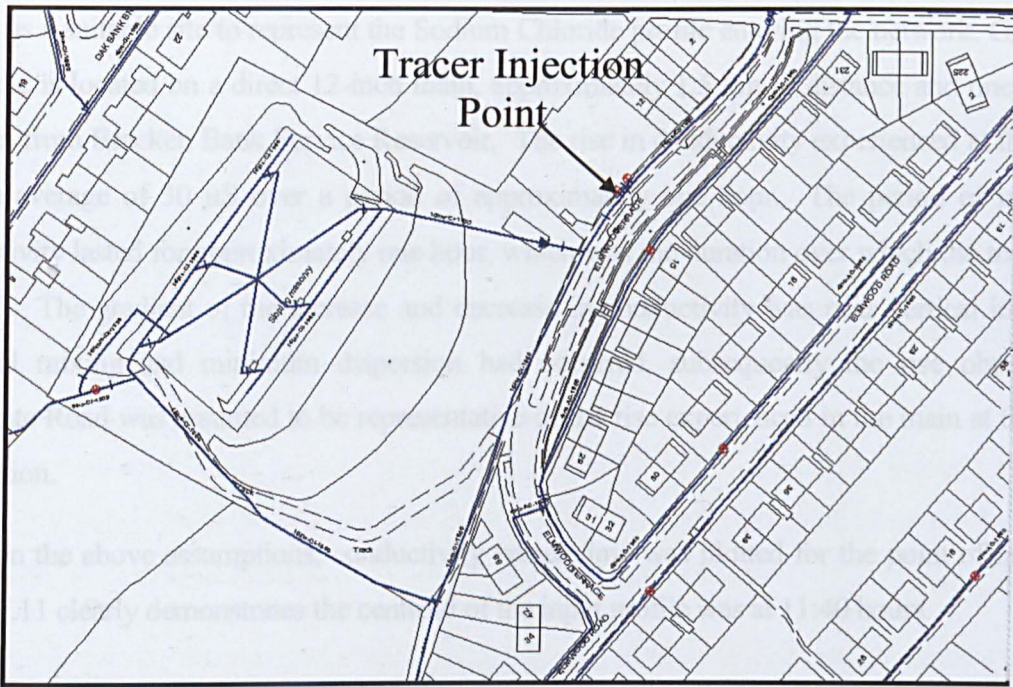


Figure 7.10 Tracer injection point

The delivery pipe from the pump was connected to the main via an existing fitting formerly used for pressure measurement. An isolation valve beneath the fitting allowed installation of the equipment to be carried out under pressure.

Before the tracer studies were carried out, a trail injection of tracer was undertaken. It was found that the water quality instrument at the Service Reservoir site did not give a representative profile of the Sodium Chloride profile entering the system. This was due to insufficient mixing time to produce a homogeneous Sodium Chloride concentration before reaching the monitor. The next site downstream was therefore chosen located on the 12-inch main feeding the network. Traces from this site showed that the injection rate was ideal for purpose.

7.3.2.4.1 Results of Tracer Study

The conductivity was measured downstream of the injection point at a number of key measurement locations distributed throughout the network. This data was recorded and transferred to a spreadsheet for subsequent analysis.

Although there was a water quality monitor at the Service Reservoir three metres downstream of the injection point, this was found not to give a representative profile of the Sodium Chloride loading entering the system, due to insufficient mixing time failing to produce a homogenous Sodium Chloride concentration. An alternative site, Greengate Road (site ID 71007) was therefore

chosen as a suitable site to represent the Sodium Chloride profile entering the network. Greengate Road site is located on a direct 12-inch main, approximately 1.5 km in distance and one hour in duration from Bracken Bank Service Reservoir. The rise in conductivity experienced at this point was an average of 30 μS over a period of approximately one hour. The period of increased conductivity lasted for approximately one hour, which was the duration over which the tracer was injected. The gradient of the increase and decrease in conductivity was near vertical indicating maximal mixing and minimum dispersion had occurred, subsequently the rise observed at Greengate Road was assumed to be representative of the rise experienced in the main at the point of injection.

Based on the above assumptions, conductivity versus time was plotted for the point of injection. Figure 7.11 clearly demonstrates the centroid of the input profile was at 11:40 hours.

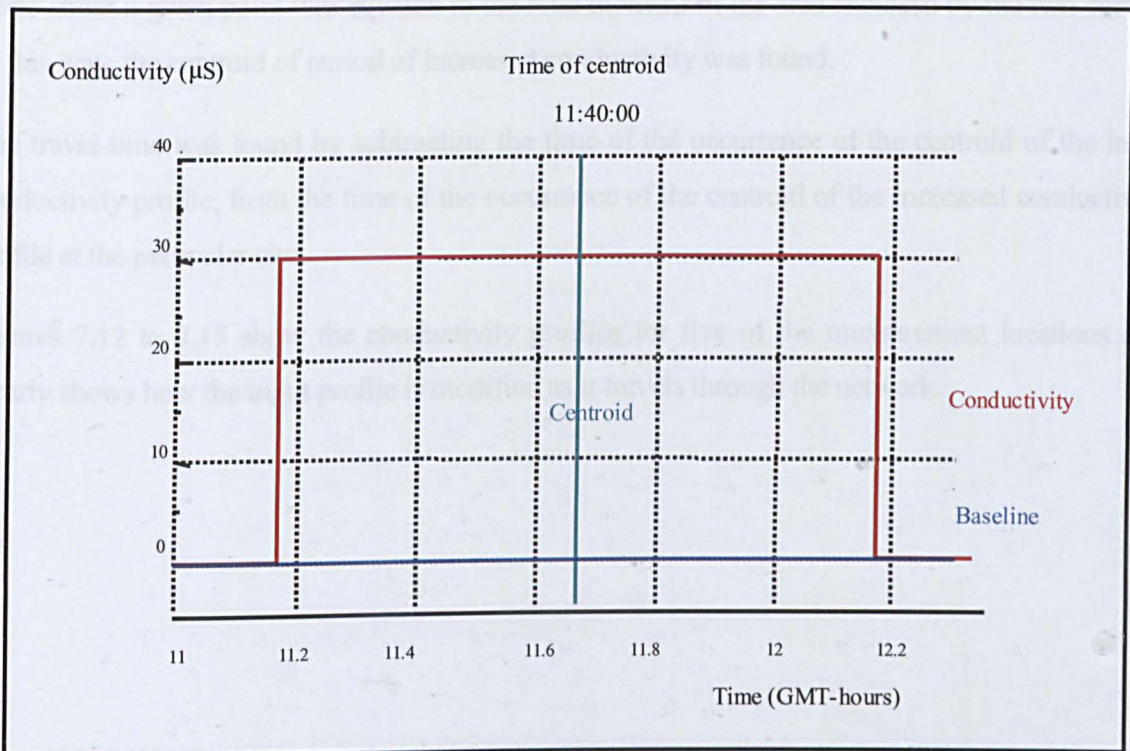


Figure 7.11 Centroid of tracer input profile

7.3.2.5 Calculation of Travel Time

Travel time to a particular measurement site is defined as the time from the centroid of the input profile to the time of the centroid of the increased conductivity profile for that site.

7.3.2.6 Calculation of Centroid

The rise in the conductivity was analysed at a number of sites downstream of the tracer injection point.

A baseline was established beneath the period of conductivity increase, to indicate what the conductivity level would be without the tracer input. This was determined as the base level conductivity before the rise and the base level conductivity after the rise.

The area between the conductivity profile and the base line was divided into elemental strips and the area of each strip was calculated in μS seconds. The sum of the moments of these elemental strips about a given point was equated to the total moment of the area enclosed by the two curves. In this way, the centroid of period of increased conductivity was found.

The travel time was found by subtracting the time of the occurrence of the centroid of the input conductivity profile, from the time of the occurrence of the centroid of the increased conductivity profile at the particular site.

Figures 7.12 to 7.15 show the conductivity profiles for five of the measurement locations and clearly shows how the input profile is modified as it travels through the network.

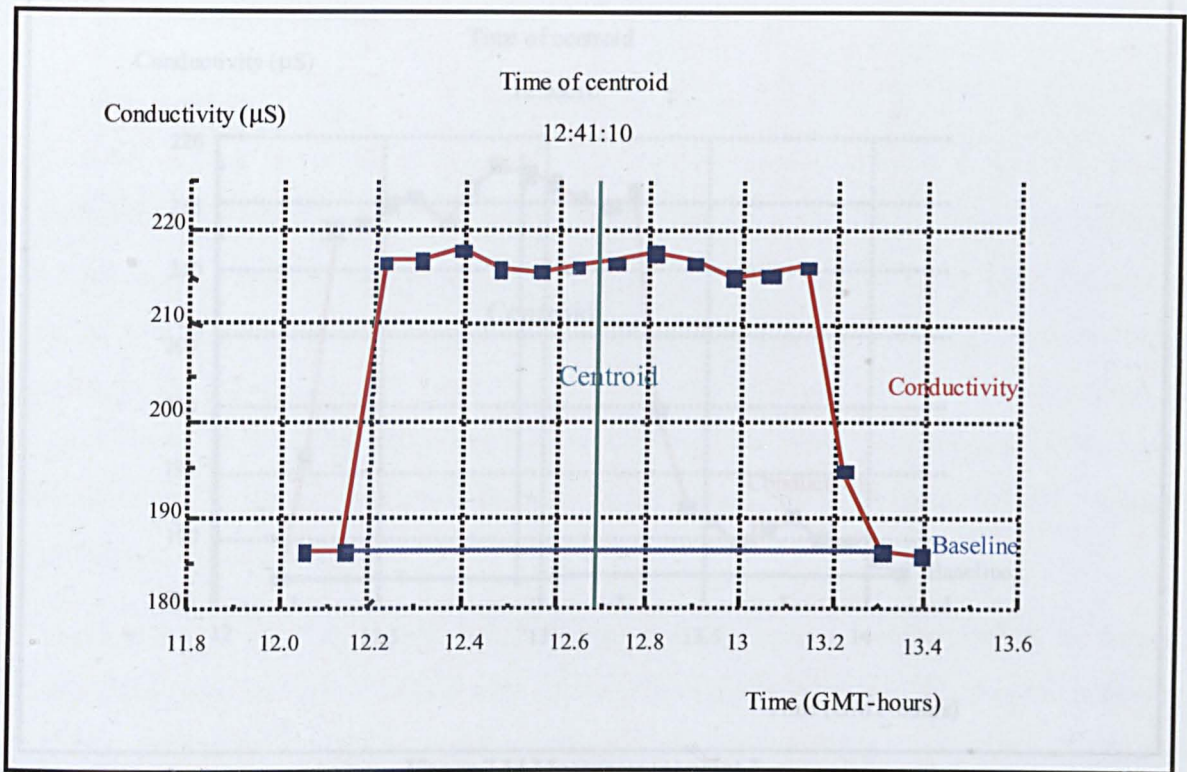


Figure 7.12 Measurement point 1

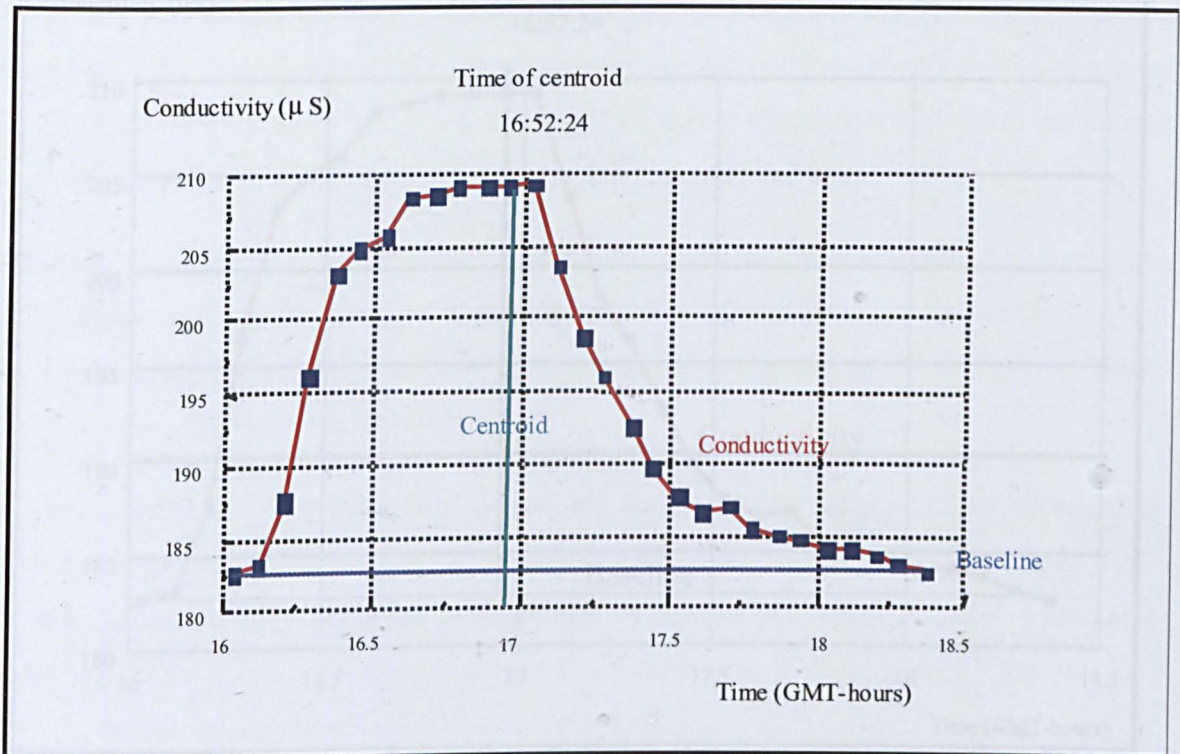


Figure 7.13 Measurement point 2

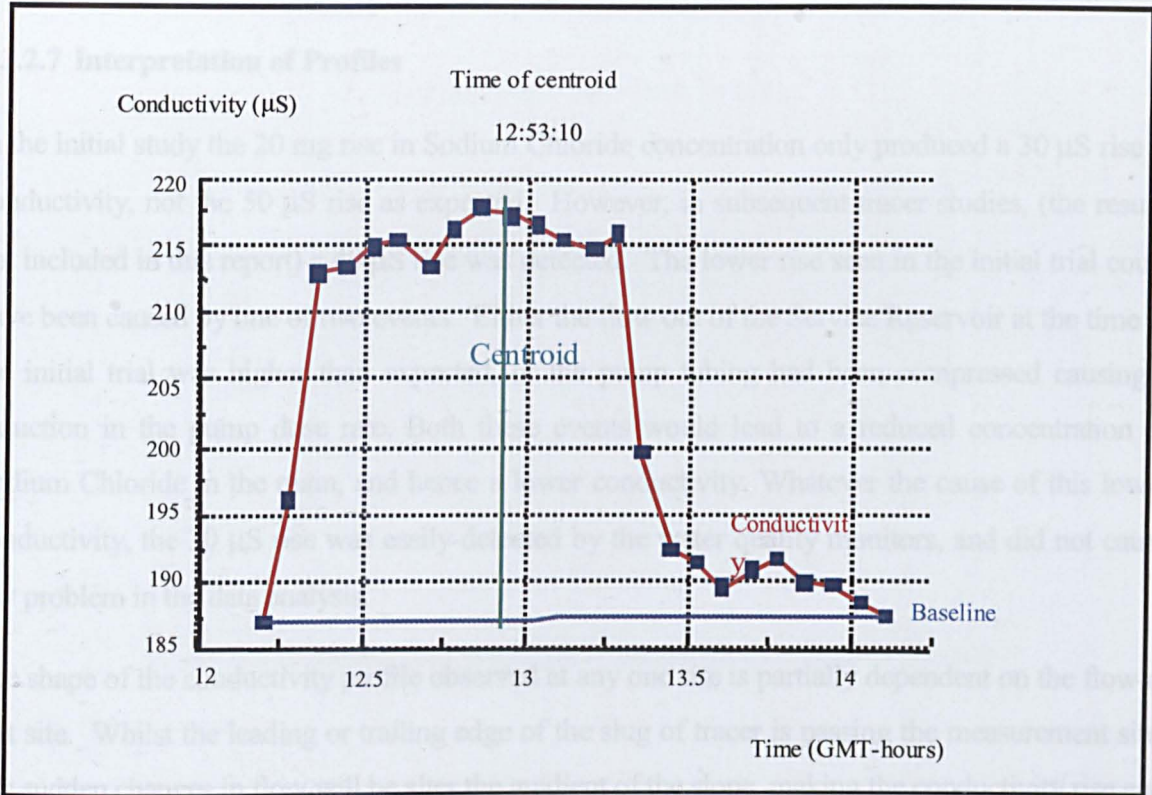


Figure 7.14 Measurement point 3

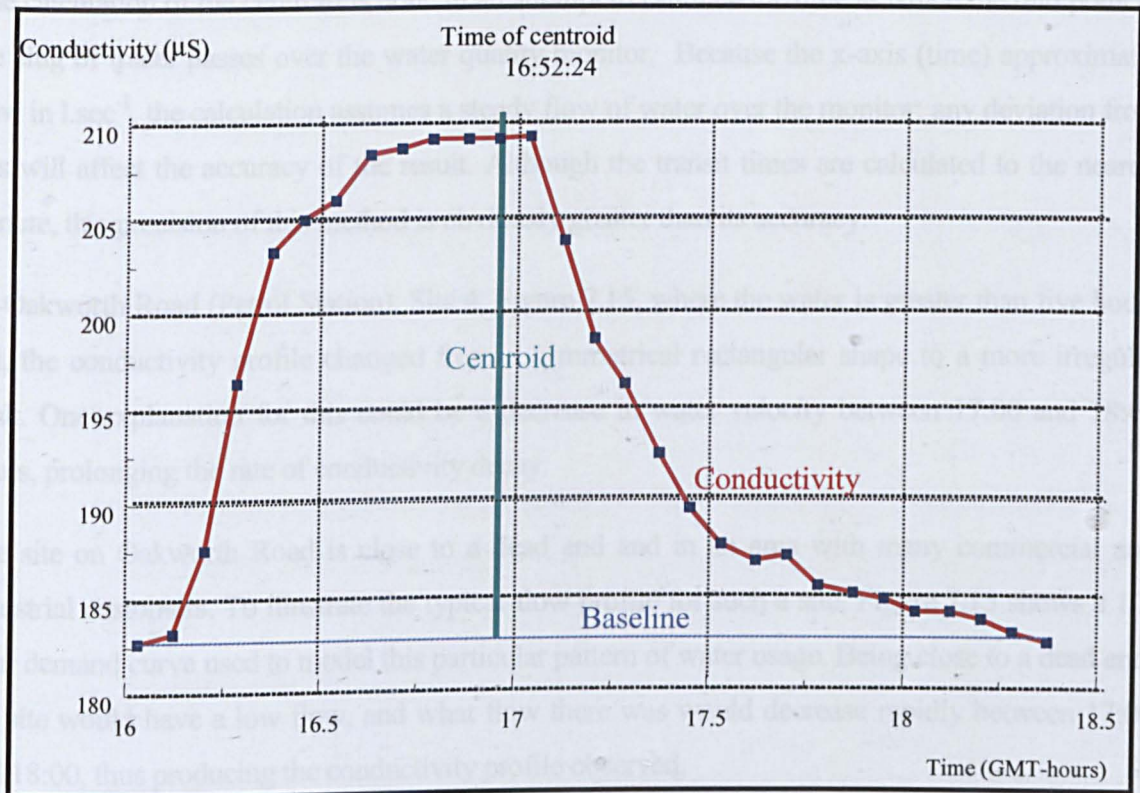


Figure 7.15 Measurement point 4

An Excel spreadsheet was designed to calculate the centroid.

7.3.2.7 Interpretation of Profiles

In the initial study the 20 mg rise in Sodium Chloride concentration only produced a 30 μS rise in conductivity, not the 50 μS rise as expected. However, in subsequent tracer studies, (the results not included in this report) a 40 μS rise was detected. The lower rise seen in the initial trial could have been caused by one of two events. Either the flow out of the Service Reservoir at the time of the initial trial was higher than expected, or the pump tubing had been compressed causing a reduction in the pump dose rate. Both these events would lead to a reduced concentration of Sodium Chloride in the main, and hence a lower conductivity. Whatever the cause of this lower conductivity, the 30 μS rise was easily detected by the water quality monitors, and did not cause any problem in the data analysis.

The shape of the conductivity profile observed at any one site is partially dependent on the flow at that site. Whilst the leading or trailing edge of the slug of tracer is passing the measurement site, any sudden changes in flow will be alter the gradient of the slope, making the conductivity rise and fall either steeper or more gradual.

The calculation of the centroid is done in an attempt to estimate the time at which the mid point of the slug of tracer passes over the water quality monitor. Because the x-axis (time) approximates flow in $\text{l}\cdot\text{sec}^{-1}$, the calculation assumes a steady flow of water over the monitor; any deviation from this will affect the accuracy of the result. Although the transit times are calculated to the nearest minute, this precision of this method is obviously greater than its accuracy.

At Oakworth Road (Petrol Station), Site 4, Figure 7.15, where the water is greater than five hours old, the conductivity profile changed from a symmetrical rectangular shape to a more irregular peak. One explanation for this could be a decrease in water velocity between 17:00 and 18:00 hours, prolonging the rate of conductivity decay.

The site on Oakworth Road is close to a dead end and in an area with many commercial and industrial customers. To illustrate the typical flow profile for such a site, Figure 7.15 shows a 10-hour demand curve used to model this particular pattern of water usage. Being close to a dead end, the site would have a low flow, and what flow there was would decrease rapidly between 17:00 and 18:00, thus producing the conductivity profile observed.

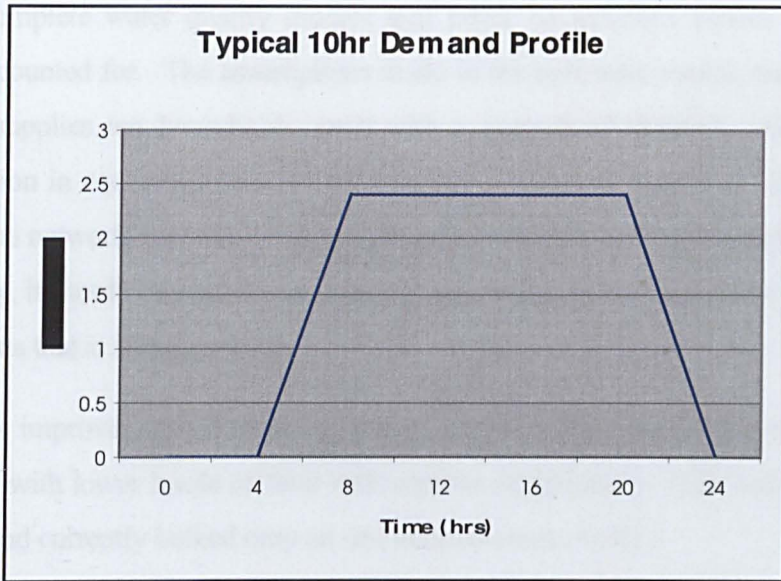


Figure 7.16 10 Hour demand curve used at commercial and industrial premises

The shape of the conductivity profiles can be affected by a number of other hydraulic conditions in the network.:

A site containing a number of water fractions with different ages could reduce the overall conductivity rise.

Changes in the direction of flow at a site could result in a number of conductivity peaks, or a single rise extended over a longer period, depending on the nature of flow.

Sites with older water will tend to deviate more from the input profile, because the slug of tracer has been in the distribution system longer and the chances of it encountering hydraulic condition with the potential to cause deviations are greater. In addition, older water tends to lie at the ends of systems where there is increased meshing and mixing of water.

A highly meshed network would increase dispersion and mixing in the water making the profile less symmetrical, this may or may not affect the overall conductivity rise.

Calibration of complete water quality models will never be achieved unless all demands are measured and accounted for. The assumptions made in the hydraulic model, for example, that a particular main supplies ten households, each with a normalised demand, will never be true. Accurate calibration in mains with low flows therefore will not be achieved. However, for the larger mains in the network, certainly down to six inch, where there is a reasonable flow twenty-four hours per day, it should be possible to obtain a very good calibration and the lower main sizes should provide data that is fit for purpose.

As the technology improves, and if metering becomes more widely acceptable in the UK, model accuracy in pipes with lower levels of flow will improve significantly. This will be aided by the fact that the demand currently bulked onto an end node of a main will be

7.9 Diagnostic Model

The author developed the functionality of the conservative propagation model in reverse, to determine the origin of where a concentration of substance entered the distribution network, the *Diagnostic Model*.

The Diagnostic model is used for calculating possible points of origin of polluting material. The diagnostic module uses the same basic input as the propagation simulation, but simulates backwards in time. The simulation is based on a user specified time dependent concentration of a substance measured at a single location in the network.

The measured concentration or concentration profile is used as a boundary condition together with the velocity profile from the basic quality simulation. At the start time of the simulation, the concentration of the substance to be traced is initialised by setting the values of the measured concentration in the downstream end of the inlet pipe(s) attached to the corresponding node. Values elsewhere are set to zero.

The diagnostic model uses the propagation functionality but the simulation is made using decreasing time, i.e.:

Periodic hydraulic conditions to start the diagnostic simulation are taken from the end time of the basic hydraulic simulation period

Periodic hydraulic conditions to end the diagnostic simulation are taken from the start time of the basic hydraulic simulation period

Existing propagation functions (equations (7.5) and (7.6)) were adapted, i.e. the concentration at time $t-dt$ is a function of the concentration at time t .

For a zero order reaction,

$$n = 0 : C(t - \Delta t) = C(t) - k\Delta t \quad (7.10)$$

For a first order reaction:

$$n = 1 : C(t - \Delta t) = C(t).e^{-k\Delta t} \quad (7.11)$$

Where:

K is overall decay rate constant (s⁻¹).

C is concentration (-).

The concentration at a time $(t-dt)$ is then a function of the concentration at time t . The propagation functionality solves along the lines in the $t-x$ (time-chainage) plane with slope:

$$\frac{dx}{dt} = V \quad (7.12)$$

Because the concentration is the only unknown variable, the linear equation for each calculation point between nodes is:

$$C_1(t) = f(C_1(t-dt) + (C_2(t-dt) - C_1(t-dt)) \cdot \frac{V \cdot dt}{dx}) \quad (7.13)$$

Where:

C_1 and C_2 are substance concentration in neighbouring calculation points
 f is the propagation function

Using the same principal backward in time the equation becomes:

$$C_1(t - dt) = f^{-1} \left(C_1(t) + (C_2(t) - C_1(t)) \cdot \frac{V \cdot dt}{dx} \right) \quad (7.14)$$

Where, f^{-1} is the *inverse* propagation function

Therefore the concentration at time $t-dt$, $C_1(t-dt)$ was found by solving equation 7.14.

In the diagnostic simulation, all points are considered as possible points of ingress. The method is applied in a loop over all nodes. The solution is found by calculating the concentration of substance in the downstream end of each inlet pipe. These concentrations are referred to as C_i in the i^{th} inlet pipe.

If the node is the one with the known / measured time series, C_i is set to the actual value(s) entered. Otherwise, C_i is calculated from:

$$C_i = \frac{\sum (Q_j \cdot C_j)}{Q_i} \quad (7.15)$$

Where:

Q_j is the flow rate in the j^{th} outlet pipe

C_j is the concentration in the j^{th} outlet pipe

Q_i is the flow rate in the i^{th} inlet pipe

The summation includes all outlet pipes attached to the node and the local consumption. The numerical solution method is similar to the one described in section 7. However, for the diagnostics module, the simulations are performed backwards in time.

The Diagnostic model can be used to identify where a substance or a discoloured water event originated, or, almost as importantly, where it did *not*.

7.4 Basis of the Propagation model

The propagation model can calculate the concentration of up to nine substances simultaneously. However, one of the ‘substances’, by default, must be age. The diagnostic model can only simulate one substance at a time.

7.4.1 Conservative and Non-conservative propagation, and Age

All the functionality has been coded into a single model entity and the output depends on what the user tells the model to do e.g. how the substances are defined; trace, linear or exponential decay. Conservative propagation is a trace substance, non-conservative propagation is linear or exponential substance and Age is a growth law using negative linear decay.

The simulation includes diffusion and convective transport of substances in a network. Further, the interconnected changes in concentration of different substances are included.

The simulation is made via two steps. The first step is a quasi-stationary simulation of the water flow in the network using the hydraulic engine. This creates a database that includes the calculated velocity for all calculation points, in all pipes, at all time-steps.

The second step is the simulation of the substance flow. This simulates the concentration of substances with respect to time and location.

The following assumptions are made in order to calculate the concentration as a function of position and time:

The decomposition rate of a substance per unit length and per time can be described by the term $-k_A C_A^{n_A}$. Further, it is assumed that the decomposition rate can be related to the volume of water.

It is assumed that growth of one substance (B) may be due to a proportional decomposition of another substance (A). In other words, it is possible, due to chemical reactions, that the concentration of one substance B is increased per unit length and per time by the term $-k_{trans} k_A C_A^{n_A}$.

The decomposition of substance concentration along the pipe is ignored (only in this section).

If these assumptions are combined and defined as the left side of equation (7.2), the basic equation expressing the coupled decay / growth of two non-conservative substances A and B is expressed as:

$$\frac{\partial(AC_B)}{\partial t} + \frac{1}{\rho} \frac{\partial(QC_B)}{\partial x} = -k_B AC_B^{n_B} + k_{trans} (k_A AC_A^{n_A}) \quad (7.16)$$

Where:

- A is the cross-sectional area of pipe.
- Q is the mass flow rate.
- ρ is the density of water.
- C_A is the concentration of substance A.
- C_B is the concentration of substance B.
- k_B is the decay rate constant, $k_{v,n}$, for substance B.
- n_A is the exponent (order reaction) for substance A.
- n_B is the exponent (order reaction) for substance B.
- t is time.
- x is the 'chainage' (the accumulated length of pipe)

k_{trans} is the transformation factor expressing the increased amount of substance B relative and due to the decomposed amount of substance A (representing the stoichiometry of the reaction)

With reference to section 7.2, it is seen that the solution to equation 7.16 along a particle path: $\Delta x / \Delta t = V$ is respectively:

$$n_B = 0: \quad C_B(t) = C_B(t_0) - k_B(t - t_0) - k_{trans} \Delta C_A \quad (7.17)$$

$$n_B = 1: \quad C_B(t) = C_B(t_0) e^{-k_B(t-t_0)} - k_{trans} \Delta C_A \quad (7.18)$$

Where:

t is actual time (s)

t_0 is latest reference time (s), corresponding to a known / calculated concentration

ΔC_A is the decomposed amount (concentration) of substance A between time t_0 and time t .

If the last term in equations 7.17 and 7.18 is ignored, and subscript B substituted by A, the decomposed amount of substance A due to time is:

$$n_A = 0: \quad \Delta C_A = -k_A(t - t_0) \quad (7.19)$$

$$n_A = 1: \quad \Delta C_A = C_A(t_0)(e^{-k_A(t-t_0)} - 1) \quad (7.20)$$

Where: k_A is decay rate constant for substance A.

The last term in Equation 7.19 or 7.20 is included when condition (a) and (b) below are both fulfilled:

$$(a) \quad \Delta C_A > C_{\min}$$

$$(b) \quad \Delta C_A < 0$$

Where C_{\min} is the minimum concentration of substance A necessary for the transformation process from substance A, into substance B, to take place.

Condition (b) is always fulfilled, when $k_A > 0$, which is the normal case.

Figure 7.17 and Figure 7.18 show the 0 and 1st order of reaction for a substance provided that no interconnected changes are taking place, i.e. where $k_{\text{trans}} = 0$.

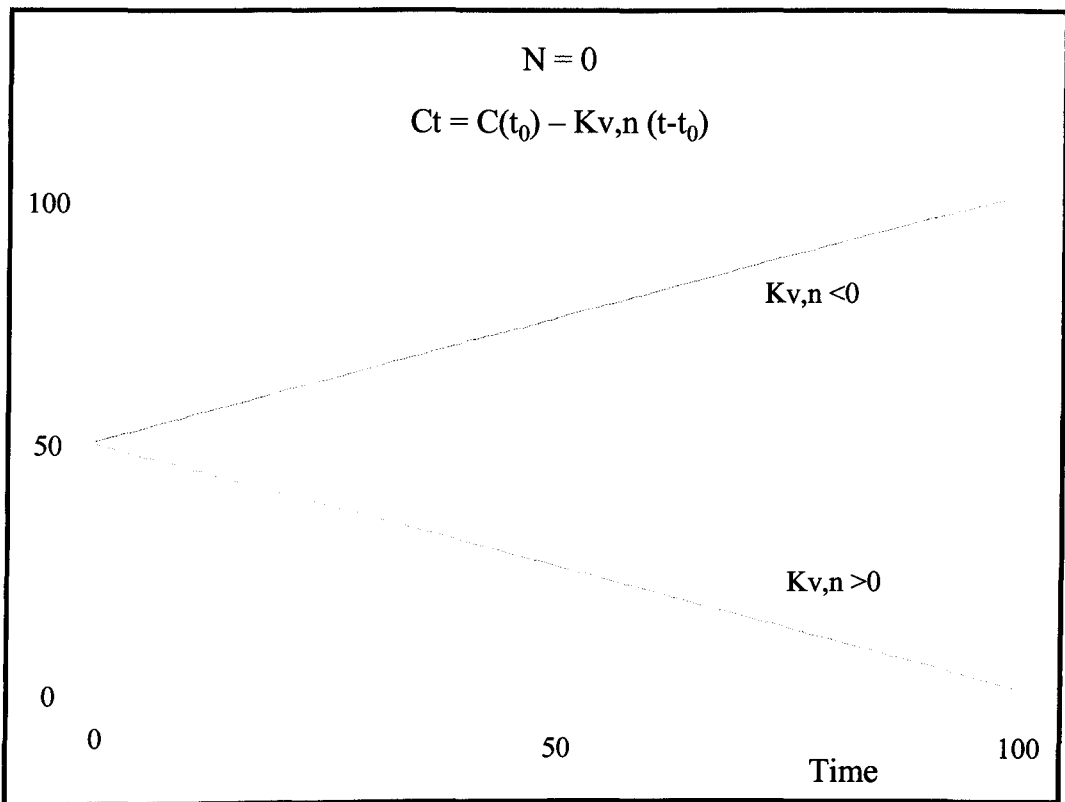


Figure 7.17 Zero order reaction.

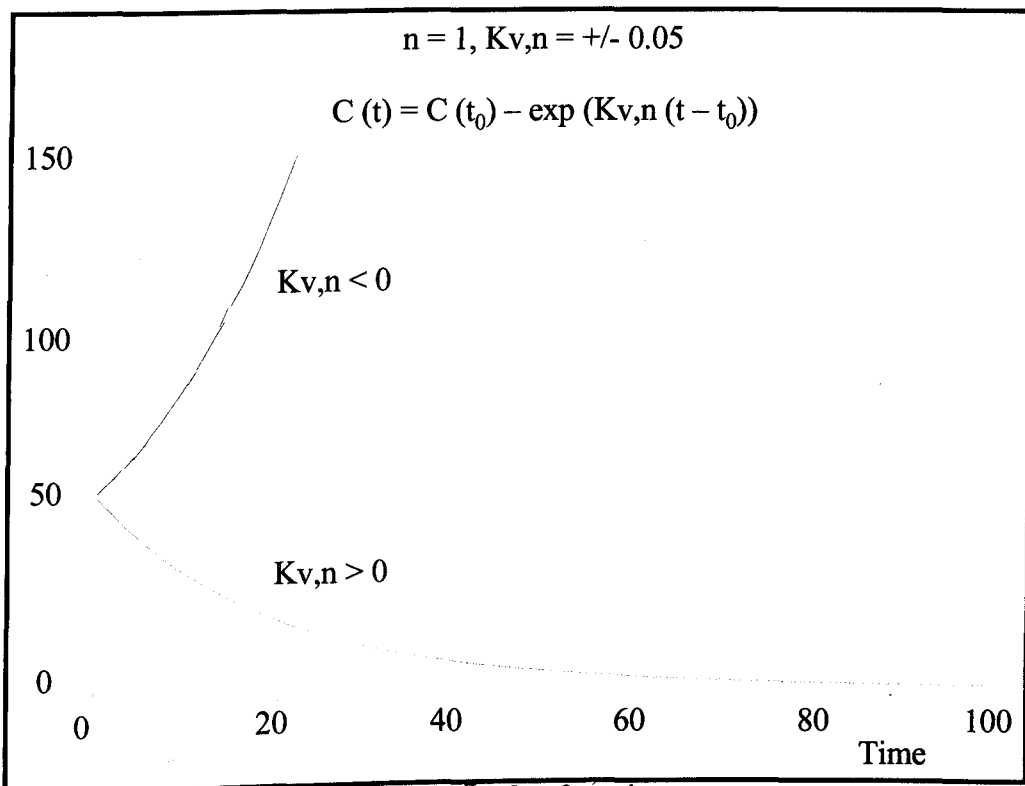


Figure 7.18 1st order of reaction.

Figure 7.19 shows an example for three substances at a point in the network with zero velocity relative to bulk flow (i.e. where there is stagnant water).

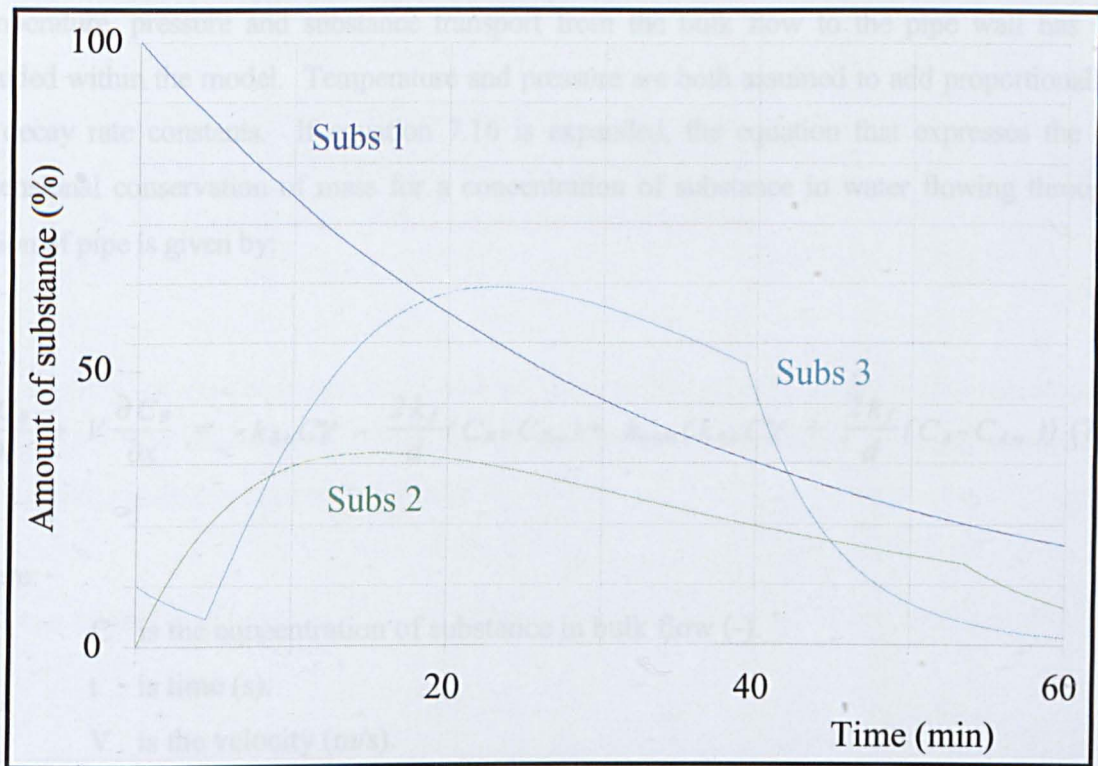


Figure 7.19. Coupled decay / growth of substances.

Where:

- n = 1 for all three substances (1'st order reaction)
- C_{\min} = 20 for both substances 1 and 2
- $k_{trans,1}$ = (transformation from subs, 1 into subs 2)
- $k_{trans,2}$ = (transformation from subs, 2 into subs. 3)
- k_1 = 0.0005
- k_2 = 0.002
- k_3 = 0.003

In the example, substance 1 decays creating substance 2. When the concentration of substance 2 exceeds 20 mg.l^{-1} , substance 3 is formed via the decay of substance 2. When the concentration of substance 2 is reduced to below 20 mg.l^{-1} again, substance 3 decays exponentially. The model can be used, for example, to determine Trihalomethane residuals or how much nitrate will be produced from a given amount of ammonia by nitrifying bacteria (or any other mechanism).

7.4.2 Temperature, Pressure and Transport to Pipe Wall

Temperature, pressure and substance transport from the bulk flow to the pipe wall has been included within the model. Temperature and pressure are both assumed to add proportionally to the decay rate constants. If equation 7.16 is expanded, the equation that expresses the one-dimensional conservation of mass for a concentration of substance in water flowing through a section of pipe is given by:

$$\frac{\partial C_B}{\partial t} + V \frac{\partial C_B}{\partial x} = -k_{B,b} C_B^{n_B} - \frac{2k_f}{d} (C_B - C_{B,w}) + k_{trans} (k_{A,b} C_A^{n_A} + \frac{2k_f}{d} (C_A - C_{A,w})) \quad (7.21)$$

Where:

- C is the concentration of substance in bulk flow (-).
- t is time (s).
- V is the velocity (m/s).
- x is the chainage (m).
- kb is the decay rate constant in the bulk flow (s-1) (Equation 11).
- kf is the mass transfer coefficient (m²/s).
- d is the inner pipe diameter (m).
- Cw is the concentration of substance at the pipe wall (-).
- A, B as indexes, refer to substances A and B respectively.

The additional terms in equation 7.21 as compared to equation 7.16 accounts for transport of the substance between bulk flow and pipe wall. The remaining part of the expression is in agreement with equation 7.4 except that $k_{v,n}$ has been added the effect of pressure and temperature as follows:

$$k_b = k_{v,n} + \alpha(T - T_0) + \beta(p - p_0) \quad (7.22)$$

Where:

$k_{v,n}$ is the decay rate constant (user defined)

T is the measured (or user defined) temperature ($^{\circ}\text{C}$).

T_0 is a user defined global reference temperature ($^{\circ}\text{C}$).

p is the actual (measured) pressure (mwc).

p_0 is a user defined global reference pressure (mwc).

The user may specify the temperature and, or, pressure to be less than the global reference values. This is so the user can define a decay or growth law or positive or negative effects of temperature / pressure in same equations.

The mass transfer coefficient k_f (*Liou and Kroon., 1987*) in Equation (7.19) is calculated from:

$$k_f = Sh \frac{D}{d} \quad (7.23)$$

$$Sh = 0.023 Re^{0.83} Sc^{0.333} \quad \text{for } Re \geq 2300 \quad (7.24)$$

$$Sh = 3.65 + \frac{0.0668(d/L)(ReSc)}{1 + 0.04((d/L)(ReSc))^{2/3}} \quad \text{for } Re < 2300 \quad (7.25)$$

$$Re = \frac{Vd}{\nu} \quad (7.26)$$

$$Sc = \frac{\nu}{D} \quad (7.27)$$

Where:

Sh is the Sherwood Number (Dimensionless).

Re is the Reynolds Number (Dimensionless).

Sc is the Schmidt Number (Dimensionless).

D is the molecular diffusivity of substance in water (m^2/s).

ν is the kinematic viscosity of water (m^2/s).

L is the pipe length (m).

Note that for a particular substance, k_f is a function of pipe diameter, flow velocity, and temperature as k_f affects diffusivity and viscosity. Assuming that the reaction of substance at the pipe wall is first order with respect to the wall concentration C_w , and that it proceeds at the same rate as substance is transported to the wall, results in the following mass balance for substance at the wall:

$$k_f (C - C_w) = k_w C_w \quad (7.28)$$

Where: k_w is pipe wall decay rate constant (m/s).

Solving equation (20) for C_w and substituting it into equation (13) for each pipe in the network gives:

$$\frac{\partial C_{B,i}}{\partial t} + V_i \frac{\partial C_{B,i}}{\partial x_i} = -K_{B,i} C_{B,i}^{m_B} + k_{trans} K_{A,i} C_A^{n_A} \quad (7.29)$$

Where: i is a subscript indicating the i 'th pipe in the network.

K is the overall decay constant (s^{-1}).

Equation (21) is similar to Equation (4) with k_A and k_B ($k_{v,n}$) substituted by the overall decay constants for substance A and B respectively:

$$K_i = k_b + \frac{2 k_{fi} k_w}{d_i(k_w + k_{fi})} \quad (7.30)$$

Where: k_{fi} is mass transfer coefficient for the i^{th} pipe (m/s), (equation 7.18).

7.4.3 Effect of Variables

This section provides a sensitivity analysis for the variable parameters within the model. By definition, the properties of a conservative substance are such that its concentration does not change with time or location other than because of dilution effects. The model therefore does not permit the user to apply k values or any other of the factors that would affect concentration via reactions or increased decay due to temperature or pressure variations. Figure 7.20 shows the substance properties configuration dialogue box. All the variable parameters are greyed out and are therefore not accessible to the user.

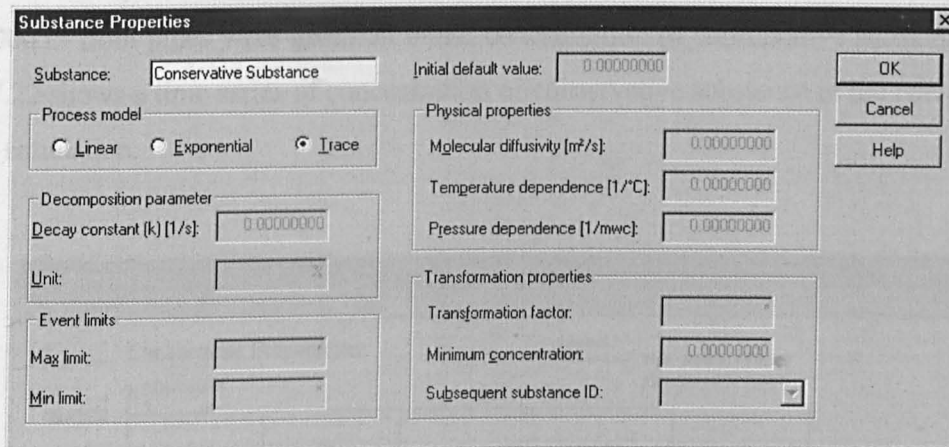


Figure 7.20 The substance properties configuration dialogue box

The model determines changes in concentration of a conservative substance therefore via dilution calculations from pipe and node flows obtained from the hydraulic engine. Figure 7.21 shows the flow time series for two pipes. Pipe P-0005 has a flow of 4 l/s and P-0006 has no flow at all.

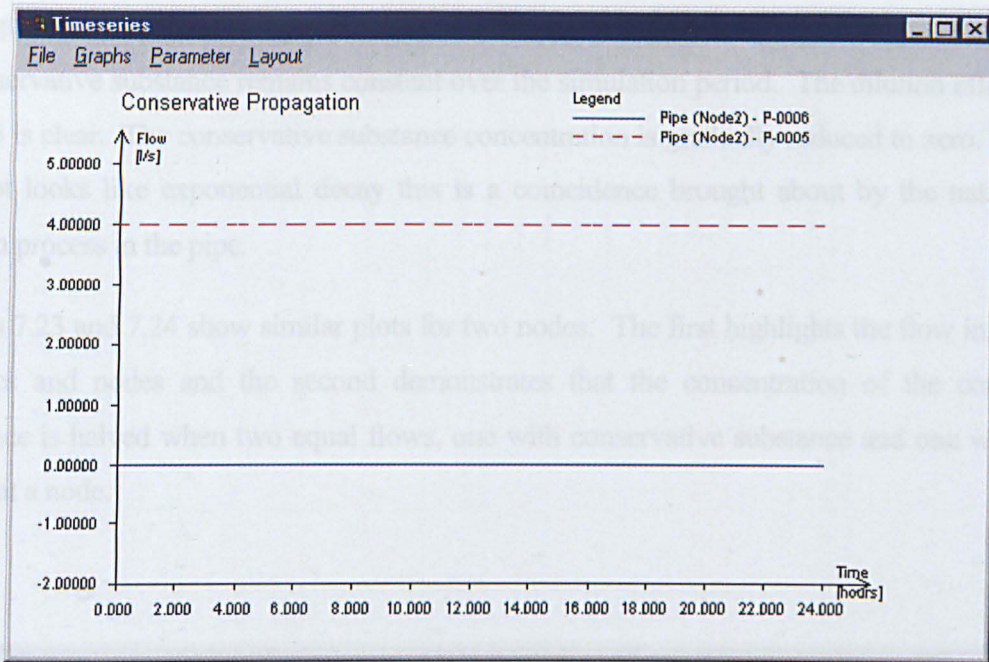


Figure 7.21 Time series for 2 pipes – one with and one without flow

The blue trace shows that pipe P-0006 has no flow. The red trace highlights the $4.0 \text{ l}\cdot\text{s}^{-1}$ flow in pipe P-0005. Both pipes were given an initial concentration of conservative substance of 100%. Figure 7.22 shows a time series of concentration of conservative substance in the two pipes over a 24-hour simulation.

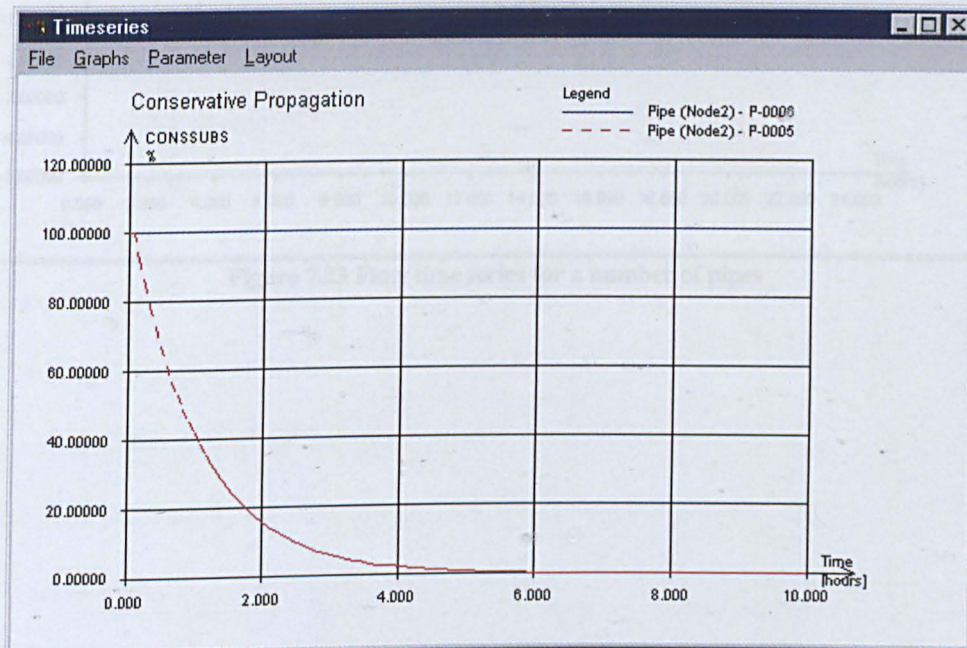


Figure 7.22 Plot of concentration of conservative substance

The figure clearly shows that for the pipe P-0006 with no flow, i.e. no dilution, the concentration of conservative substance remains constant over the simulation period. The dilution effect in pipe P-0005 is clear. The conservative substance concentration is gradually reduced to zero. Although the plot looks like exponential decay this is a coincidence brought about by the nature of the dilution process in the pipe.

Figures 7.23 and 7.24 show similar plots for two nodes. The first highlights the flow in a number of pipes and nodes and the second demonstrates that the concentration of the conservative substance is halved when two equal flows, one with conservative substance and one without are mixed at a node.

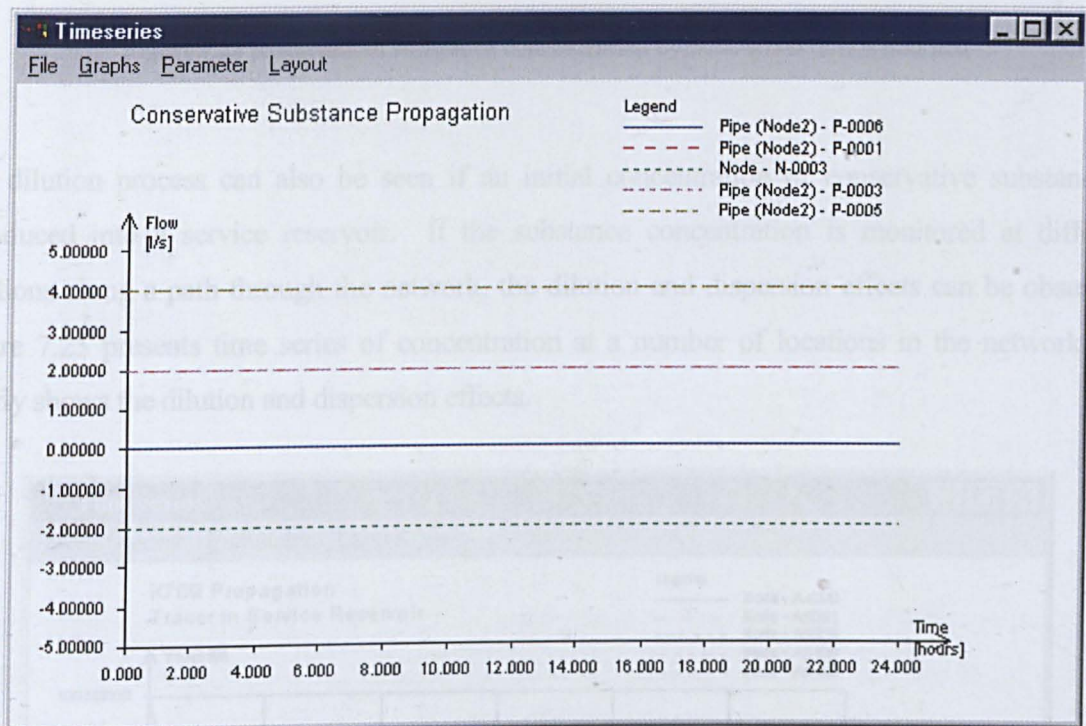


Figure 7.23 Flow time series for a number of pipes

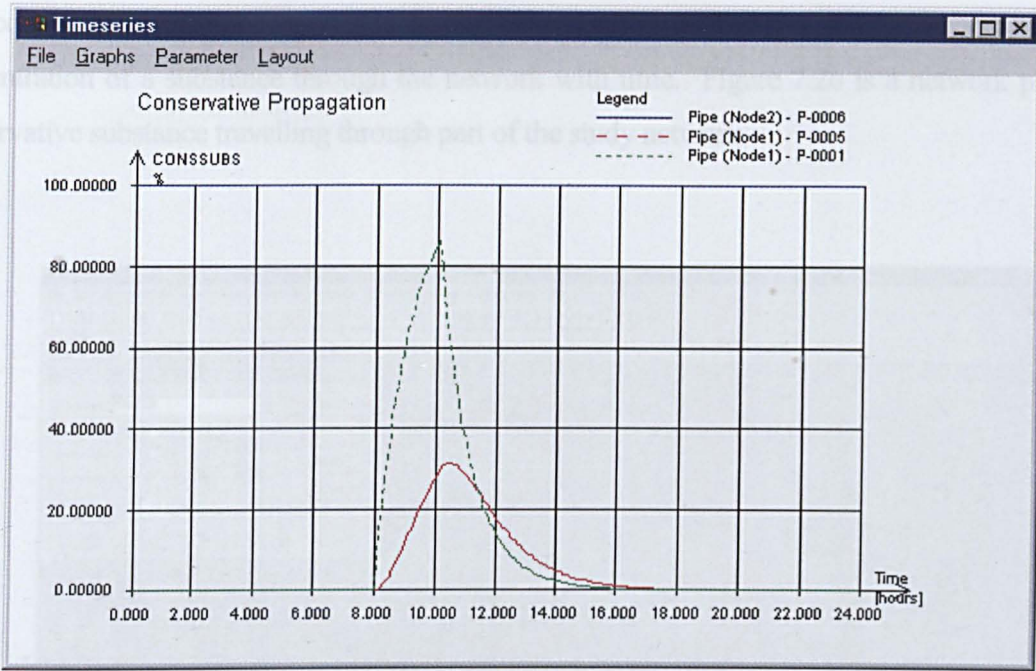


Figure 7.24 Reduction of substance concentration by 50% when flow is doubled

The dilution process can also be seen if an initial concentration of conservative substance is introduced into a service reservoir. If the substance concentration is monitored at different locations along a path through the network, the dilution and dispersion effects can be observed. Figure 7.25 presents time series of concentration at a number of locations in the network that clearly shows the dilution and dispersion effects.

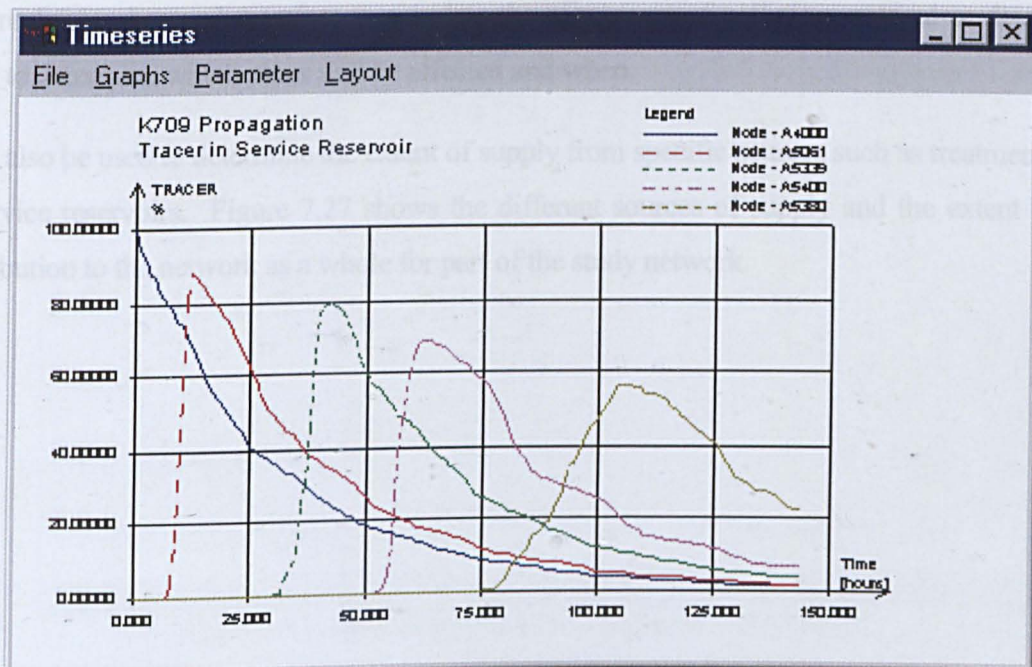


Figure 7.25 Substance concentration at a number of locations in the network

The conservative substance propagation functionality can be used to follow the progress and concentration of a substance through the network with time. Figure 7.26 is a network plot of a conservative substance travelling through part of the study network.

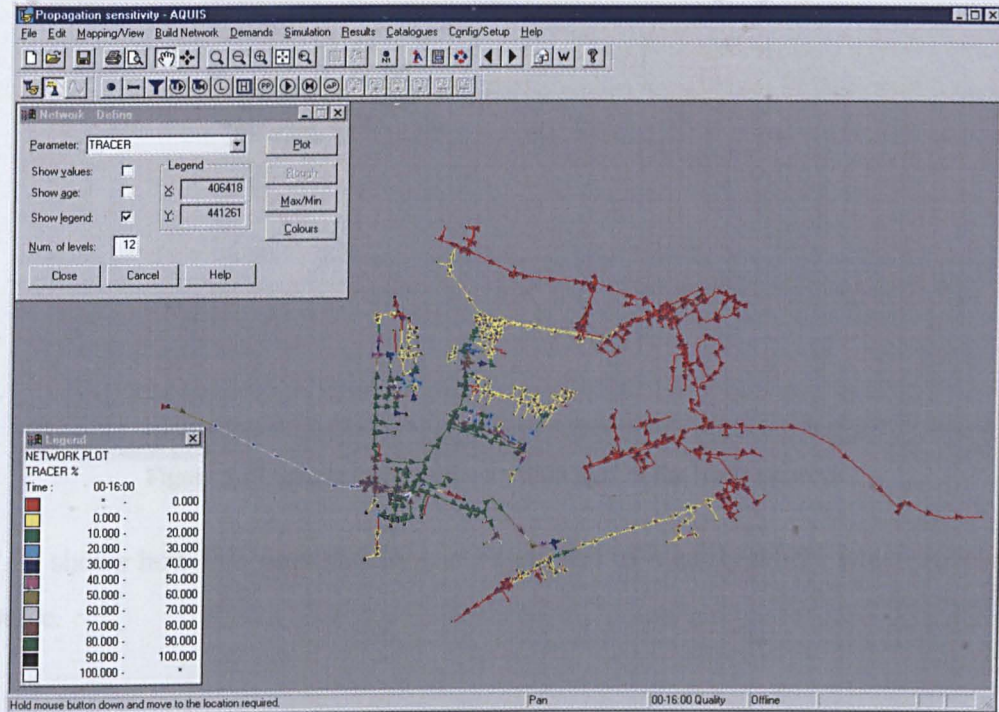


Figure 7.26 Conservative substance location and concentration

The model can be used therefore to simulate the movement of polluting material or discoloured water to determine which pipes will be affected and when.

It can also be used to determine the extent of supply from specific sources such as treatment plants or service reservoirs. Figure 7.27 shows the different sources of supply and the extent of their contribution to the network as a whole for part of the study network.

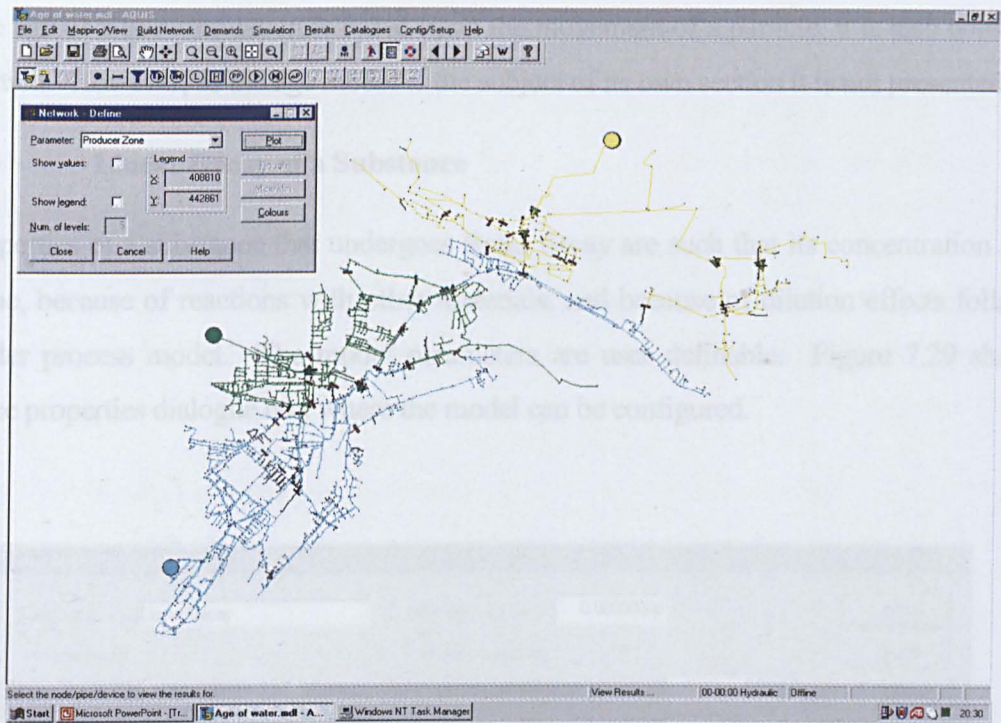


Figure 7.27 Source contributions within part of the study network

Figure 7.28 shows how this functionality can be utilised to identify where water travels from a single source.

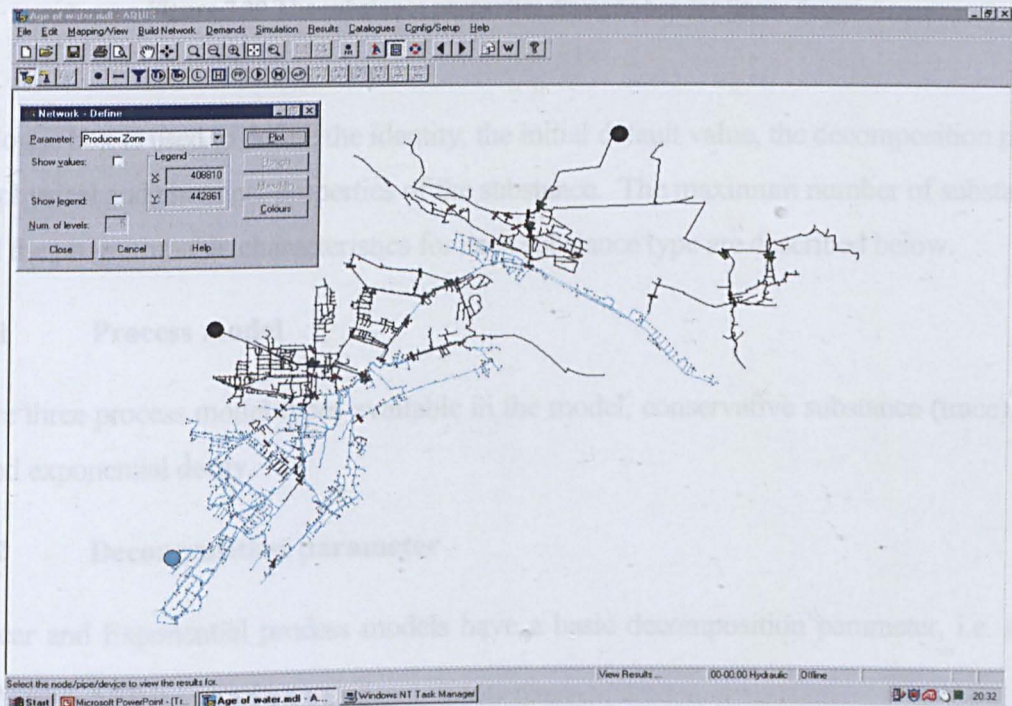


Figure 7.28 Identification of extent of supply from a single source.

Because age of water simulation is based upon the movement of a particle, it is also conservative propagation. However, as the age model is the subject of its own section it is not presented here.

7.4.3.1 Linear Decay of a Substance

The properties of a substance that undergoes linear decay are such that its concentration changes with time, because of reactions with other materials, and because of dilution effects following a zero order process model. The model parameters are user definable. Figure 7.29 shows the substance properties dialogue box where the model can be configured.

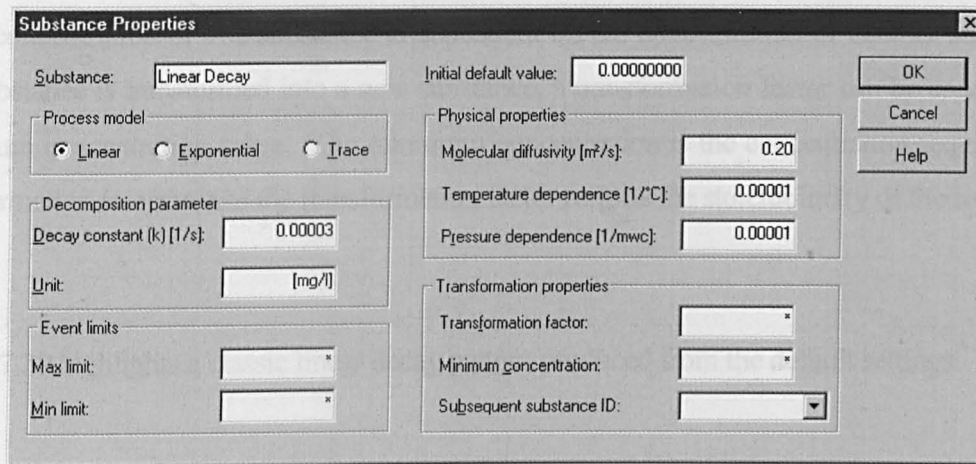


Figure 7.29 The substance properties dialogue box for linear decay

The dialogue box is used to define the identity, the initial default value, the decomposition process, and the physical and chemical properties of the substance. The maximum number of substances is four and the programmable characteristics for each substance type are described below.

7.4.3.1.1 Process model

There are three process model types available in the model, conservative substance (trace), linear decay and exponential decay.

7.4.3.1.2 Decomposition parameter

The Linear and Exponential process models have a basic decomposition parameter, i.e. a basic decay constant, k_v, n

7.4.3.1.3 Physical properties

Values can be entered for the molecular diffusivity, temperature and pressure effects. These three values are automatically incorporated with the basic decay constant into an actual decay constant for the bulk flow.

It is possible to assign the units of the substance concentration by entering a text string. However, this unit is only used as a label for the output data. The calculation is not affected by the choice of unit.

7.4.3.1.4 Transformation properties

If the concentration of one substance is dependent on the concentration of another substance i.e. one substance is transformed into a new substance, a transformation factor can be specified and a minimum concentration value. The minimum concentration is the concentration required for the transformation to occur and the transformation factor reflects the stoichiometry of the reaction.

Figure 7.30 highlights a classic linear decay pattern produced from the default settings.

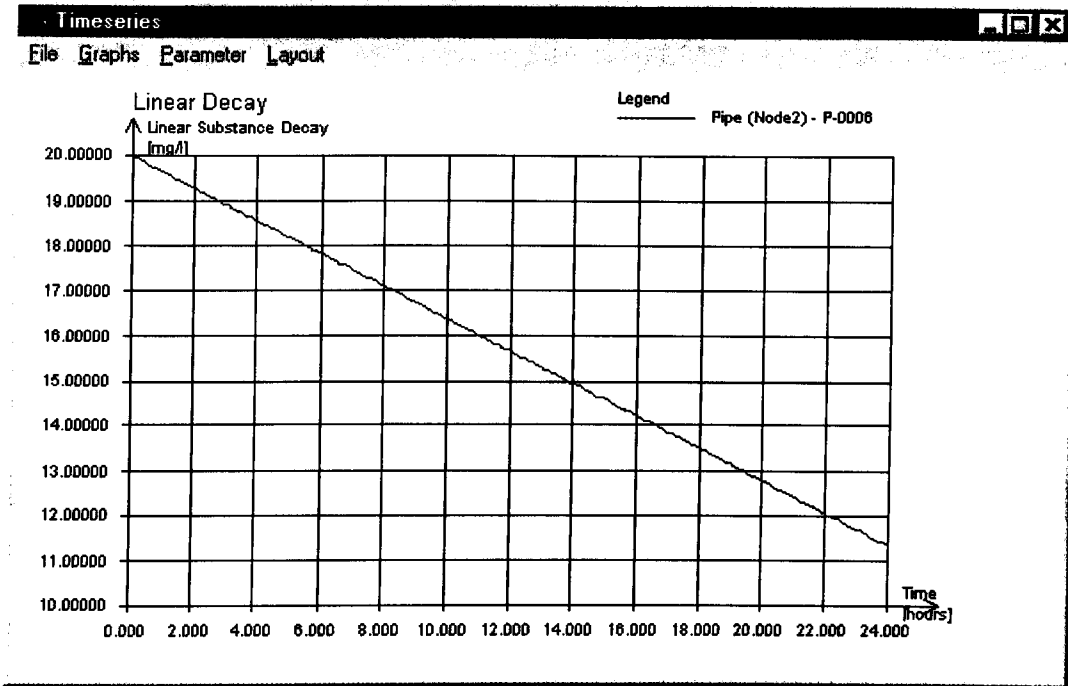


Figure 7.30 A linear decay pattern produced from the process model default settings

The decay constant determines the rate of decay, i.e. the slope of the line. Figures 7.31 to 7.36 demonstrate the effect of varying the decay constant.

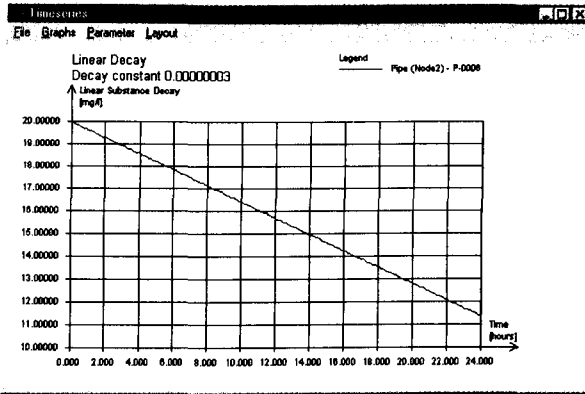


Figure 7.31 Decay rate constant = 0.00000003

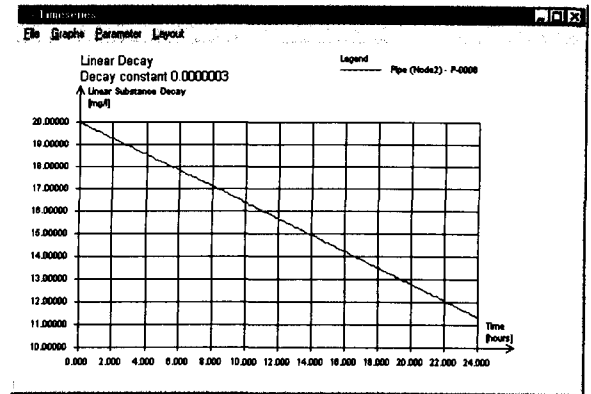


Figure 7.32 Decay rate constant = 0.0000003

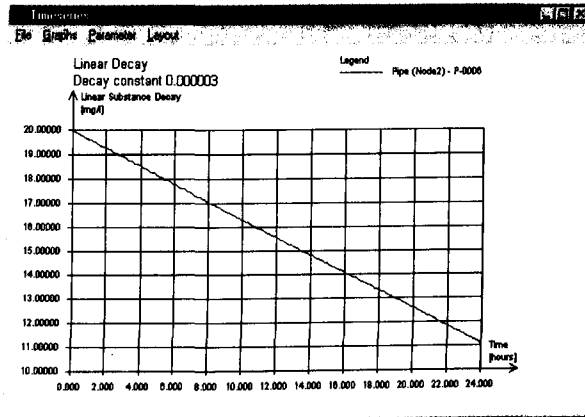


Figure 7.33 Decay rate constant = 0.0000003

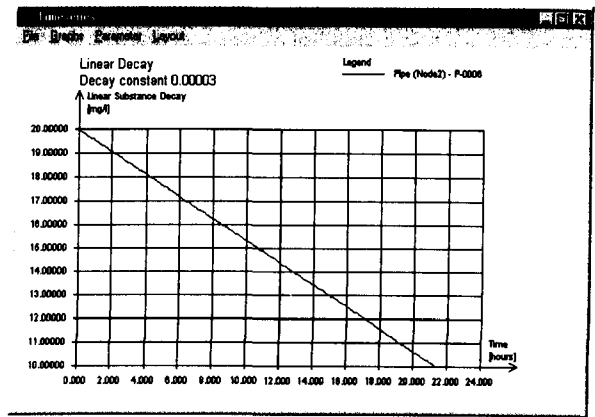


Figure 7.34 Decay rate constant = 0.000003

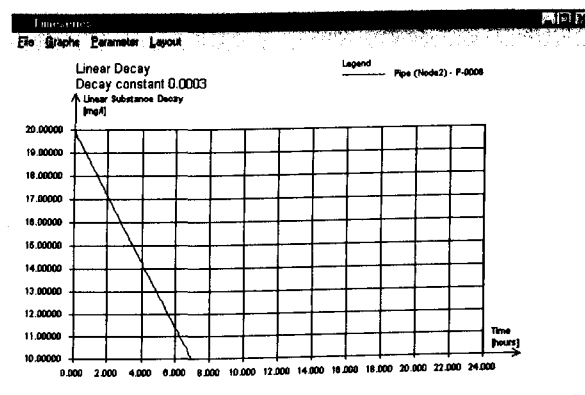


Figure 7.35 Decay rate constant = 0.0003

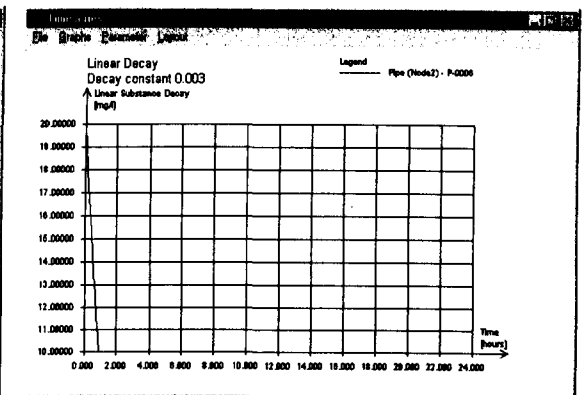


Figure 7.36 Decay rate constant = 0.003

The variance in the decay constant required to produce the changes seen in the figures is six orders of magnitude. This makes the decay constant extremely flexible with almost infinite configurable values.

Temperature and pressure can both be accounted for in the model.

Temperature and pressure are both assumed to add proportionally to the decay rate constants. This is achieved by adjusting $K_{v,n}$ to include terms for temperature and pressure as shown in equation 7.31 below.

$$k_b = k_{v,n} + \alpha(T - T_0) + \beta(p - p_0) \quad (7.31)$$

Where:

$k_{v,n}$ is the decay rate constant (user defined)

α is the temperature dependency factor

β is the pressure dependency factor

T is the measured (or user defined) temperature ($^{\circ}\text{C}$).

T_0 is a user defined global reference temperature ($^{\circ}\text{C}$).

p is the actual (measured or calculated) pressure (mwc).

p_0 is a user defined global reference pressure (mwc).

The effect of global reference temperature on the bulk flow decay rate is shown in figures 7.37 to 7.42.

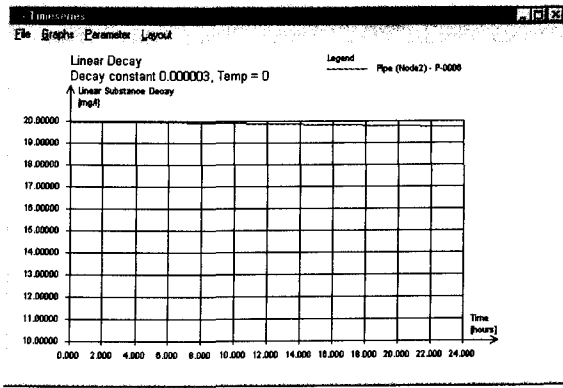


Figure 7.37 Temperature = 0 °C

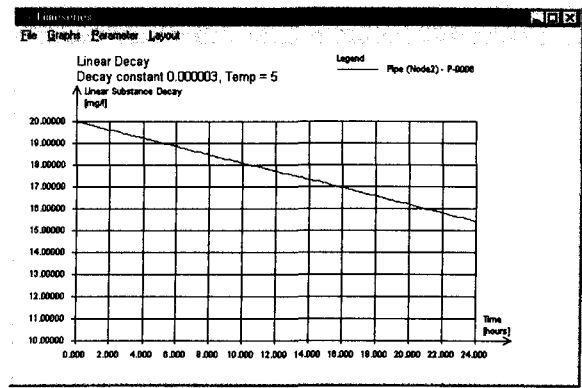


Figure 7.38 Temperature = 5 °C

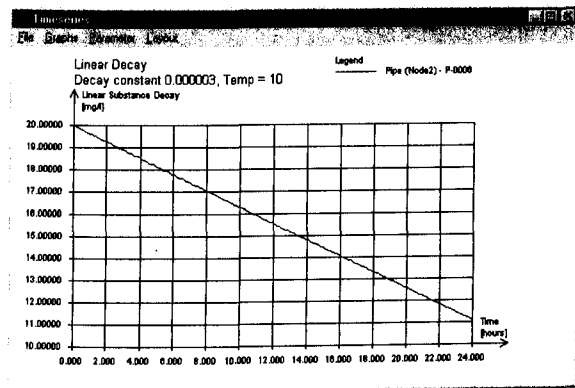


Figure 7.39 Temperature = 10 °C

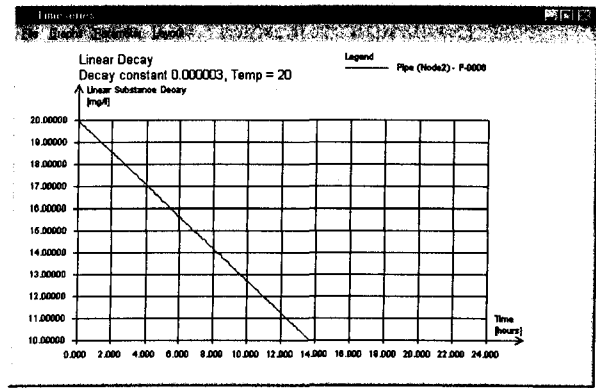


Figure 7.40 Temperature = 20 °C

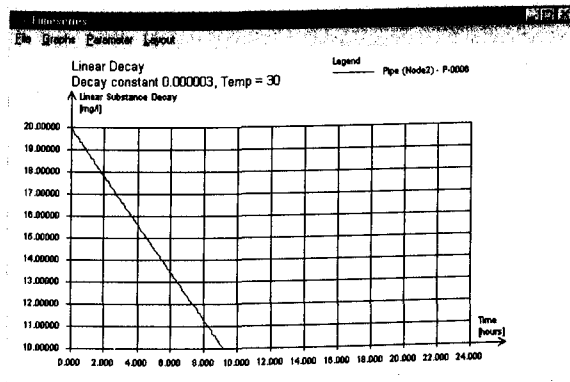


Figure 7.41 Temperature = 30 °C

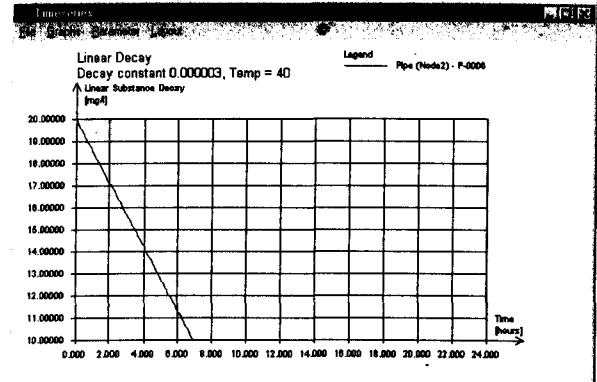


Figure 7.42 Temperature = 40 °C

The dependency effect of temperature is also configurable, as the true effects are not yet fully understood. This permits the user to apply a range of effects due to differing temperature and pressure and to apply real values when they are determined. Figures 7.43 to 7.48 demonstrate the effect of temperature dependency.

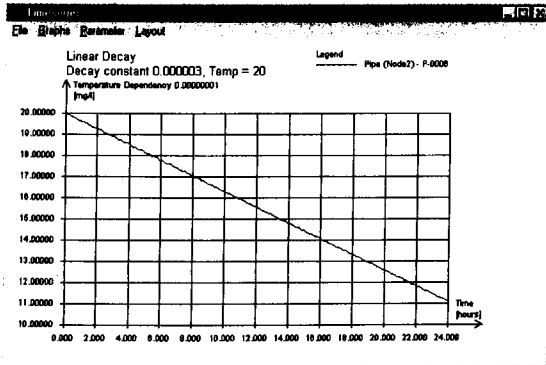


Figure 7.43 Temperature dependency=0.0000001

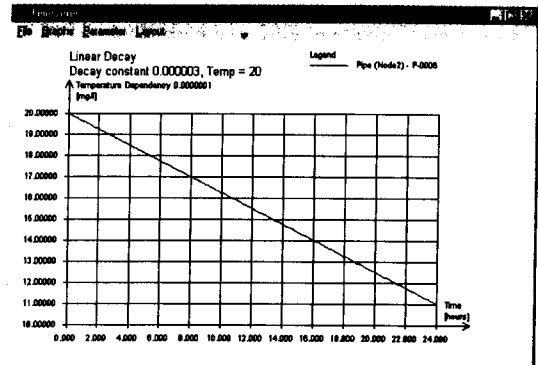


Figure 7.44 Temperature dependency=0.0000001

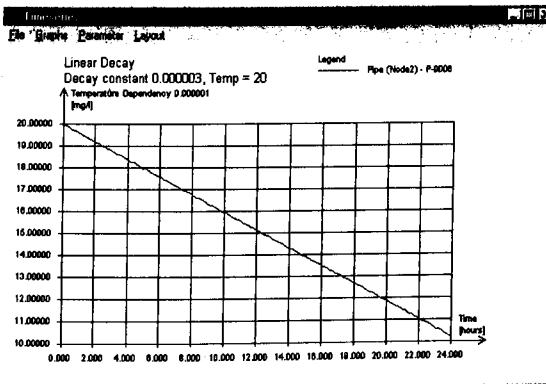


Figure 7.45 Temperature dependency=0.000001

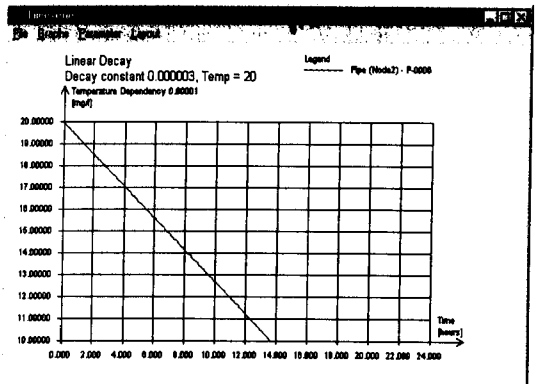


Figure 7.46 Temperature dependency=0.00001

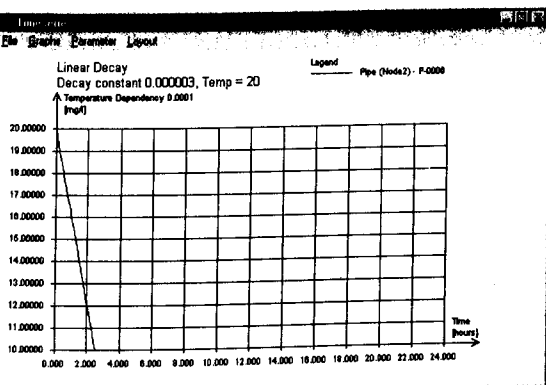


Figure 7.47 Temperature dependency=0.0001

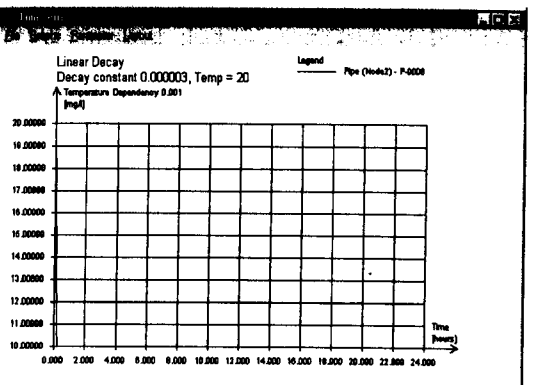


Figure 7.48 Temperature dependency=0.001

Figures 7.49 to 7.54 demonstrate the effect of pressure dependency

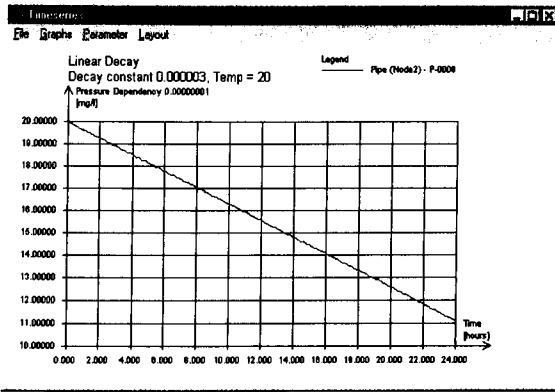


Figure 7.49 Pressure dependency = 0.0000001

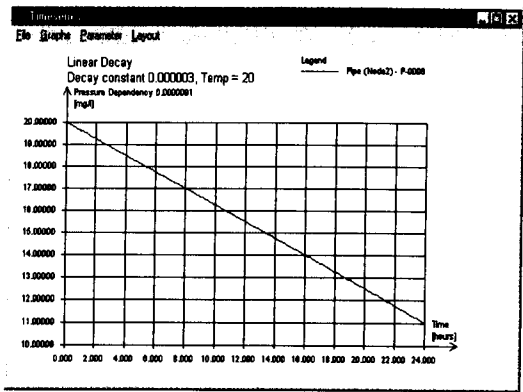


Figure 7.50 Pressure dependency = 0.0000001

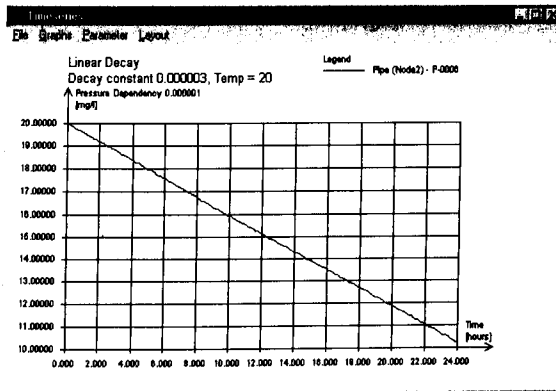


Figure 7.51 Pressure dependency = 0.000001

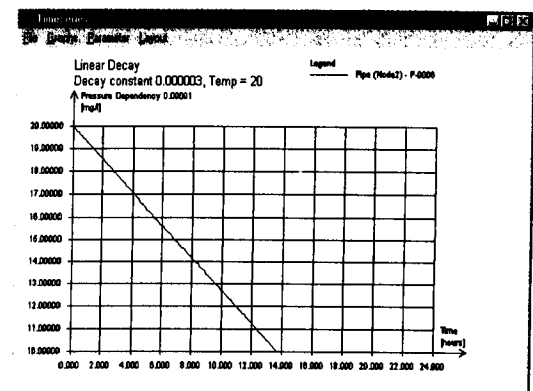


Figure 7.52 Pressure dependency = 0.00001

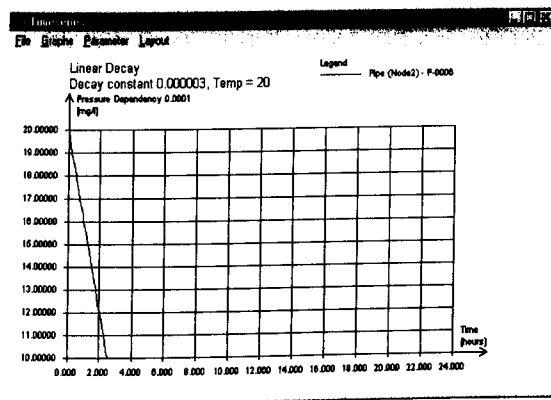


Figure 7.53 Pressure dependency = 0.0001

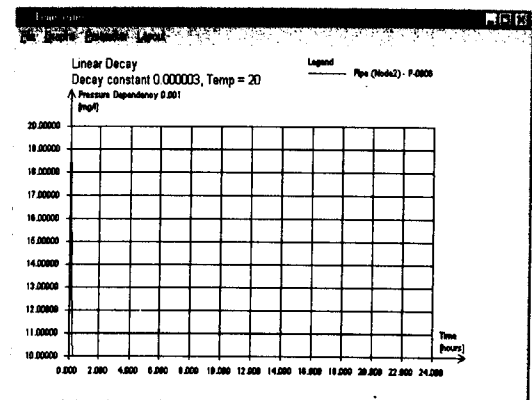


Figure 7.54 Pressure dependency = 0.001

Reactions with / at the pipe wall are accounted for by inclusion in the model of a pipe wall coefficient K_w .

Assuming that the reaction of substance at the pipe wall is first order with respect to the wall concentration C_w , and that it proceeds at the same rate as substance is transported to the wall, the mass balance for substance at the wall can be represented by:

$$k_f (C - C_w) = k_w C_w \quad (7.32)$$

Where: k_w is pipe wall decay rate coefficient ($\text{m}\cdot\text{s}^{-1}$).

Figures 7.55 to 7.60 demonstrate the effect of varying the pipe wall coefficient k_w

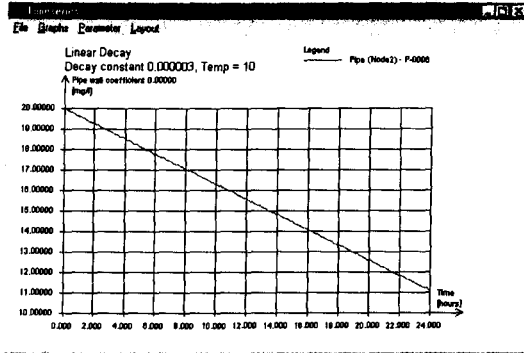


Figure 7.55 $K_w=0.00$

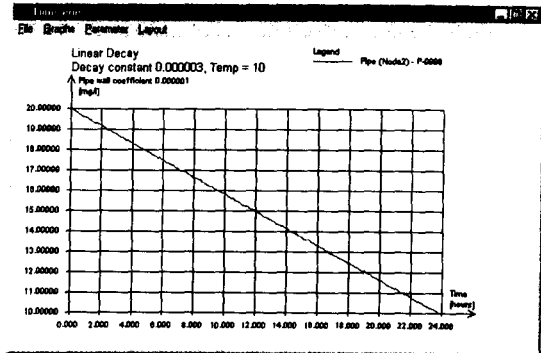


Figure 7.56 $K_w=0.000001$

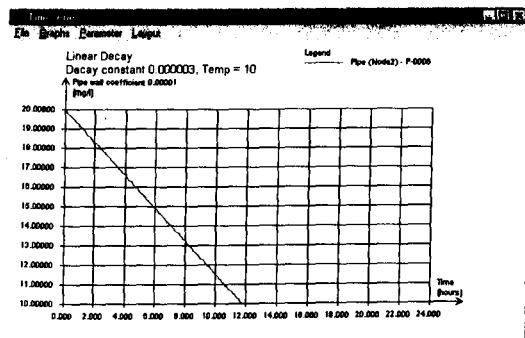


Figure 7.57 $K_w=0.00001$

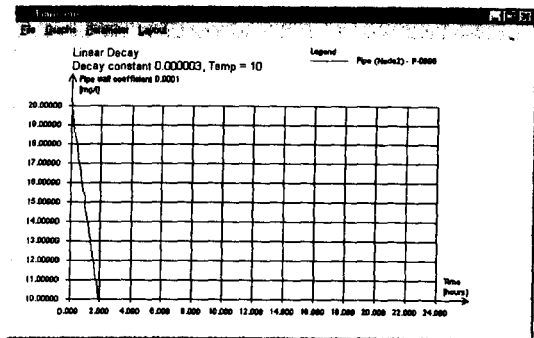


Figure 7.58 $K_w=0.0001$

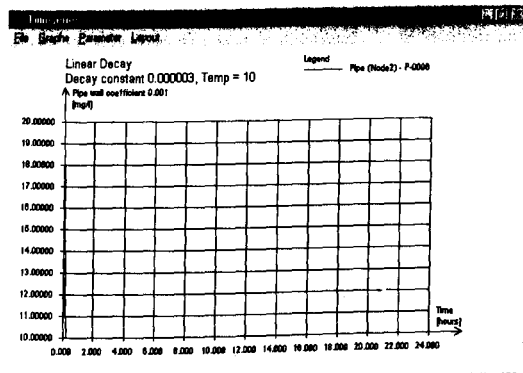


Figure 7.59 $K_w=0.001$

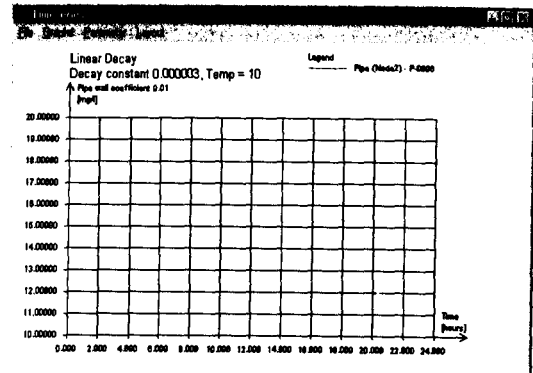


Figure 7.60 $K_w=0.01$

As explained earlier, because the mass transfer coefficient k_f , 7.19, is calculated using the equation

$$k_f = Sh \frac{D}{d}$$

Where:

$$Sh = 0.023 Re^{0.83} Sc^{0.333} \quad \text{for } Re \geq 2300$$

$$Sh = 3.65 + \frac{0.0668(d/L)(ReSc)}{1 + 0.04((d/L)(ReSc))^{2/3}} \quad \text{for } Re < 2300$$

$$Re = \frac{Vd}{\nu}$$

$$Sc = \frac{\nu}{D}$$

Where:

Sh is the Sherwood Number (Dimensionless).

Re is the Reynolds Number (Dimensionless).

Sc is the Schmidt Number (Dimensionless).

D is the molecular diffusivity of substance in water (m^2/s).

ν is the kinematic viscosity of water (m^2/s).

L is the pipe length (m).

For a particular substance, k_f is a function of pipe diameter, flow velocity, and temperature as it affects diffusivity and viscosity. The effect of the molecular diffusivity value therefore will be minimal when viewed as an incorporated change in the decay constant K_b . Figures 7.61 to 7.63 highlight this.

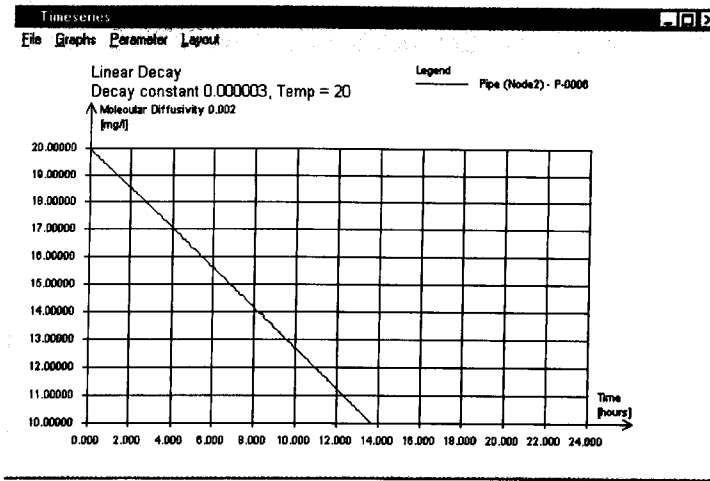


Figure 7.61 Molecular diffusivity 0.001

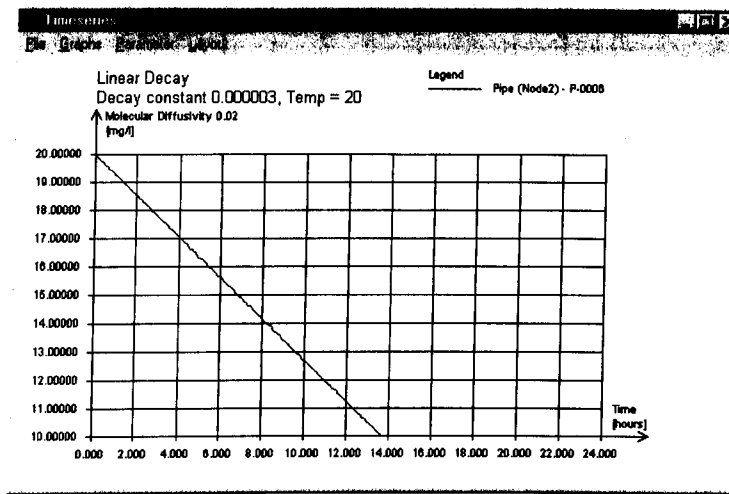


Figure 7.62 Molecular diffusivity 0.02

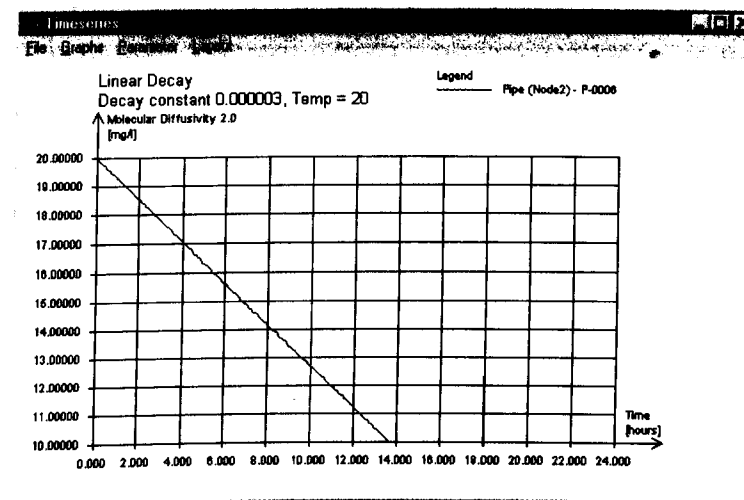


Figure 7.63 Molecular diffusivity 2.0

The above effects can be combined to provide more model flexibility thereby making it easier to calibrate models for different networks with differing properties. Figure 7.64 shows the combined effect of the decay constant and pipe wall coefficient at a temperature of 20 °C.

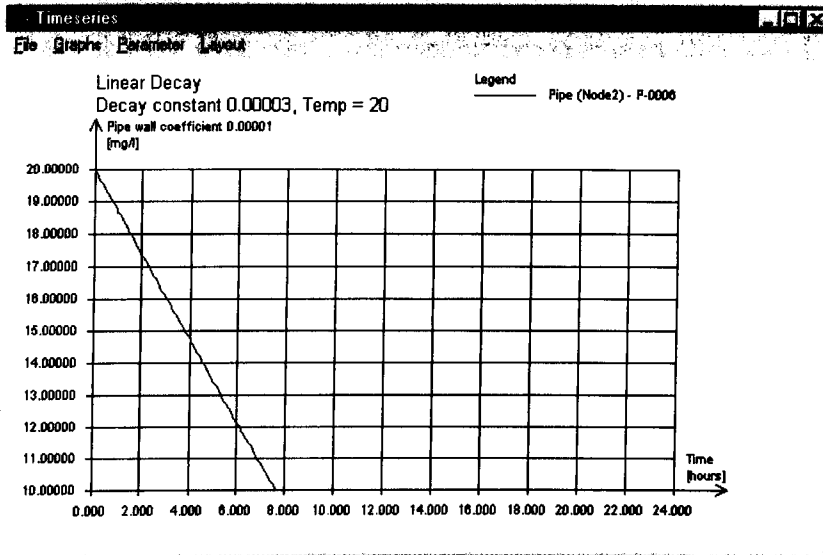


Figure 7.64 The combined effect of the decay constant and pipe wall coefficient at a temperature of 20 °C

Figure 7.65 demonstrates the effect of changing the temperature to 30 °C

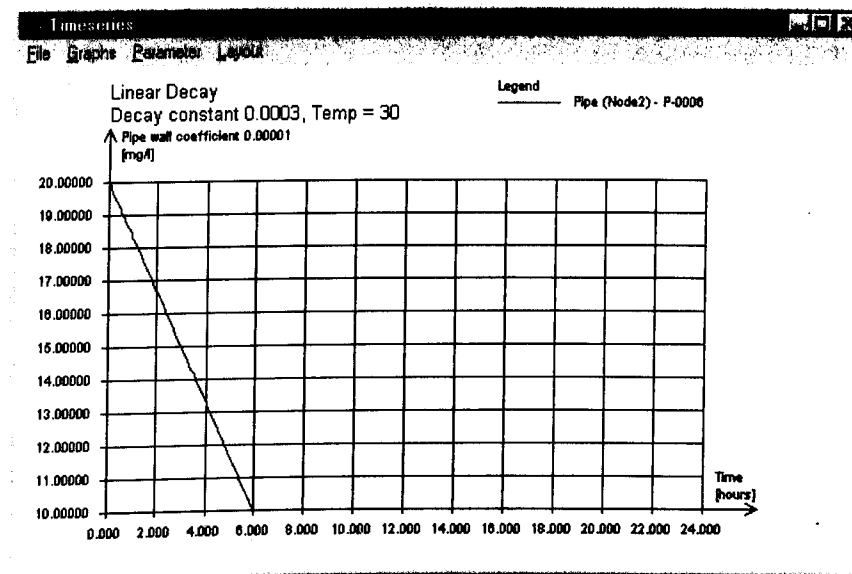


Figure 7.65 The effect of increasing temperature to 30 °C

The effect is clear as the decay is seen to increase significantly. Figure 7.66 shows the superimposed effect of adding a pressure dependency.

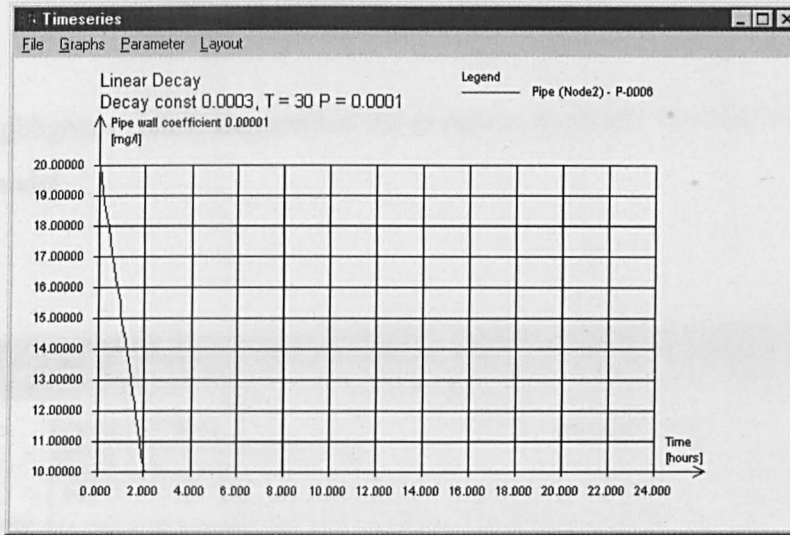


Figure 7.66 The superimposed effect of adding a pressure dependency

7.4.3.2 Exponential Decay of a Substance

The properties of a substance that undergoes exponential decay are such that its concentration changes with time, reactions with other materials following a 1st order process model, and as a result of dilution effects. The model parameters are user definable. Figure 7.67 shows the substance properties dialogue box where the model can be configured.

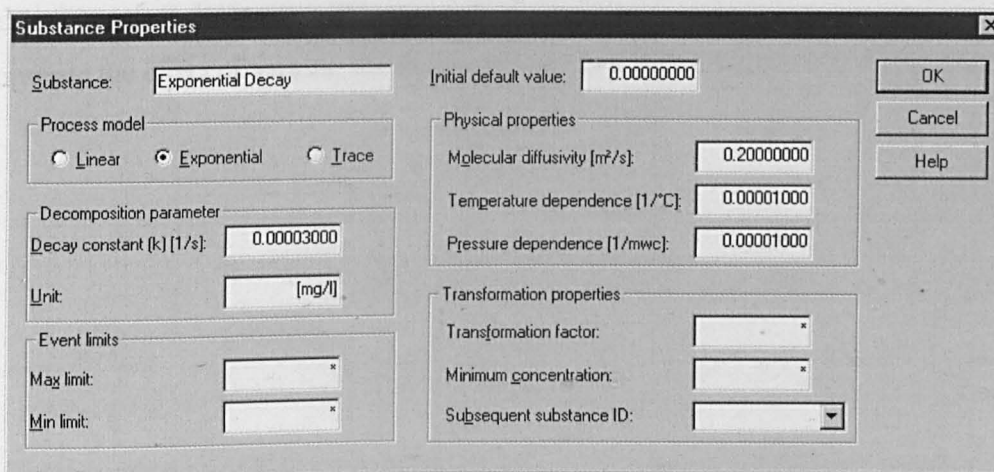


Figure 7.67 Shows the substance properties dialogue box where the model can be configured

The exponential process model has a basic decomposition parameter, i.e. a basic decay constant, $k_{v,n}$ that can be modified by the cumulative effects of a number of other parameters including pipe wall coefficient, temperature and pressure.

Figure 7.68 highlights a classic exponential decay pattern produced from the default settings of the linear decay model.

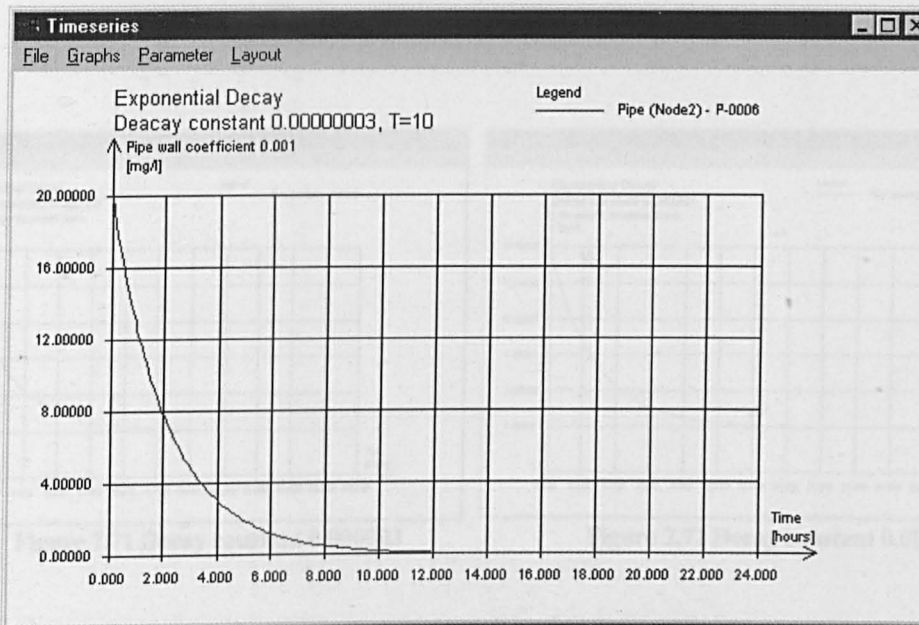


Figure 7.68 Classic exponential decay pattern produced using the linear decay model default settings

The decay constant determines the rate of decay, i.e. the slope of the curve. Figures 7.69 to 7.74 demonstrate the effect of varying the decay constant at a given temperature and pressure.

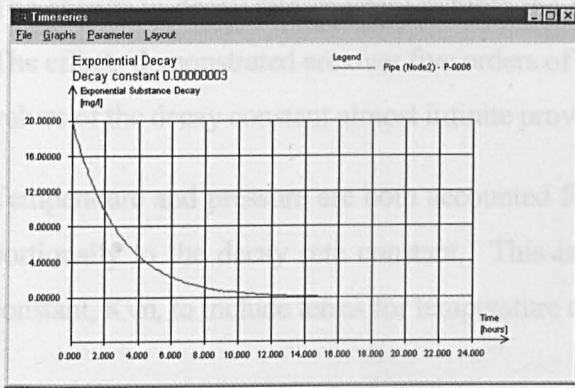


Figure 7.69 Decay constant 0.00000003

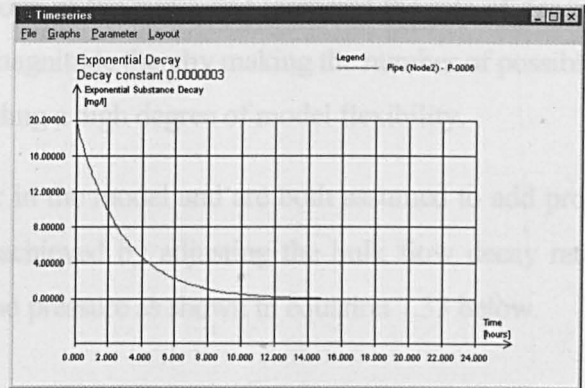


Figure 7.70 Decay constant 0.0000003

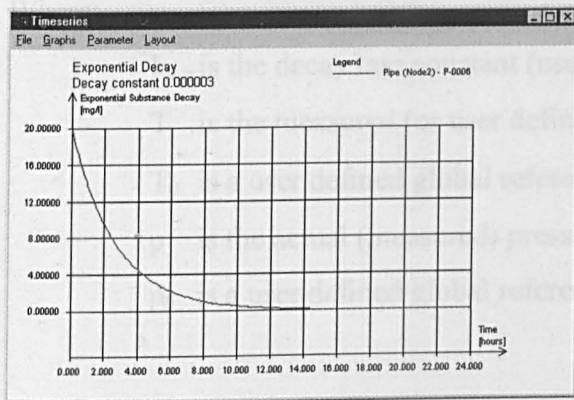


Figure 7.71 Decay constant 0.000003

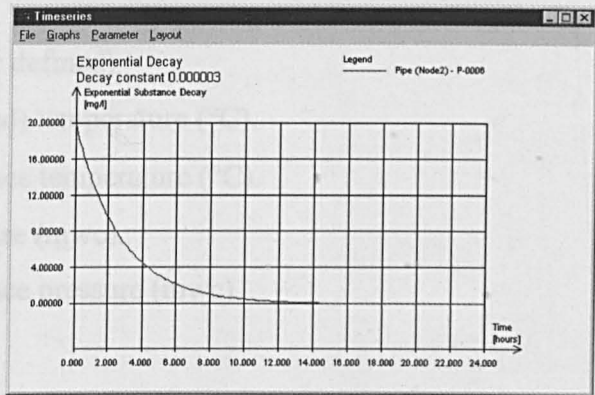


Figure 7.72 Decay constant 0.00003

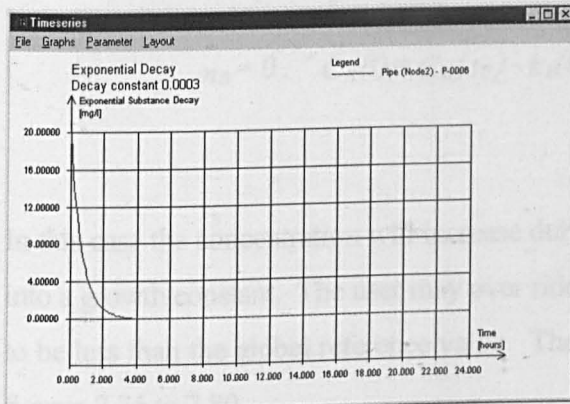


Figure 7.73 Decay constant 0.0003

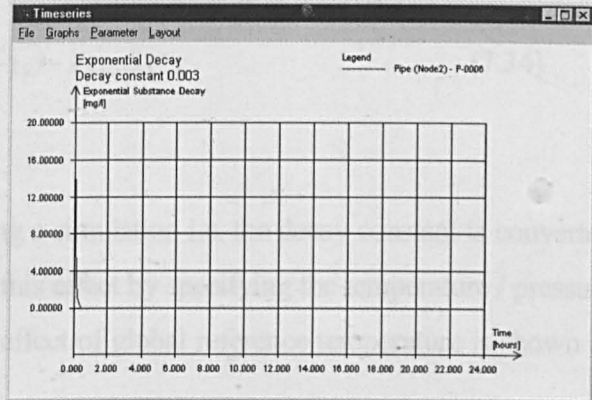


Figure 7.74 Decay constant 0.003

The change in decay rate constant reduces the slope of the curve and increases the rate of decay. The effects demonstrated are over five orders of magnitude thereby making the number of possible values of the decay constant almost infinite providing a high degree of model flexibility.

Temperature and pressure are both accounted for in the model and are both assumed to add proportionally to the decay rate constant. This is achieved by adjusting the bulk flow decay rate constant, $K_{v,n}$, to include terms for temperature and pressure as shown in equation 7.33 below.

$$k_b = k_{v,n} + \alpha(T - T_0) + \beta(p - p_0) \quad (7.33)$$

Where:

$k_{v,n}$ is the decay rate constant (user defined)

T is the measured (or user defined) temperature (°C).

T_0 is a user defined global reference temperature (°C).

p is the actual (measured) pressure (mwc).

p_0 is a user defined global reference pressure (mwc).

The decay rate constant in the bulk flow is allowed to be negative due to the last two terms in the following equation:

$$n_B = 0: C_B(t) = C_B(t_0) - k_B(t - t_0) - k_{trans} \Delta C_A \quad (7.34)$$

In this case the concentration will increase during a simulation i.e. the decay constant is converted into a growth constant. The user may over ride this effect by specifying the temperature / pressure to be less than the global reference value. The effect of global reference temperature is shown in figures 7.75 to 7.80

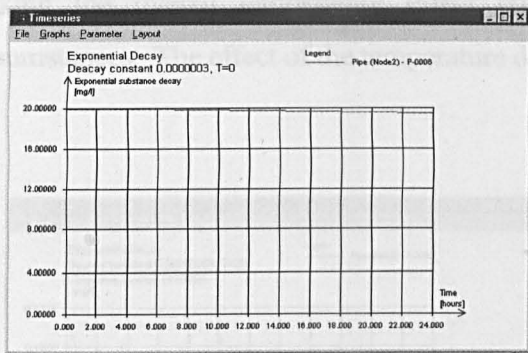


Figure 7.75 Global reference temperature = 0°C

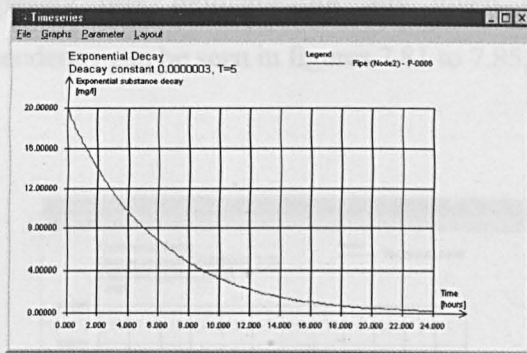


Figure 7.76 Global reference temperature = 5°C

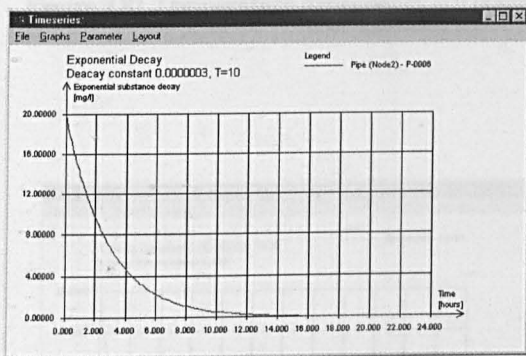


Figure 7.77 Global reference temperature = 10°C

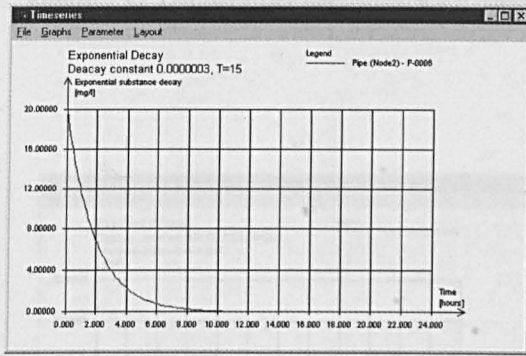


Figure 7.78 Global reference temperature = 15°C

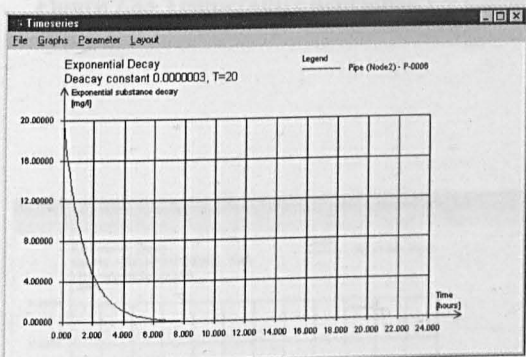


Figure 7.79 Global reference temperature = 20°C

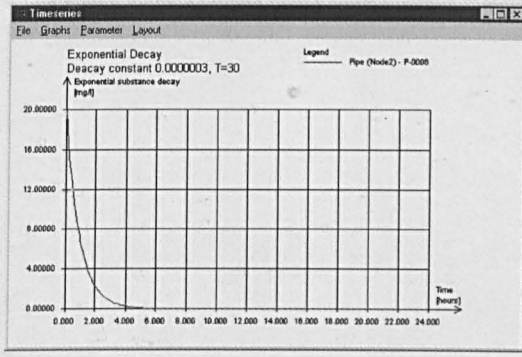


Figure 7.80 Global reference temperature = 30°C

A temperature change is assumed to add proportionally to the decay rate. However, as this may not be the case for all reactions, applying a temperature dependency coefficient can modify the effect. This factor changes the magnitude of effect a given change in temperature has thereby

making the overall temperature effect completely user definable for any given set of circumstances. The effect of the temperature dependency can be seen in figures 7.81 to 7.85.

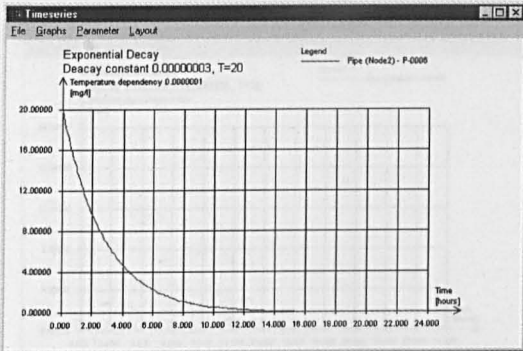


Figure 7.81 Temperature dependency = 0.0000001

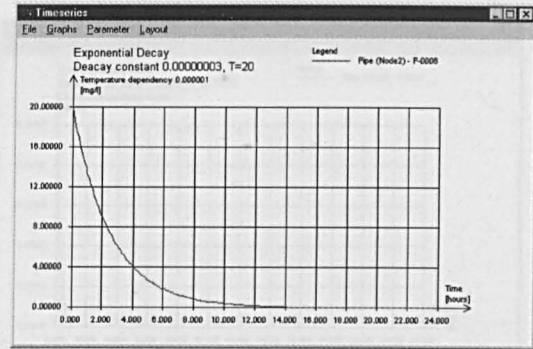


Figure 7.82 Temperature dependency = 0.0000001

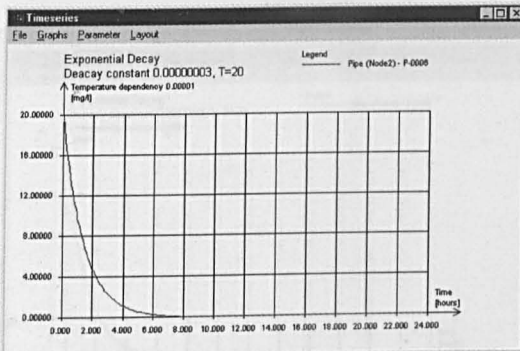


Figure 7.83 Temperature dependency = 0.00001

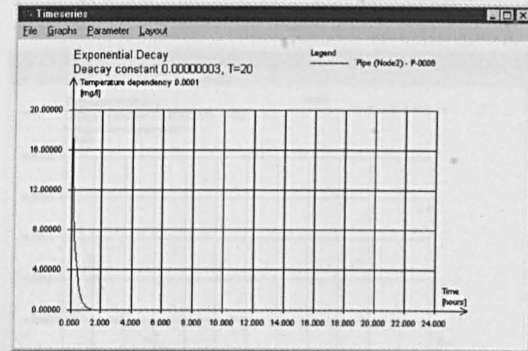


Figure 7.84 Temperature dependency = 0.0001

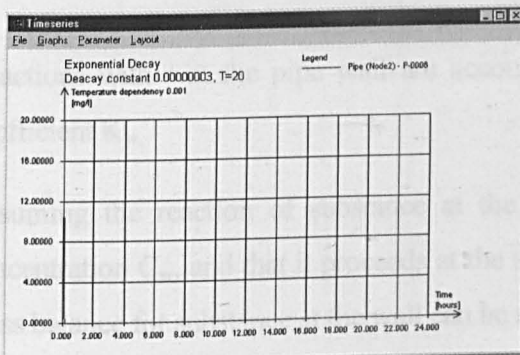


Figure 7.85 Temperature dependency = 0.001

Similar functionality is available for pressure. Figures 7.86 to 7.89 show the effect of the pressure dependency coefficient.

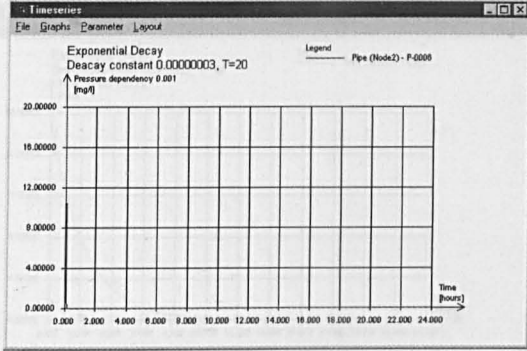


Figure 7.86 Pressure dependency = 0.001

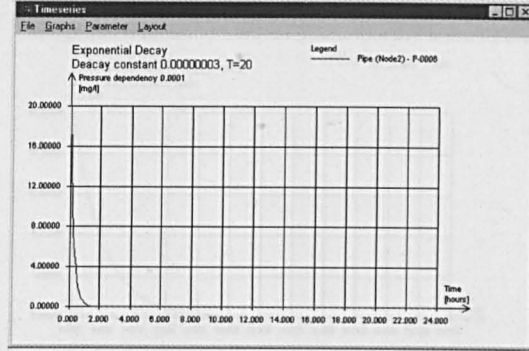


Figure 7.87 Pressure dependency = 0.0001

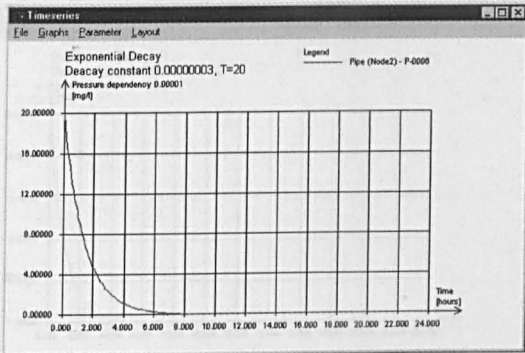


Figure 7.88 Pressure dependency = 0.00001

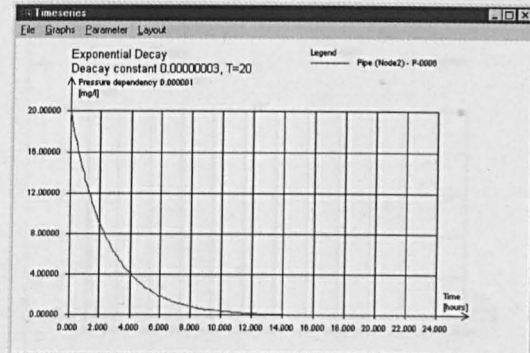


Figure 7.89 Pressure dependency = 0.000001

Reactions with / at the pipe wall are accounted for by inclusion in the model of a pipe wall coefficient K_w .

Assuming the reaction of substance at the pipe wall is first order with respect to the wall concentration C_w , and that it proceeds at the same rate as substance is transported to the wall, the mass balance for substance at the wall can be represented by:

$$k_f (C - C_w) = k_w C_w \quad (7.35)$$

Where: k_w is pipe wall decay rate coefficient (m/s).

Figures 7.90 to 7.93 demonstrate the effect of varying K_w

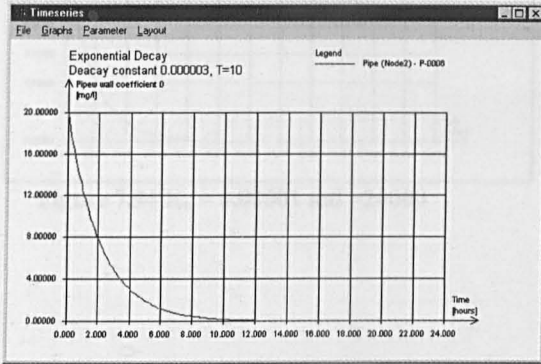


Figure 7.90 Pipe wall decay rate 0.0

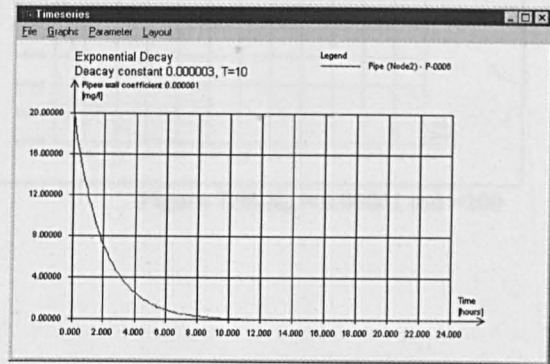


Figure 7.91 Pipe wall decay rate 0.0000001

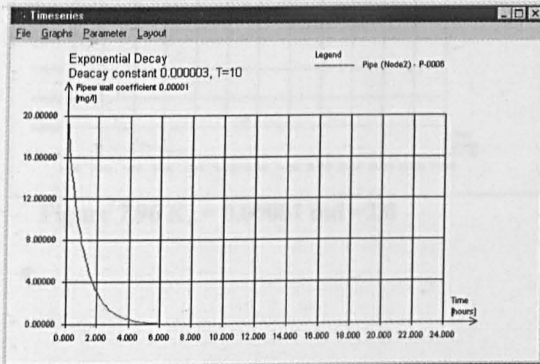


Figure 7.92 Pipe wall decay rate 0.00001

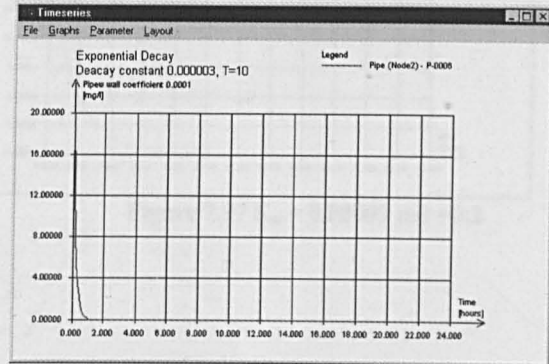


Figure 7.93 Pipe wall decay rate 0.0001

The effect of the molecular diffusivity value therefore will be minimal when viewed as an incorporated change in the decay constant K_b . Figures 7.94 through 7.98 highlight this clearly.

The above effects can be combined to provide extreme model flexibility thereby making it easier to

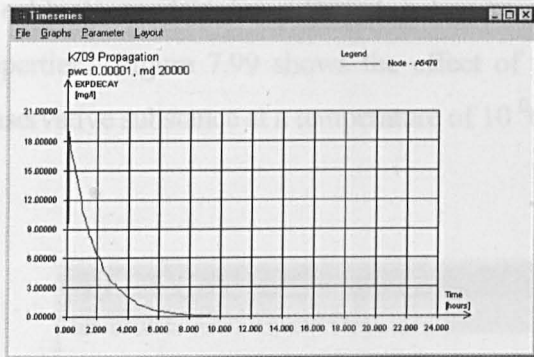


Figure 7.94 $K_w = 0.00001$ $md = 20000$

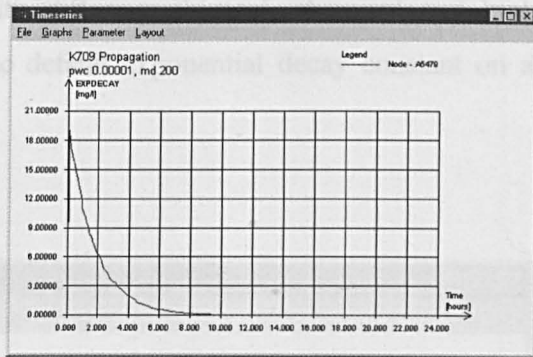


Figure 7.95 $K_w = 0.00001$ $md = 200$

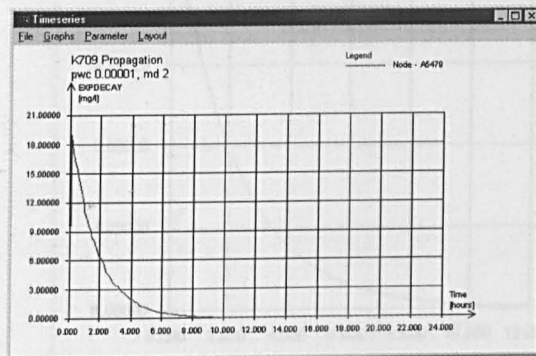


Figure 7.96 $K_w = 0.00001$ $md = 2.0$

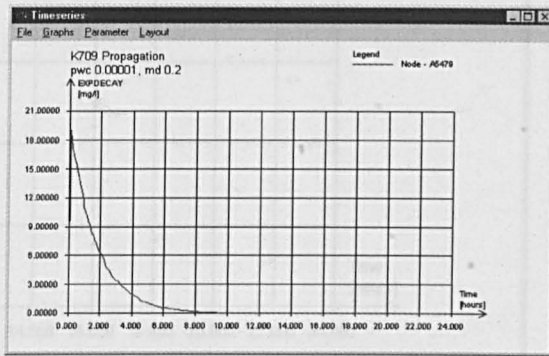


Figure 7.97 $K_w = 0.00001$ $md = 0.2$

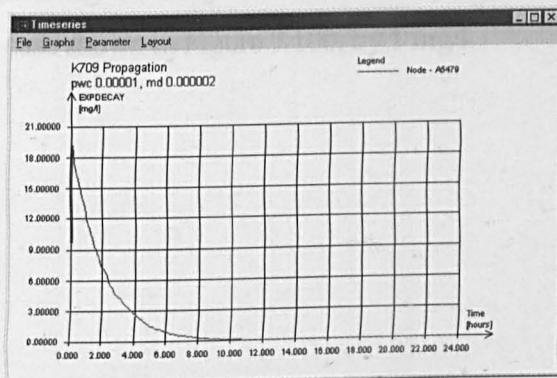


Figure 7.98 $K_w = 0.00001$ $md = 0.000002$

The much larger effects of the other coefficients swamp the small effect of the contribution from the molecular diffusivity when combined in the overall decay constant.

The above effects can be combined to provide extreme model flexibility thereby making it easier to calibrate models for different networks with differing physical, chemical and biological properties. Figure 7.99 shows the effect of the default exponential decay constant on a non-conservative substance at a temperature of 10 °C.

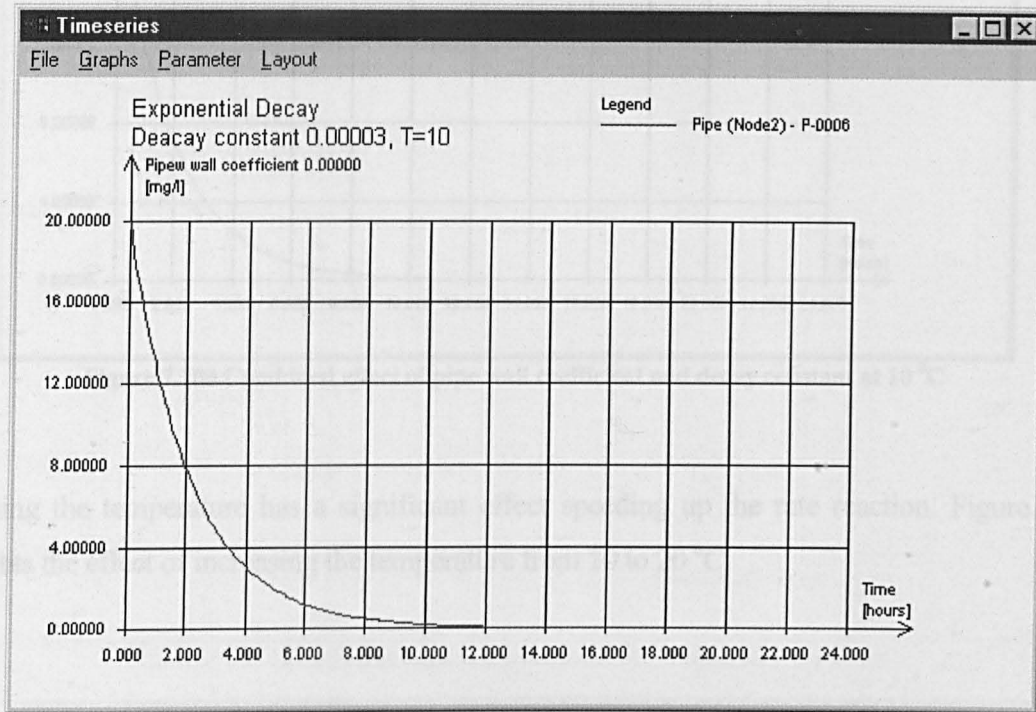


Figure 7.99 The effect of the decay constant at a temperature of 10 °C

Combining the decay constant and the pipe wall coefficient reduces the substance concentration, in the case shown in Figure 7.100, by 1 mg/l.

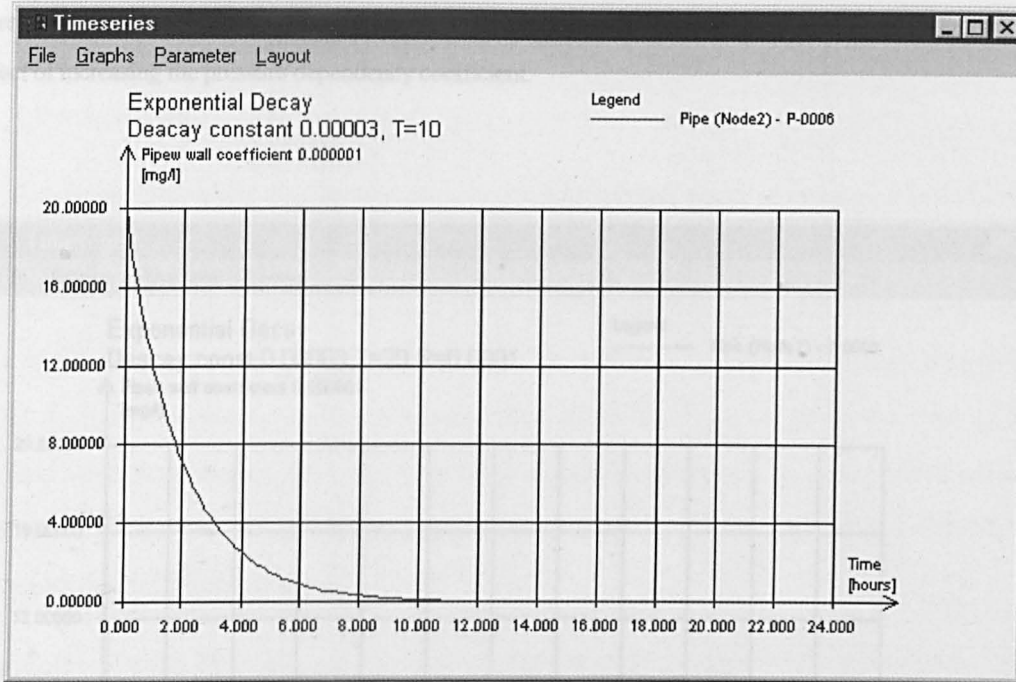


Figure 7.100 Combined effect of pipe wall coefficient and decay constant at 10 °C

Increasing the temperature has a significant effect speeding up the rate reaction. Figure 7.101 highlights the effect of increasing the temperature from 10 to 20 °C.

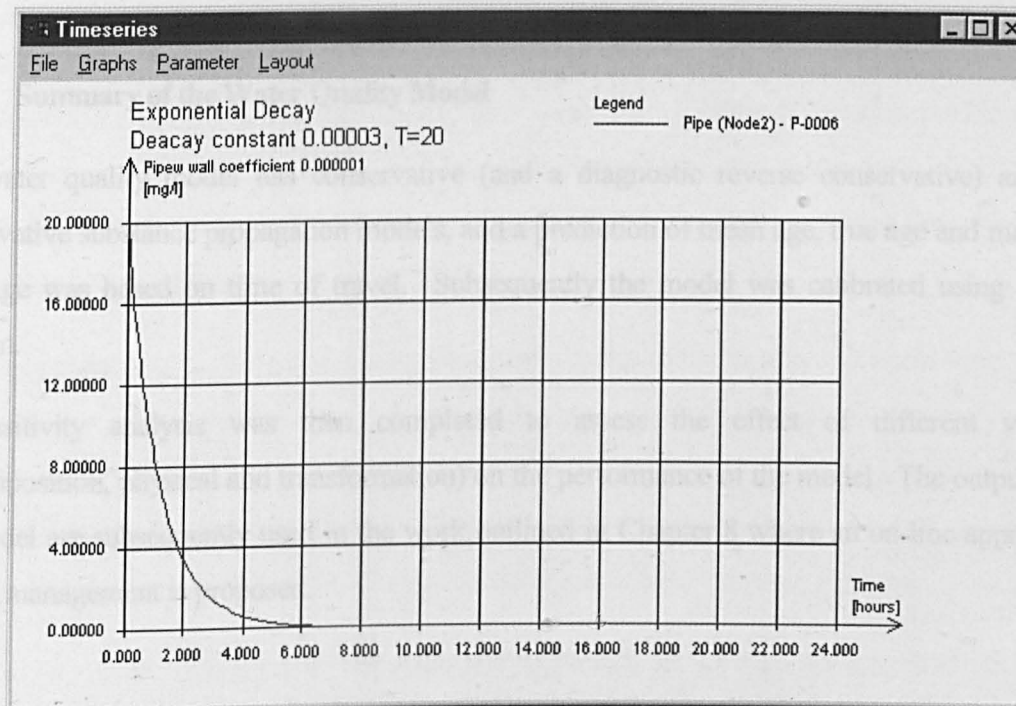


Figure 7.101 The effect of increasing the temperature from 10 to 20 °C

If a pressure coefficient is now superimposed, the effect on the overall decay rate is significant. Figure 7.102 depicts the effect of increasing the pressure dependency coefficient.

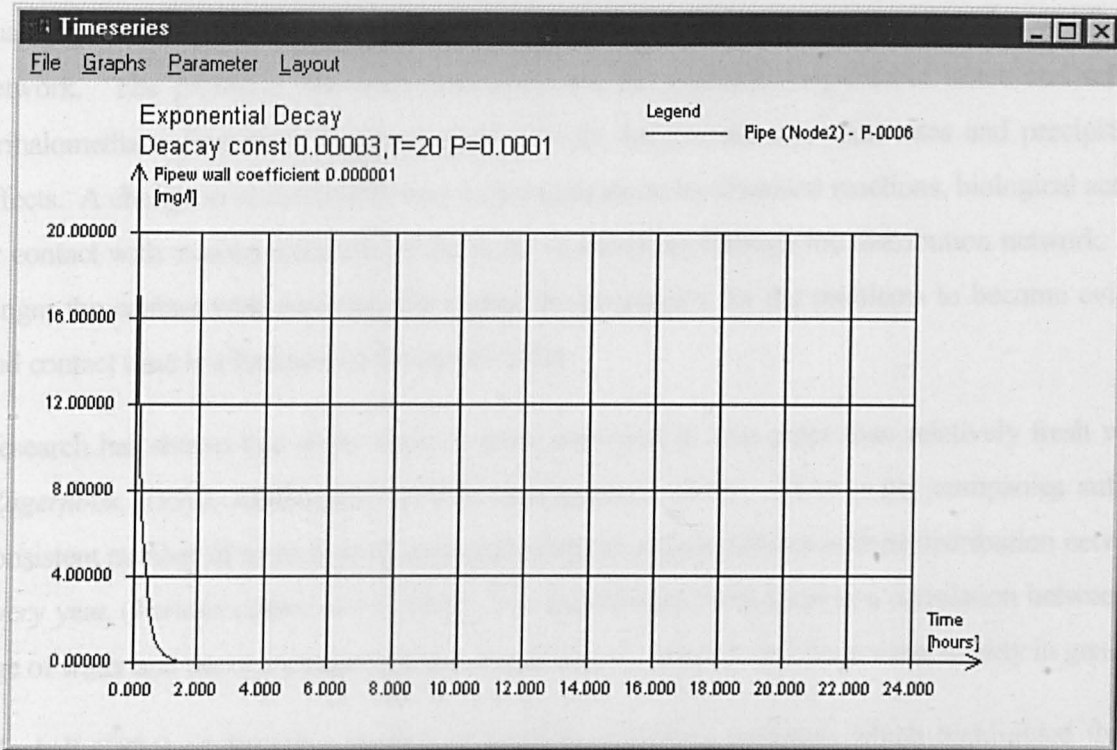


Figure 7.102 The effect of increasing the pressure dependency coefficient

7.4.4 Summary of the Water Quality Model

The water quality model has conservative (and a diagnostic reverse conservative) and non-conservative substance propagation models, and a prediction of mean age, true age and maximum age. Age was based on time of travel. Subsequently the model was calibrated using a tracer solution.

A sensitivity analysis was then completed to assess the effect of different variables (decomposition, physical and transformation) on the performance of the model. The outputs from the model are subsequently used in the work outlined in Chapter 8 where an on-line approach to system management is proposed.

7.5 Age of Water

7.5.1 Background

Age is an important water quality parameter. Newly treated water may have a potential for water quality problems that may only become evident when the water ages within the distribution network. The problems manifest themselves as, for example, unpalatable tastes and odours, Trihalomethane formation, bacteriological activity, heightened corrosion rates and precipitation effects. A change in water quality may be brought about by chemical reactions, biological activity or contact with various materials as the water is travelling through the distribution network. The longer the contact with materials the higher the propensity for the problems to become evident, and contact time is a function of the age of water.

Research has shown that older water is more corrosive to iron pipes than relatively fresh water. (*Zagerholm, 1996*), (*Mallevalle, (1987)*), (*Burlingame,(1995)*). Most water companies suffer a consistent number of unaccounted for bacteriological sample failures in their distribution networks every year. (*Various contributors, 1992*) It is hypothesised that there is a correlation between the age of water and the occurrence of these unsatisfactory samples and poor water quality in general.

Machell (1991), undertook a review of existing modelling packages which highlighted that, in general, only simple mathematics were used whereby the mean of the individual water ages merging at the node were used to represent the age of water at the node. In reality however, it is not possible to mix ages in this way to produce a mean age. If water 10 hours old is blended with an equal amount of water of 2 hours old the resultant mixture is not 6 hours old. The important thing is that half the water is five times as old as the rest of the water reaching the node and will have different characteristics.

Mean age calculated in this manner may be a useful guide in that it might provide *some* evidence of older water within the network (if for example the mean is much higher than expected) but it does not allow the identification of the older water components or where or how they originate. Nor does this simple approach allow for flow reversals within pipes or water entering the network that has already aged, for example, in a service reservoir or long transmission main.

Taken to its logical conclusion, by using this simple method volumes of water with a high age, that may have extremely poor quality characteristics, can be present in, or moved around, a network and not be identified using current age calculation models.

The objective of developing the age functionality in this study was therefore to provide a model that could more accurately assess the age of water within a distribution network by providing information about the constituent age components that contribute to the mean age. The model takes into account flow reversals and ageing in service reservoirs and along transmission mains.

Because several flows with several individual age components may combine at many different nodes, the computational power required to identify all component ages simultaneously would be a major constraint. In order to get round this problem, a limit of nine user-defined age component bands that may be determined at each node was introduced. Also, initial age conditions can be imposed as global or individual pipe characteristics in order to reduce the number of iterations required to attain a solution and to lower simulation time.

It is proposed that an entire network can be assigned a *component age profile* and that the shape of the profile can be used to predict whether a network might suffer problems such as taste and odour, higher than normal corrosion rates or bacteriological activity.

In order to further the understanding of the hydraulic / age of water / bacteriological activity relationships a 'biological' model has also been proposed and is being developed (7.6.1).

The information gleaned by applying the models will provide new insights into the relationships between the hydraulic and water quality characteristics of any water distribution network.

7.5.2 Age Calculations

7.5.2.1 Retention Time

If a water particle enters a pipe at time t_0 , and the bulk flow velocity in the pipe is known, the computational power required to calculate how long it takes the particle to travel down the pipe is very small. The mathematics involves only a pipe length / flow rate relationship to determine how long the particle of water takes to go from one end of the pipe to the other. If the time at which the particle emerges from the pipe is t_1 , then $(t_1 - t_0)$ is called the *retention time* of the water particle in the pipe. The model can determine retention times in individual pipes. Figure 7.103 shows a plot of part of the study network with individual pipes coloured to reflect the retention times depicted in the key.

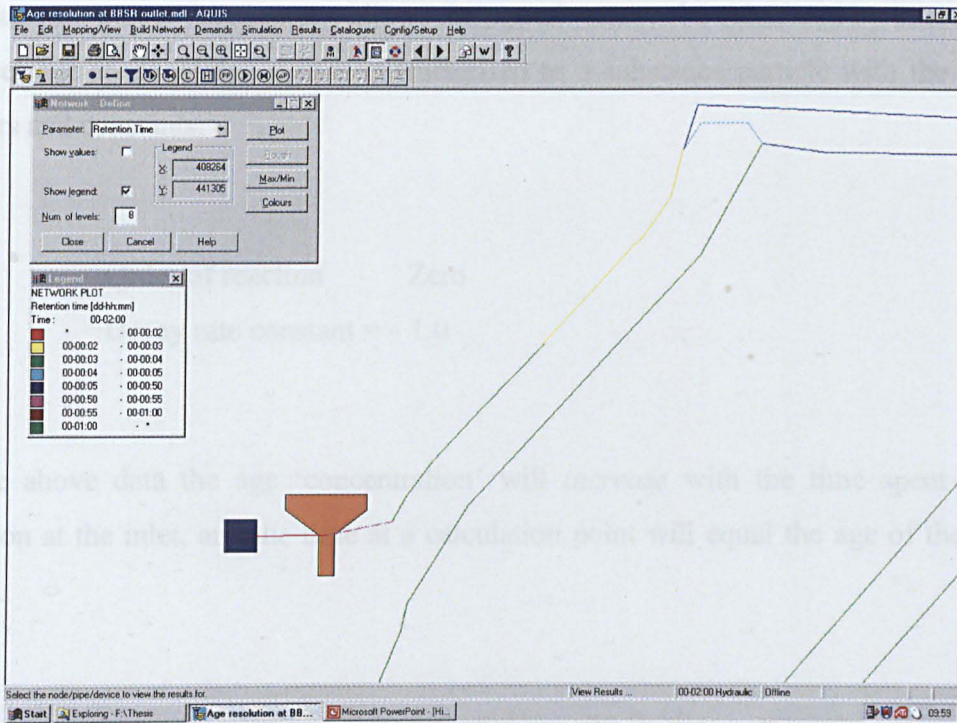


Figure 7.103 Plot of retention times in pipes

It can be seen that the first three pipes from the outlet of the service reservoir have the following retention times:

- 1 51 – 52 minutes
- 2 3 – 4 minutes
- 3 2 – 3 minutes
- 4 2 – 3 minutes

This gives a total retention time for of between 58 and 62 minutes.

7.5.2.2 Age of Water

To obtain the total time a water particle has been retained in a series of pipes is a question of summing the retention times in all the pipes the water particle has travelled through. This sum of times is called the *age* of the water particle.

The propagation model functionality allows the age of water to be calculated by introducing the 'substance' age at all inlets. Age is characterised as a substance particle with the following parameters and constants:

Order of reaction Zero

Decay rate constant = - 1.0

Using the above data the age 'concentration' will *increase* with the time spent from the introduction at the inlet, and the time at a calculation point will equal the age of the water at that point.

Substituting $k_{v,n} = -1$ into equation 7.6 we get:

$$C(t) = C(t_0) + (t - t_0)$$

Figure 7.104 depicts how this relationship is translated into a linear growth law relating time to age concentration.

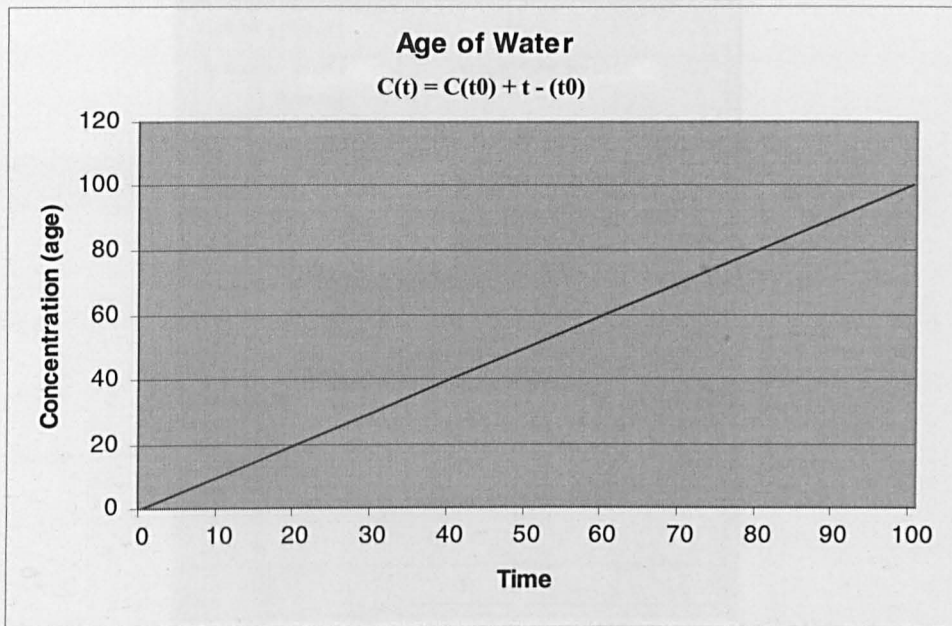


Figure 7.104 Simulated age of water

It is necessary to specify the age of the water in all nodes supplying the network as model boundary conditions; this includes inlet nodes and service reservoirs. The boundary conditions are defined using dialogue boxes. The default value is zero for all boundary conditions.

Using the default setting provides information on how the water ages purely as a function of the network modelled and not as a result of transmission time to the network or storage prior to reaching the network inlet node.

If the inlet to the network *is* at a node connected to another network or transmission main the incoming water age can be input as a boundary condition via the node dialogue box shown in Figure 7.105.

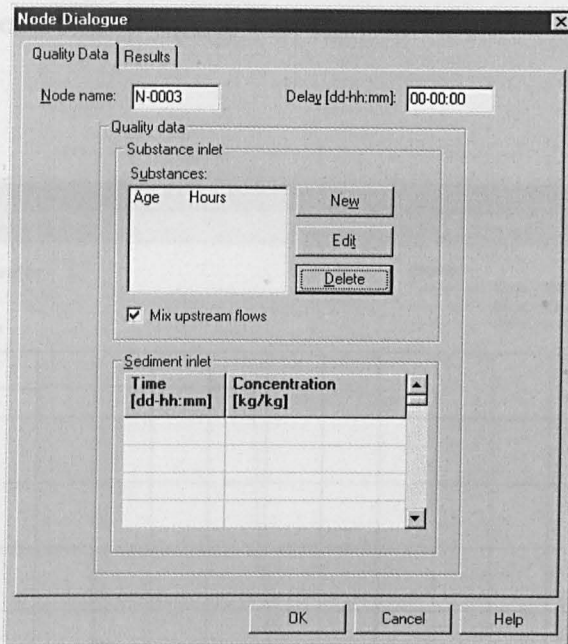


Figure 7.105 Dialogue box for age boundary condition at an inlet node

The water age at an inlet node can be a constant value or a time series. Because age is treated as a substance, an age profile can be defined for the incoming age ‘concentration’ as a substance inlet characteristic as shown in Figure 7.106.

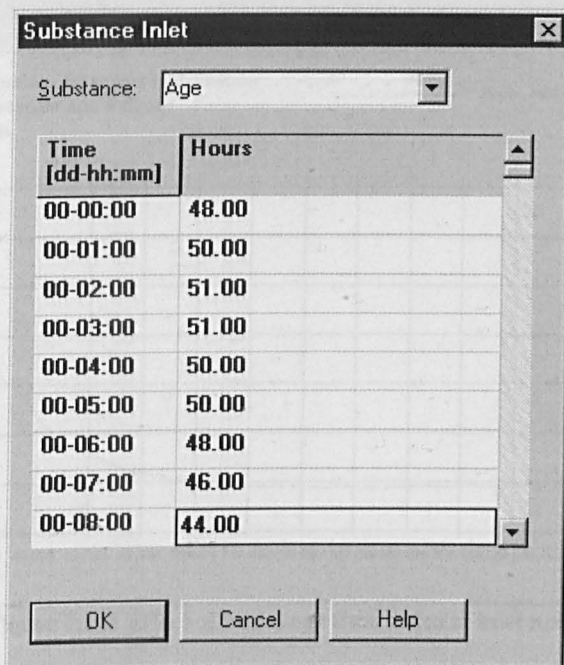


Figure 7.106 Substance (age) configuration at an inlet node

Figure 7.107 shows the detail of a configured inlet age time series.

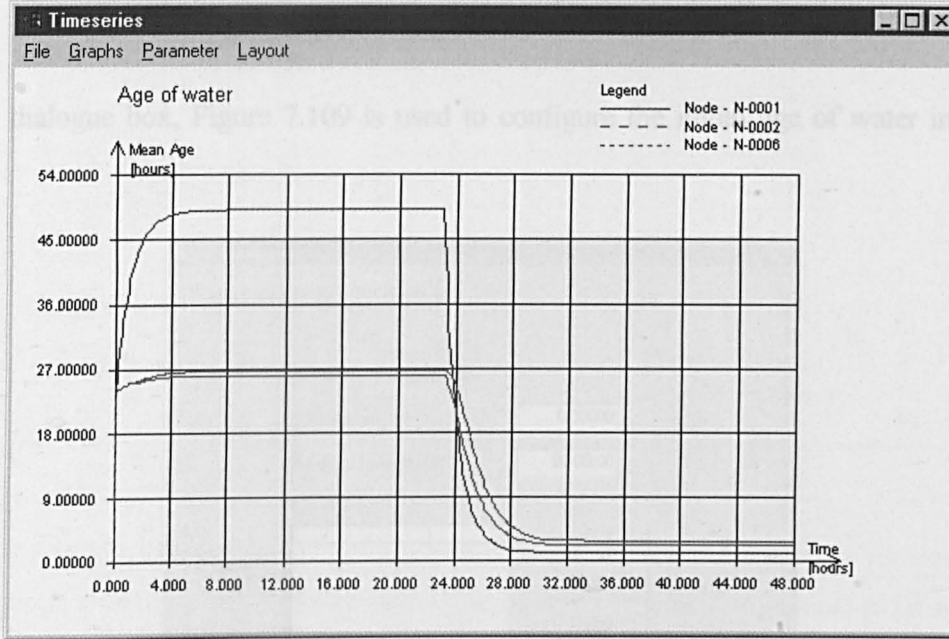


Figure 7.107 Age time series definition at an inlet node

Figure 7.108 shows how a constant inlet age of 48 hours can be applied to one node.

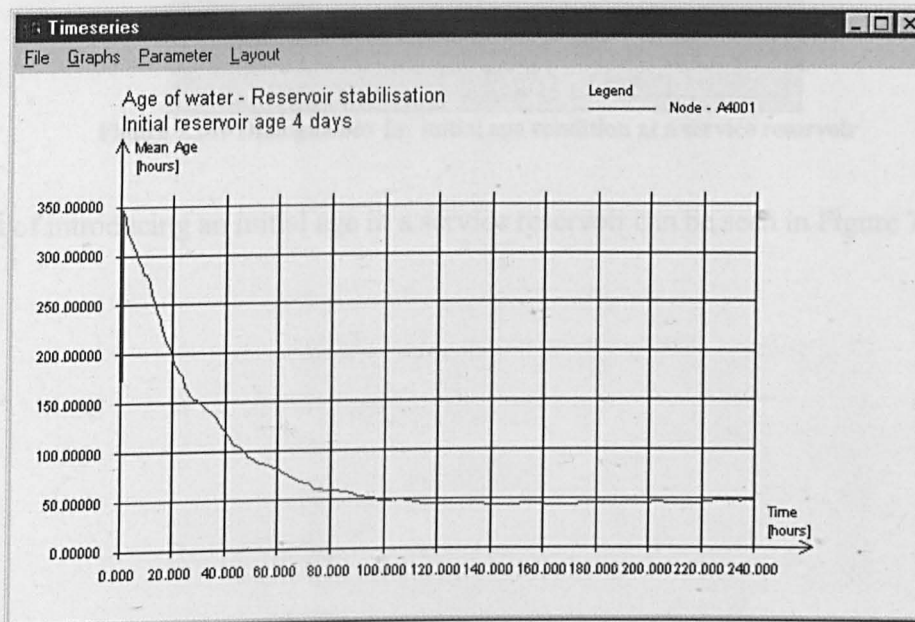


Figure 7.108 Effect of initial age time series at inlet node

The figure shows how an age of 48 hours at the inlet node and an initial age of 24 hours for 2 other nodes. After 24 hours the inlet age is reset to zero and the age in the other two pipes decreases to the mean age.

A similar dialogue box, Figure 7.109 is used to configure the initial age of water in a service reservoir.

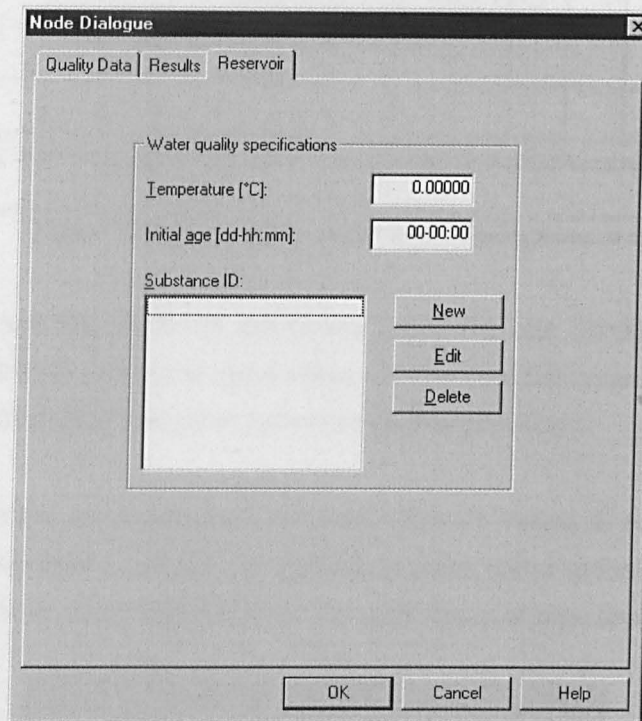


Figure 7.109 Dialogue box for initial age condition at a service reservoir

The effect of introducing an initial age in a service reservoir can be seen in Figure 7.110

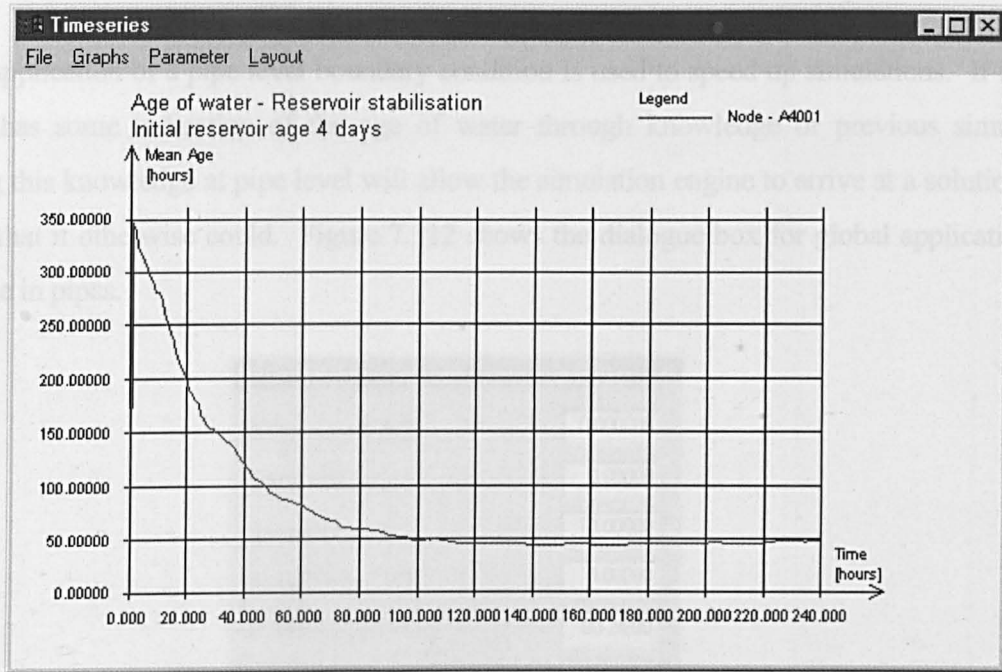


Figure 7.110 Initial age in a reservoir resolving to mean age

Because the initial age has been set artificially high, the age leaving the service reservoir decreases until it stabilises at its true mean value. In this case the mean value is 50 hours. This indicates a long turnover time and potential water quality problems.

The configured age time series can be a constant value or varied to reflect the incoming age profile. Initial age conditions can also be applied to pipes either globally or at individual pipe level. Figure 7.111 shows the dialogue boxes for application at pipe level.

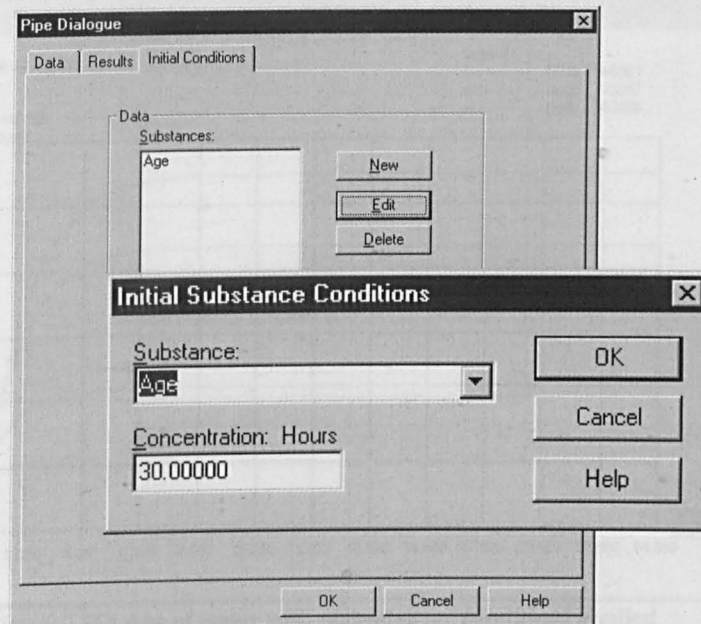


Figure 7.111 The dialogue boxes for application of initial age at pipe level

Global application of a pipe level boundary condition is used to speed up simulations. If the user already has some indication of the age of water through knowledge or previous simulations applying this knowledge at pipe level will allow the simulation engine to arrive at a solution more quickly than it otherwise could. Figure 7.112 shows the dialogue box for global application of a water age in pipes.

Default Values	
Pipe wall coeff. [m/s]:	0.000000
DBOC level [micro g/l]:	0.000000
Temperature [°C]:	10.000000
Turbidity level [FTU]:	0.000000
Default age [dd-hh:mm]:	00-24:00
Initial sediment fraction:	0.000000
Unspecified parameter [conc/value]:	0.000000
<input type="checkbox"/> Roughness dependency	
<input type="button" value="OK"/> <input type="button" value="Cancel"/> <input type="button" value="Help"/>	

Figure 7.112 The dialogue box for global application of water age in pipes

Figures 7.113 and 7.114 highlight the effect of applying a global pipe factor.

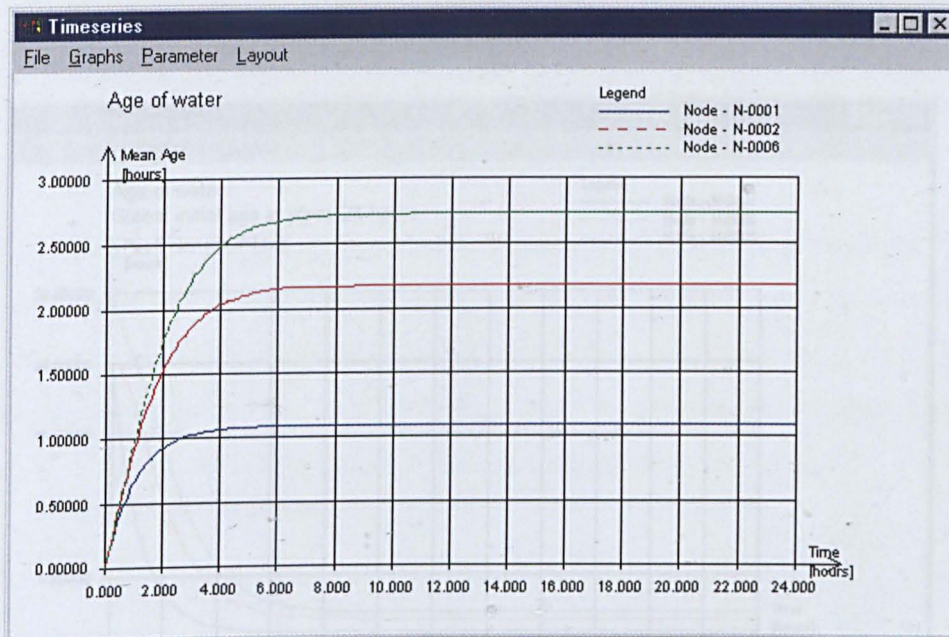


Figure 7.113 Age of water with no initial age conditions applied

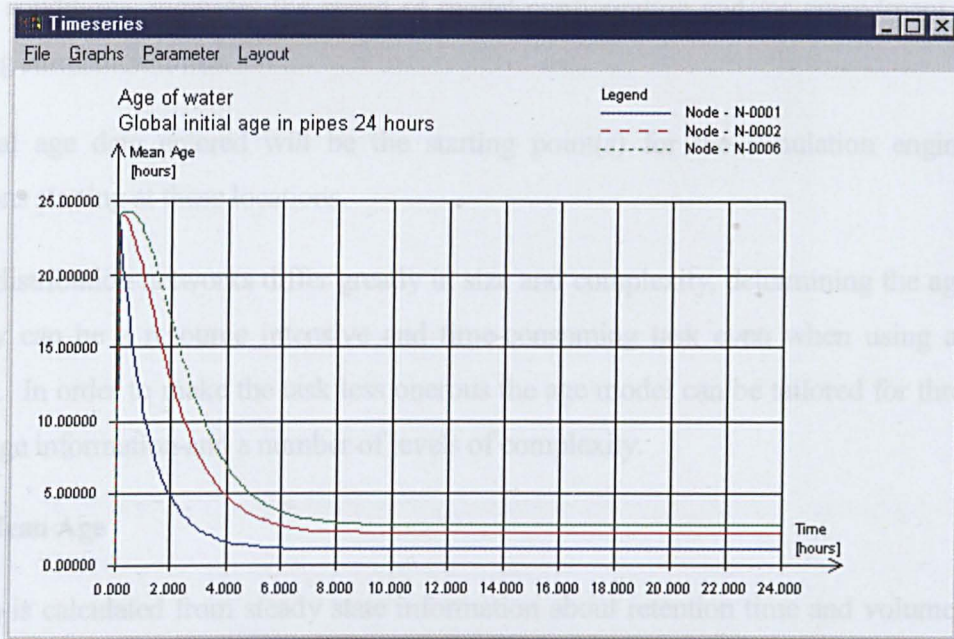


Figure 7.114 Effect of global application of an initial water age to pipes

The first figure is a time series of mean age in three pipes. The second is the same time series after a global application of an initial pipe age of 24 hours. To demonstrate how the individual pipe level condition can be applied Figure 7.115 shows the effect of changing the initial age of one of the pipes to 30 hours.

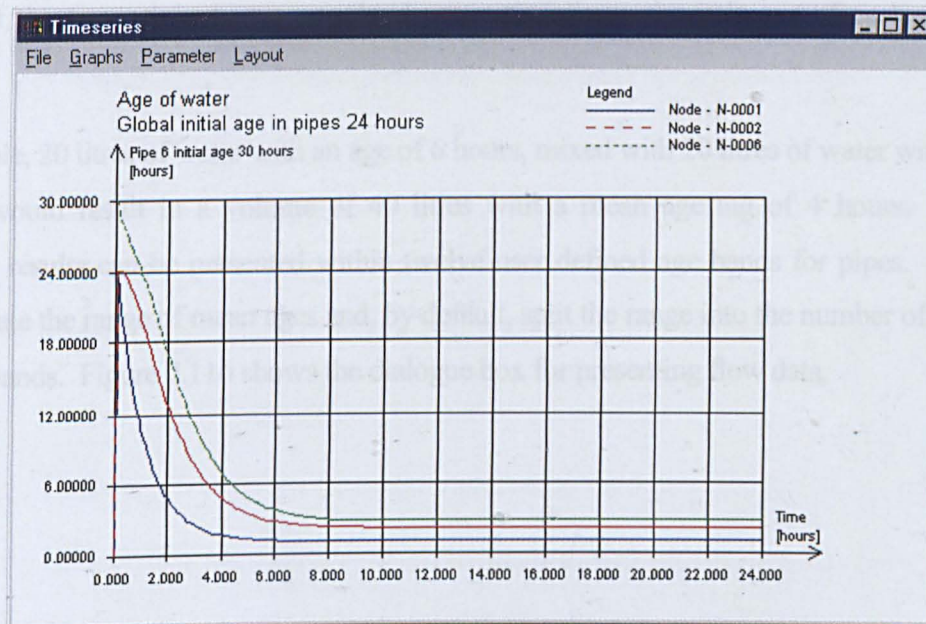


Figure 7.115 Effect of changing initial pipe age at pipe level

A combination of globally applied pipe conditions along with specific inlet node, service reservoir and pipe conditions, increases the speed of model configuration and /or amendment as well as decreasing simulation times.

The initial age data entered will be the starting point(s) for the simulation engine for age calculations starting at these locations.

Because distribution networks differ greatly in size and complexity, determining the age of water accurately can be a resource intensive and time-consuming task even when using a powerful computer. In order to make the task less onerous the age model can be tailored for three specific types of age information and a number of levels of complexity.

7.5.3 Mean Age

Mean age is calculated from steady state information about retention time and volume of all the water(s) merging at a node from one or more different pipes. In case of quasi-dynamic simulations the procedure is repeated for each time step. The mean age of water in a service reservoir is based on the assumption that the water in the reservoir is completely mixed at all times. The model automatically calculates the mean age in every pipe during every simulation for all time steps.

Mean age is a simple solution where two or more volumes of water of different ages are mixed into a single volume. The new volume is then tagged with a new age value calculated from the average of the two original ages weighted proportionally to the volumes of each original age category.

For example, 20 litres of water with an age of 6 hours, mixed with 20 litres of water with an age of 2 hours, would result in a volume of 40 litres with a mean age tag of 4 hours. Mean age simulation results can be presented within twelve user defined age bands for pipes. The model will calculate the range of mean ages and, by default, split the range into the number of configured reporting bands. Figure 7.116 shows the dialogue box for presenting flow data.

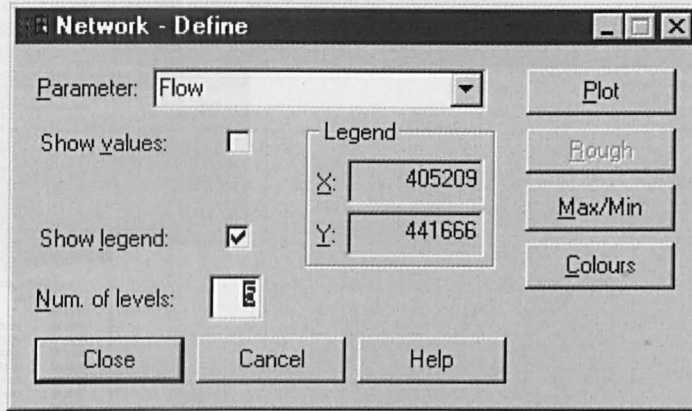


Figure 7.116 The dialog box for configuration of the presentation of flow data

By choosing the Max/Min button and entering the required values the age reporting bands can be configured. Figure 7.117

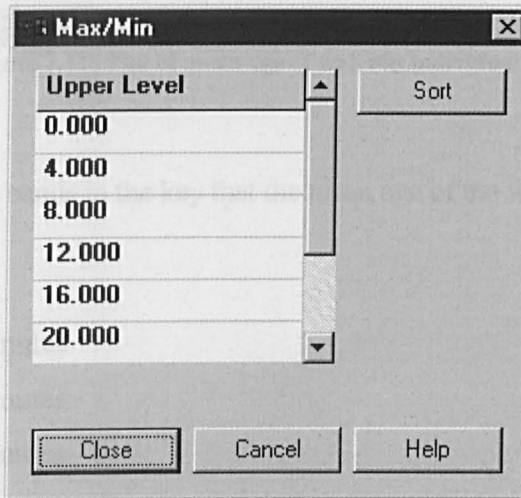


Figure 7.117 The dialog box for configuring mean age reporting bands

Figure 7.118 is a representation of the mean age in the same pipes in the study network as those shown for retention time.

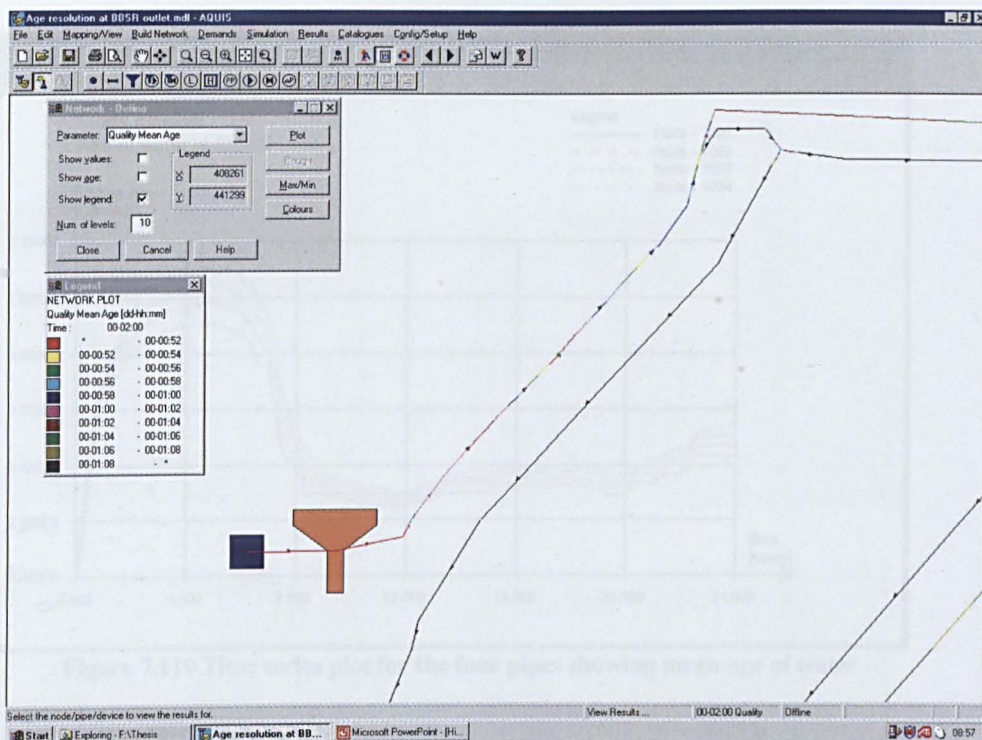


Figure 7.118 Plot of mean age of water in individual pipes

It can be seen from the age bands in the key that the mean age of the water in each pipe is:

- 1 0 – 56 minutes
- 2 56 – 58 minutes
- 3 58 – 60 minutes
- 4 60 – 62 minutes

This is in agreement with the summed retention times (58 to 62 minutes) for the three pipes. This type of presentation of results is adequate for a rapid overview, even of the entire network, but not specific enough for detailed analysis. Each plot represents results for a single time step. Time series graphs show how the mean age changes with time in the pipes (or at nodes) reflecting changing flow conditions. The mean age for any time step can be obtained from this time series output. Figure 7.119 is a time series plot for the same four pipes in Figure 7.118.

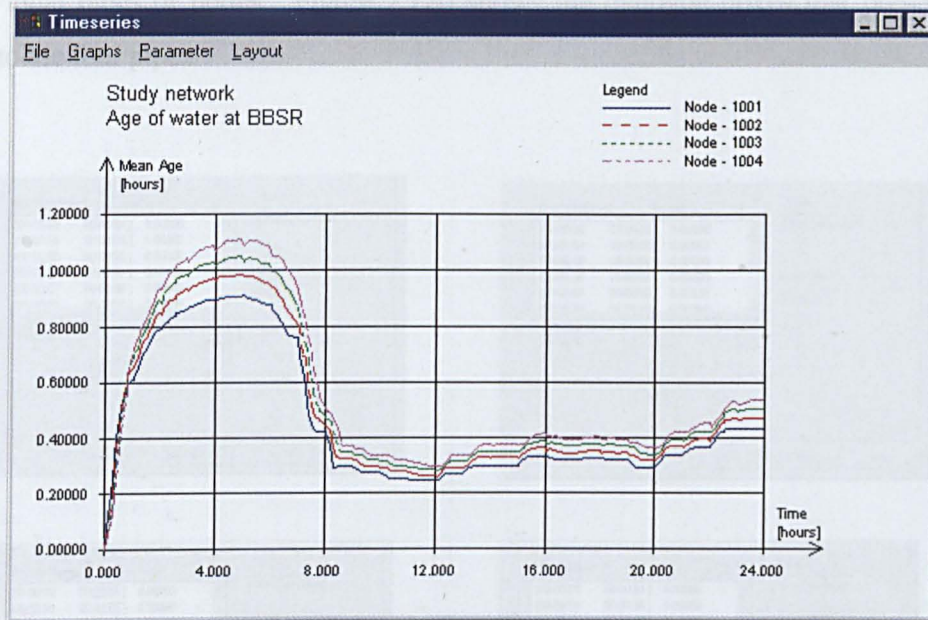


Figure 7.119 Time series plot for the four pipes showing mean age of water

The results are more specific but still require extrapolation from the graph. At 02:00 the extrapolated figures for mean age of water are:

- 1 0.8 hours
- 2 0.825 hours
- 3 0.89 hours
- 4 0.94 hours

These figures equate to mean age values of:

- 1 48 minutes
- 2 50 minutes
- 3 52 minutes
- 4 55 minutes

The results therefore are in good agreement with retention time calculations and mean age network plots results. Actual mean ages for individual pipes may be obtained from results dialogue boxes

for individual pipes or nodes. Figure 7.120 shows the dialogue boxes that present mean ages specific to the four pipes.

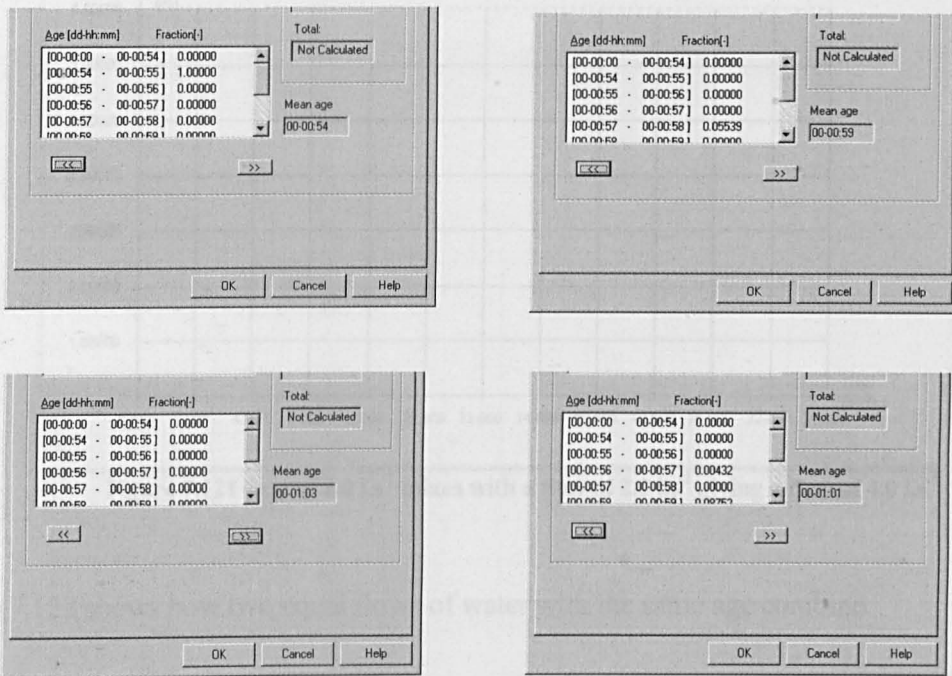


Figure 7.120 Dialogue boxes presenting mean age specific to the four pipes

The retention time calculation was between 58 and 62 minutes and the mean age calculations gave a result of 54 to 62 minutes, which agrees. These results are for a series of pipes with only one inlet and outlet, so no mixing of different flows or ages of water occurs.

Meshed distribution networks however contain, by definition, a large number of pipes that are interconnected and mixing does occur.

7.5.3.1 Mixing of Flow and Age

The model calculates the mean age of the water particles at points where mixing occurs. This can be clearly demonstrated using the model. Figures 7.121 to 7.126 show how the model, to give a mean age, mixes flows and age.

Figure 7.121 is the hydraulic component of the calculation comprising of two equal flows combining to make a single flow twice the magnitude of the original.

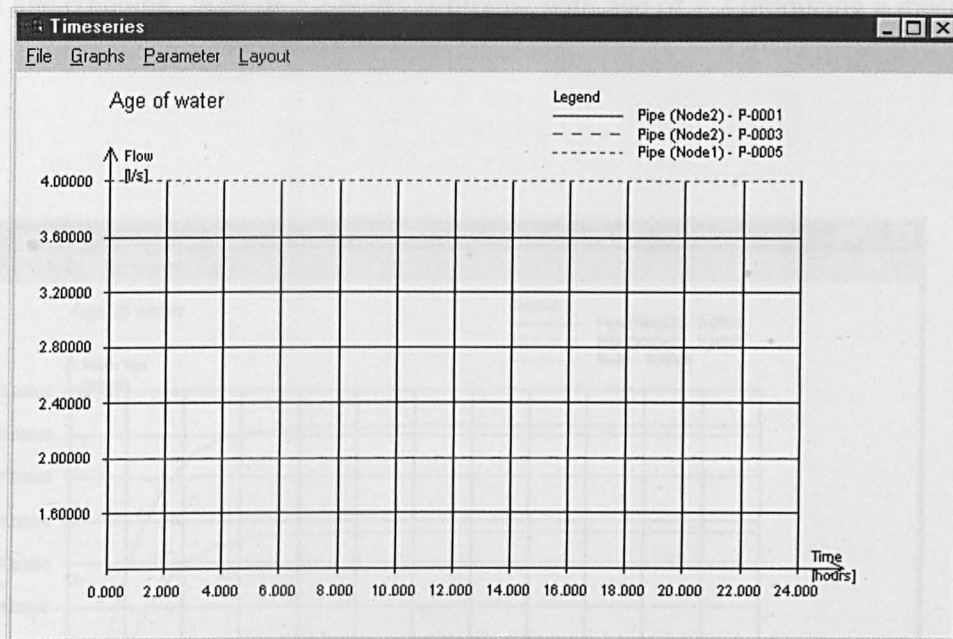


Figure 7.121 flow of 2.0 L s^{-1} mixes with a flow of 2.0 L s^{-1} giving a flow of 4.0 L s^{-1}

Figure 7.122 shows how two equal flows of water with the same age combine.

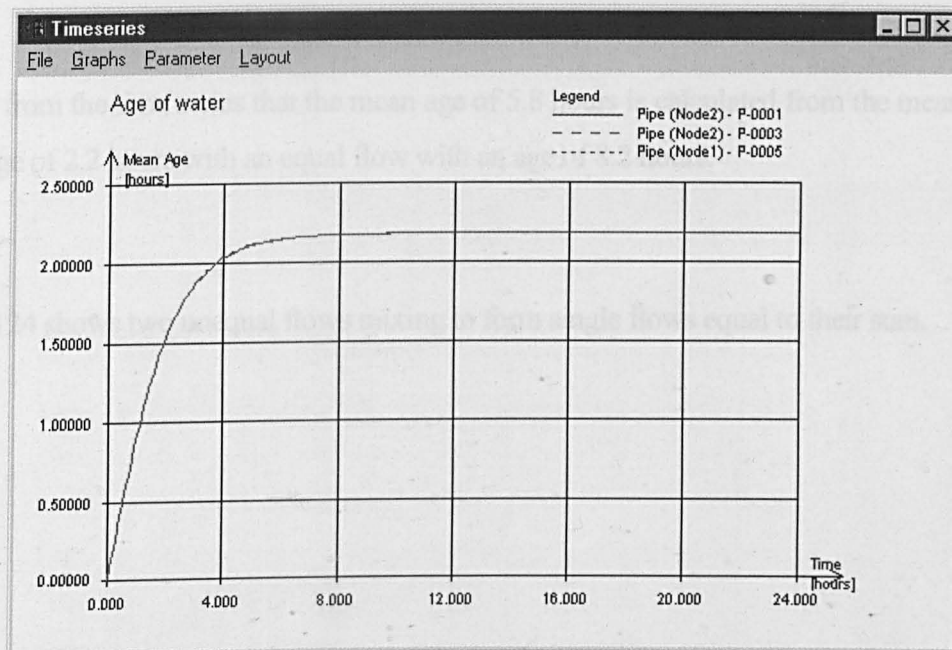


Figure 7.122 Age 2.2 hours mixes with age 2.2 hours giving mean age of 2.2 hours

Because the two combining flows are the same magnitude and the individual ages are the same, the mean age is the same as the individual ages. Figure 7.123 shows how two equal flows of

different age combine. Age of 2.2 hours is mixing with age of 8.2 producing a mean age of 5.8 hours.

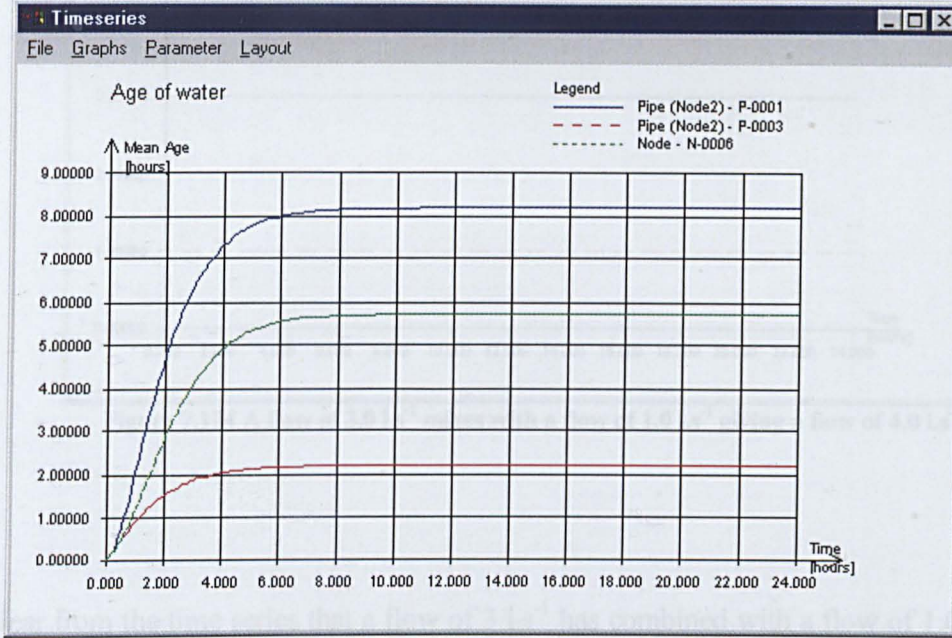


Figure 7.123 Combining two equal flows of different age

It is clear from the time series that the mean age of 5.8 hours is calculated from the mean of a flow with an age of 2.2 hours with an equal flow with an age of 8.2 hours.

Figure 7.124 shows two unequal flows mixing to form single flows equal to their sum.



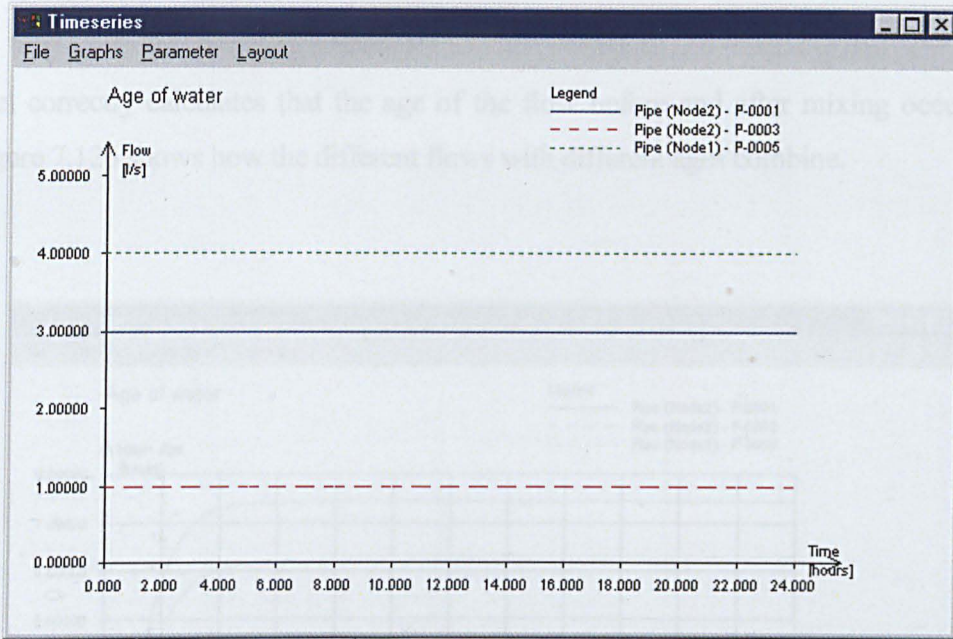


Figure 7.124 A flow of 3.0 l.s^{-1} mixes with a flow of 1.0 l.s^{-1} giving a flow of 4.0 l.s^{-1}

It is clear from the time series that a flow of 3 l.s^{-1} has combined with a flow of 1.0 l.s^{-1} to give a combined flow of 4.0 l.s^{-1} . Figure 7.125 shows how the different flows with the same age combine.

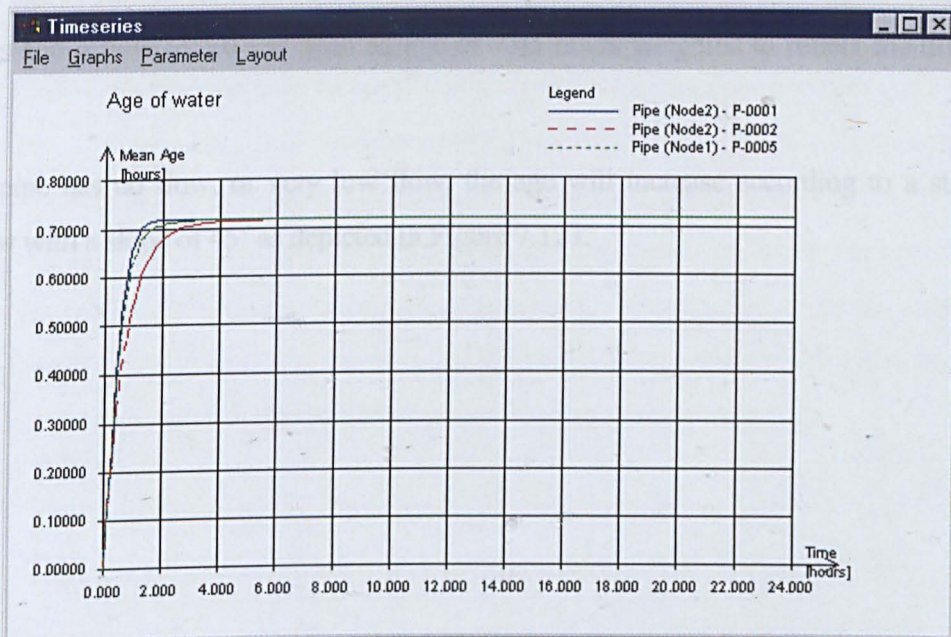


Figure 7.125 Age of two equal flows with same age (0.72 hrs) mix to give mean age (0.72 hrs)

The model correctly calculates that the age of the flow before and after mixing occurs are the same. Figure 7.126 shows how the different flows with different ages combine.



Figure 7.126 Age of 7.45 hours mixes with age of 2.70 hours giving mean age of 6.25 hours

The mean age of 6.25 hours is obtained by calculating the mean of a flow of 1.0 l.s^{-1} and an age of 2.70 hours and a flow of 3.0 l.s^{-1} with an age of 7.45 hours weighted to reflect the difference in flow.

Where a pipe has no flow, or very low flow, the age will increase according to a straight-line growth law with a slope of 45° as depicted in Figure 7.127.

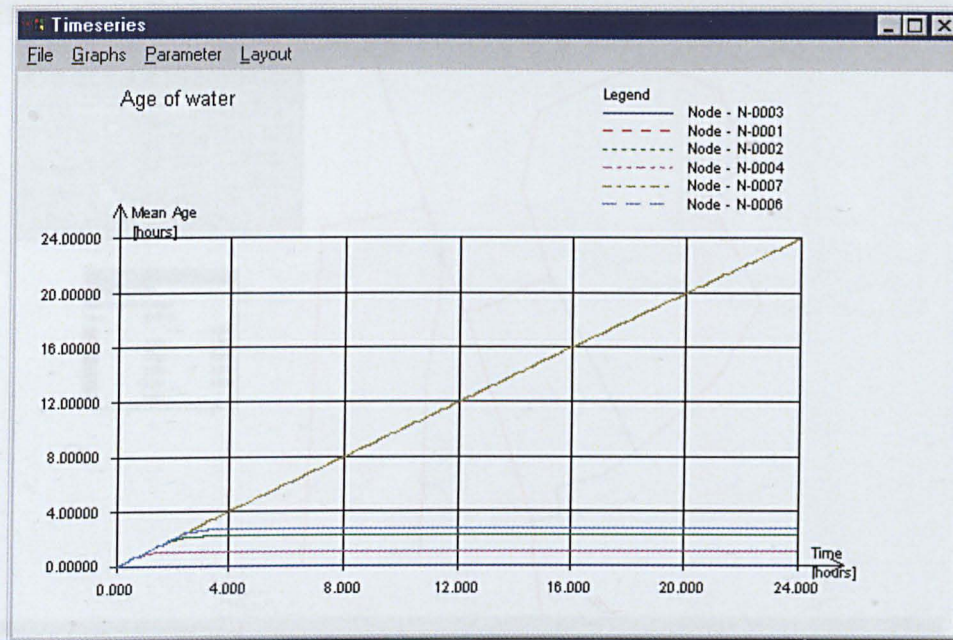


Figure 7.127 Mean age in a pipe with no flow (brown trace)

The diagrams above demonstrate that the model correctly calculates the mean age for the different possible combinations of flows and age.

7.5.3.2 Flow Reversals and Age

It is possible to have an area of the network where the water is held in a specific pipe, or pipes, some of which is unable to escape because demands in more than one direction compete against each other resulting in a tidal flow and associated flow reversals. As a rule, consumer demands vary little from day to day because of habitual use so overall network hydraulic conditions do not vary to a great extent over a 24-hour period thereby making these tidal areas semi permanent. In such areas the water can attain high age values resulting in associated aesthetic, chemical, and biological deterioration. The model can be used to identify such areas and the effect they have on the age of water. Figure 7.128 shows a flow reversal site in the study network.

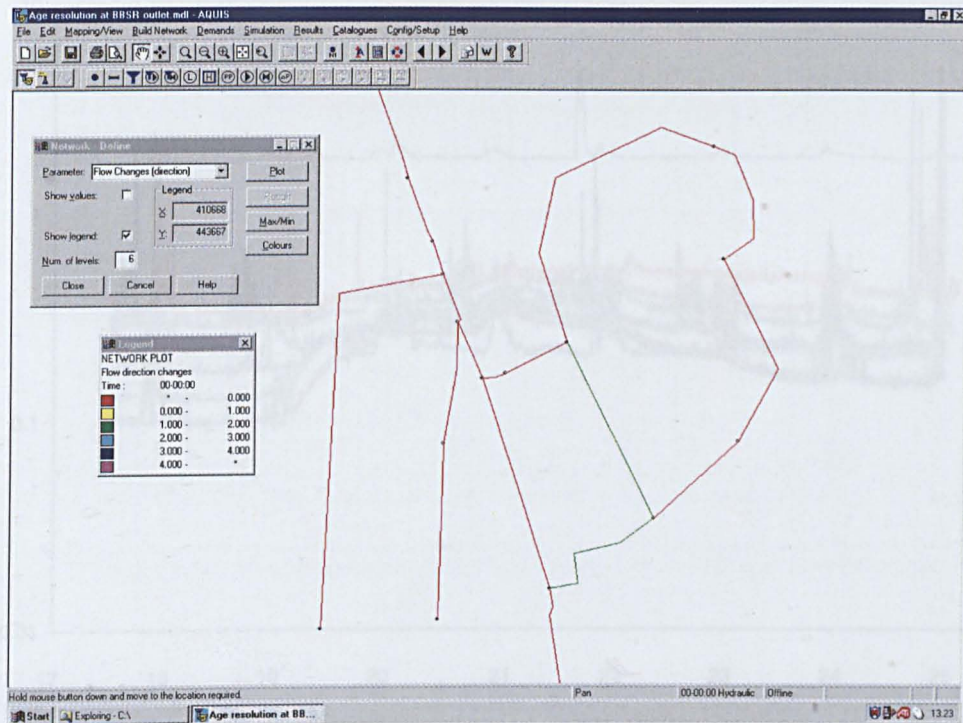


Figure 7.128 Flow reversal site within the study network

This is important because an unusual demand, such as a burst or higher than normal industrial use, or operational change may break this tidal behaviour releasing water to blend with that contained in other parts of the network resulting in a mixture of waters of very different ages and characteristics. This type of event can also cause a long-lived water quality event such as turbid or discoloured water. Figure 7.129 shows turbidity data measured during and following a burst event.

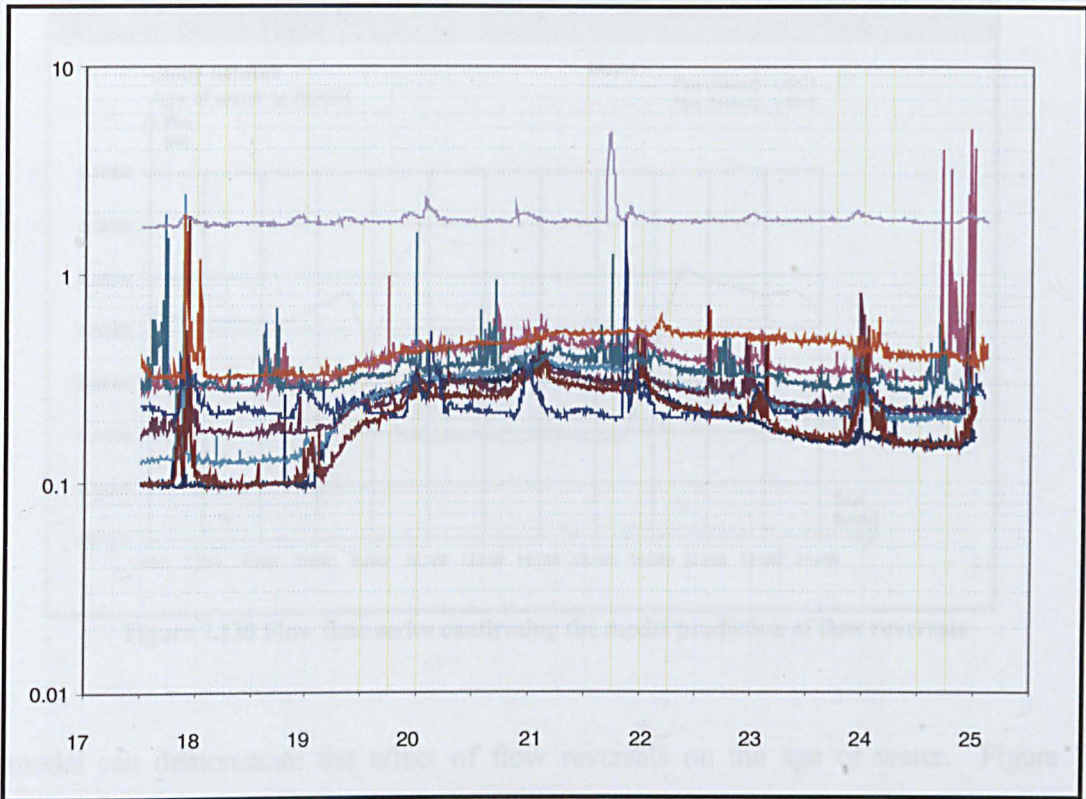


Figure 7.129 Turbidity effects of a burst event

It can be seen that the turbidity generated lasts for several days in some pipes. These are the pipes with low flow characteristics and sediments of low specific gravity that are easily suspended in the bulk water flow.

The model has calculated that the two pipes coloured green in Figure 7.128 suffer two flow reversals per 24-hour period. Figure 7.130 is a time series of flow in the two pipes concerned that confirms the model prediction.

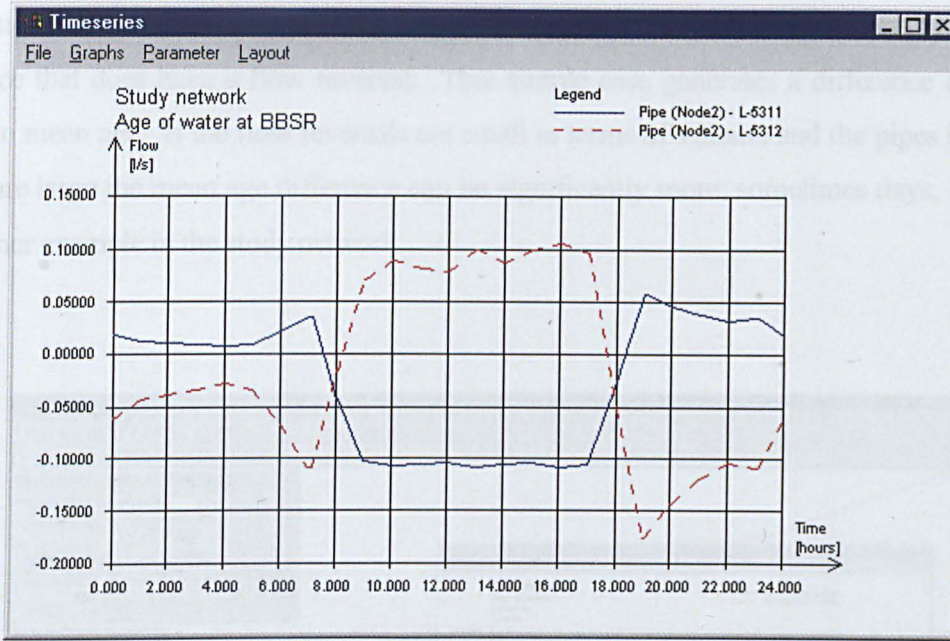


Figure 7.130 Flow time series confirming the model prediction of flow reversals

The model can demonstrate the effect of flow reversals on the age of water. Figure 7.131 highlights the difference in age pattern in a pipe with and without flow reversals.

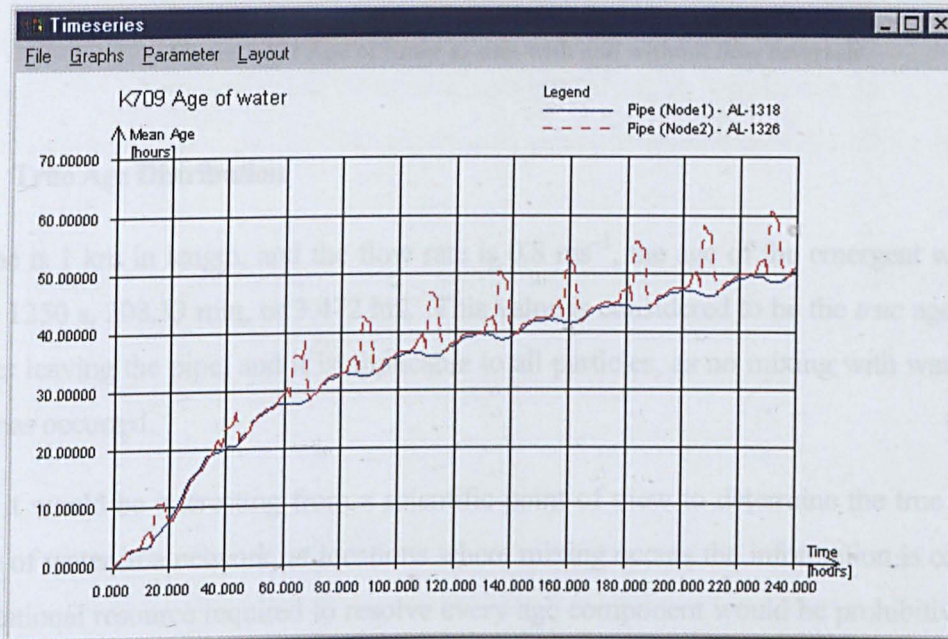


Figure 7.131 Age of water at sites with and without flow reversals

The blue trace is a pipe with similar flow rate but no flow reversal compared to the pipe with the red trace that does have a flow reversal. This simple case generates a difference of around ten hours in mean age. If the flow reversals are small in terms of volume and the pipes in which they occur are large the mean age difference can be significantly more; sometimes days. Figure 7.132 is another example in the study network.

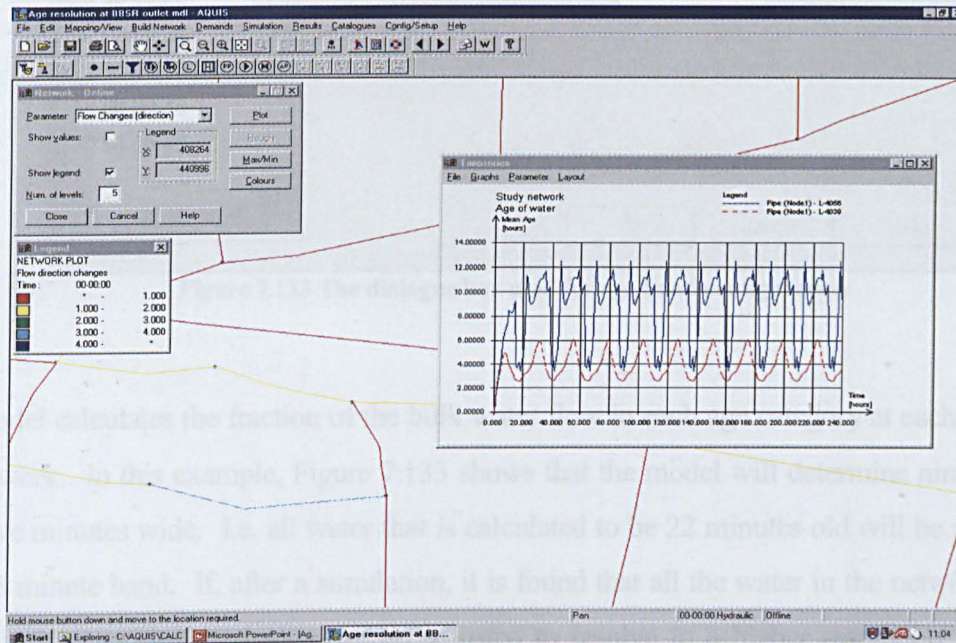


Figure 7.132 Age of water at sites with and without flow reversals

7.5.4 True Age Distribution

If a pipe is 1 km in length, and the flow rate is 0.8 ms^{-1} , the age of the emergent water particles will be 1250 s, 208.33 min, or 3.472 hrs. This value is considered to be the *true* age of the water particles leaving the pipe, and it is applicable to all particles, as no mixing with water of another age(s) has occurred.

Whilst it would be interesting from a scientific point of view to determine the true age of every particle of water in a network, at locations where mixing occurs the information is complex. The computational resource required to resolve every age component would be prohibitively high and simulation times would be excessive. In order to minimise calculation time therefore, all age categories at a particular node are assigned to one of up to nine user-definable "age bands". The user is allowed to choose up to 9 different age intervals that define the upper and lower limits of the bands. Figure 7.133 shows the dialogue box used for age band configuration.

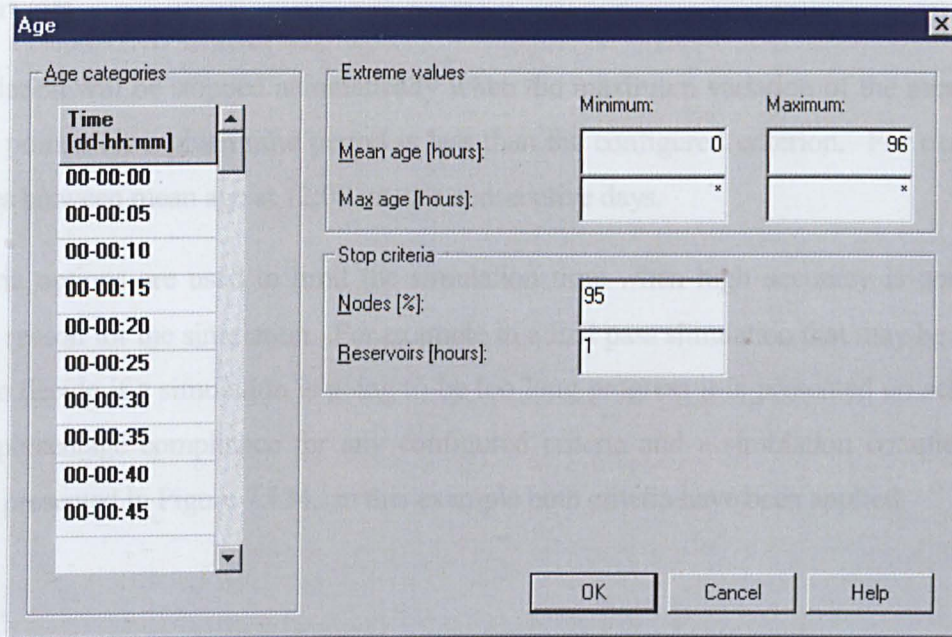


Figure 7.133 The dialogue box used for age band configuration

The model calculates the fraction of the bulk water flow in each age category at each node within the network. In this example, Figure 7.133 shows that the model will determine nine age bands, each five minutes wide. I.e. all water that is calculated to be 22 minutes old will be placed in the 20 – 25 minute band. If, after a simulation, it is found that all the water in the network falls into just 2 or 3 categories the bands can be adjusted to resolve to a higher resolution within these categories only. Also, individual or unusual age bands can be located and investigated in detail. By an iterative process, it is therefore possible to get very detailed analysis of the age of water in any part of a network.

In order to minimise simulation time the age dialogue box also allows the configuration of age simulation stop criteria. The stop criteria may be applied to nodes and / or service reservoirs and are used to halt a simulation when the stop criteria are met.

For Nodes

The simulation will be stopped automatically when the model identifies that, at any time step, the configured percentage of nodes has a mean age that is not less than the previous simulation period. I.e. the model has resolved the mean age in the configured percentage of nodes. The model takes into account dead end nodes with no demand, where the actual age criteria will never be satisfied.

For Reservoirs

The simulation will be stopped automatically when the maximum variation of the mean age at a particular point over a given time period is less than the configured criterion. For example, the differences between mean age at 12:00 on two consecutive days.

The criteria options are used to limit the simulation time when high accuracy is not the most important reason for the simulation. For example in a first pass simulation that may be very long. In order to decide if a simulation is going to be too long progress it is presented on screen along with the percentage compliance for any configured criteria and a simulation completion time. These are presented in Figure 7.134. In this example both criteria have been applied.

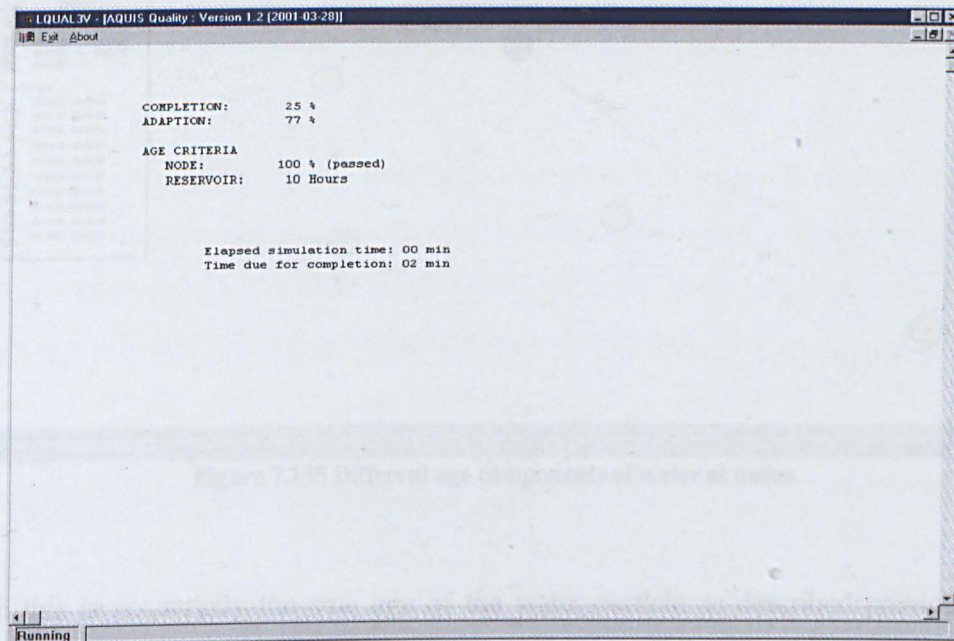


Figure 7.134 Presentation of simulation progress and completion time scale

It can be seen that, at the moment of the screen capture, the node criteria had been met but the simulation was continuing because the reservoir criterion had not yet been met. The ten hours reported against this criterion means that over a 24-hour simulation period the mean age in the reservoir had changed by ten hours. I.e. the model had not yet resolved the age in the reservoir to the required accuracy.

Using this more detailed age analysis in conjunction with mean age analysis it is possible, via an iterative process, to obtain very accurate age information for a particular area of a network.

Figure 7.135 shows the proportion of water in each configured age band for a number of nodes in the study network.

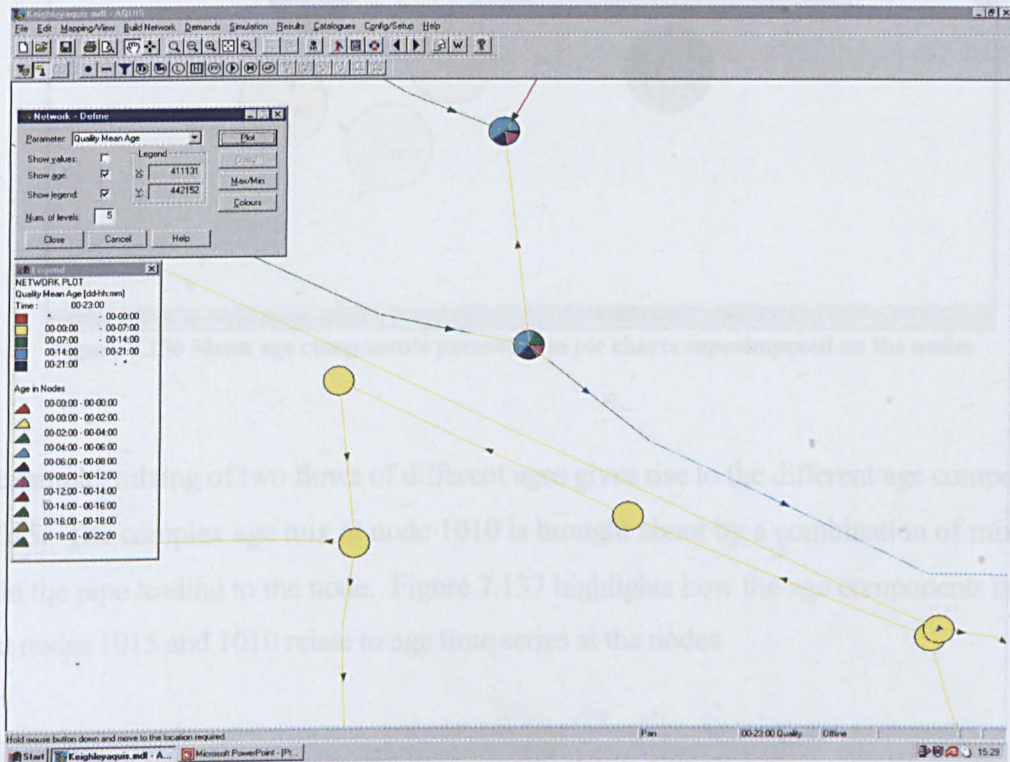


Figure 7.135 Different age components of water at nodes

Although this is not strictly the *true age* of the water particle as described previously, it is a significant improvement on other models providing much more information about age components distributed around a network. Figure 7.136 depicts the component parts of the mean age represented as pie charts superimposed on the nodes.

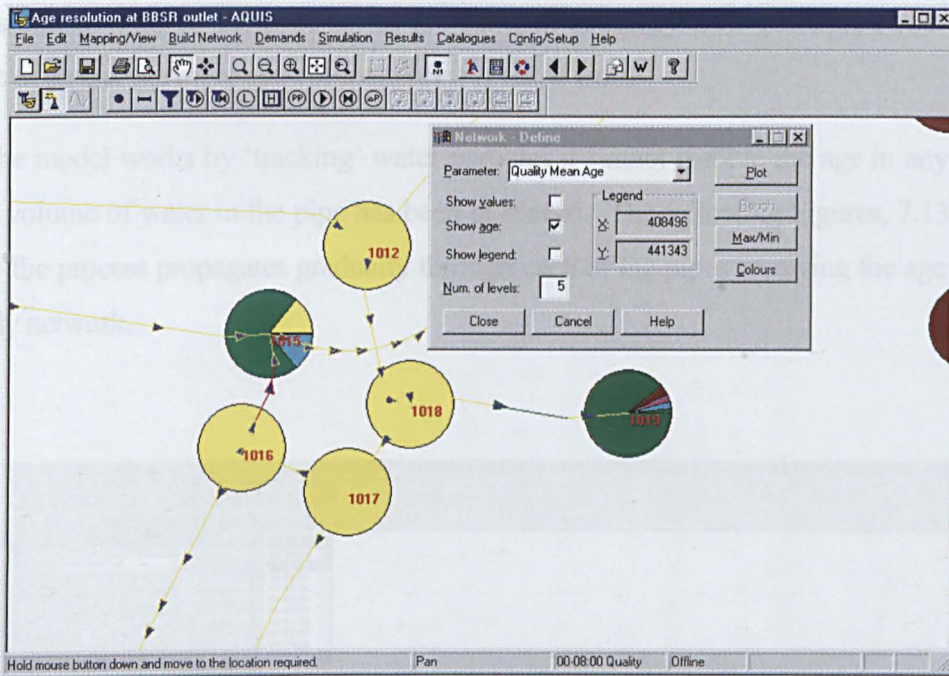


Figure 7.136 Mean age components presented as pie charts superimposed on the nodes

In this example, mixing of two flows of different ages gives rise to the different age components at node 1015. The complex age mix at node 1010 is brought about by a combination of mixing and ageing in the pipe leading to the node. Figure 7.137 highlights how the age components of the pie charts at nodes 1015 and 1010 relate to age time series at the nodes.

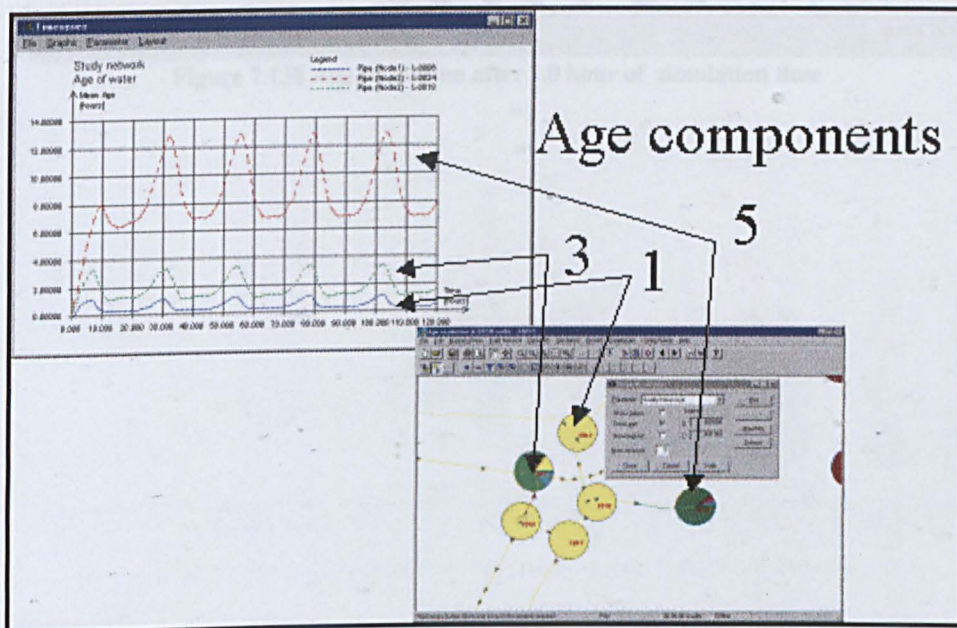


Figure 7.137 The relationship between age components and age time series for 3 nodes

The peaks and troughs in the time series are brought about by flow patterns. The higher the magnitude of a peak the more age components it is made from.

Because the model works by 'tracking' water particles it cannot resolve the age in any pipe until the whole volume of water in the pipe has been displaced. The following Figures, 7.138 to 7.141 show how the process propagates gradually through each of the pipes resolving the age in an area of the study network.

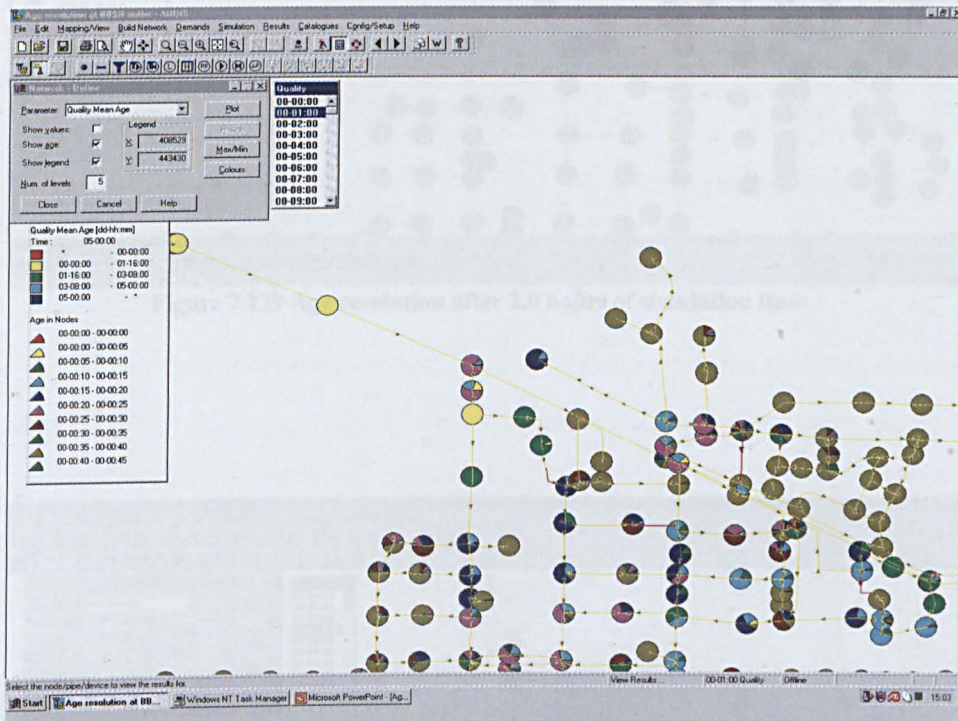


Figure 7.138 Age resolution after 1.0 hour of simulation time

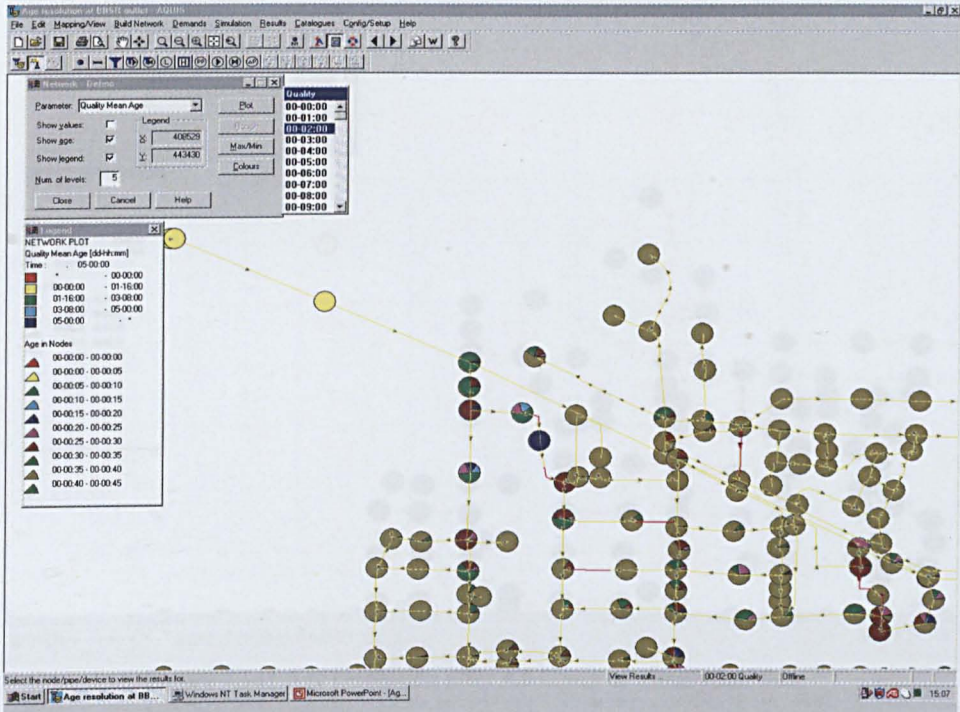


Figure 7.139 Age resolution after 2.0 hours of simulation time

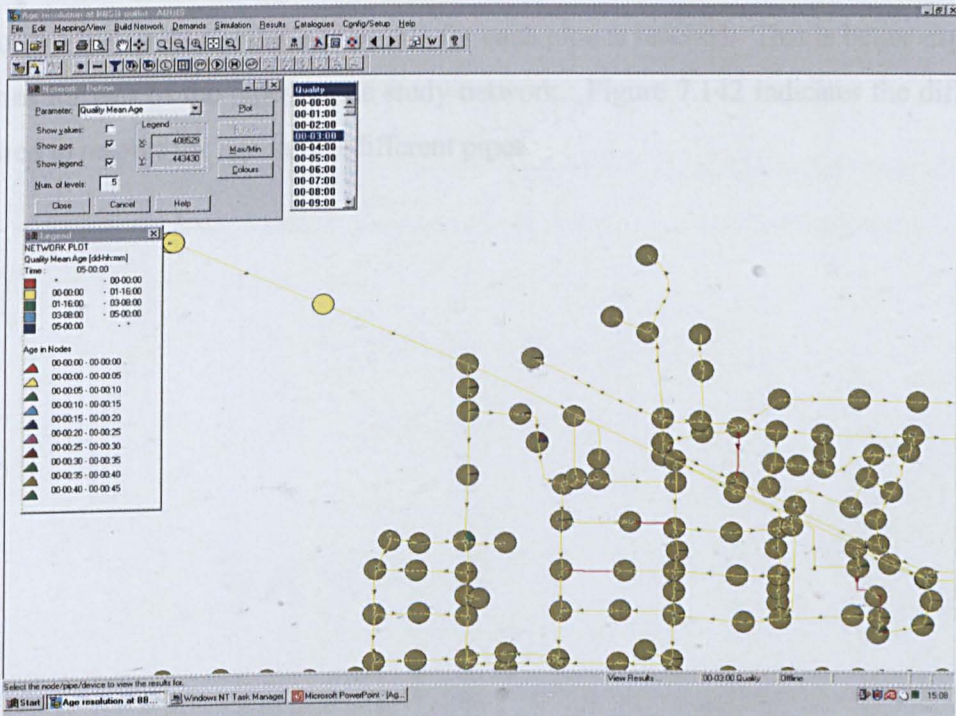


Figure 7.140 Age resolution after 3.0 hours of simulation time

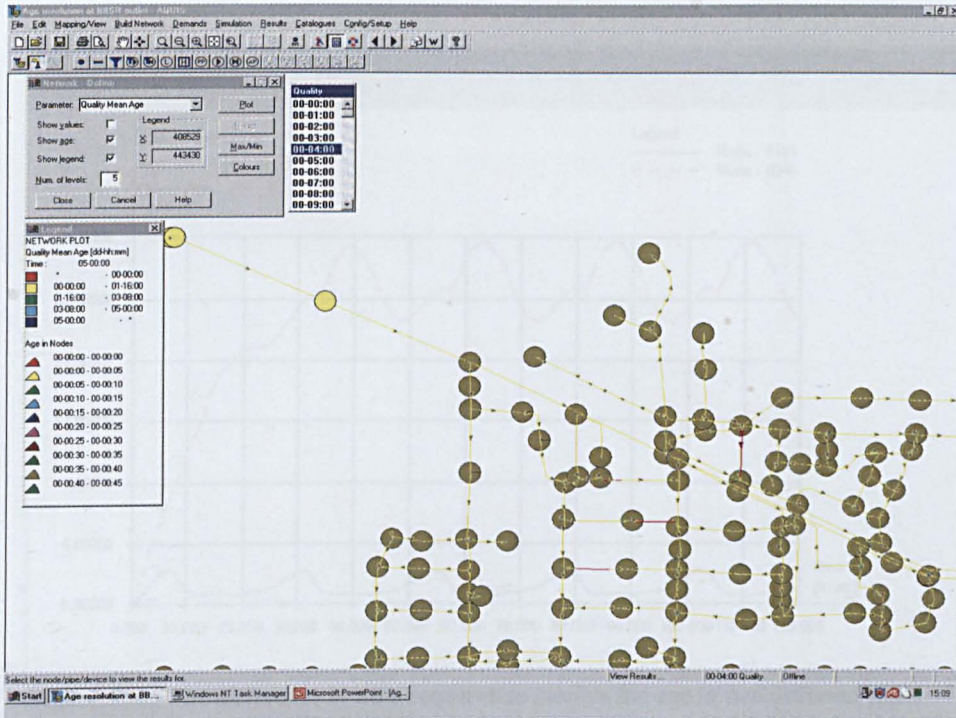


Figure 7.141 Age resolution after 4.0 hours of simulation time

The figures show how the model sees many different ages and complex mixes at the beginning of the simulation. This is because many of the pipes have different flow rates so take different lengths of time to displace their contents. As the simulation progresses each pipe is resolved in the direction of the flow until a stable age profile for each pipe is reached. This is better explained by a time series for two of the pipes in the study network. Figure 7.142 indicates the difference in time required to resolve the age in two different pipes.

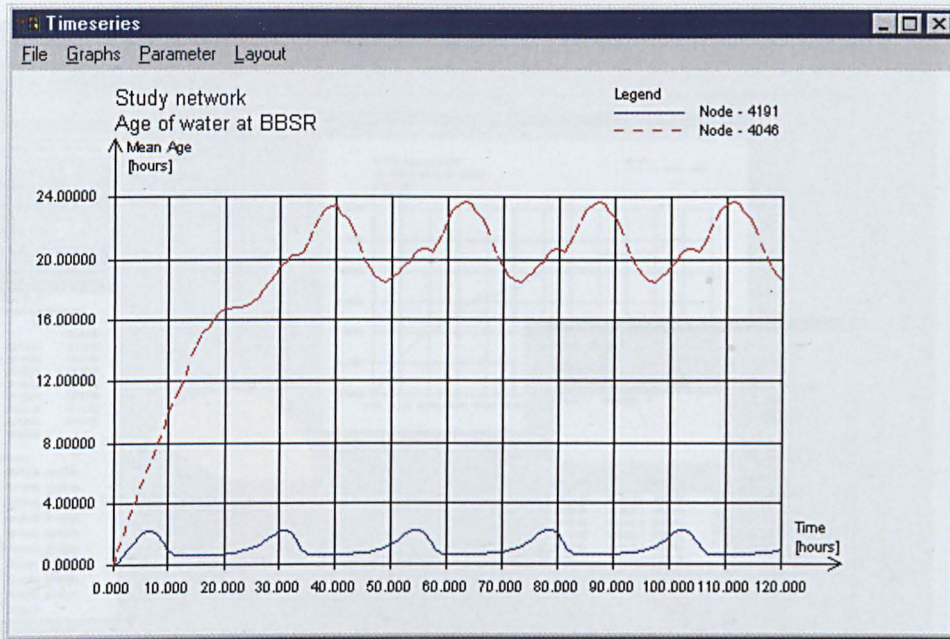


Figure 7.142 The difference in time required to resolve the age in two different pipes

It is evident that the age at node 4191 is resolved after just a few minutes whereas the age reaching node 4046 is not fully resolved for over 40 hours. It is necessary therefore to ensure all the water in the network has been displaced at least once before using simulation results for the pipes and nodes at the extremities.

7.5.4.1 Relationship between Mean and True Age

In order to demonstrate how the model resolves the age of water, and how the mean age and true age components are related, a series of screen shots following a simulation is presented on the following pages. Figures 7.143 to 7.149.

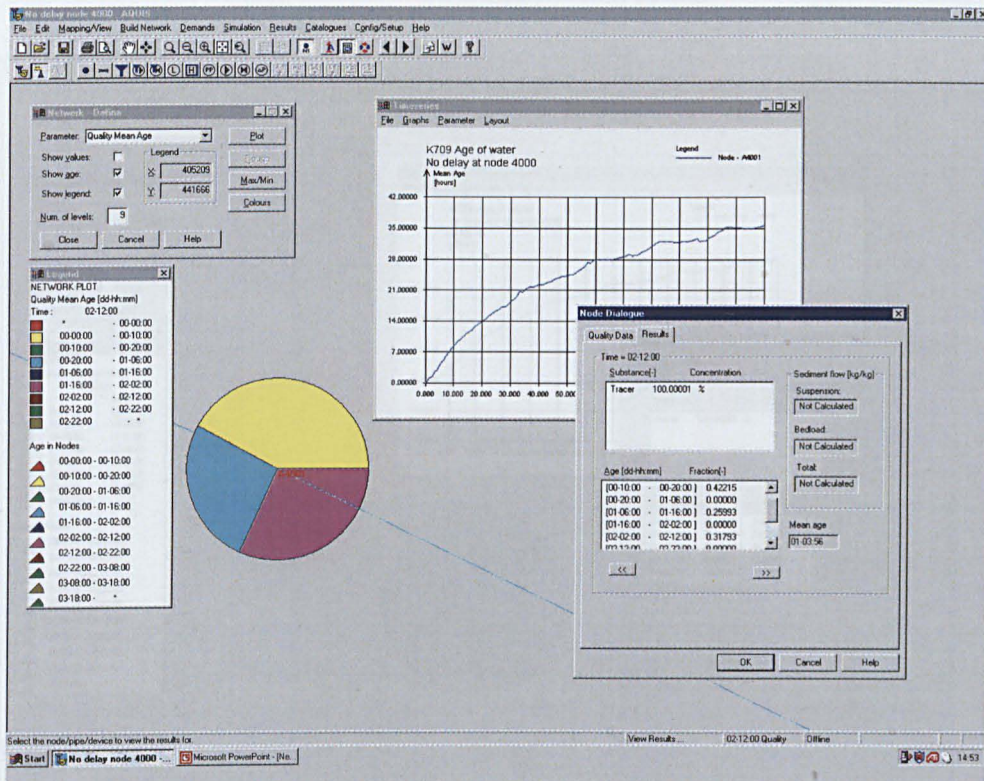


Figure 7.143 Age components at Node 4001 after 2 days 12 hours simulation

The figure shows that both pipes are colour coded blue, indicating that the mean age, after 2 days 12 hours of simulation time, is between 1 day 6 hours and 1 day 16 hours. The mean age in the node dialogue box shows that the mean age at the node is in fact 1 day 3 hours and 56 minutes. The pie chart on the node that reports the true age contributions to the mean has three components:

- 0 days 10 hours to 0 days 20 hours
- 1 day 6 hours to 1 day 16 hours
- 2 days 2 hours to 2 days 12 hours

The node results dialogue box also shows that the mean age is comprised of three components with percentage compositions of 0.42215, 0.25993 and 0.31793 respectively.

A 24-hour delay was added before Node 4001. Figure 7.144 shows how this is translated by the model.

12-hour delay has been correctly applied. Figure 7.145 is a plot of age after twelve hours of simulation. The simulation is running on a network with a 24-hour delay at node 4000.

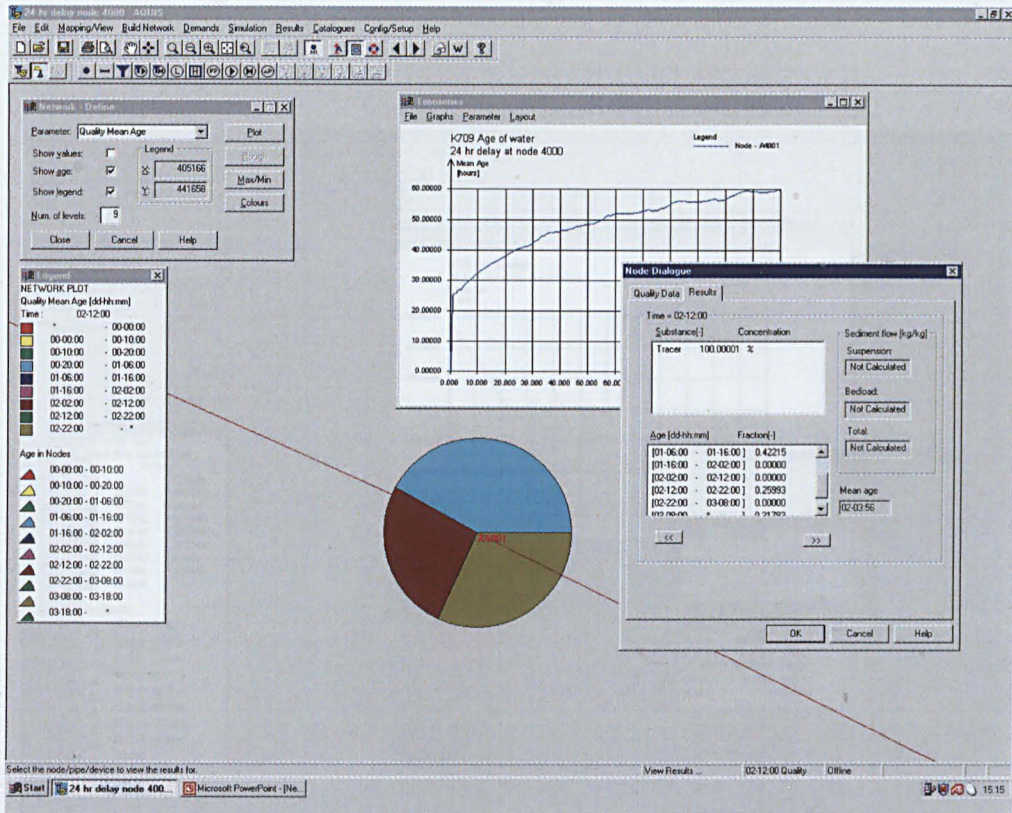


Figure 7.144 Age components at Node 4001 after 3 days and 12 hours of simulation

The figure shows that both pipes are colour coded brown, indicating that the mean age, after 2 days 12 hours of simulation time, is between 2 days 2 hours and 2 days 12 hours. The mean age in the node dialogue box shows that the mean age at the node is in fact 2 days 3 hours and 56 minutes.

The pie chart on the node that reports the true age contributions to the mean has three components:

- 1 day 6 hours to 1 day 16 hours
- 2 days 12 hours to 2 days 22 hours
- 3 days 8 hours to 3 days 18 hours

The node results dialogue box also shows that the mean age is comprised of three components with percentage compositions of 0.42215, 0.25993 and 0.31782 respectively. It is clear that the model has calculated the age component percentages exactly as for the previous example and the

12-hour delay has been correctly applied. Figure 7.145 is a plot of age after twelve hours of simulation time following an increase of the initial age in the service reservoir to ten days.



Figure 7.145 Age components at Node 4001 after 12 hours of simulation

After 12 hours simulation time the mean age at Node 4001 is 7 days 10 hours 3 minutes. The fraction of water in 10-day age band (original service reservoir water) is 0.81. The fraction is reflected in the green portion of the pie chart on the node.

The simulation was continued to monitor how the age and the fraction of water in the 10-day age band changed with time. Figure 7.146 shows the results after 24 hours of simulation.

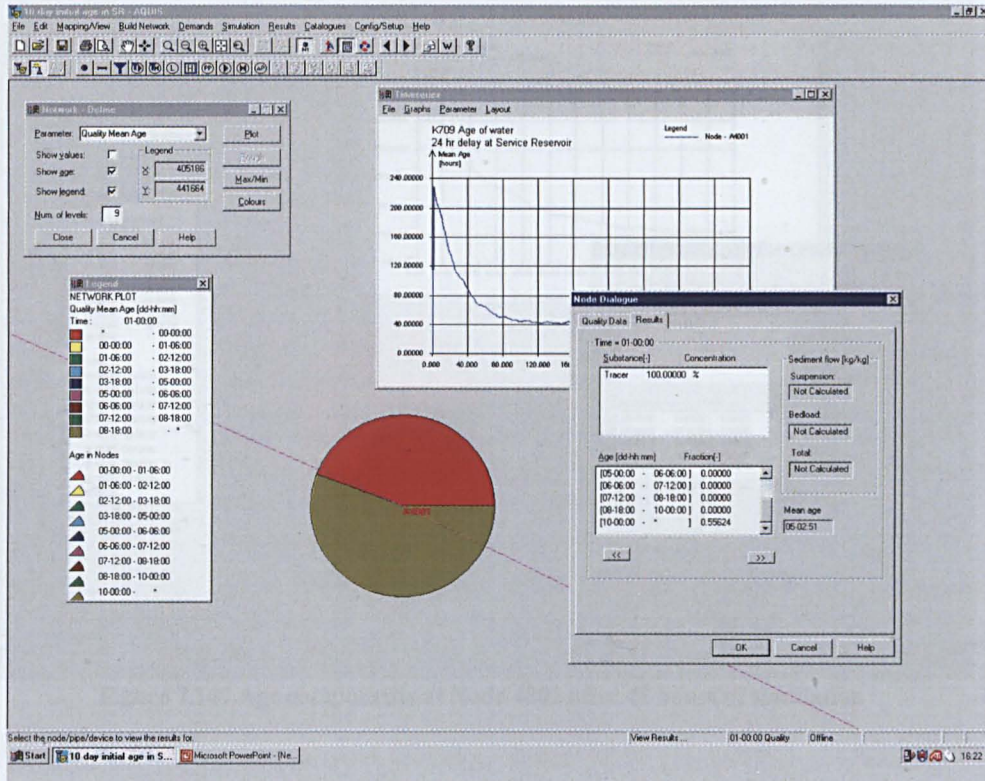


Figure 7.146 Age components at Node 4001 after 24 hours of simulation

With a ten-day initial age in service reservoir, and after 24 hours simulation time, the mean age at Node 4001 is 5 days 2 hours 13 minutes. The fraction of water in 10-day age band (original service reservoir water) is 0.556.

Figure 7.147 shows the results after 48 hours of simulation.

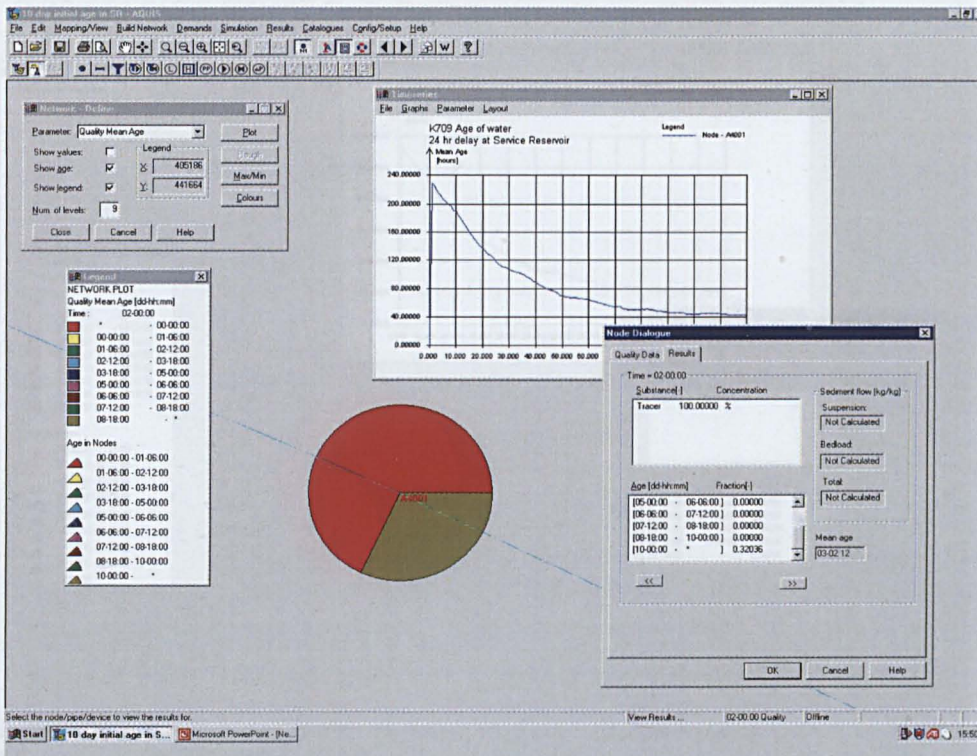


Figure 7.147 Age components at Node 4001 after 48 hours of simulation

With a ten-day initial age in service reservoir, and after 28 hours simulation time, the mean age at Node 4001 is 3 days 2 hours 12 minutes. The fraction of water in 10-day age band (original service reservoir water) is 0.32.

Figure 7.148 shows the age results after 72 hours of simulation.

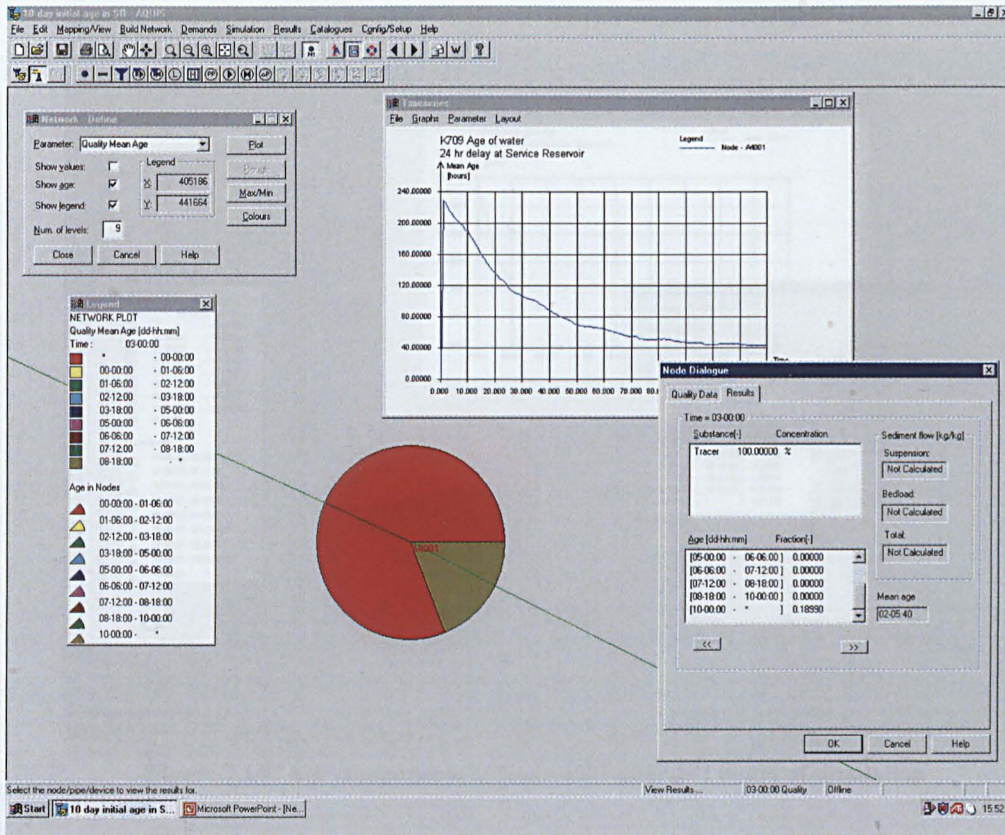


Figure 7.148 Age components at Node 4001 after 72 hours of simulation

With a ten-day initial age in the service reservoir, and after 72 hours simulation time, the mean age at Node 4001 is 2 days 5 hours 40 minutes. The fraction of water in 10-day age band (original service reservoir water) is 0.19.

Figure 7.149 shows the age results after 192 hours of simulation.

7.5.5 Maximum Age

The maximum age identifies when the oldest water may be found within the network. The output design includes a list of the ten oldest occurrences of water within the network but the ages are not calculated. Figure 7.150 shows an extract from the simulation output file identifying the ten occurrences of the maximum aged water in the network.

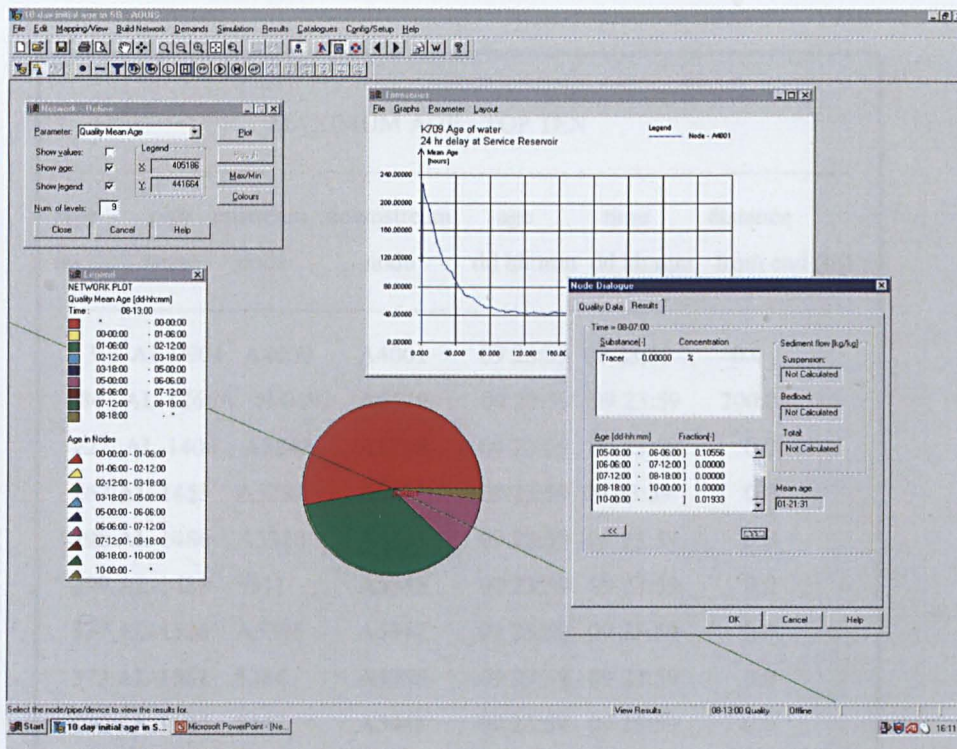


Figure 7.149 Age components at Node 4001 after 192 hours of simulation

With a ten-day initial age in the service reservoir, and after 192 hours simulation time the mean age at Node 4001 is 1 day 21 hours 31 minutes. The fraction of water in 10-day age band (original service reservoir water) is 0.02. Essentially, the age has now stabilised following an exponential decay of the original water volume in the service reservoir. The decay is clearly highlighted in the time series plot on each of the figures.

As well as the mean age and true age components of the mean age, the model can detect where the oldest water in the network can be found.

7.5.5 Maximum Age

The maximum age identifies where the oldest water may be found within the network. The output design includes a table of the ten oldest occurrences of water within the network but the ages are not calculated. Figure 7.150 shows an extract from the simulation output file identifying the ten occurrences of the maximum aged water in the network.

MAXIMUM AGE - TOP TEN						
pipe no.	pipe name	upstream node	downstream node	age dd hh:mm	time dd hh:mm	distance from end (m)
3	AL-0904	A4000	A4001	09 23:59	09 23:59	0.0
84	AL-1266A	N-0492	A5519	09 23:59	09 23:59	200.0
222	AL-1406	A5242	A5708	09 23:59	09 23:59	0.0
267	AL-1453	A5281	A5283	09 23:59	09 23:59	0.0
298	AL-1486	A5310	A5355	09 23:59	09 23:59	0.0
299	AL-1488	5311	A5346	09 23:59	09 23:59	0.0
337	AL-1526	A5346	A5347	09 23:59	09 23:59	0.0
372	AL-1561	5384	A5385	09 23:59	09 23:59	0.0
436	AL-1624	5447	A5448	09 23:59	09 23:59	0.0
464	AL-1655	A5478	A5479	09 23:59	09 23:59	40.0

Figure 7.150 Maximum age top ten occurrences from output file

Maximum age is calculated from the age of water entering a node from all pipes connected to it. In the case of quasi-dynamic simulations, this procedure will be repeated for each time step, but the maximum age of water in service reservoirs is updated based on knowledge about the size of the actual time step. During a quasi-dynamic simulation, the maximum age that occurred at each node is stored thereby allowing the oldest water to be tracked.

The parts of the network containing water of the maximum age therefore can easily be located. Figure 7.151 shows how a network plot can be used to study an entire network.

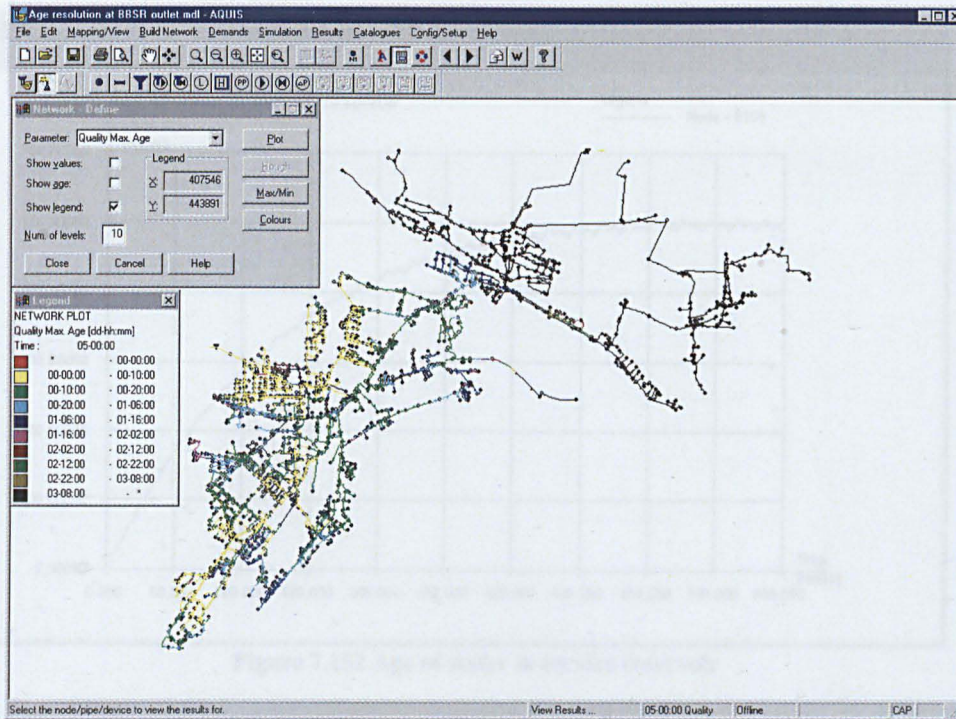


Figure 7.151 Maximum age across part of the study network

It is relatively simple to identify that the zone to the NorthEast has older water than any other section of this part of the network. This is because the service reservoir has a low turnover rate and the water ages considerably before entering the network. It can be seen from Figure 7.152 that the mean age in the service reservoir supplying this part of the network is 150 hours.

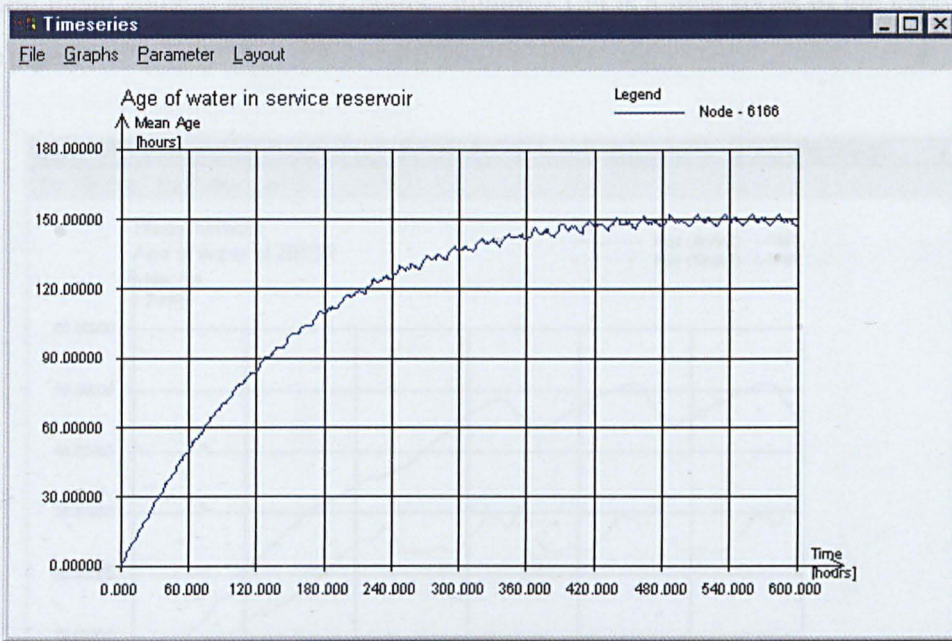


Figure 7.152 Age of water in service reservoir

The difference between mean and maximum age at a particular location gives an indication of where pockets of older water are travelling. Figure 7.153 is a time series of mean age in two pipes in the study network.

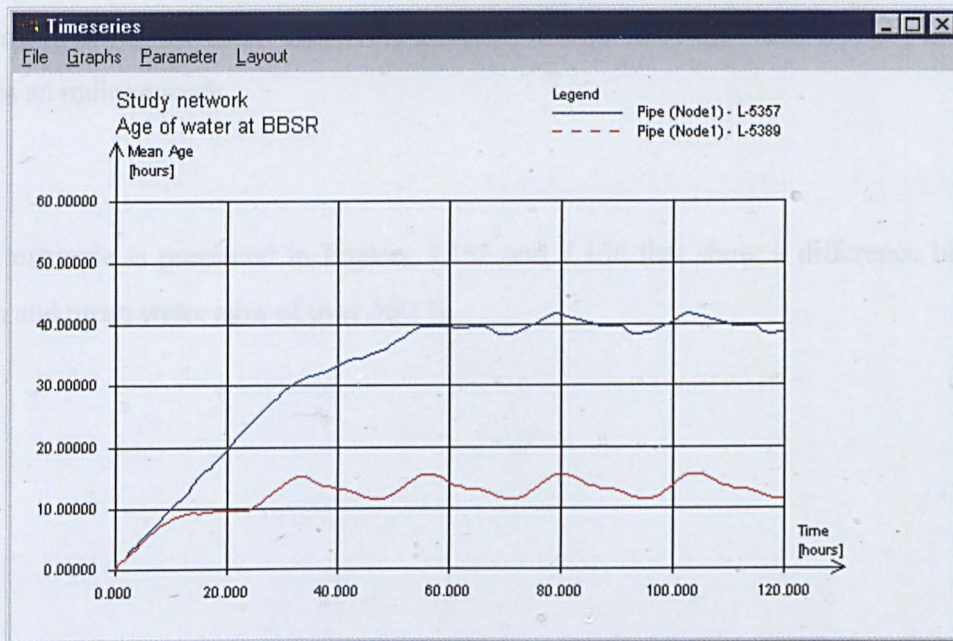


Figure 7.153 Time series of mean age in two pipes

The highest mean value is around 42 hours. Figure 7.154 is a time series of the maximum age in the same pipes.

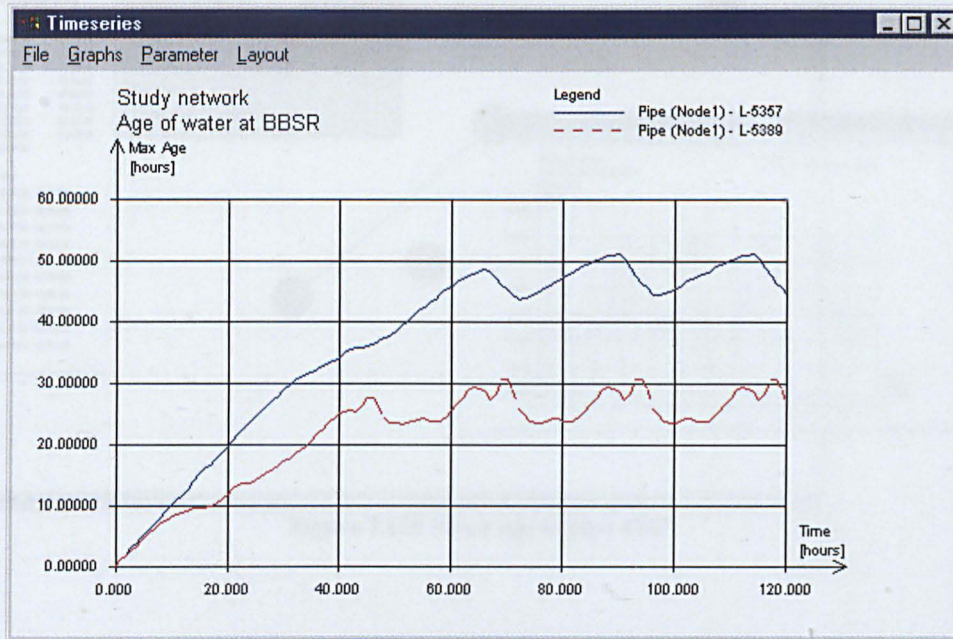


Figure 7.154 Time series of maximum age in two pipes

It can be seen that the maximum age is over 50 hours, an increase of 10 hours when compared to the mean age. The maximum age of water travelling through these two pipes is some 20% older than the maximum of the mean age. This indicates that some of the water flowing into this pipe does so via an indirect route.

A second example is presented in Figures 7.155 and 7.156 that show a difference between the maximum and mean water ages of over 360 %.

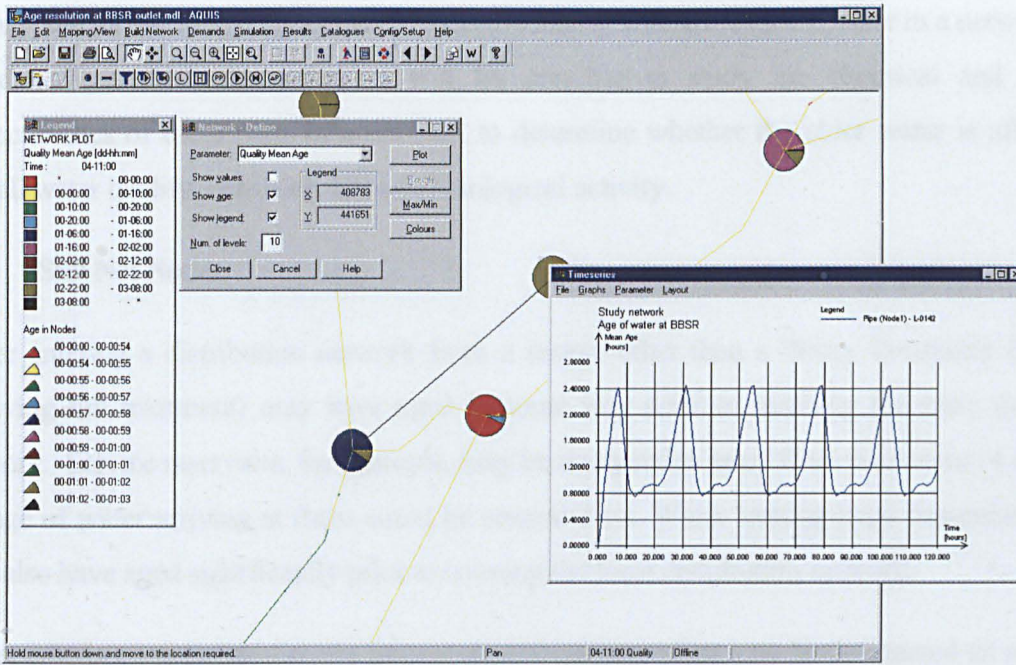


Figure 7.155 Mean age in pipe 4142

The maximum of the mean age in pipe 4142 is 2.4 hours. Figure 7.156 shows the maximum age profile in the same pipe.

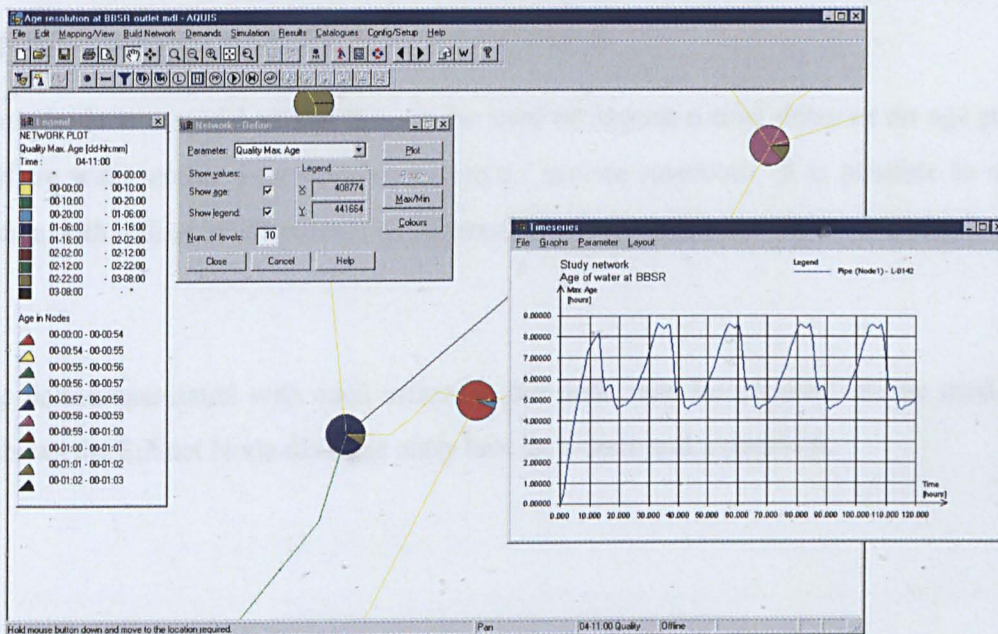


Figure 7.156 Maximum age in pipe 4142

The maximum age can be seen to be 8.7 hours.

Clearly, the model functionality can be used to identify where the oldest water in a network can be found. With this information, it will be possible to study the chemical and biological characteristics of these parts of a network to determine whether the older water is affecting the overall water quality, corrosion rates and biological activity.

7.5.6 Sub Net Nodes

Water entering a distribution network from a source other than a Water Treatment Plant (after receiving full treatment) may have aged in some way prior to reaching the entry point to the network. Service reservoirs, for example, may be many miles away from the source of supply and the age of water arriving at them could be several days. Water leaving large transmission mains may also have aged significantly prior to entering the local distribution network.

Water entering a network directly from a water treatment plant may be designated an age of zero as it can, for all intents and purposes, be declared to be “new” following clarification, filtration and disinfection. However, water from a service reservoir or source other than the treatment works is also assumed to have an age of zero in most current models. The calculated ages of water in a network where this occurs can therefore be very misleading.

In order to overcome this problem and allocate all sources a realistic age at the beginning of a water quality simulation, a Sub Net Node was developed.

A sub net node is a model utility that can be used to impose a time delay or an age profile any node where water enters a network or a source / service reservoir. It is possible to simulate a whole area with inflow, outflow and time dependent consumption in a single or multiple nodes.

Specific delays associated with each inflow to the node may be imposed on the model. Figure 7.157 shows the Subnet Node dialogue entry box for a node and a reservoir.

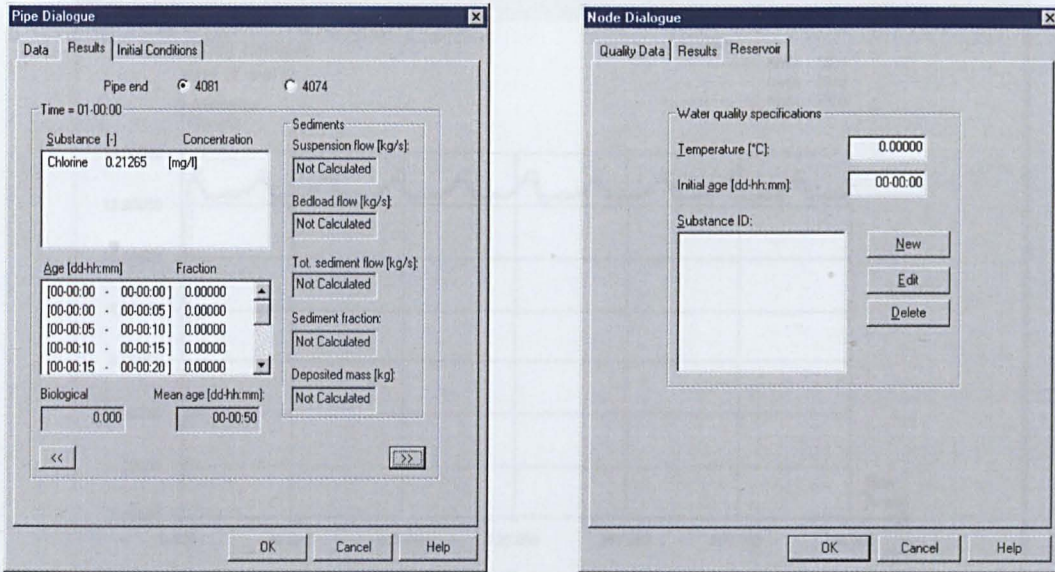


Figure 7.157 Subnet Node dialogue boxes for node and reservoir

Figure 7.158 shows age time-series for three nodes in series when no delay is imposed on the first one.

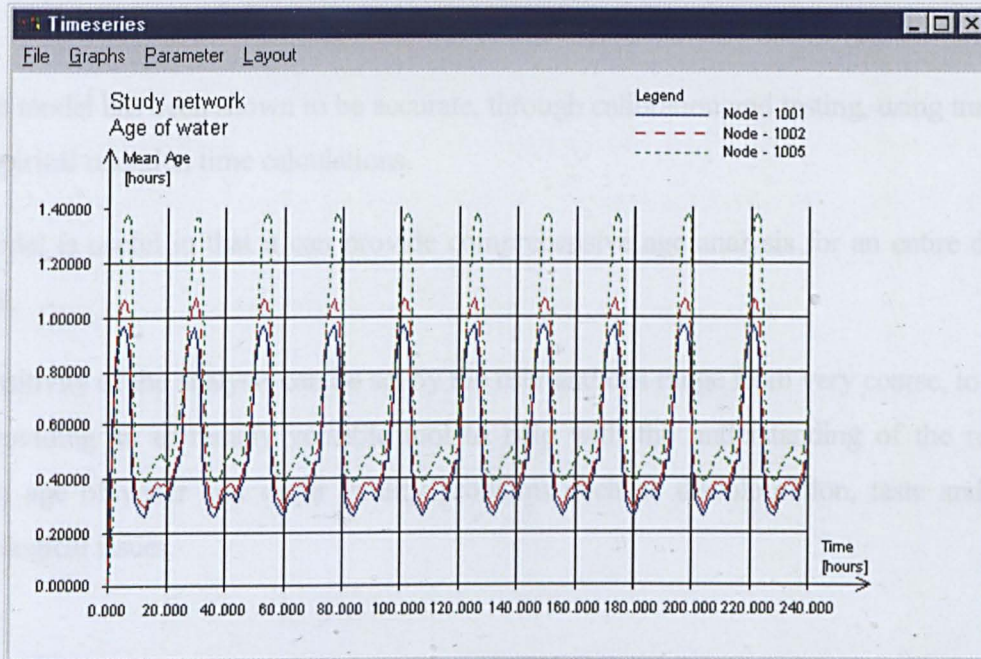


Figure 7.158 Subnet Node with no delay

Figure 7.159 shows the same three nodes but with a 12-hour delay imposed on the inlet node.

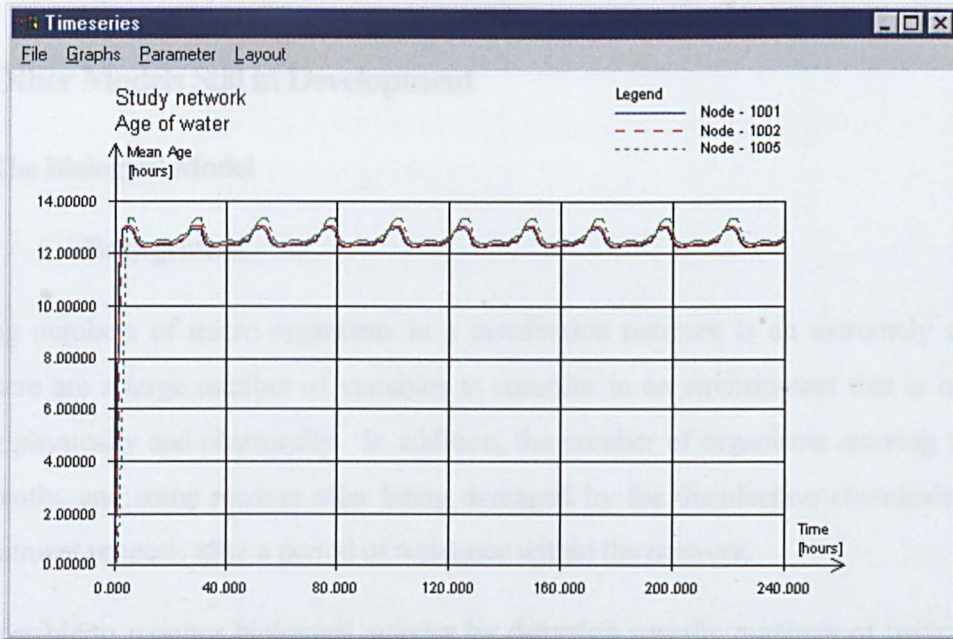


Figure 7.159 Subnet Node with a 12-hour delay imposed

It can be seen that the age profiles have all been shifted by 12 hours. The use of a sub net node does not alter the age profile. It adds a delay constant that is applied over the whole profile and to every node downstream.

7.5.7 Summary of Age Model

The age model has been shown to be accurate, through calibration and testing, using tracer studies and empirical retention time calculations.

The model is useful in that it can provide comprehensive age analysis for an entire distribution network.

The sensitivity of the analysis can be set by the user and can range from very coarse, to extremely fine, providing an extremely versatile tool to help with the understanding of the relationship between age of water and water quality problems such as discolouration, taste and odour or bacteriological issues.

7.6 Other Models Still in Development

7.6.1 The Biological Model

7.6.1.1 Background

Modelling numbers of micro-organisms in a distribution network is an extremely complicated task. There are a large number of variables to consider in an environment that is continuously changing physically and chemically. In addition, the number of organisms entering the network varies greatly, and some recover after being damaged by the disinfection chemicals during the water treatment process, after a period of residence within the network.

It is impossible to monitor biological activity by detecting specific numbers of bacteria at a rate that would provide a window of opportunity for process control via feedback to the water treatment process above that which already exists through regulatory sampling. However, research effort *has* been put into continuous high speed bacteriological monitoring (*Joret et al., 1989*), (*Colin, 1994*), but the technology is not yet fully developed and, because it depends on growing live organisms, there will always be a significant minimum time delay before a result can be obtained making it inappropriate for process control loop technologies.

It would be useful however, if the conditions that favour bacteriological growth / re-growth could be monitored to provide surrogate information that could be used to model a network to provide a better understanding of where in the network micro-organisms might be more active. Operational controls or changes could then be made to minimise the conditions that favour biological activity. The biological model described in this section attempts to do exactly that.

It is possible for organisms to enter a distribution network by other means other than the water treatment process, for example as a result of burst pipes. If the burst is sufficiently large, the resulting pressure drop in parts of the network may result in cavitation and draw foreign material into the network.

The different species of micro-organisms present in the network are opportunistic and population dynamics can change very quickly depending on conditions prevailing at a given time (*Banks, 1998*). The vast majority of organisms in a distribution network are harmless, but routine sampling often results in the isolation of organisms that is indicative of faecal contamination.

Water utilities worldwide suffer from these unsatisfactory bacteriological samples, often for no apparent reason. *Machell, 1993*, found that water utilities in France experienced almost identical bacteriological failure patterns as those in the UK.

In the United Kingdom, the bacteriological quality of drinking water has improved tremendously over the last decade due to improved water treatment processes and control and the sealing, regular cleaning and management of service reservoirs. However, each year many companies still suffer unsatisfactory bacteriological samples in their distribution networks.

The biological model for this study was developed therefore as an attempt to try to better understand the reasons for the sporadic failures. It was designed from first principals taking into account several important factors that are related to the basic survival needs of micro-organisms such as food supply, and turbidity that provides protection from disinfectant. Environmental elements, for example, temperature and the level of disinfectant residual are included. It relates to hydraulic parameters also. These include shear stress, transient pressure fluctuations and cavitation, and the 'roughness' of the internal pipe surface.

The model differentiates between the potential for biological activity in different pipes by applying a positive or negative bias to the growth constant in an exponential growth equation. The need for the exponential relationship is to provide a large difference in the characteristics of each pipe in order to classify the pipes into groups with different relative activity potentials.

The potential model has not been designed therefore to predict numbers of organisms in the network, rather how probable it is that a pipe will be more biologically active compared to other pipes in the same network. Good reasons for not trying to predict numbers of bacteria include:

The majority of bacteria in a distribution network live in the biofilm phase. The relationship between density of biofilm and the numbers of cells found in the planktonic phase is not fully understood, nor is the mechanism for this cell release.

A distribution main is very difficult to sample methodically. Variables such as tap type, length and material of sample line, flow out of tap whilst sampling, length of time tap is flushed before sampling, and tap disinfection method will all affect the final number of bacteria collected in the water sample.

Many methods are available to count bacteria in a water sample, and all give significantly different results.

7.6.1.2 Model Description

The potential for biological activity is given by equation 7.36.

$$N = N_0 e^{k(T-T_0)} \quad 7.36$$

Where:

- N_0 is the configured potential at $T = T_0$ (-)
- k is a constant calculated by the model. The initial value is 1.0 ($^{\circ}\text{C}^{-1}$)
- T is the temperature ($^{\circ}\text{C}$)
- T_0 is a configured reference temperature, at which change in growth potential approaches zero ($^{\circ}\text{C}$)

The potential is calculated at each time step for every pipe.

7.6.1.3 Model Configuration

The model is configured by accessing a number of different dialogue boxes from the main screen shown in Figure 7.160.

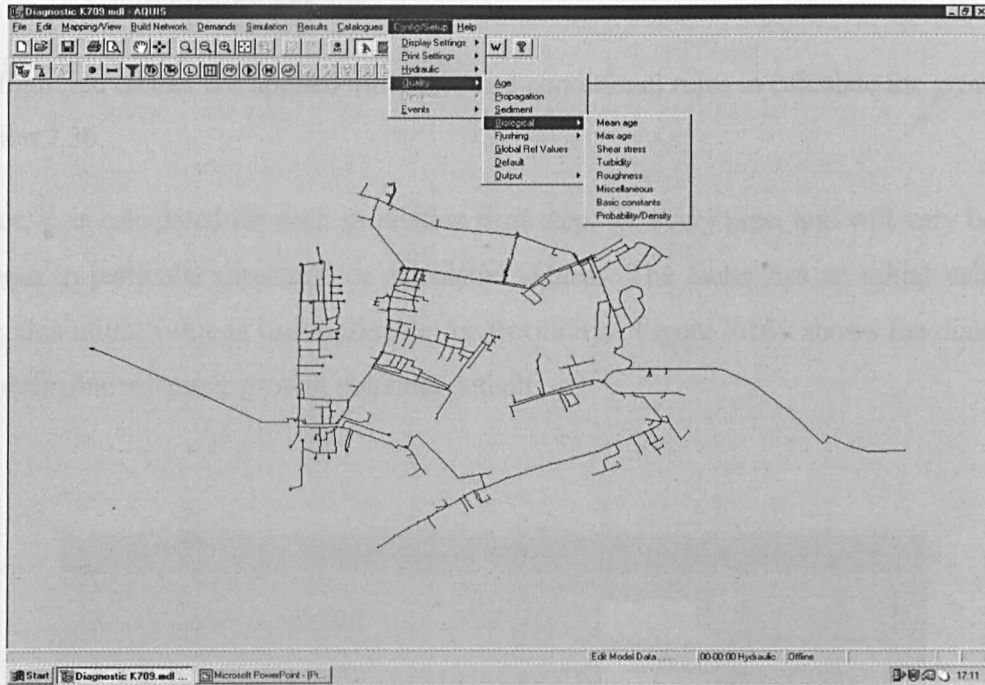


Figure 7.160 Menu structure to the Biological model dialogue boxes

The model takes account of the effect of a combination of any, or all, the following factors:

- Pipe roughness coefficient
- Dissolved oxygen drop (Indication of bacteriological re-growth)
- BDOC (Bio-degradable organic carbon)
- Free chlorine
- Bulk flow reversal(s)
- Transient pressure effects
- Cavitation
- Velocity (As shear stress)
- Turbidity
- Age of the water
- Temperature

The factors are included in the model by using pre programmed default values and / or user specified values for each parameter.

7.6.1.4 Configurable Factors

The configurable factors are applied via tables and conditional rules to calculate the growth factor in Equation 7.36.

The factor, k , is calculated for each simulation time step, for every pipe, and will vary because of the changes in particular measured or calculated values. The factor has an initial value of 1.0 however, this initial value is user definable for flexibility. Figure 7.161 shows the dialogue box with the definable reference growth potential default.

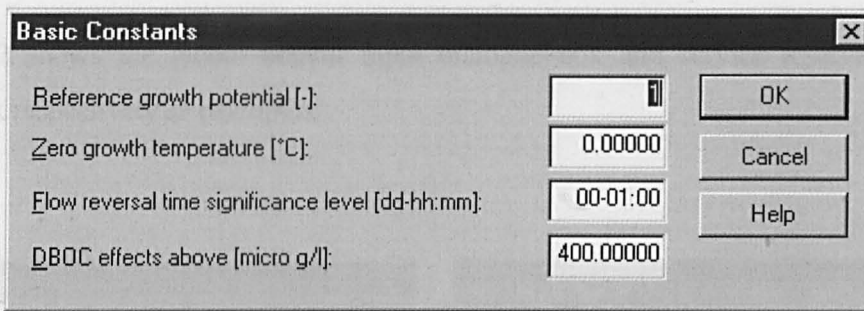


Figure 7.161 The default reference growth potential, k .

During the simulation, the factor is successively multiplied by sub-factors that are defined below (7.6.1.4.1). The final growth factor includes effects of temperature, turbidity, mean age, pipe roughness, dissolved oxygen drop, assimilable organic carbon, level of corrosion, free chlorine, flow reversals (within the last user defined time period), transients, cavitation, shear stress and maximum age.

7.6.1.4.1 Sub Factors

Each sub factor has an individual effect on the overall growth factor k . All are user definable in the model.

7.6.1.4.1.1 Temperature

Water temperature was accurately measured throughout the study distribution network using a Platinum Resistance Temperature Detector incorporated into water quality instruments. (Chapter 4).

Although most temperature variation occurs seasonally, increased water temperature is also seen as a function of age and therefore location (Banks. 1997). Between-site variations of three to four degrees Centigrade were detected in the study distribution network.

Because temperature increases the rate of biological and chemical reactions, higher water temperature will decrease the generation time for bacteria and increase the decay rate of free chlorine. Both these factors contribute to potentially higher numbers of bacteria in the biofilm and planktonic phases. Temperature can be allocated globally or at individual pipe level. This flexibility was necessary because it was observed that some water mains are coincident with sewers or other assets containing hot effluents that heat up the surrounding ground. Water mains very close to such a sewer can suffer local heating up to more than 40 degrees Centigrade.

Temperature can be assigned globally as a default temperature or at individual asset level.

Figure 7.162 shows the global default input dialogue box and service reservoir temperature dialogue box respectively as examples.

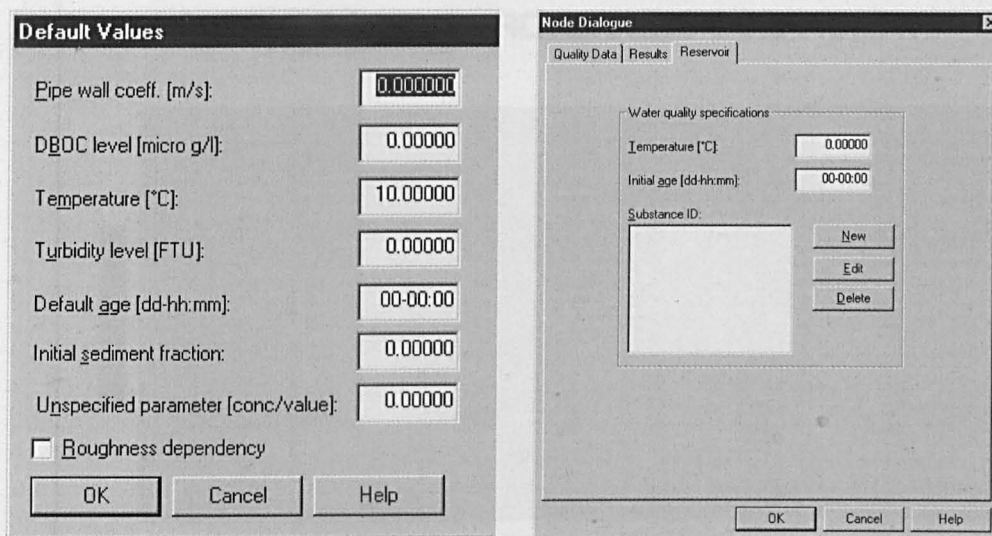


Figure 7.162 Global default, and service reservoir temperature dialogue boxes

7.6.1.4.1.2 Pipe Roughness Coefficient

Pipe material is a significant factor when considering the biological colonisation of a water main. In the initial stages of a colonisation, an unlined cast iron main will provide an uneven surface that is easier for bacteria to adhere to compared to a synthetic pipe such as plastic or MDPE. (Verran, 1997).

Corroded cast iron mains have a larger surface area and a higher chlorine demand than a synthetic main. (Chevallier et al., 1990)

The pits and cavities in a tuberculated iron main offer the biofilm physical, as well as chemical protection from chlorine. The extent of corrosion in a water main may be expressed as a roughness coefficient. This value of the coefficient is indicative of the level of tuberculation in the water main.

The roughness of the internal pipe wall is a very important factor because new iron pipes with smooth internal surfaces have relatively fewer sites for microbial colonisation than do corroded iron pipes. The available surface area increases dramatically as the corrosion mechanism begins, reaching a maximum when the corrosion products reduce the internal diameter of the pipe to a point beyond which further corrosion actually reduces the available surface area again. Figure 7.163 shows the relationship between surface area versus roughness coefficient.

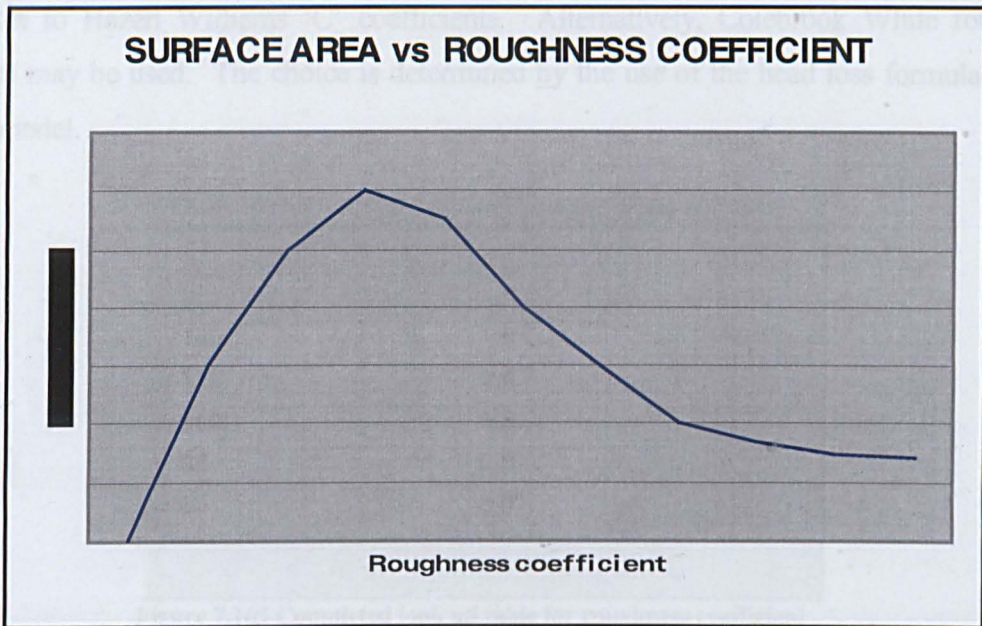


Figure 7.163 Hypothetical surface area vs. roughness coefficient profile

A similar argument would apply to other pipe materials such as plastics to which, for example, manganese and iron salts adhere. These pipes will also be colonised by bio-film that will affect the available surface area and will provide protection for organisms. The effect may however be much less for these materials than for iron. The first set of parameters therefore specifies the dependence on pipe roughness coefficient.

The default table entries for this parameter can be seen in Figure 7.164 below.

Roughness or C - Value [mm] or [-]	Factor [-]
*	1.00000

Figure 7.164 Default look up table for roughness coefficient factor

Figure 7.165 shows a configured look up table for roughness coefficient factors. In this case, the table relates to Hazen Williams ‘C’ coefficients. Alternatively, Colebrook White roughness coefficients may be used. The choice is determined by the use of the head loss formulae in the hydraulic model.

Roughness or C - Value [mm] or [-]	Factor [-]
140	1.0
120	2.0
100	3.0
80	5.0
60	2.0

Figure 7.165 Completed look up table for roughness coefficient

The model determines the roughness coefficient for each pipe from the hydraulic model and looks up the relevant factor from the table.

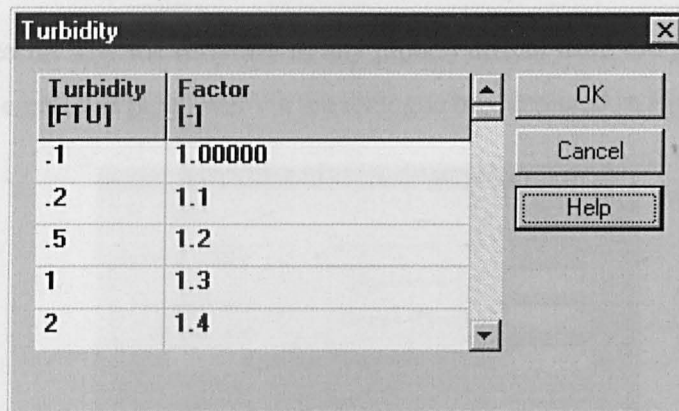
7.6.1.4.1.3 Turbidity

Measurements in the study network highlighted a number of areas where the turbidity exceeded the bulk flow turbidity by a significant amount (Khan et al, 2000). This was thought to be due to

the accumulation and the continual disturbance of corrosion by-products and colloidal matter by unusual demand patterns.

Turbid water indicates high levels of suspended matter that, if organic in nature, could indicate a high nutritional content. Turbid water containing suspended organic matter will also have a higher chlorine demand.

Turbidity protects biological organisms from the effects of disinfectant and provides sites for colonisation. It is therefore an important parameter to consider. The dependency dialogue box for turbidity allows the user to compile a relationship between turbidity and its contribution to the overall affect on biological activity. Figure 7.166 depicts a configured table for dependence on turbidity.



Turbidity [FTU]	Factor [-]
.1	1.00000
.2	1.1
.5	1.2
1	1.3
2	1.4

Figure 7.166 A configured table for dependence on turbidity

The table is configured by entering the appropriate values in both columns. If the table is used with the default values, a factor of 1.0 is used for all levels of turbidity.

The bulk flow turbidity in the water entering the network is specified as a default value, obtained from measurement or user defined. It is entered into the model via the dialogue box shown in Figure 7.167

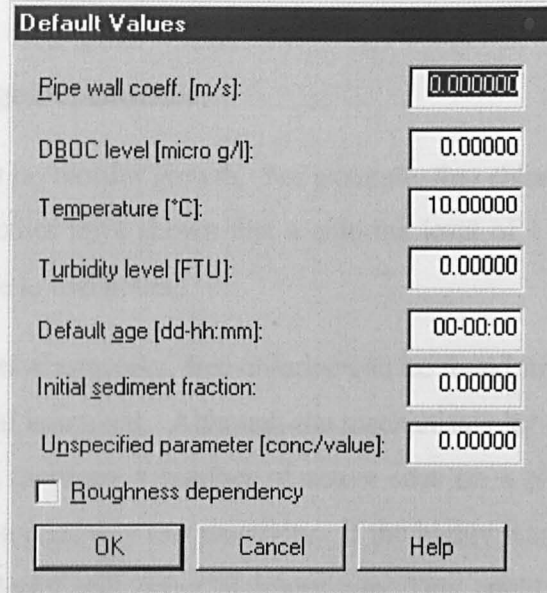


Figure 7.167 The Default Values dialogue box where bulk flow turbidity is entered

If there is a valid reason why the turbidity in any pipe(s) differs from the global value, individual pipe values may be entered at pipe level via the dialogue box depicted in Figure 7.168.

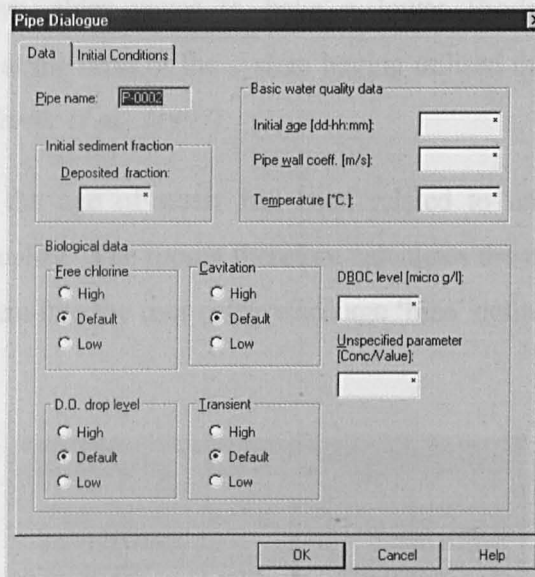


Figure 7.168 Pipe level data entry dialogue box

This dialogue box allows the user to over ride the global configuration data.

7.6.1.4.1.4 Mean Age Dependence

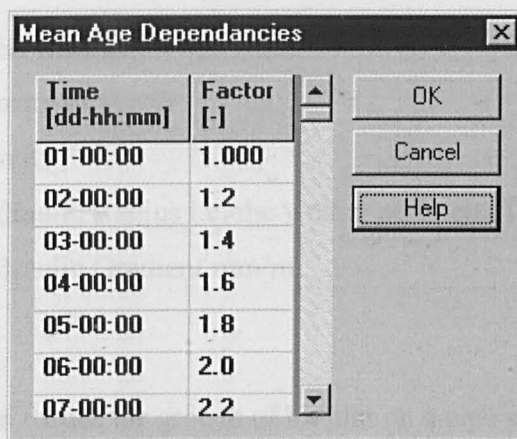
Age has an indirect effect on biofilm growth. For example, free chlorine residuals in water decay with time. Laboratory studies have shown that a chlorine level of 1.0 mg.l^{-1} will decay to $<0.1 \text{ mg.l}^{-1}$ over a period of one to two hours.

However, in real distribution networks, free chlorine can be found in parts of the network where the age of water is several hours old. Although the mechanisms for chlorine decay are not fully understood, it is thought there are a number of active sites on a pipe wall, some with a high reactivity and some with a relatively low reactivity. If the highly reactive sites are saturated with chlorine, then chlorine decay will occur at lower rate. This could explain why free chlorine residuals occur in parts of the network where the water is several hours old. (UKWIR, 1997).

Water temperature has been seen to increase with age (Banks, 1997). Consequently, the rate of biological and chemical reactions will also increase.

Some research has shown older water to have a lower Biodegradable Organic Carbon concentration, the biofilm at the head of the system having utilised the available carbon first (LE Chevallier et al., (1991), Carter et al., (1997)).

As discussed previously, the age of water has been related to unsatisfactory bacteriological samples, and poor water quality. The model therefore calculates the mean age at every time step for every pipe and sorts them into the user categorised age 'bins' defined in column one of Figure 7.169.



Time [dd-hh:mm]	Factor [-]
01-00:00	1.000
02-00:00	1.2
03-00:00	1.4
04-00:00	1.6
05-00:00	1.8
06-00:00	2.0
07-00:00	2.2

Figure 7.169 Configured mean age dependency table

7.6.1.4.1.5 Maximum Age Dependence

Because the age of the water is related to poor water quality, it is a logical assumption that the oldest water will produce the maximum effect. The maximum age is calculated in the same manner as for the mean age. Figure 7.170 shows a configured dependency table.

Time [dd-hh:mm]	Factor [-]
04-00:00	1.0
08-00:00	2
12-00:00	3
16-00:00	4
20-00:00	6
24-00:00	8

Figure 7.170 Configured maximum age dependency table

7.6.1.4.1.6 Dependence on shear stress

Shear stress is determined in $N.m^{-2}$ and is calculated using:

$$r = P g R S \quad 7.37$$

Where:

r = Shear Stress

P = Density of Water

g = Gravity

R = Hydraulic Radius i.e. the wetted perimeter $D/4$

S = Hydraulic Gradient mm/m

Very high shear stresses can reduce the growth of biofilm on a pipe surface. However, stresses of this magnitude are not common in a distribution system. Pipes have a design capacity of around $1 m.sec^{-1}$; above this, the head losses are excessive. Networks are usually designed so ideally, velocities do not exceed $0.7 m.sec^{-1}$. Where growth does occur in an environment with high shear

stresses, the structure of biofilm will adapt to these conditions. The intracellular matrix will become denser and the bonds stronger, making the cells in the biofilm less prone to sloughing.

If a biofilm is grown in low shear stress conditions, and is suddenly subject to greater stresses, more cell loss will occur (Stoodley, 1997).

Shear stress at the pipe wall is responsible for sloughing of biofilm and corrosion products. A high shear stress will limit the growth of biofilm and minimise localised particulate build up from corrosion mechanisms. The model at each time step for each pipe calculates the shear stress in column one. The shear stress values are sorted into user ranges defined in column one. Figure 7.171 depicts a completed dependency table for effect of shear stress.

Turbidity [FTU]	Factor [-]
.1	1.00000
.2	1.1
.5	1.2
1	1.3
2	1.4

Figure 7.171 Dependency table for effect of shear stress with default settings

The values in column one of the tables are upper limits to which the factor in column two is applied to the factor k . The band width (difference between individual upper limits, or column one values, can be as small or as large as the user wishes thereby increasing or decreasing the sensitivity as required for individual applications. The last factor value in column two is applied to any value in column one above the last numerical entry.

The tables approach allows the user to define any type of relationship between the measured parameter and the applied factor. This means that as new data becomes available the model can be modified accordingly.

It is possible to calibrate a model of this kind and get a very good match between predicted and measured results. There is a chance however, that such results are correct for the wrong reasons or even by chance. One use of such a model is to continually fine-tune model data and the contributions of individual parameters by using increasingly real data from the field to ensure

correct cause effect relationships for the parameters. For example, if turbidity is measured continuously at many sites in a network, and the relationship between level of turbidity and biological activity is accurately defined over a period, this data can be entered into the model. Doing this may result in erroneous model predictions that have to be amended by changing another parameter to make the predictions accurate again. By an iterative process, the model continually evolves until it is certain as is practicable that the correct results are being obtained for the right reasons i.e. all the defined relationships are corrected to reflect measured data thereby calibrating the model.

All the above factors can be applied globally or at individual pipe level. There are other factors accounted for by the models that are only applied globally because they relate to all pipes in a network simultaneously.

7.6.1.4.2 Miscellaneous Dependencies

The model considers the remaining parameters as miscellaneous dependencies. The effects of these parameters are complex and may be positive or negative. For example, high free chlorine residual will have a significant positive effect on reducing biological activity therefore the factor value may be a fraction say, 0.2, that will reflect this in the model. Similarly, low chlorine residual would have a much lesser effect on the biological population and may have a factor value of 0.8 that still produces an overall negative effect on the growth potential.

Therefore, each parameter has two user-definable factors: one for a high level impact and one for low-level impact providing for maximum model flexibility. Figure 7.172 shows the high and low factors defined in the Miscellaneous Dependencies dialogue box.

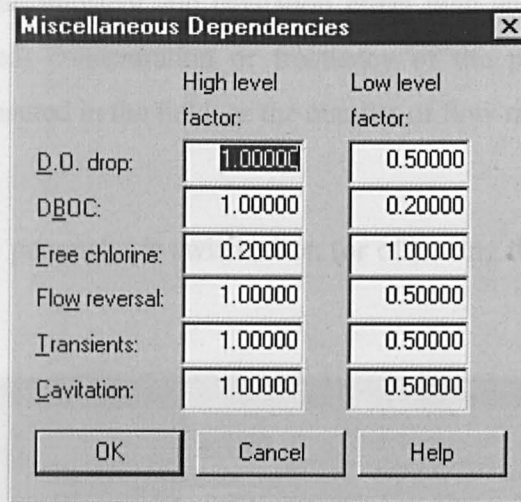


Figure 7.172 Miscellaneous Dependencies dialogue box

Table 7.1. shows how this configuration information is translated into the model input file.

	Dissolved Oxygen	DBOC	Free	Flow	Transient	Cavitation
	drop		Chlorine	reversal	pressure	
Yes	1	1	0.2	1	1	1
No	0.5	0.2	1	0.5	0.5	0.5

Table 7.1 Miscellaneous dependency information from the model input file

In some networks, the individual dependencies for each factor will differ because of pipe material, age of network and the source of water, corrosion propensity and for many other reasons. The dependency dialogue box is therefore fully user configurable allowing any size of high or low factor to be applied.

The definition high or low is at the user discretion, because what is regarded for example, as high free chlorine in some networks would be regarded as quite modest in others. Similarly, a network that is used to relatively high chlorine residuals may have a biological explosion should that residual drop to a level that in other network would be considered high. Due consideration must therefore be given to the magnitude of the factors.

The constant 'k' in the growth potential equation, 7.36, is multiplied successively by the user defined values depending on whether, in reality, a parameter level is high or low.

Free chlorine, BDOC residual, transient and cavitation effect high and low levels relate to the actual (measured / calculated) concentration or frequency of the parameter. For example, concentration of chlorine measured in the field, or the number of flow reversals determined by the hydraulic engine.

High and low effect for each parameter is switched on (or off) using the dialogue box shown in Figure 7.173.

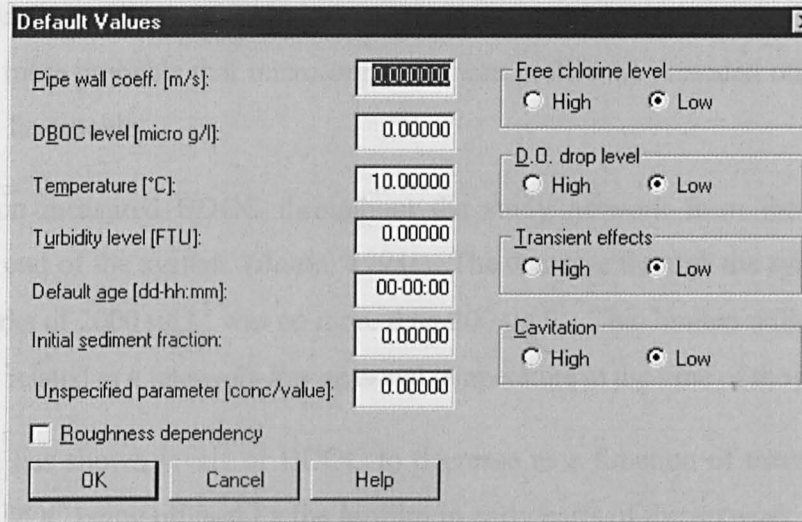


Figure 7.173 High and low effect switches

This dialogue box is also used to define the incoming turbidity level in the bulk water flow. The reason for inclusion of the miscellaneous dependencies is presented in the following sections.

7.6.1.4.2.1 Free Chlorine

The bactericidal effect of free chlorine is well documented and chlorine has been the most widely used disinfectant for water treatment since the mid 20th Century. The presence of free chlorine will inhibit the re-growth of micro-organisms and the effect in terms of equation 7.36 needs to be negative so the applied factor will be less than 1.0. The higher the free chlorine the more negative the growth effect should be so the nearer to zero the factor will be.

Free-living bacteria are more sensitive to chlorine than those living within a biofilm are. The polysaccharide matrix, holding the biofilm together, is thought to offer protection to the bacteria living within the film (*Wingender, 1997*). Networks with well-established biofilms might therefore require a low effect factor compared to those with sporadic or weakly established biofilms.

Free chlorine decays as a function of time and pipe material at a rate dependent on water temperature. Where the water is relatively old, higher factors may be required compared to areas of the network where the flow rate is relatively high.

7.6.1.4.2.2 Biodegradable Organic Carbon (BDOC)

Biodegradable organic carbon is a food source for micro-organisms and, if there is only a low level of food available, growth will be limited. The main reason for the inclusion of this parameter is that the availability of BDOC will be reflected in the microbiological population dynamics. The more food the more probable that micro-organisms can proliferate provided other environmental factors are also favourable.

An investigation measured BDOC throughout the study network from the treatment plant, through to the end of the system (*Banks, 1993*). The decrease through the system, from a level leaving the works of $2000 \mu\text{g.l}^{-1}$ was no more than $200 \mu\text{g.l}^{-1}$. This limited utilisation of nutrients may have been related to a relatively low seasonal temperature at the time of the study.

Other research has shown levels of BDOC to decrease as a function of travel time through a network, the BDOC being utilised by the biofilm in early parts of the network. (*UKWIR, 1995*). It is important therefore to determine whether BDOC levels are constant or diminishing through the network.

The effect of BDOC concentration on the model is determined by two switches. The first, a simple on / off switch is defined based on the overall level of nutrient availability. If the incoming bulk water flow has a level of BDOC that does not support or enhance bacteriological growth the effect can be switched off completely using the dialogue box shown in Figure 7.174.

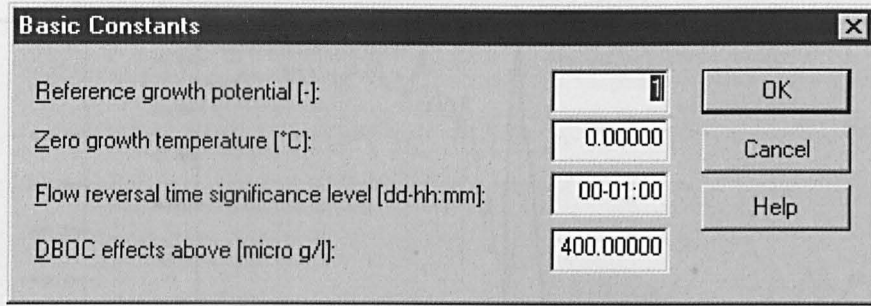


Figure 7.174 Basic Constants dialogue box.

In this case the BDOC residual must exceed 400 mg.l^{-1} to have any effect at all. Above this value the high and low level factor boxes are used as before. A high BDOC will attract a high level factor that will be > 1.0 to produce an increased biological potential and a moderate BDOC residual would attract a factor that produced a more modest effect on the overall biological potential.

7.6.1.4.2.3 Transient effects and Cavitation

Pumps, particularly those without surge damping devices can cause pressure transients when switching on and off. Valve operations or any phenomenon that can cause a sudden change in flow may also generate transient pressure effects. These pressure surges may be capable of “exploding” biofilm into the planktonic phase and into supply.

The effects of transient pressure waves can cause disruption to biofilm layers within the pipe network and disturb accumulated sediments entraining them in the bulk flow (Keevil, 1995). Figure 7.175 shows the effect on turbidity of a sudden pressure change in the study network.

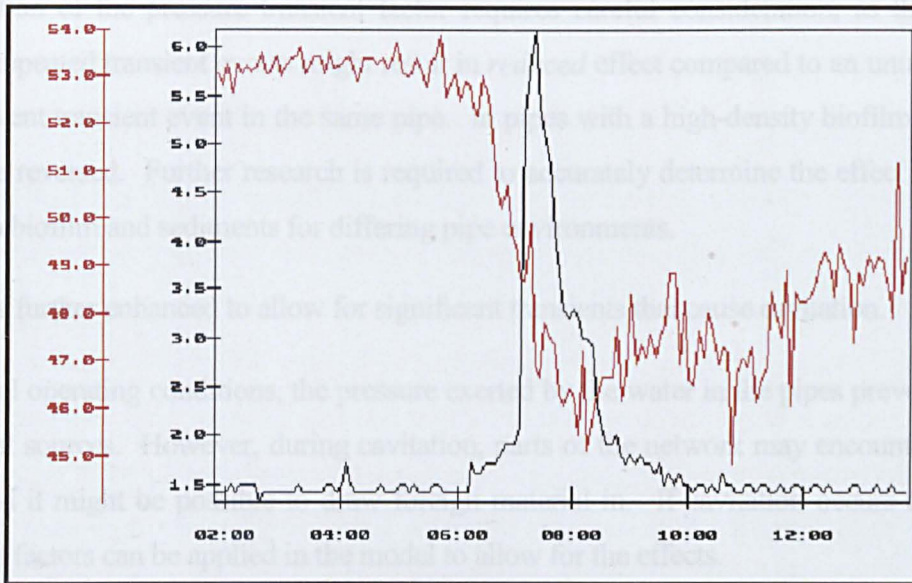


Figure 7.175 Pressure vs turbidity

The water quality instruments detailed in Chapter 4 collected the data depicted in this figure. The resultant increase in turbidity may last for several hours in low flow velocity pipes as shown in Figure 7.176. (Khan, 2000).

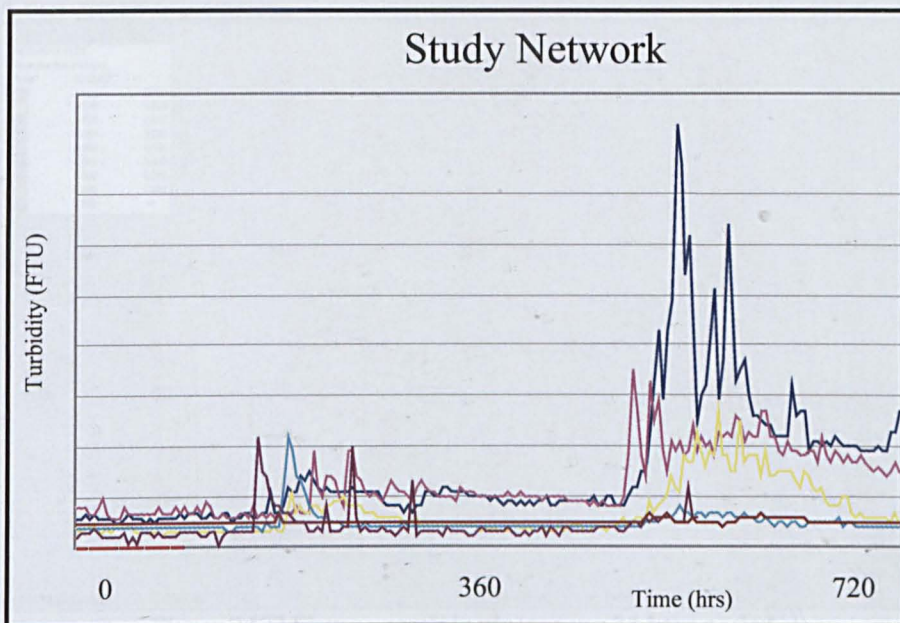


Figure 7.176 Time series showing duration of turbidity event following a burst main

The application of the pressure transient factor requires careful consideration, as the effect of continually repeated transient events might result in *reduced* effect compared to an unusual or low frequency event transient event in the same pipe. In pipes with a high-density biofilm, this effect may even be reversed. Further research is required to accurately determine the effect of pressure transients on biofilm and sediments for differing pipe environments.

The model is further enhanced to allow for significant transients that cause cavitation.

Under normal operating conditions, the pressure exerted by the water in the pipes prevents ingress from external sources. However, during cavitation, parts of the network may encounter negative pressures and it might be possible to draw foreign material in. If cavitation occurs therefore, a second set of factors can be applied in the model to allow for the effects.

7.6.1.4.2.4 Flow Reversals

The hydraulic model determines the number of flow reversals for every time step for every pipe in a network. Figure 7.177 highlights the pipes in a section of the study network that suffer flow reversals and the number of times the flow reverses direction over a 24-hour period.

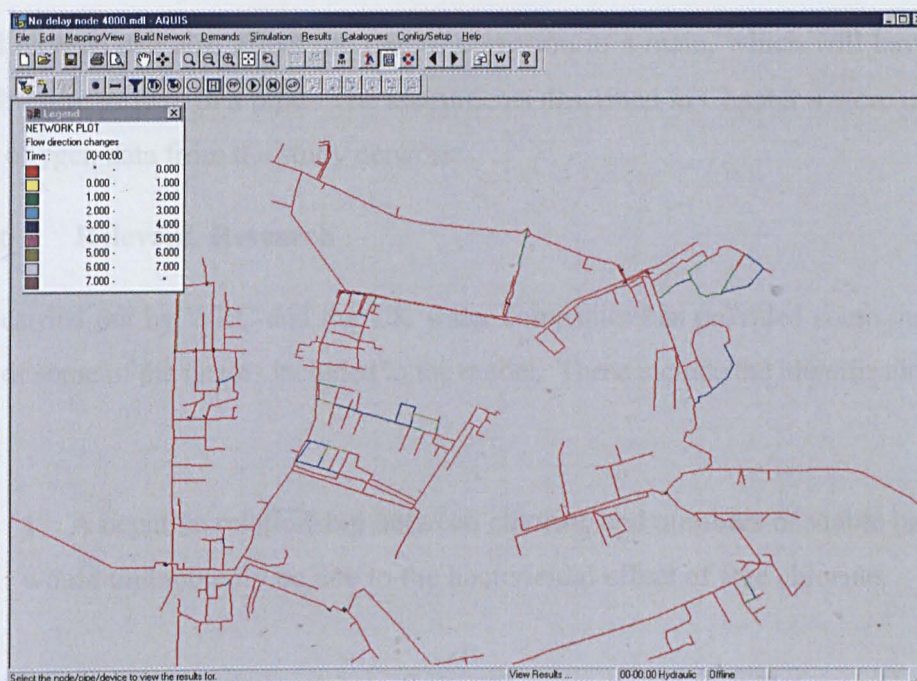


Figure 7.177 Flow reversals in pipes over a 24-hour period

Reversals in flow can re-suspend sediments, increasing the turbidity, and potentially redistributing a food source. If the changes in velocity are severe enough, they can also disrupt the biofilm and

transport cells into the planktonic phase. Water in pipes experiencing frequent reversals in flow, can also suffer from excessive age, this can have the effect of increasing water temperature, decreasing chlorine residual and increasing biofilm formation.

The relative effect of a flow reversal(s) will be dependent on the time between successive reversal events. If there are several flow reversals per day in a pipe the effect will be negligible as the frequency of the events make it a continuous characteristic of the pipe. A very low frequency, or an unusual flow event, will have a much more significant impact, especially in low velocity mains where sediments may have accumulated and biofilm may have colonised to a greater extent than would be the case in a pipe with higher flow velocity. This factor therefore requires analysis of the flow reversal patterns in the network and a scientific approach to the magnitude of the relative effect entered in the look up table before its application.

7.6.1.4.2.5 Dissolved Oxygen

Dissolved oxygen can affect the rate of growth and respiration in aerobic bacteria. However in an environment of low oxygen, the species present in the biofilm will change to adapt to the low or absent oxygen, i.e. the proportion of anaerobes will increase.

Dissolved oxygen can also affect the rate of corrosion in a main, which will have an indirect effect on biofilm growth in a pipe. The instruments described in Chapter 4 were used to obtain dissolved oxygen data from the study network.

7.6.1.4.2.6 Relevant Research

Research carried out by WRC and the UK water companies has provided some insight into the relevance of some of the factors included in the model. These include the identification of:

- 1 A negative relationship between chlorine and numbers of viable bacteria. This would undoubtedly be due to the bactericidal effect of free chlorine.
- 2 A positive relationship to flow. This may be because Heterotrophs require a constant source of organic carbon and a reasonable supply of dissolved oxygen. Therefore, a pipe with a relatively high flow will satisfy these requirements better than a pipe with a low flow.

(On the other hand, pipes with reasonable flow rates are more likely to contain the younger water in the system and have higher free chlorine residuals so the effect will be different in any particular network).

3 A strong negative effect with respect to velocity. This is because biofilm that develops in high velocity mains tend to be more resilient to sloughing and releasing cells, than those grown in lower velocity mains.

4 The higher bacteriological failure rates appeared in the summer months when the water was warmer supporting the knowledge that temperature is a key factor in biological growth dynamics.

The variables that were shown to have a significant impact are all taken into account in the Biological model. However, further work is required to obtain a more comprehensive set of data and determine the effect of the other variables that are in the Potential model that were not considered in this first analysis.

7.6.1.5 Model Output

The most useful output from the biological model is a network plot highlighting the difference in biological potential between individual pipes. Figure 7.178 shows a plot where all the pipes have the same characteristics.

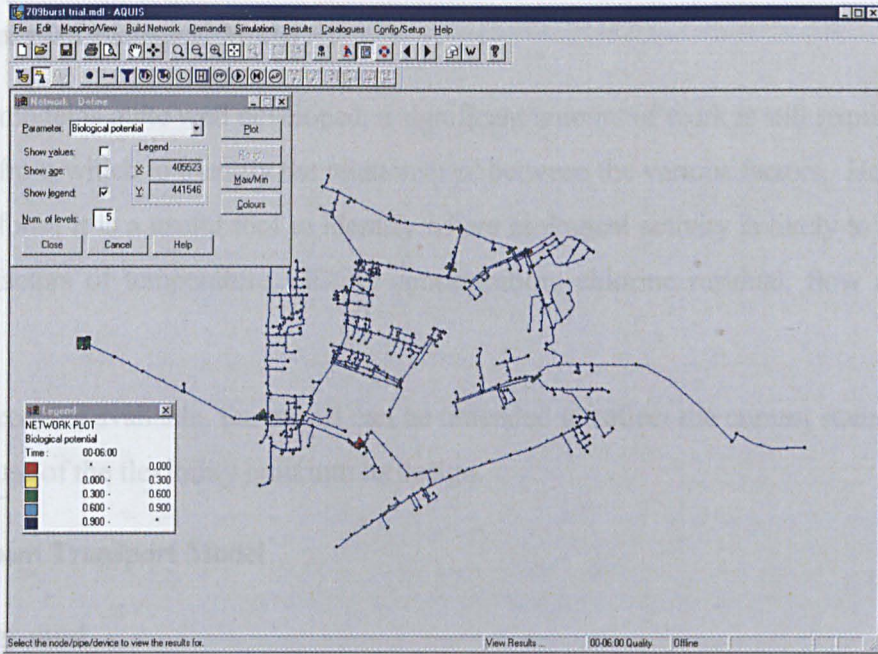


Figure 7.178 plot where all the pipes have the same characteristics

Figure 7.179 is the same plot when the temperature is raised in a single pipe to highlight how pipes with higher biological activity potential are easily identified.

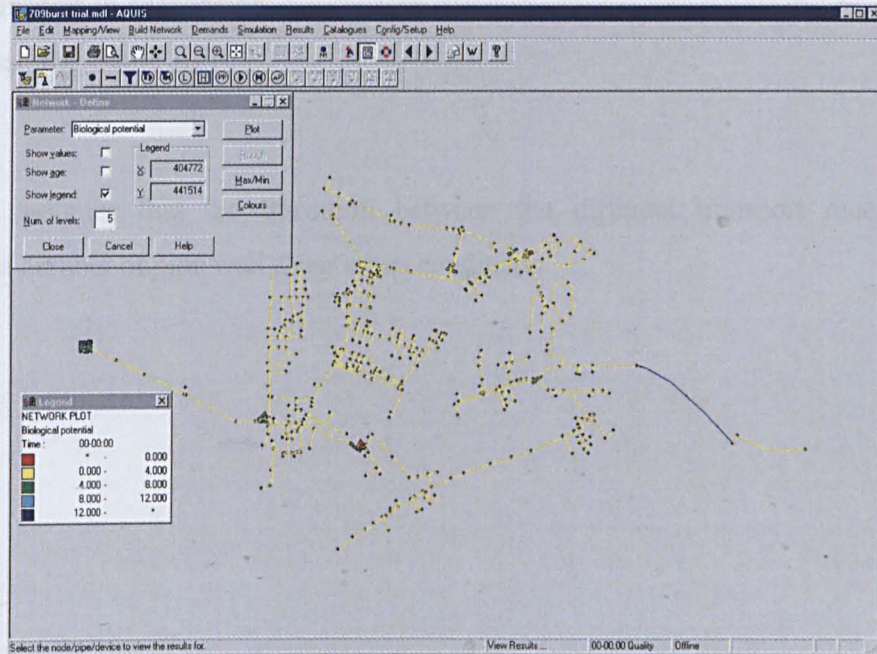


Figure 7.179 Pipes with higher biological activity potential

7.6.1.6 Summary of Biological Model

Although the model is quite well developed, a significant amount of work is still required to obtain network data from which to identify the relationships between the various factors. However, even in its current form, it is a useful tool to identify where biological activity is likely to be using the well-proven factors of temperature, BDOC concentration, chlorine residual, flow and velocity data.

As the data becomes available, the model can be amended to reflect the current state of the art in this area because of the flexibility built into its design.

7.6.2 Sediment Transport Model

7.6.2.1 Background

The sediment transport model differentiates between the following forms of particle transport:

- Settlement of suspended particles (precipitation) - no bed load transport
- Transport in suspension and by bed load movement
- Transport in suspension
- Flushing (Scouring)

The approach assumes that the transition between the different transport mechanisms is determined by a number of pipe wall shear stress conditions.

7.6.2.2 Model Description

7.6.2.2.1 General

The sediment transport model is a box model. The box model is shown in Figure 7.180

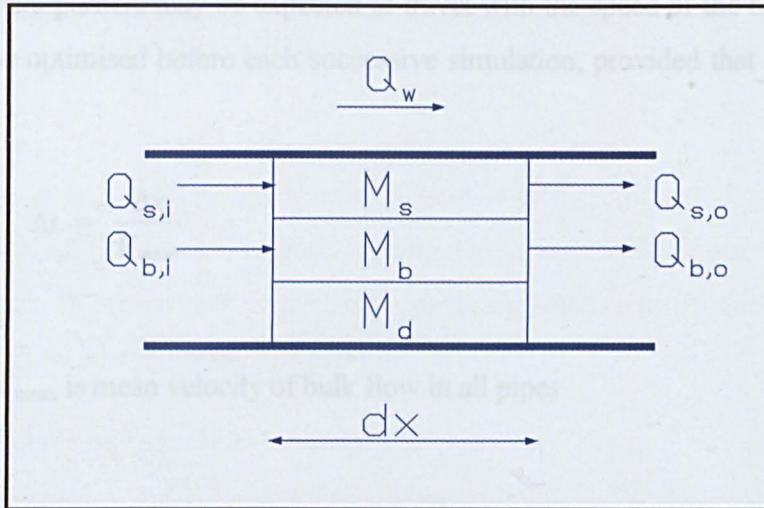


Figure 7.180 The sediment transport box model

Where:

M_s Is the mass of sediments in suspension (kg)

M_b Is the mass of sediments in bed load movement (kg)

M_d Is the mass of deposited sediments (kg)

Q_w Is the water flow ($\text{kg}\cdot\text{s}^{-1}$)

$Q_{s,i}$ Is the flow of suspended sediments entering the box ($\text{kg}\cdot\text{s}^{-1}$)

$Q_{b,i}$ Is the flow of bed load movement entering the box ($\text{kg}\cdot\text{s}^{-1}$)

$Q_{s,o}$ Is the flow of suspended sediments leaving the box ($\text{kg}\cdot\text{s}^{-1}$)

$Q_{b,o}$ Is the flow of bed load movement leaving the box ($\text{kg}\cdot\text{s}^{-1}$)

dx Is the length of box, $dx = \Delta x$ (m)

Each pipe in the model is divided into a number of boxes. The model will calculate the mass of sediment entering the box, leaving the box, and present within the box in the three phases; suspension, bed load and deposited mass, at every simulation time step.

7.6.2.2.2 Numerical Solution

The velocity of a sediment particle is a function of the actual time step and the length of boxes:

$$V_{\text{particle}} = 0.5 \cdot \Delta x / \Delta t \quad 7.37$$

Because a sediment particle may be expected to travel with the speed of the bulk flow, the actual time step may be optimised before each successive simulation, provided that default time step is accepted:

$$\Delta t = \frac{\Delta x}{V_{\text{mean}}} \quad 7.37$$

Where: V_{mean} is mean velocity of bulk flow in all pipes

In order to utilize *all* the information contained in the hydraulic database, the user defined time step may be overridden by automatic adjustment of the time step. At each time step the adaption, a measure for the relative agreement between velocity of bulk flow and sediment particles, is calculated from equation 3.

$$ADAPTION = \left(1 - \frac{\sigma}{N_p^{0.5}}\right) \cdot 100\% \quad 7.38$$

Where:

N_p is the number of pipes

σ is a function of the relative error in the distance travelled by a sediment particle:

$$\sigma = (\sum r_i^2)^{0.5} \quad 7.39$$

Where: r_i is the relative error:

$$r_i = \min\left(\frac{V_{bulk,i} \Delta t_i - 0.5 \Delta x}{V_{bulk,i} \Delta t_i}, 1\right) \quad 7.40$$

Where:

- $V_{bulk,i}$ is the bulk flow velocity corresponding to time step no. i
 Δt_i is actual time step 'i'

The summation in equation 4 is undertaken for all pipes at every time step. It follows by definition that the adaption is within the interval from 50% to 100%. During the simulation, the adaption is presented on screen in order that the user may determine whether to continue the simulation or amend the conditions to improve the results.

7.6.3 Forms of Particle Transport

7.6.3.1 Precipitation

Under conditions of low flow velocity, the forces acting on the particles will be such that most particulate matter will settle over a certain period to the bottom of the pipe and form a 'deposited bed'. This precipitation phase occurs when the flow velocity is below a minimum, V_{min} .

In this phase, the mass flow of sediment leaving a box can occur only as suspended transport. No bed load transport occurs out of the box and no new particles will be suspended in the bulk flow. The suspended mass 'settles' by being transformed into bed load mass, via the decay law in equation 7.41.

$$M_s = M_{s,0} e^{-K_s(t-t_0)} \quad 7.41$$

- Where: M_s is suspended mass at time t .
 $M_{s,0}$ is suspended mass at time t_0 .
 K_s is a user defined decay rate constant

If any particles in the box exist in the bed load phase, these will settle by being transformed into deposited mass, following a similar decay law:

$$M_b = M_{b,0} e^{-K_b(t-t_0)} \quad 7.42$$

Where: M_b is bed load mass at time t
 $M_{b,0}$ is bed load mass at time t_0
 K_b is a user defined decay rate constant

A large value for K_b will cause all bed load mass will settle in one time step.

7.6.3.2 Bed Load Transport

When the flow velocity increases, some entrained particles will not be lifted into the flow, but the forces exerted upon them will cause them to be transported by rolling and / or sliding over the surface of the material in the deposited bed. This phenomenon is called bed load transport. In this phase, bed load is the dominant type of transport.

Some suspended particles may exist at this time and these will precipitate in a manner described by a decay law. Existing particles in the bed load phase may be suspended, and existing particles in the deposited phase may be entrained into the bed load phase.

Bed load transport occurs, if the velocity, V , is in the interval:

$V_{\min} < V < V_{\max}$, and the ratio

$$\frac{U_f}{W_s} < 0.4 \quad 7.43$$

Where:

U_f is shear velocity
 W_s is particle fall velocity

The maximum possible mass flow of sediments leaving a box during the bed load phase is:

$$Q_{b,o} = C_{b,max} Q_w \quad 7.44$$

Where:

- $C_{b,max}$ is the maximum suspended concentration (May's formula)
 Q_w is mass flow of water

The actual mass removed from a box by bed load transport is limited by the amount of sediments present in the bed load and deposited phases. If particles exist in suspension, these will settle into the bed load phase following a decay law, and no new particles will be suspended. The maximum possible mass flow of sediments leaving a box is for the suspended phase:

$$Q_{s,o} = C_{s,max} Q_w \quad 7.45$$

Where:

- $C_{s,max}$ is the maximum suspended concentration (Mackes formula).

The *actual* mass removed from a box by suspension transport is limited by the amount of sediments present in the suspension and bed load phases.

7.6.3.3 Suspension Transport

As the velocity increases further, hydrodynamic lift and drag forces act upon the particles that constitute the deposited bed. The forces cause the particles to be lifted and held in suspension to be transported within the bulk flow. In this phase, the transportation occurs only in suspension. Existing particles in both the bed load and deposited phases may be suspended in the bulk flow. This phenomenon is called *suspension* transport.

Suspension transport occurs if the velocity, V , is in the interval:

$$V_{\min} < V < V_{\max}, \text{ and the ratio}$$

$$\frac{U_f}{W_s} > 0.4 \tag{7.46}$$

Since particles are transported exclusively in suspension, the maximum flow of sediment leaving the box is calculated using equation. (9). The *actual* flow is limited by the amount of sediments present in any phase.

7.6.3.4 Flushing

At a certain level of flow velocity all particles within the pipe will be transported in suspension due to the high level of forces acting upon the material. This last phenomenon is defined as *flushing*. In this case particles can exist only in the suspension phase, and no bed load or deposited sediment exist.

Flushing occurs, if the velocity:

$$V > V_{\max} \tag{7.47}$$

7.6.4 Transport Criteria

7.6.4.1 Basic Parameters - General

The transport criteria are velocity based for both the settlement phase and the flushing phase. If the velocity is below a configured limit, V_{\min} , settlement occurs. If the velocity is above a configured limit V_{\max} , flushing occurs.

If the velocity is between V_{\min} and V_{\max} , either suspended transport or simultaneous bed load and suspended transport occur. A criterion to distinguish between suspended transport and transport in suspension and in bed load is defined using the ratio of shear velocity to fall velocity.

7.6.4.2 Criterion for Bed Load / Suspension

The criterion to distinguish between sediment transported in suspension or in both bed load and suspension are as follows:

The fall velocity of a particle of a given specific gravity and size is calculated from equation 13, (May, 1993). The fall velocity W_s is given by:

$$W_s = \frac{\sqrt{9\nu^2 + d^2 g \times 10^{-9} (s-1) (0.03869 + 0.0248 d) - 3\nu}}{(0.11607 + 0.074405 d) \times 10^{-3}} \quad 7.48$$

Where:

- W_s is fall velocity (m/s).
- ν is kinematic viscosity (m²/s).
- d is a user defined particle size (□m).
- g is gravitational acceleration (9.81 m/s²).
- s is a user defined specific gravity of sediments (-).

The shear velocity is related to the velocity and the friction factor by:

$$U_f = \max \left(V \sqrt{\frac{f}{8}}, U_0 \right) \quad 7.49$$

Where:

- U_f is shear velocity.
- V is mean flow velocity of water.
- f is friction factor.
- U_0 is a configurable minimal shear velocity.

The friction factor is obtained from the hydraulic model database of pressure drop, velocity and levels at pipe ends:

$$f = \frac{d_p}{2L\rho V^2} \Delta p \quad 7.50$$

Where:

- L is the pipe length (m)
- Dp is the internal pipe diameter (m)
- ρ is the density of water (kg.m^{-3})
- Δp is the pressure drop due to friction (which may include single losses and calibration factors):

$$\Delta p = p_{up} - p_{dw} + \rho g(z_{up} - z_{dw}) \quad 7.51$$

Where:

- pup is the upstream pressure from hydraulic database (Pa abs)
- pdw is the downstream pressure from hydraulic database (Pa abs)
- zup is the upstream level from hydraulic database (m)
- zdw is the downstream level from hydraulic database (m)

The criterion for sediments transport in the model as exclusively suspension is:

$$\frac{U_f}{W_s} > 0.4 \quad 7.52$$

If the ratio is below 0.4, the transport will primarily take place as bed load transport, but suspension exists too.

7.6.4.3 Maximum Suspension Transport

Macke, 1983, produced the following equation that describes the maximum limit of sediments that can be transported in suspension phase in pipes. The formula is dimensionless.

$$C_{s,max} = \frac{f^3 V_L^5 K_u}{30.4 (s-1) W_s^{1.5} A \rho} \quad 7.53$$

Where:

$C_{s,max}$ is maximum sediment concentration (kg sediments/kg water).

V_L is a configured limiting flow velocity without deposition-input (m/s).

A is cross sectional area of the part of pipe without deposited sediments (m²).

K_u is a unit conversion factor mg.l⁻¹ to kg.m⁻³

7.6.4.4.1 Maximum Bed Load Transport

The maximum bed load transport is calculated using May's equation, developed from experimental data describing the relationship between volumetric sediment concentration and the flow velocity at the limit of deposition:

$$C_{b,max} = 3.03 \cdot 10^{-2} \left(\frac{D^2}{A} \right) \left(\frac{d}{D} \right)^{0.6} \left(1 - \frac{V_t}{V} \right)^4 \left(\frac{V^2}{g(s-1)D} \right)^{1.5} \frac{K_u}{\rho} \quad 7.54$$

Where the threshold velocity is given by:

$$V_t = 0.125 \sqrt{g(s-1)d} \left(\frac{D}{d} \right)^{0.47} \quad 7.55$$

and $C_{b,max}$ is maximum volumetric sediments concentration

The formula assumes that the sediments are transported as bed load, i.e. that the limit $U_f / W_s < 0.4$ is not exceeded.

7.6.5 Model Output

The model was used to simulate a hypothetical scenario on a single leakage control zone within the study network.

The following plots, Figures 7.181 to 7.189 clearly demonstrate that the model is valid (but not calibrated).

Calibration of this type of model would have been impossible a few years ago. However, with online water quality instruments turbidity measurement would be an excellent surrogate for suspended particle flow (*Boxall, 2000*). Measurements that are more precise could be made using particle size analysis but this technology remains expensive for multiple site application.

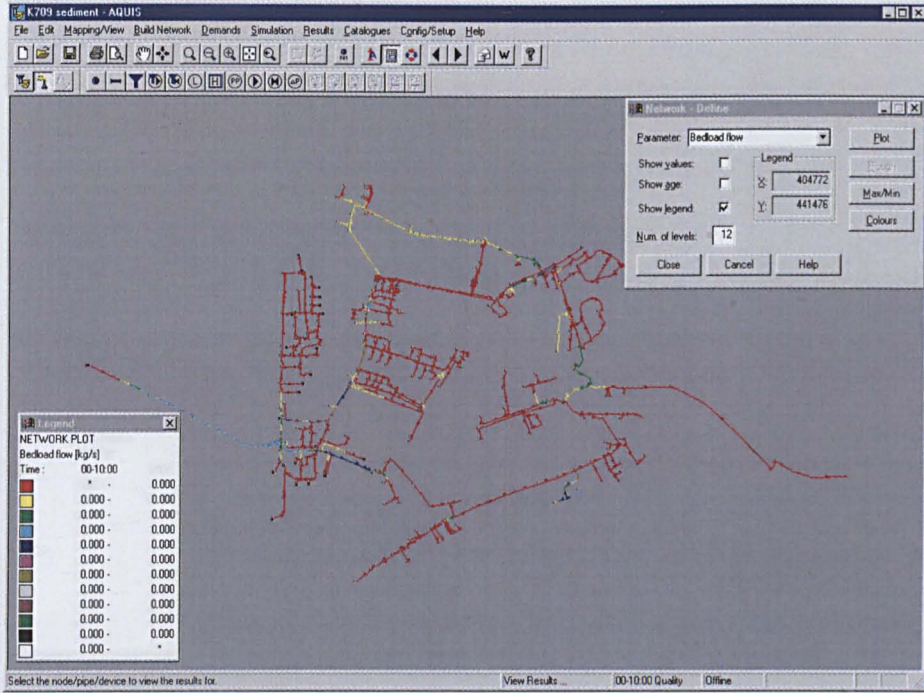


Figure 7.181 Bedload flow in LCZ K709

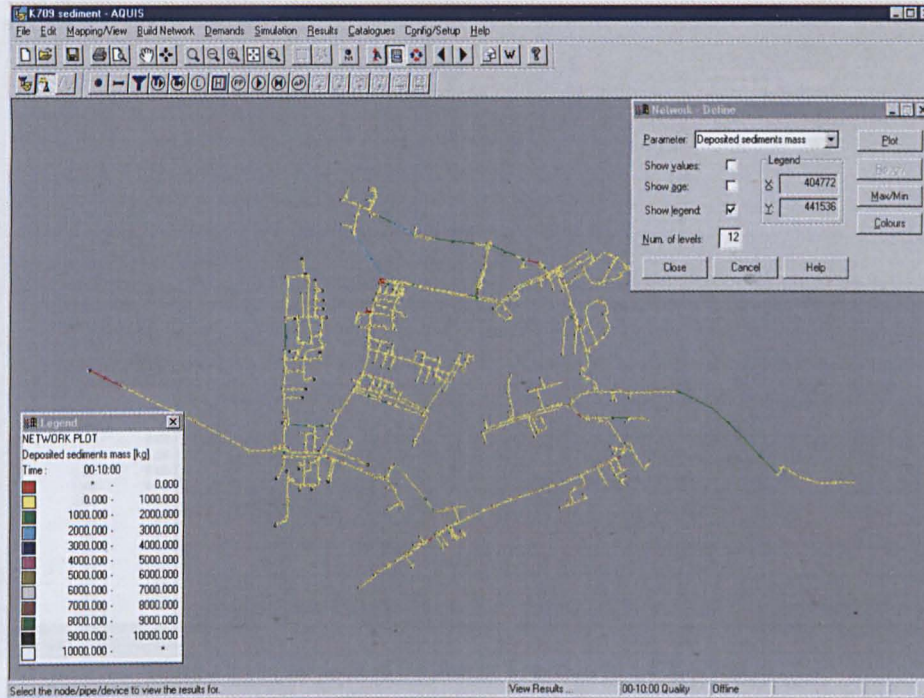


Figure 7.182 Deposited Sediment Mass in LCZ K709

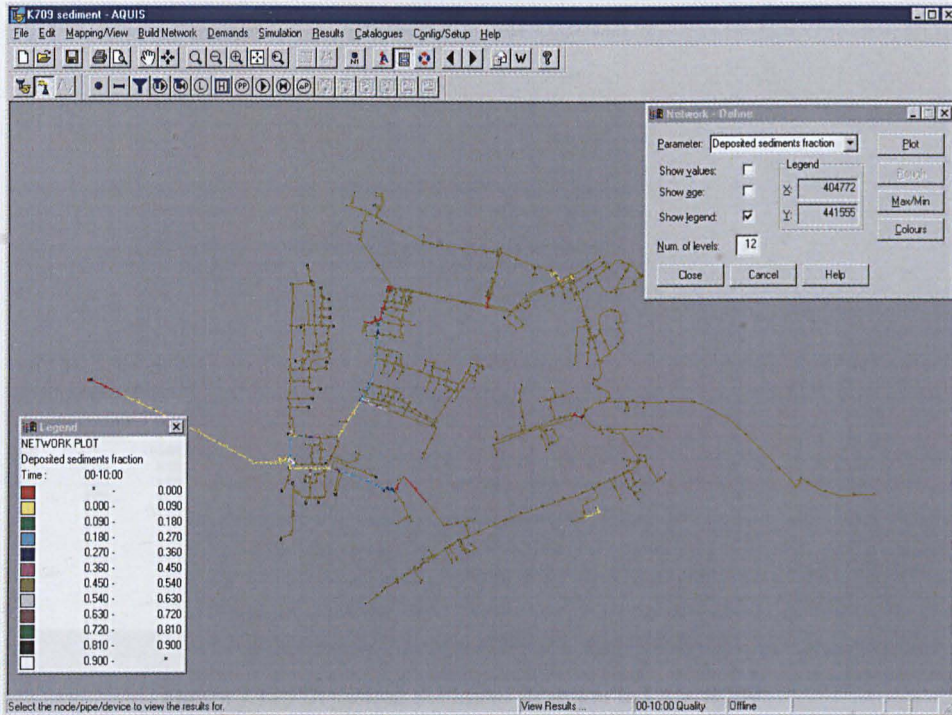


Figure 7.183 Deposited Sediment Fraction in LCZ K709

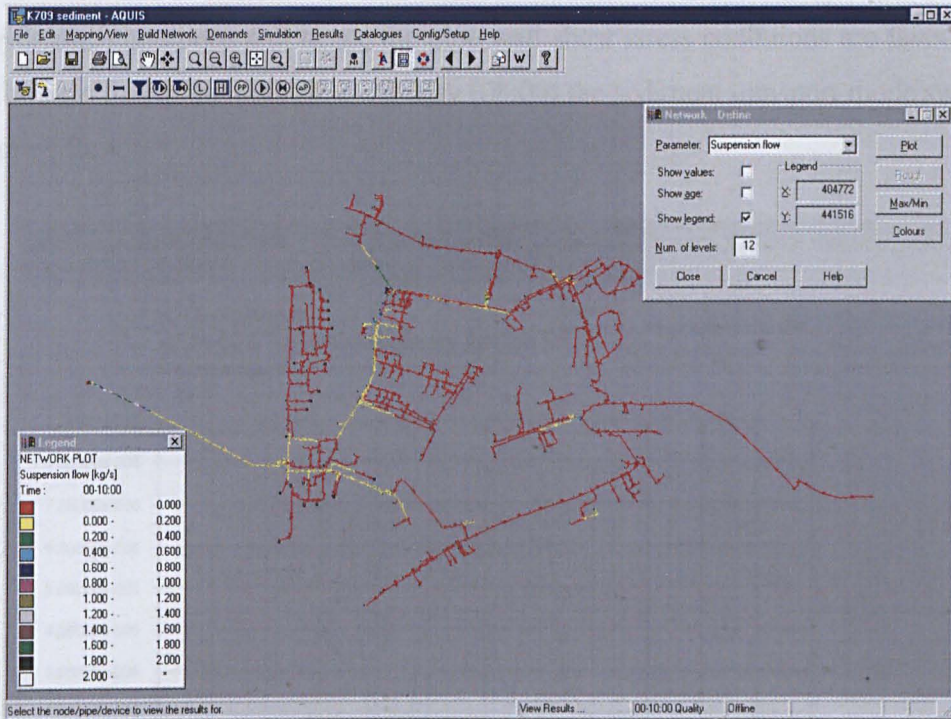


Figure 7.184 Deposited Sediment Fraction in LCZ K709



Figure 7.185 Deposited Sediment Fraction in LCZ K709

Figure 7.186 shows a time series of the mass (kg) of bedload material. The Peak bedload flow is when the hydraulic conditions and hence the pipe wall shear stress conditions are favourable. It can clearly be seen that at the peak flow velocity (08:00) the sediment transport mode switches to suspended mass flow.

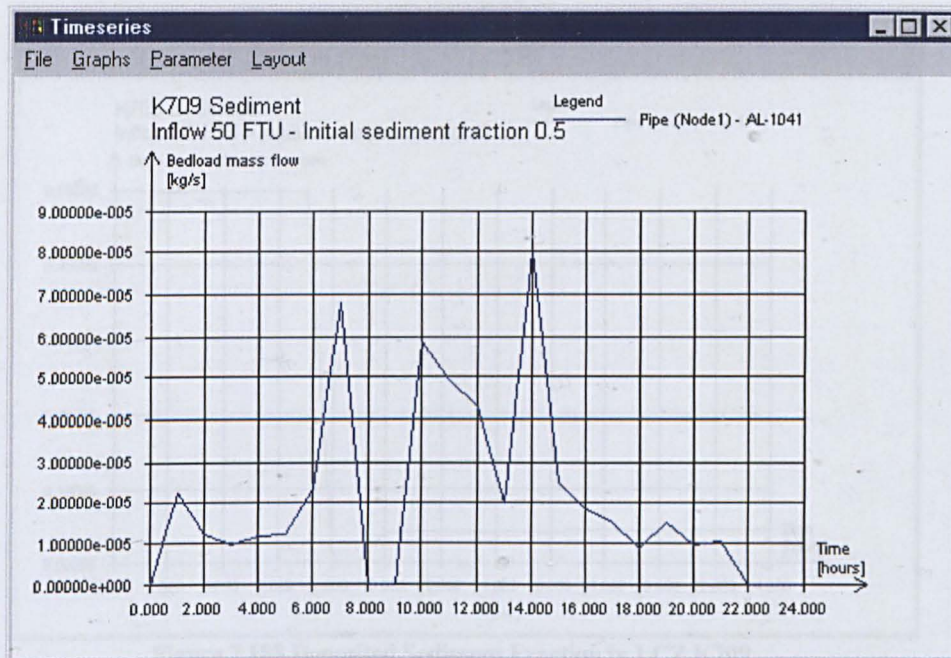


Figure 7.186 Bedload Mass Flow in LCZ K709

This is confirmed by the time series of suspended mass flow in Figure 7.187

Figure 7.189 Shows the same sudden change but in terms of the deposited sediment mass.

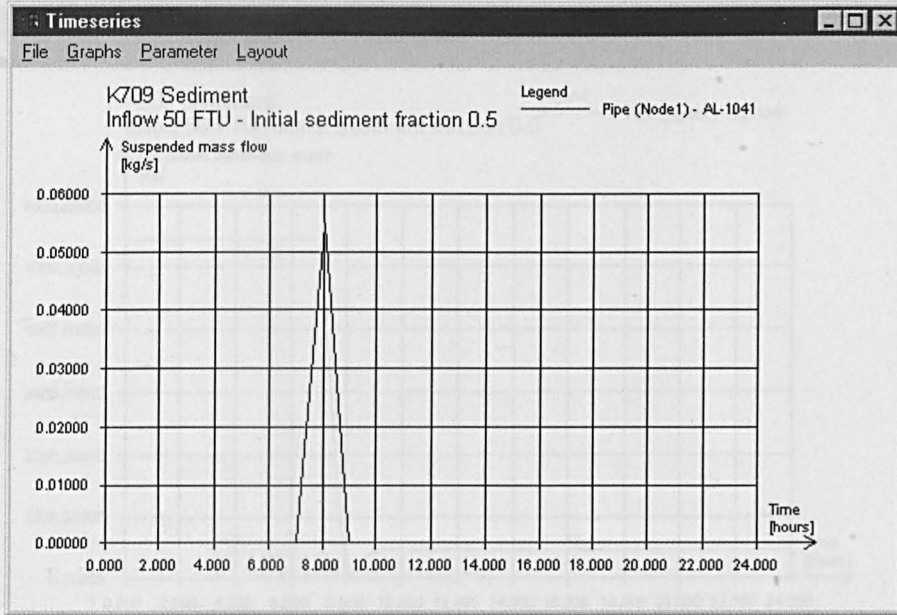


Figure 7.187 Suspended Mass Flow in LCZ K709

Figure 7.188 highlights the sudden change in deposited sediment fraction (fraction of the cross sectional area of the pipe) that occurs because of the particles being eroded into the bulk flow.

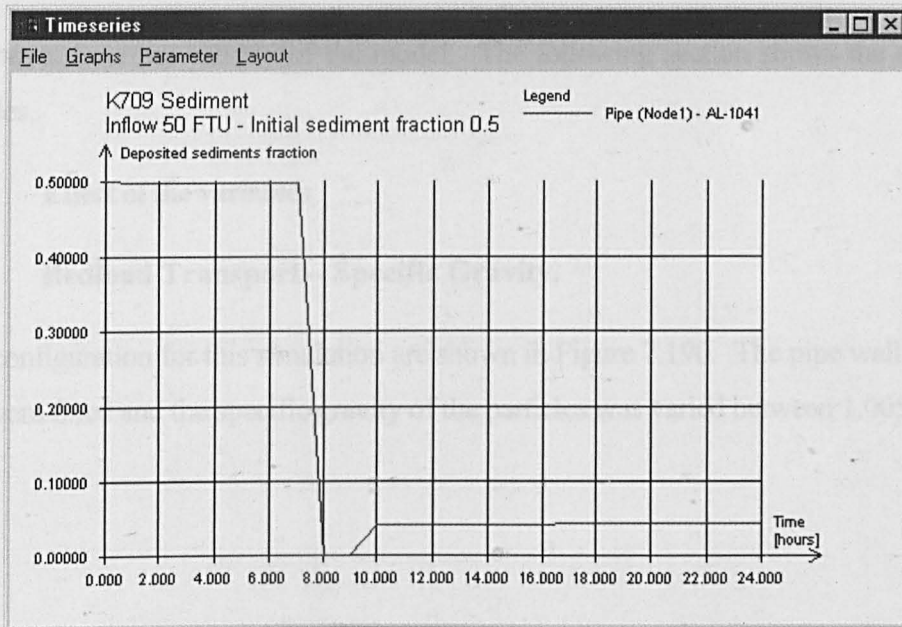


Figure 7.188 Deposited Sediment Fraction in LCZ K709

Figure 7.189 Shows the same sudden change but in terms of the deposited sediment mass.

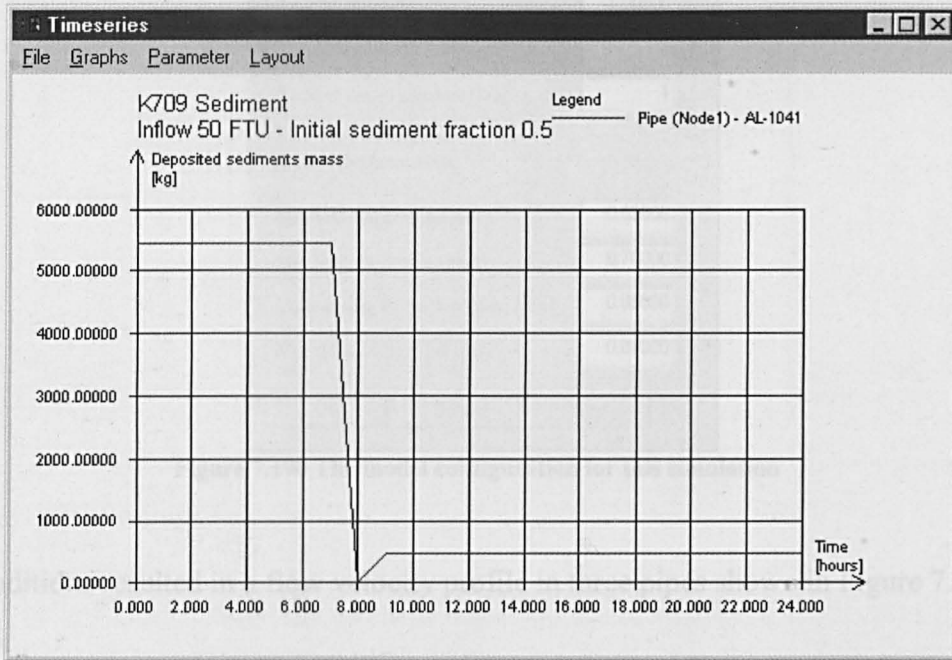


Figure 7.189 Deposited Sediment Mass in LCZ K709

In this case, all the deposited mass in this particular pipe was eroded after which new deposits started to accumulate.

The plots above show the validity of the model. The following section shows the effect of the main variables.

7.6.6 Effect of the variables

7.6.6.1 Bedload Transport – Specific Gravity.

The model configuration for this simulation are shown in Figure 7.190. The pipe wall shear stress conditions were fixed and the specific gravity of the particles was varied between 1.005 and 1.009.

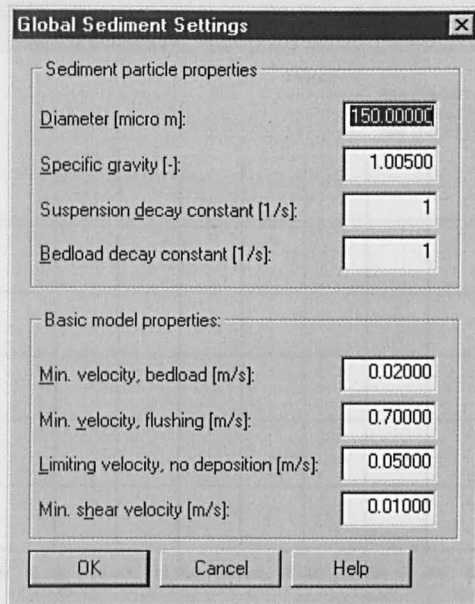


Figure 7.190 The model configuration for this simulation

These conditions resulted in a flow velocity profile in three pipes shown in Figure 7.191

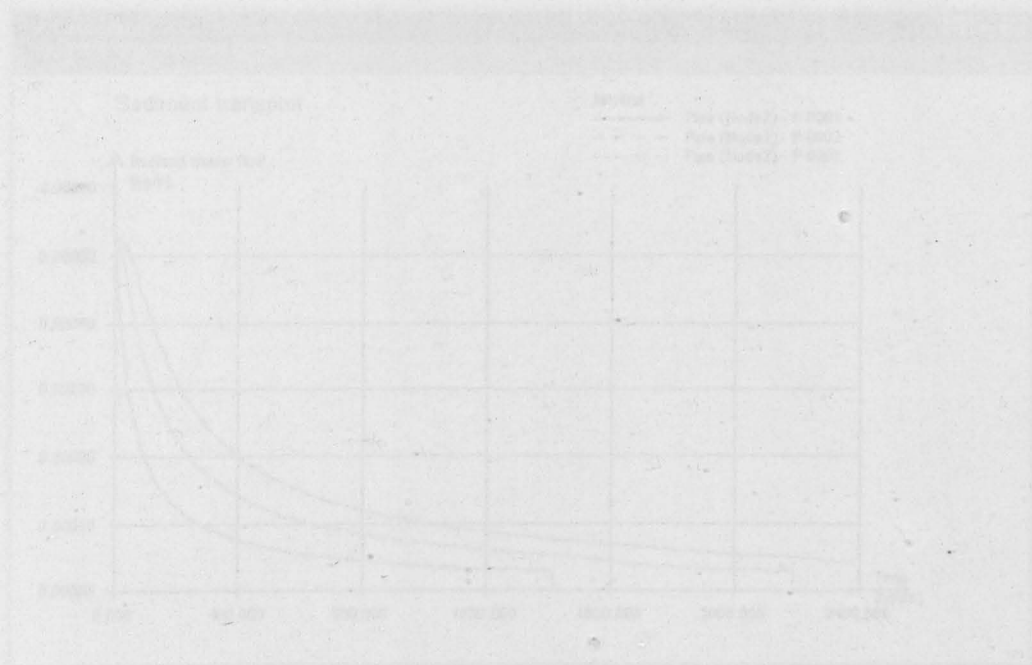


Figure 7.191 Velocity profiles for a particle specific gravity of 1.005

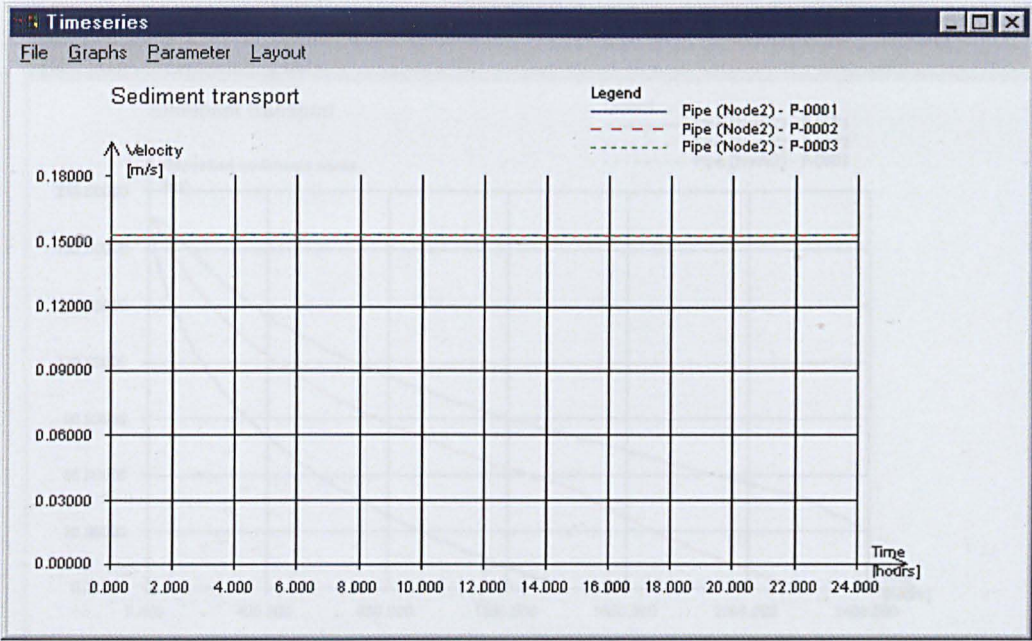


Figure 7.191 Flow velocity profile in three pipes

The effects on bedload are shown in terms of deposited sediment mass and bedload mass flow in Figure 7.192 and 7.193 for a particle specific gravity of 1.005



Figure 7.192 Bedload mass flow for a particle specific gravity of 1.005

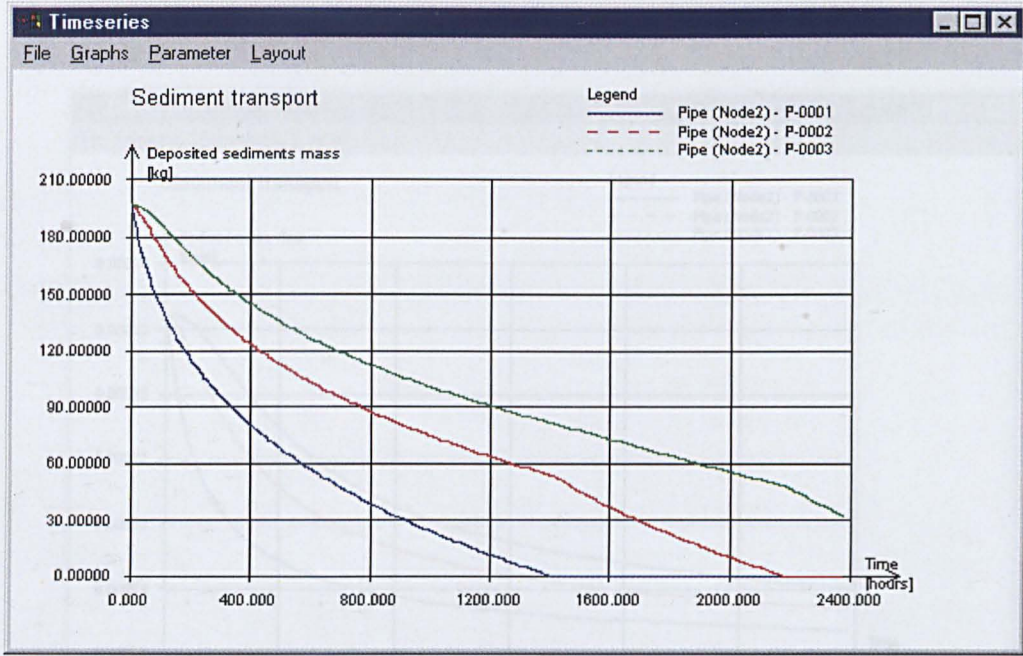
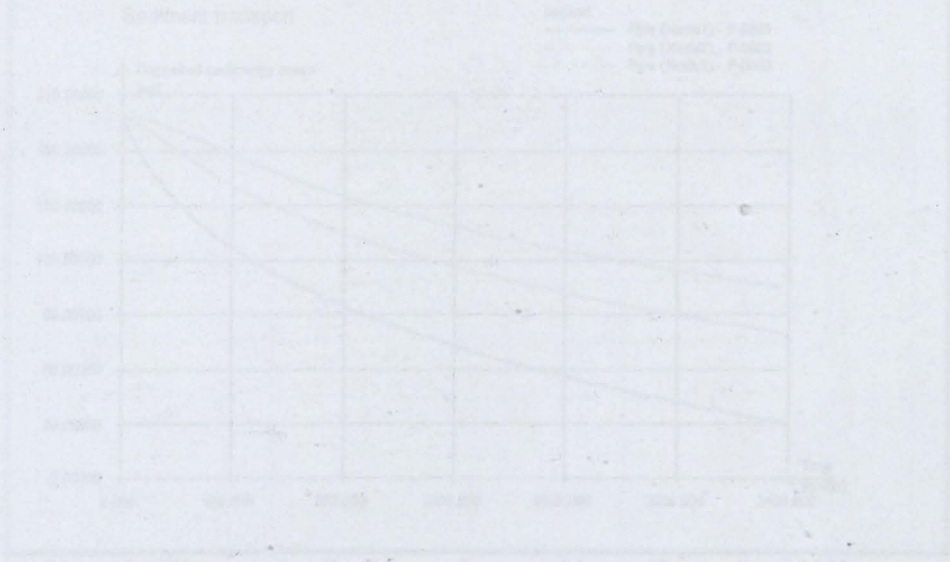


Figure 7.193 Deposited sediment mass for a particle specific gravity of 1.005

Under the same conditions, the specific gravity of the particles was changed from 1.005 through 1.009 in steps of 0.001. The final value of 1.009 shows the upper limit of the effect of changing the specific gravity in Figures 7.194 and 7.195



7.6.6.2 Bedload Transport – Particle Size

Under the
particles
deposited

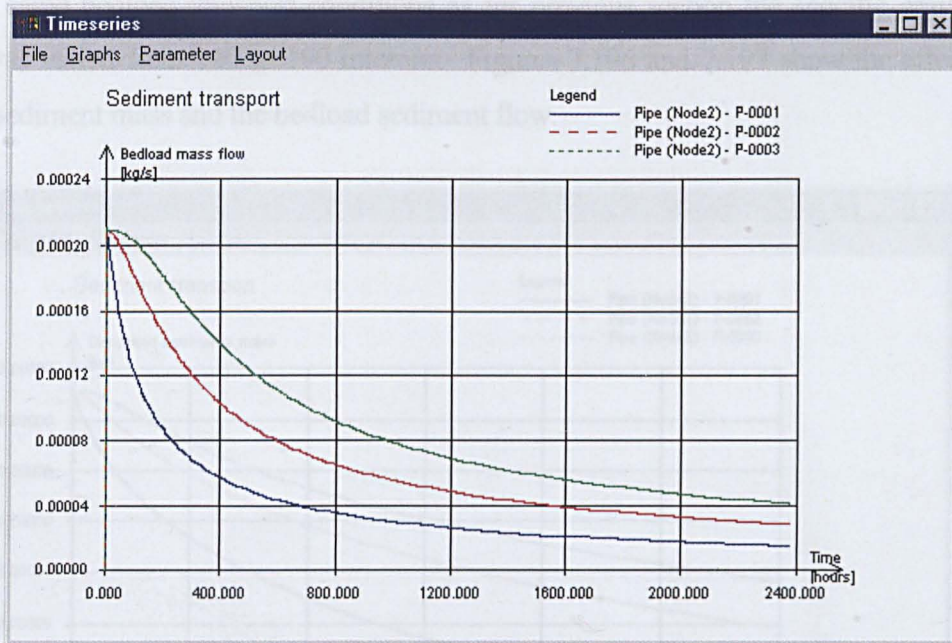


Figure 7.194 Bedload mass flow for a particle specific gravity of 1.009

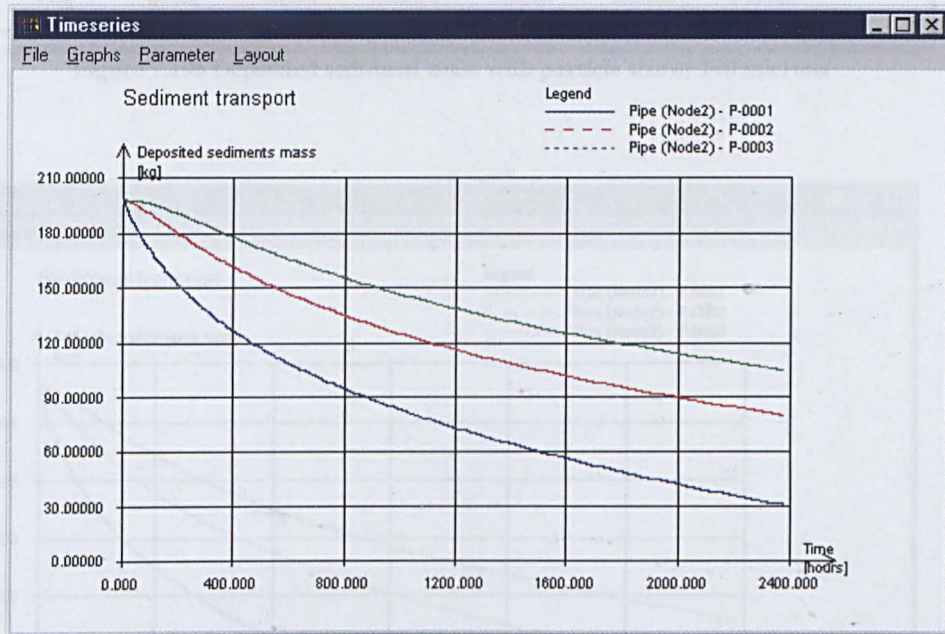


Figure 7.195 Deposited sediment mass for a particle specific gravity of 1.009

Figure 7.197 Deposited sediment mass with particle size of 150 micrometers

7.6.6.2 Bedload Transport – Particle Size

Under the same bedload transport conditions as for previous section the specific gravity of the particles was varied from 140 to 190 microns. Figures 7.196 and 7.197 show the effects on the deposited sediment mass and the bedload sediment flow.



Figure 7.196 Deposited sediment mass with particle size of 140 microns

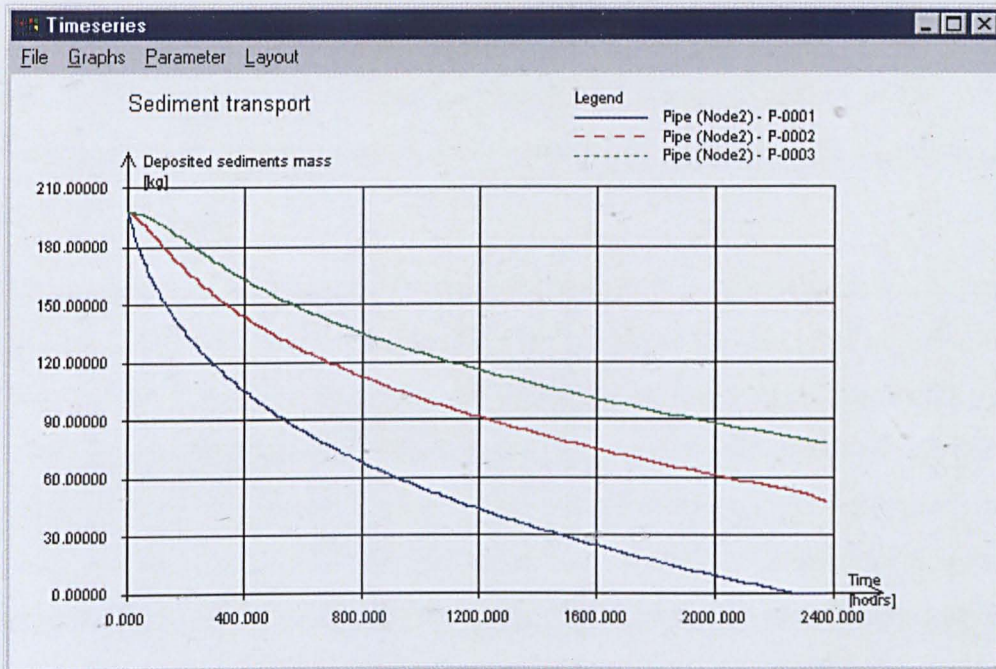


Figure 7.197 Deposited sediment mass with particle size of 190 microns

7.6.7 Summary of Sediment model

The model that has been developed has been set up in a flexible way to include aspects of settlement of suspended particles (precipitation) - no bed load transport, transport in suspension and by bed load movement, transport in suspension and flushing (scouring). The thesis has presented details of the bed load model but further work is required to obtain network data against which the model may be calibrated/validated. However, in its current form the model may be used in predictive mode to examine suspended mass flow, bed load transport and total sediment load. The model is therefore able to simulate the total mass of sediment in any pipe and of the change in cross sectional area due to the deposited sediment.

The advantage from an operational viewpoint is that the model allows the prediction of those pipes that will remain free of sediment and those pipes where the risk of discoloration due to sediment movement may be high.

Chapter 8 - Online Monitoring and Modelling

8.1 Background

There are many uncertainties to contend with when dealing with a dynamic system such as a distribution network. Even carefully planned work can produce unexpected hydraulic and water quality effects. This is because there is a lack of understanding about the prevailing hydraulic and water quality characteristics of the network at the time the work is being undertaken. The result is that simple operations such as changing the status of a valve can lead to, for example, discolouration of the water or pressure problems.

The day-to-day operation of a distribution network depends, largely, on a monitor and react philosophy. When something goes wrong, the performance monitors, usually consumers, inform the Water Company responsible. Then, operational staff investigates to determine the reason for the service failures or the customer complaints. This type of reactive management is neither effective nor efficient. By the time the company is made aware of a problem, customers are already affected and the company incurs standards of service failures that may attract legal action and, if serious enough a breach of regulations, loss of operating licence.

To demonstrate that a step change in the way distribution networks are managed is possible, the software described in the previous chapters of this thesis has been developed into an on-line network management toolkit. The toolkit is designed to provide a much greater understanding of real time distribution network performance characteristics and facilitate proactive and, ultimately, automatic control of certain essential dynamic network elements such as valves, pumps, and service reservoirs.

The system provides the operator with hydraulic and water quality information in near real time. Having such timely information permits the asset managers the luxury of knowing an event has occurred, usually before the customers are affected. In many cases distribution staff can be mobilised with prior knowledge of what they are going to deal with armed with an effective action plan. Consequently, the necessary resources are utilised efficiently and any impact on customers, the assets, and the environment are minimised. The models developed for this research were applied to the study network and used to support the management of the water supply system for the study area.

The work has clearly demonstrated the benefits of a real time modelling approach to network monitoring and management. It has also highlighted a number of issues, which need to be addressed if online, and, or, real time systems for control of distribution networks are to be viable and the potential benefits of using this technology are to be fully realised.

8.2 The Benefits of Online Modelling

Daily distribution network management and operation is mostly reactive in nature. This approach is time consuming, inefficient, and resource intensive.

For example, when a water main fails and a leak occurs, it could be a long time before the network manager is made aware from information provided by someone that actually sees or hears the water escaping. Analysis of flow measurements could highlight the presence of a burst but the frequency of data acquisition and analysis may be such that the leak may run for a month or more before being detected. Even then, analysis of flow measurement alone may not detect the presence of a leak, (*Mounce, 2002*).

Online modelling provides near real time analysis of hydraulic performance and the system can usually detect a burst main immediately it happens.

South West Water suffered a pollution incident at the Camelford Water Treatment Plant. Aluminium Sulphate was accidentally introduced into the final water tank at the works from where it entered the distribution network causing health related problems to many customers. It appears also; that some customers who were not affected claimed that they were, probably out of fear and, in some cases, the hope of compensation. The company is still in court and there is an ongoing public inquiry. The cost to the Water Company has been very high in terms of both cash and credibility.

If the company had water quality monitors installed in their networks and had the benefit of an online modelling system, they would have detected the pollution before customers were affected. This would have allowed them to proactively manage the isolation and removal of the polluting material from the network. They would have minimised the number of customers affected and been able to identify those who *were* affected with a high degree of certainty. This would have saved a great deal of money and embarrassment.

When planning distribution R&M work packages, it is of great benefit to be able to model the effects of the operations *before* they are undertaken. Rehabilitation, re-valving, pipe repairs and many other activities benefit from knowing exactly how the work will affect the network hydraulic and water quality characteristics. Careful planning using real time knowledge of the network characteristics enables the network manager to identify the best methodology for approaching the work. Impact of the effects of the work on customers and assets could be minimised and customers affected would be accurately identified allowing prior notification to be given.

As Nitrate residuals increase in source waters in the UK, the need for blending within service reservoirs and the distribution system itself are becoming a regular requirement. Water quality modelling permits the operator to calculate what proportion of which supply needs to be mixed in order to maintain water quality standards of service.

The above are just a few examples of the benefits of real time modelling and there are many more. Taken fully, a real time monitoring and modelling system can provide hydraulic and water quality information from raw water sources to the customers tap. Having this knowledge allows the company to protect resources, water treatment plants, distribution assets and customers from the effects of incidents. It will detect bursts and pollution events, predict where polluted or discoloured / turbid water will travel with time, facilitate emergency and contingency planning and allow informed decision making for network management.

The oil and gas utilities have benefited from the use of real time monitoring and modelling technologies for many years, and have developed closed loop control systems to automatically manage many of their network operations thereby significantly reducing operating costs. This chapter clearly demonstrates that the technology is transferable to water pipe networks. Examples of real events detected and managed using the study network online system are detailed in Appendix A, and examples of pollution incident management are presented in section 8.10.

8.3 Online System Development

8.3.1 Model Development

Prior to the development of this online system, distribution network modelling was predominantly a desktop exercise using historic network data that represented a specific time window of network

characteristics, usually a twenty-four hour period or less. Figure 8.1 Shows the workflow associated with the traditional desktop approach.

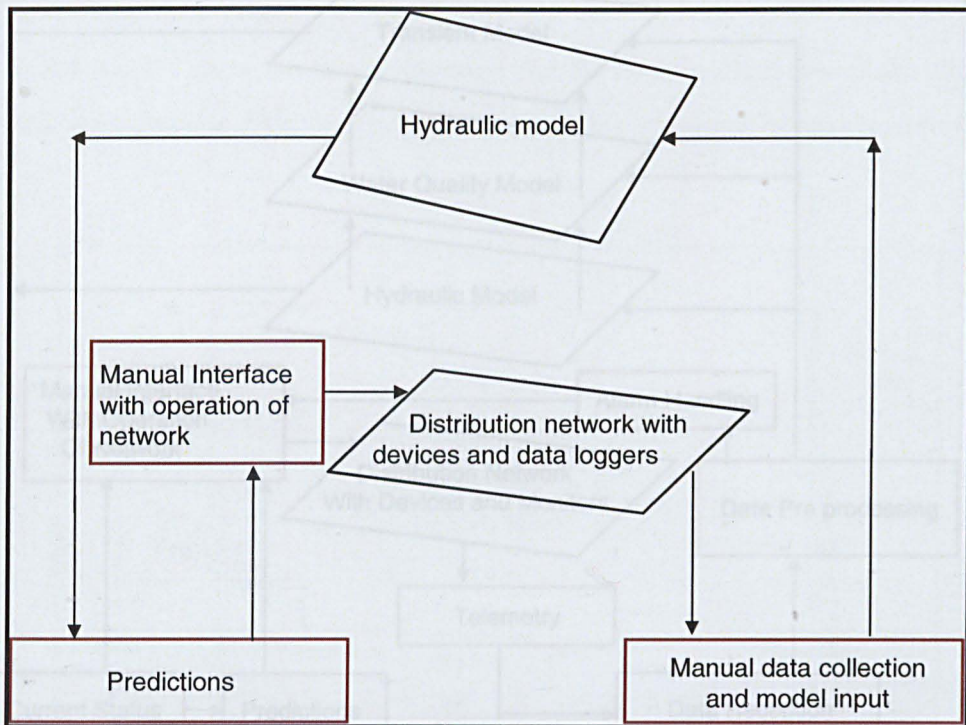


Figure 8.1 Data flow for traditional desktop approach to modelling

Integrating the hydraulic, water quality and transient models achieved the first stepped improvement and automatic network data capture was then added. The data gathering consisted of a single reading from each measurement site to capture the current network status at these points. This data was used to simulate the network characteristics at every other point, thereby obtaining an overall picture of the characteristics of the entire network.

Figure 8.2 highlights the difference this made to data handling for modelling purposes when compared to Figure 8.1

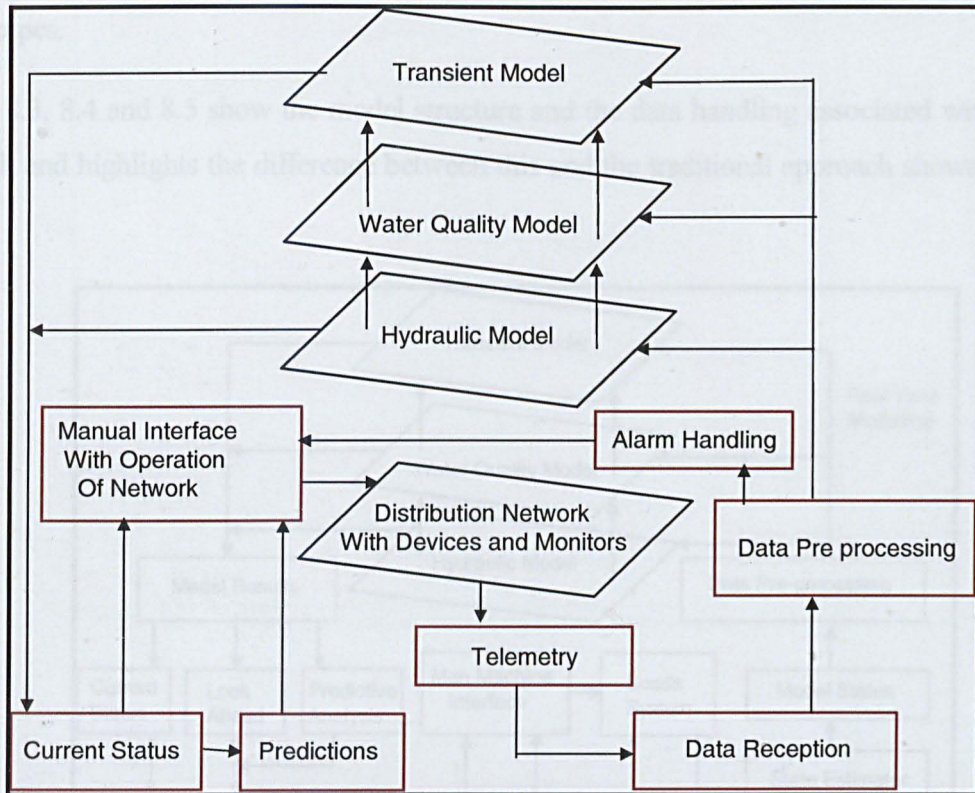


Figure 8.2 Data flow for offline modelling via new approach

One of the most important features of the improved system was the ability to automatically gather network data at any time from a central location. Effort was still required however to manage the system and input data into the models. Much of the data pre-processing was manual. It was found that, because single readings were taken from the measurement points that the hydraulic model sometimes failed to converge on a solution. The cause was found to be inadvertently capturing one or more transient values that were not representative of the true state at the point of measurement or in the network in general. This problem was overcome by introducing automatic data pre-processing that smoothed the data capturing transient measurements replacing them with other, average values, or using the last known good measured value.

The final stage of development resulted in complete integration of models, data pre-processing and model input. Alarm handling was developed and added to provide early warning of system events and to make running of specific simulations automatic under certain circumstances for example, when the field instruments detected polluting material. Alarm handling was made very flexible and programmable in order that disparate system events could be associated with each other to produce a 'complex' alarm. For example, if a pressure fluctuation in one part of the system

corresponded with the change of direction of flow in another part of the network the probable cause would be a burst or discoloured water detection might be associated with flow reversals in certain pipes.

Figures 8.3, 8.4 and 8.5 show the model structure and the data handling associated with the new approach and highlights the difference between this and the traditional approach shown in Figure 8.1.

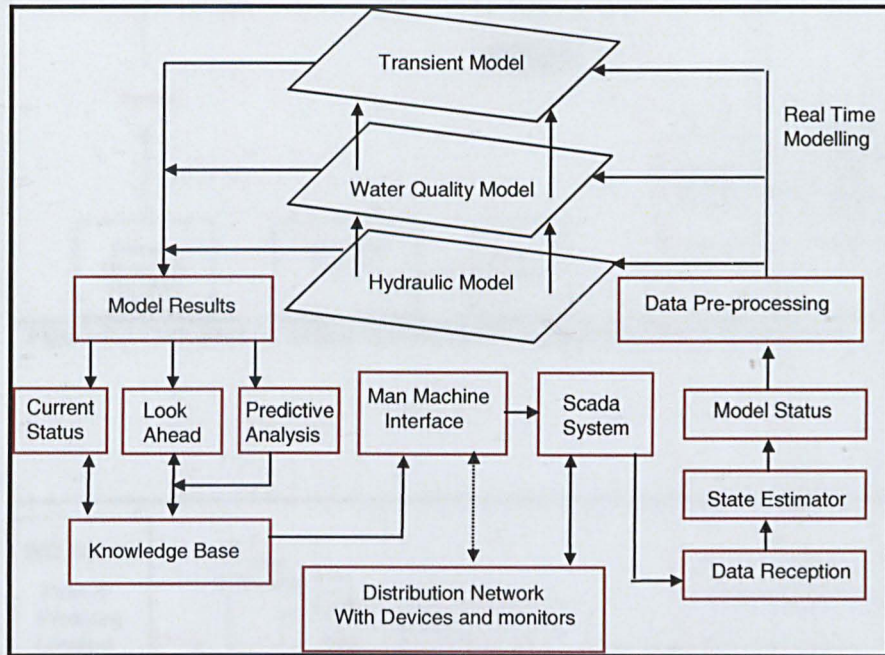


Figure 8.3 Data flow for online modelling in new approach

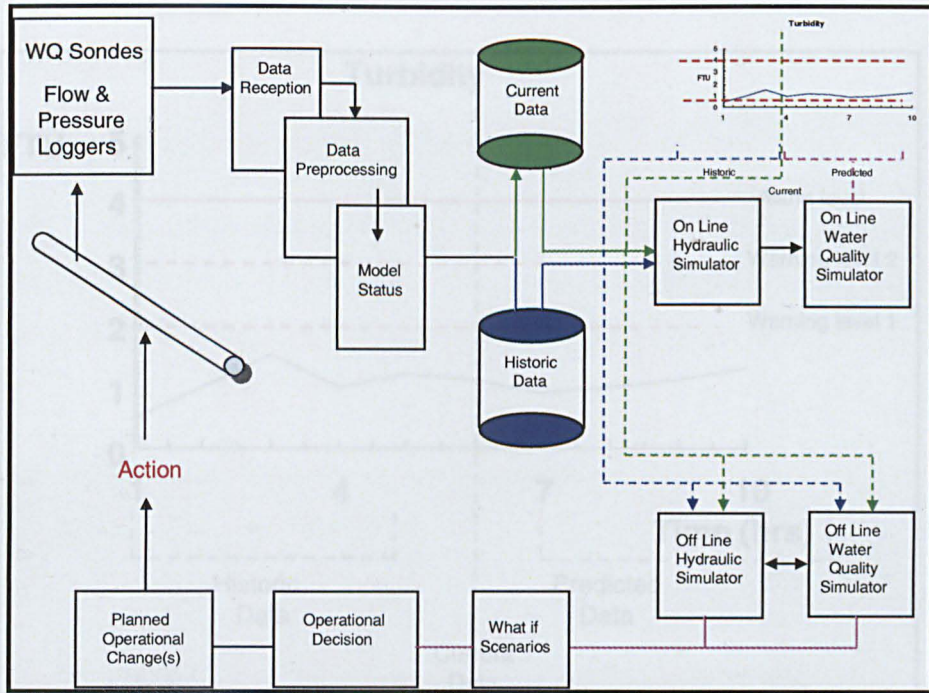


Figure 8.4 Data flow for online modelling in new approach normal operation

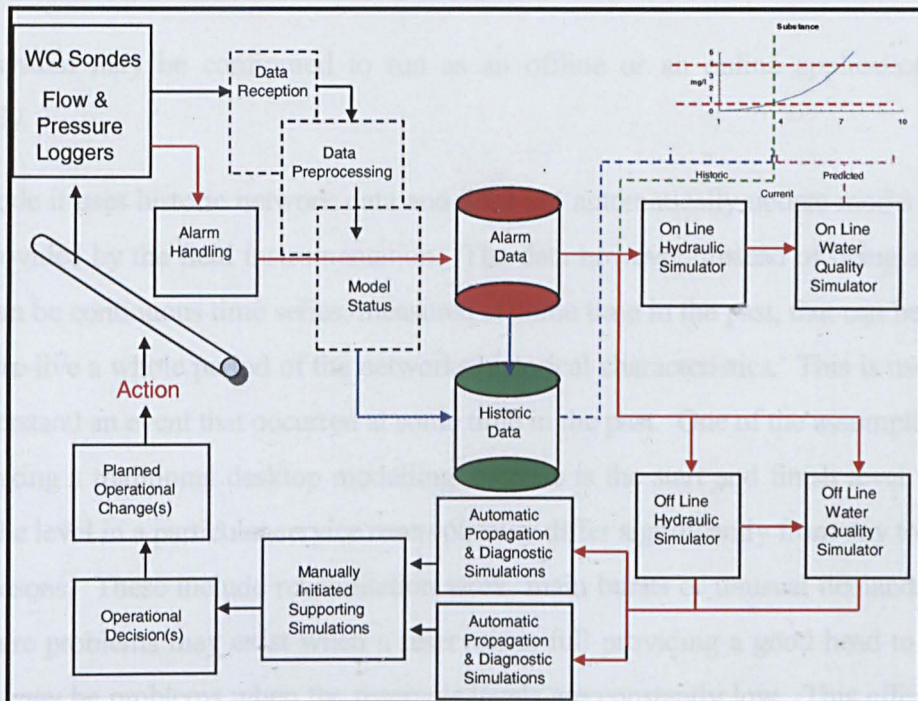


Figure 8.5 Data flow for online modelling in new approach alarm condition

Figure 8.6 highlights the detail of the measurement currently being processed, in this case turbidity, clearly showing the historic, current future data split and the alarm and warning levels for this parameter.

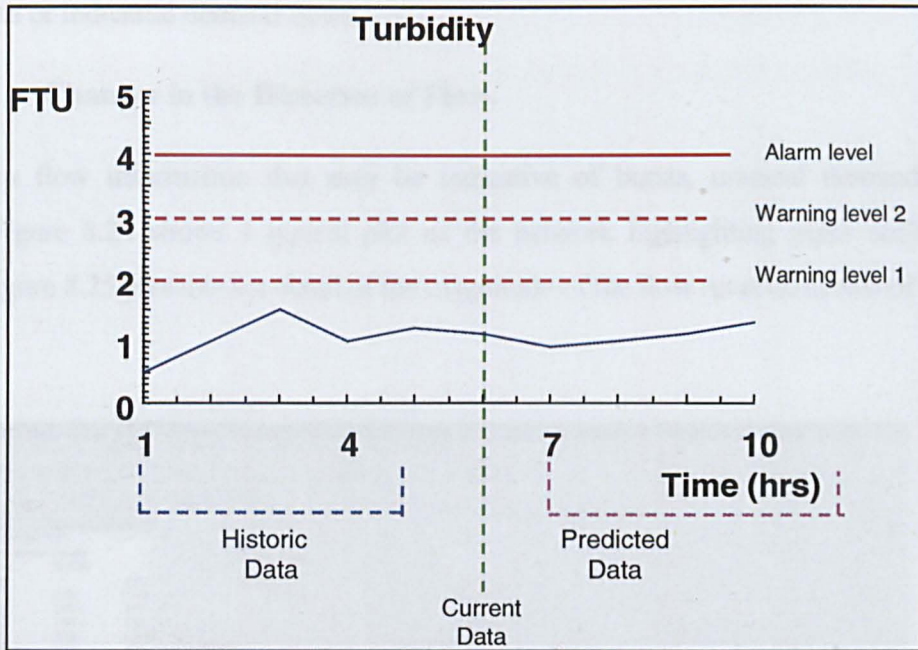


Figure 8.6 Detail of the turbidity data currently being processed

8.3.2 Mode of Operation

The online system may be configured to run as an offline or an online application or both simultaneously.

In off line mode it uses historic network data and does not automatically update model boundary conditions provided by the field instrumentation. The data however, instead of being a snapshot on one day can be continuous time series, measured at some time in the past, that can be read into the model to re-live a whole period of the networks historical characteristics. This is useful when trying to understand an event that occurred at some time in the past. One of the assumptions made when undertaking a traditional desktop modelling exercise is the start and finish level in service reservoirs. The level in a particular service reservoir may differ significantly from day to day for a number of reasons. These include rehabilitation work, main bursts or unusual demands. Whilst no low-pressure problems may exist when a reservoir is full providing a good head to drive the system, there may be problems when the reservoir levels are constantly low. This effect may be magnified if there are multiple reservoirs supplying a network.

In online mode, all the modelling functionality is available, but the system additionally automatically gathers, pre-processes and uses data from the instrumentation continually updating the user screen with the latest network characteristics and, based on 'normal' operational patterns,

The model is used in offline mode to determine if the network can support extra development such as new houses or industrial demand flows.

8.4.3.3.1.2 Changes in the Direction of Flow.

Provide extra flow information that may be indicative of bursts, unusual demands or zone breaches. Figure 8.24 shows a typical plot of the network highlighting pipes suffering flow reversals. Figure 8.25 presents the detail of the magnitude of the flow reversal in two of the pipes.

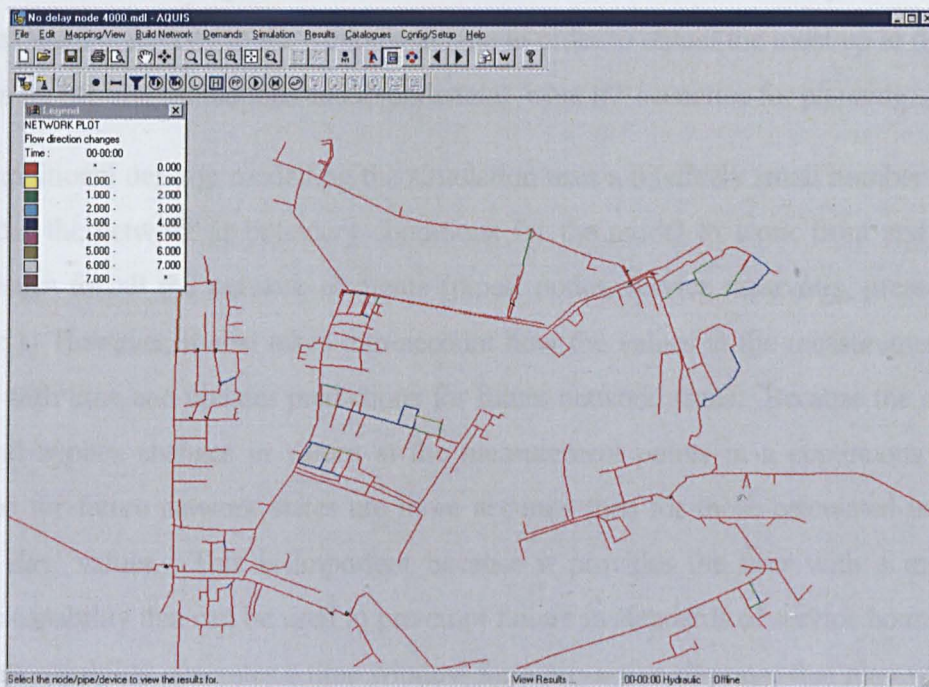


Figure 8.24 Flow reversals

the predicted future network states. Online modelling uses actual reservoir levels where measured, or computed levels where not measured, over extended period simulations thereby taking into account a much wider variation of network characteristics during a simulation period.

The online process is automatic and continuous, controlled by a timer that determines how often a simulation is initiated. The timer always initiates a hydraulic simulation but water quality, sedimentation and diagnostic simulations are optional. The operator's screen is automatically updated each time a simulation cycle is completed.

When one machine is configured to run online and the other offline, data may be transferred from the online machine directly to the offline machine in order to obtain the most up to date boundary conditions for off-line simulations and to undertake 'what if?' scenarios for planning.

As with traditional desktop modelling the simulation uses a relatively small number of measured values from the network as boundary conditions for the model to work from and predicts the characteristics for all the network elements (pipes, nodes, service reservoirs, pressure reducing valves etc.). However, it also takes into account how the values at the measurement points are changing with time and updates predictions for future network status. Because the online model detects and applies changes in values at the measurement points in a continuous fashion, the predictions for future network states are more accurate than for those calculated using a set of 'standard day' values. This is important because it provides the user with a more accurate predictive capability that can be used to pre-empt failure in standards of service hours before they occur. This capability generates a time window for a pro-active response that may well allow the network operators to prevent the event from having an impact at all or, at the least, minimise the effects of the event on the network.

Figure 8.7 depicts an online screen shot of the showing the historic, current and future (predicted) flow at a particular node in the study network. The screen background displays the pressure profile across the entire network.

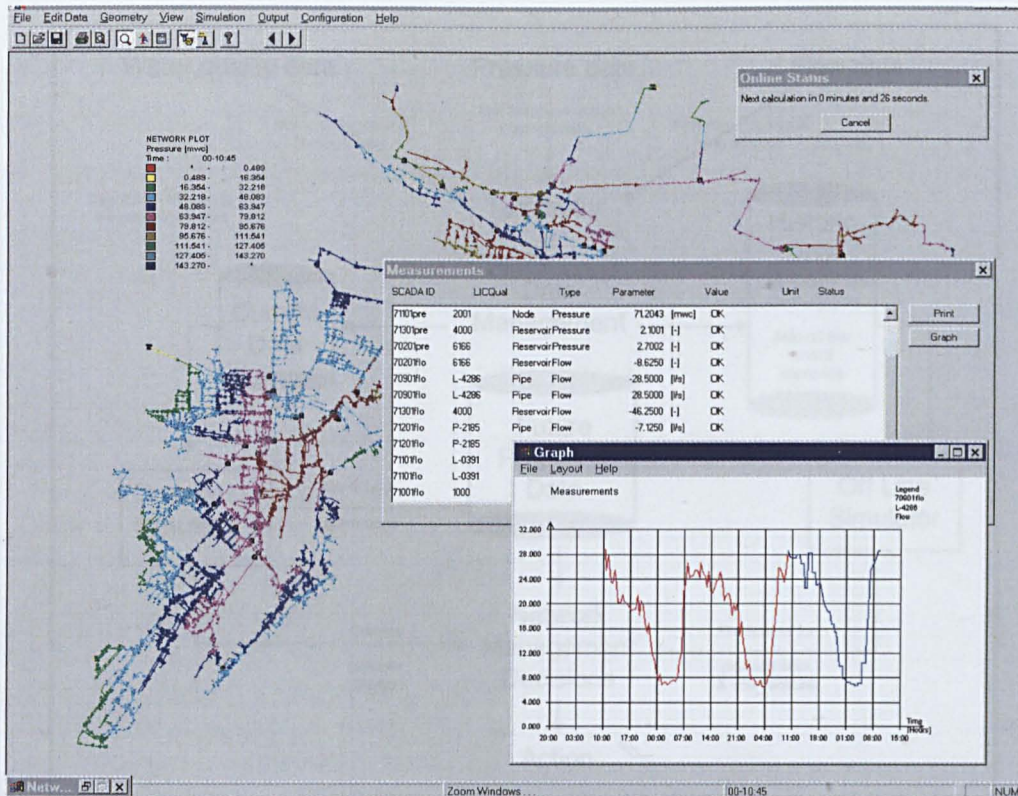


Figure 8.7 Online screen showing a time series of historic, current and future predicted pressure

Having this capability also allows the detection and location of mains bursts. The author proposed further development of the study network model in order to concentrate on this aspect of online modelling, (Machell, 1997). The proposal was incorporated into an EPSRC WITE Framework project and the development and the findings were reported in a PhD Thesis, (Mounce, 2002)

Figure 8.8 depicts how the online model manages data when network characteristics are within normal operating parameters.

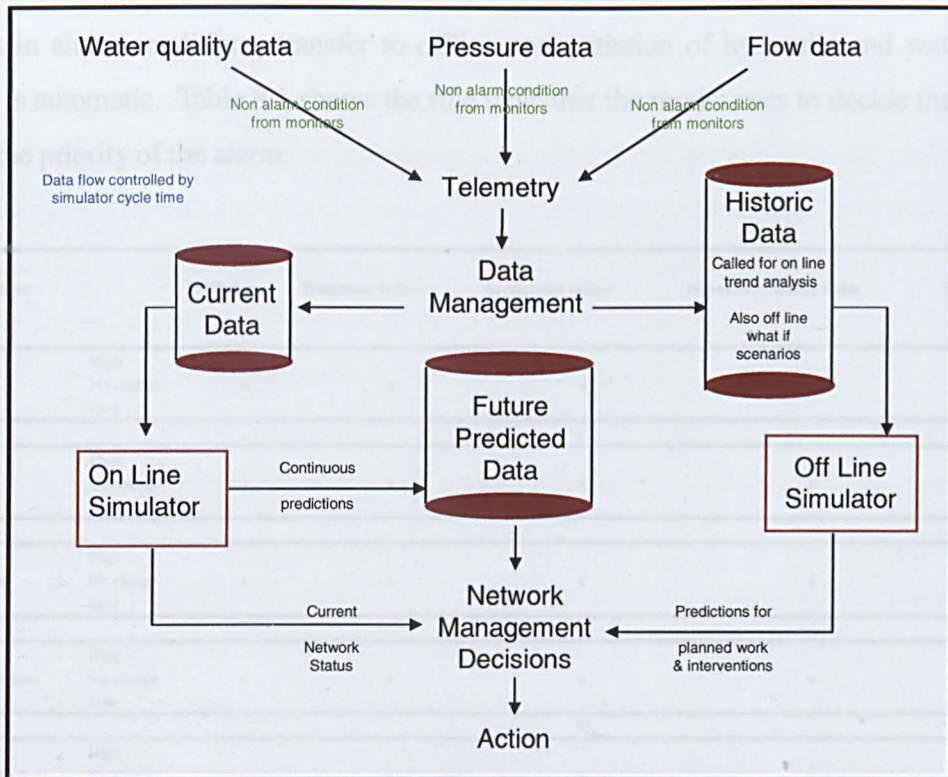


Figure 8.8 Online model data management under normal operating conditions

Figure 8.9 shows how this data management changes when an alarm condition is active.

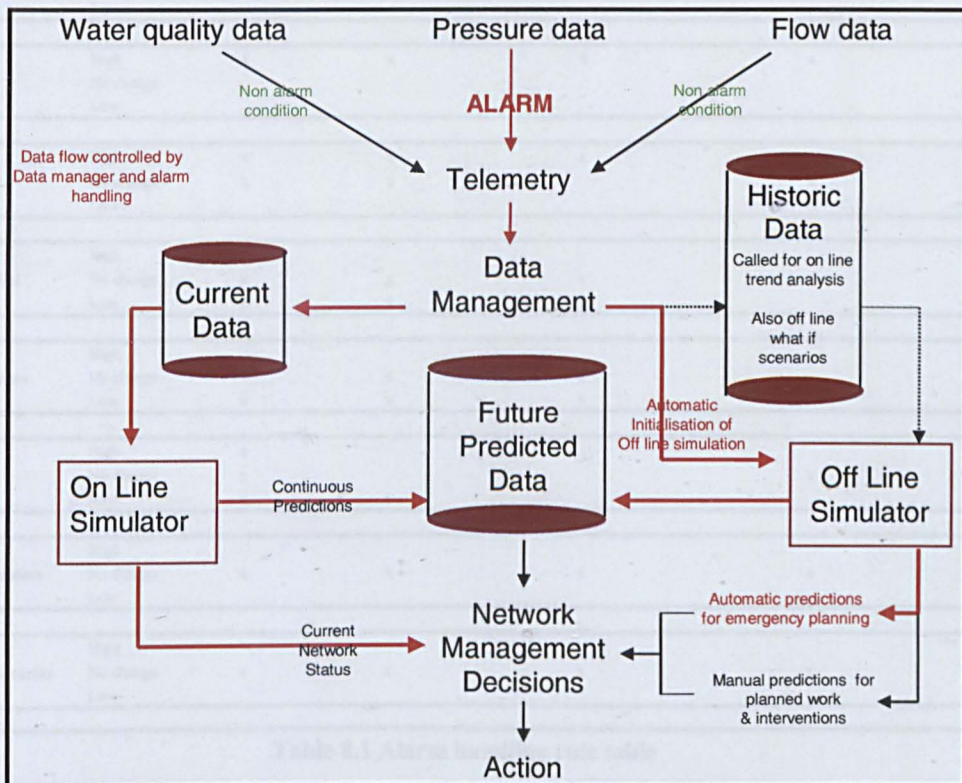


Figure 8.9 Model data management when an alarm condition is active

Under certain alarm conditions, transfer to offline and initiation of hydraulic and water quality simulation is automatic. Table 8.1 shows the rule table that the model uses to decide the probable cause and the priority of the alarm.

Alarm prioritiser		Pollution	Treatment failure	Service Res failure	Discoloured/turbid water	Burst
Pressure	High					
	No change	x	x	x	x	
	Low					x
Flow	High					x
	No change	x	x	x	x	
	Low					
Reservoir level	High					
	No change	x	x	x	x	x
	Low					x
Reservoir inlet flow	High					x
	No change	x	x	x	x	x
	Low					
Reservoir outlet flow	High					x
	No change	x	x	x	x	x
	Low					
Reservoir overflow	Overflow					
	Normal	x	x	x	x	x
Pressure	High					
	No change	x	x	x	x	
	Low					x
Turbidity	High	x	x	x	x	x
	No change					x
	Low					
Conductivity	High	x	x	x	x	x
	No change	x	x		x	x
	Low					
Redox potential	High					
	No change	x	x	x	x	x
	Low	x	x	x		
Dissolved oxygen	High					
	No change		x	x	x	
	Low	x	x	x		
pH	High	x				
	No change	x			x	x
	Low	x	x			
Water temperature	High					
	No change	x	x	x	x	
	Low					
Cabinet temperature	High					
	No change	x	x	x	x	
	Low					

Table 8.1 Alarm handling rule table

High priority alarms result in automatic transfer of the latest boundary condition to the offline model and automatic hydraulic, age and propagation simulation activation. Whilst this action is in process, the online system continues to operate and present the current network characteristics. This provides the user with a regular update of the current network characteristics and allows the use of offline modelling to undertake scenario planning. For example, if a burst occurs how to minimise the impact on the network, isolate the burst and continue to feed the rest of the network with minimal loss of water. In the case of pollution ingress, the user can identify how to isolate the pollutant in a particular area of the network, how to maintain supply to the rest of the network, and how to flush the polluting material out of the isolated part of the network. As the system can be used for 'what if?' scenarios, it can be used for contingency planning. Scenarios such as pollution of a service reservoir may be simulated and operational tasks pre-recorded for use in such an event. This allows rapid identification of critical assets such as valves and timing of their opening or closure in order to manage the event effectively and efficiently.

Simulation results are output in tabular or graphical output styles. Tabular information output is useful in that it can be configured to provide just the output required for example, all pipes with a water age in excess of three days or, all nodes where pressures fall below the regulatory limit value. Output files contain summary statistics for each time step of a simulation. As output files are very large and contain a wide variety of information, it is not realistic to try to present one here and an extract is not meaningful.

The graphical output is a representation of tabular output that can take a number of forms that allow the user to assimilate information that would be impossible from a tabular output file. Figure 8.10 For example, shows a part of the study network.

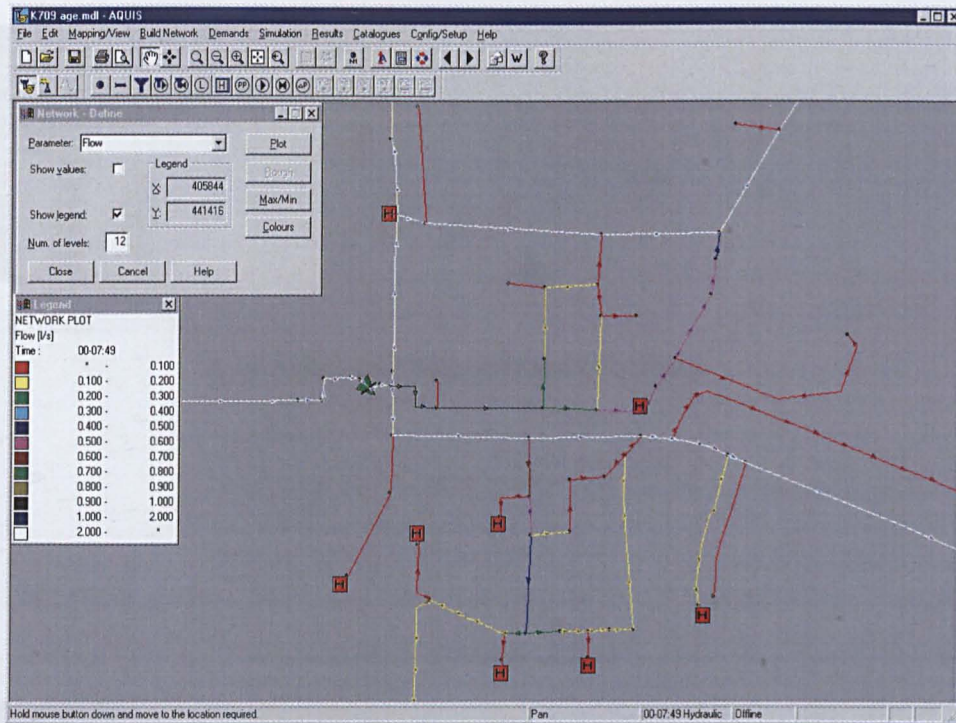


Figure 8.10 Graphical output for part of the study network

On this one screen, the user can determine the magnitude and direction of flow in the pipes, where the pressure reducing valve and hydrants are located. Other output styles viewed simultaneously provide specific information. Figure 8.11 for example, shows the pressure drop associated with a pressure-reducing valve.

8.3.3 The Transient Model

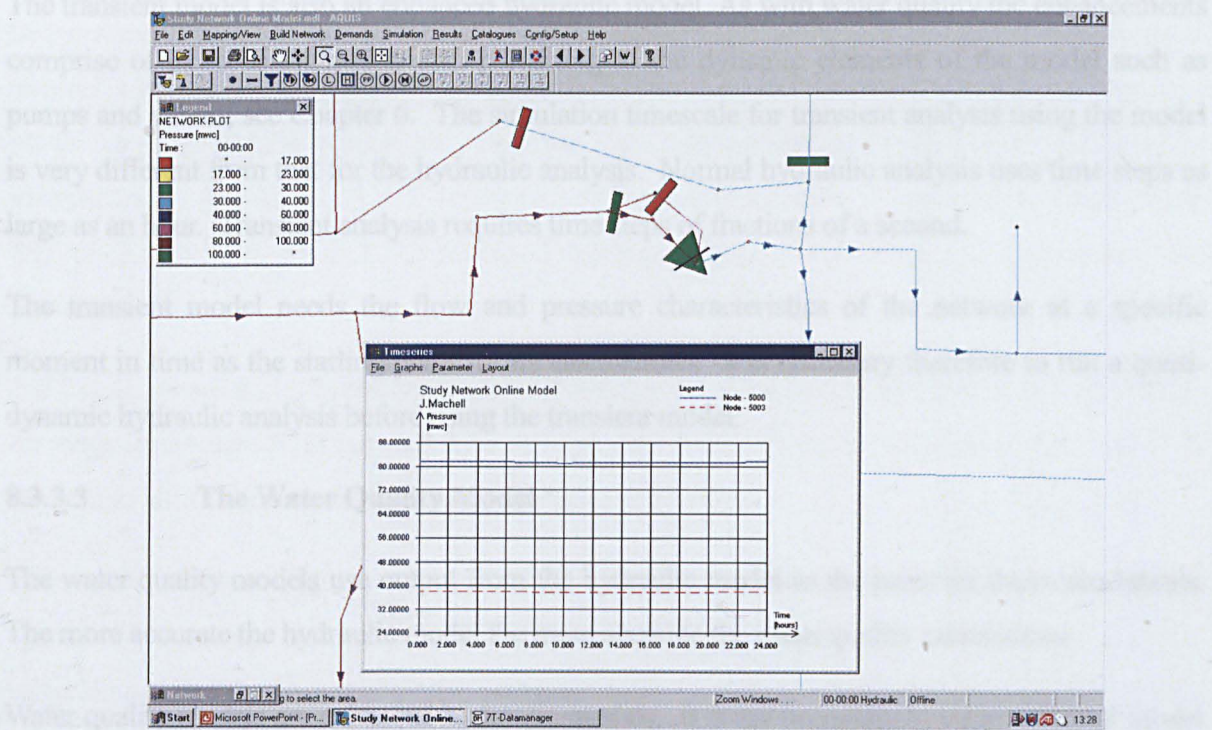


Figure 8.11 Pressure drop associated with a pressure-reducing valve

The timely collection of network data and this graphical output of information gleaned from the data is the key to rapid assimilation of very large amounts of operational data that give the user the capability to better understand and proactively manage the network.

8.3.3 The Online Model

The online model comprises of three sub models: Hydraulic, Transient and Water Quality. The following section describes each in detail.

8.3.3.1 The Hydraulic Model

A hydraulic network model of the system is the minimum prerequisite for online modelling. The hydraulic model, Chapter 5.1, describes the hydraulic model building process. Facilities are available to import models from other systems or to build new models from scratch.

8.3.3.2 The Transient Model

The transient model is also an enhanced hydraulic model. As with water quality the enhancements comprise of extra configuration data pertaining to the dynamic elements of the model such as pumps and valves, see Chapter 6. The simulation timescale for transient analysis using the model is very different from that for the hydraulic analysis. Normal hydraulic analysis uses time steps as large as an hour. Transient analysis requires time steps of fractions of a second.

The transient model needs the flow and pressure characteristics of the network at a specific moment in time as the starting point for its calculations. It is necessary therefore to run a quasi-dynamic hydraulic analysis before using the transient model.

8.3.3.3 The Water Quality Model

The water quality models use output from the hydraulic model as the basis for their calculations. The more accurate the hydraulic model the more accurate the water quality calculations.

Water quality models are enhanced hydraulic models. It is not necessary to build a special model for water quality modelling. The enhancement is in the form of additional data that is input via the graphical user interface or by writing directly into the input files. There are occasions when this method is appropriate.

8.4 The On-line System

The study network online system consists of four main parts: field instrumentation, communications hardware and software, online hardware and software, and offline hardware and software.

8.4.1 Field Instrumentation

Two types of field instruments gather network data for the online system.

The hydraulic instruments were Spectrascan Microlog 4T, originally installed for leakage control purposes. They provided flow and pressure data at pre-set intervals on a continuous basis. They are standard products commercially available. The instruments are programmable from a remote location.

Water quality instruments gather water quality data (and also pressure). The water quality instruments were developed in conjunction with Solomat, a subsidiary of Neotronics, later to become part of the Zellweger Empire. These instruments are sophisticated. They are multi channel, can be programmed to take readings at any time period down to 1 second, and they have alarm handling and, in conjunction with the data management software, pollution fingerprint identification capabilities. A full description of the instrumentation is presented in Chapter 4 of this document. A local P.S.T.N based telemetry system contacted the instruments and downloaded data.

For this application, the instruments were all configured to take measurements every 15 minutes for each parameter. This was because the cycle time of the online system was been set to 30 minutes to allow the outstations to be contacted individually.

The water quality instruments measure pH, conductivity, redox potential, pressure, turbidity, dissolved oxygen and temperature. The determinants were chosen because they were the most robust / reliable measurements readily available at the time, which could be applied in the hostile environment of distribution networks with minimal development costs. Figure 8.12 shows how the instruments are installed into a water main via an under-pressure T and valve assembly.

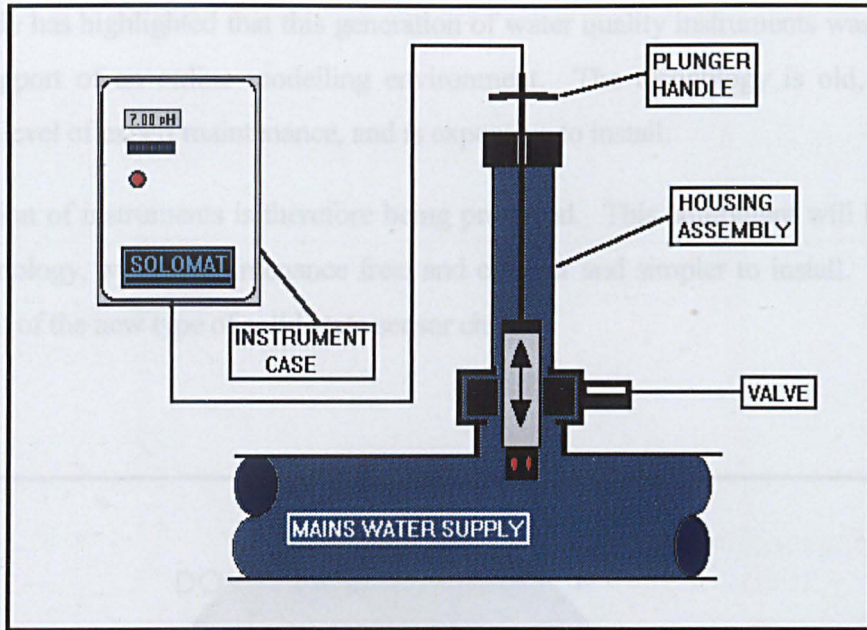


Figure 8.12 Instrument installation detail

Detailed installation requirements are shown in Figure 8.13

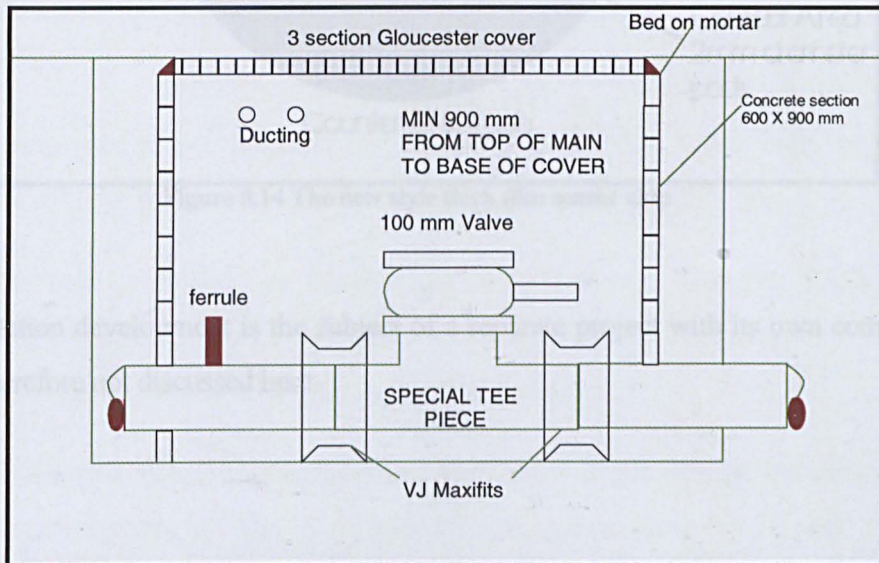


Figure 8.13 Detail of installation

The instruments can generate high and low alarms for all channels. An alarm results in the instrument ringing the operator and, or, initiating an automatic sampling machine which can take water samples at any time throughout the incident which generated the alarm. Full details of the instrumentation are described in Chapter 4.

The online study has highlighted that this generation of water quality instruments was not wholly suitable for support of an online modelling environment. The technology is old, not robust, requires a high level of expert maintenance, and is expensive to install.

A new generation of instruments is therefore being produced. This equipment will be based on thick film technology, will be maintenance free, and cheaper and simpler to install. Figure 8.14 shows the detail of the new type of solid-state sensor chip.

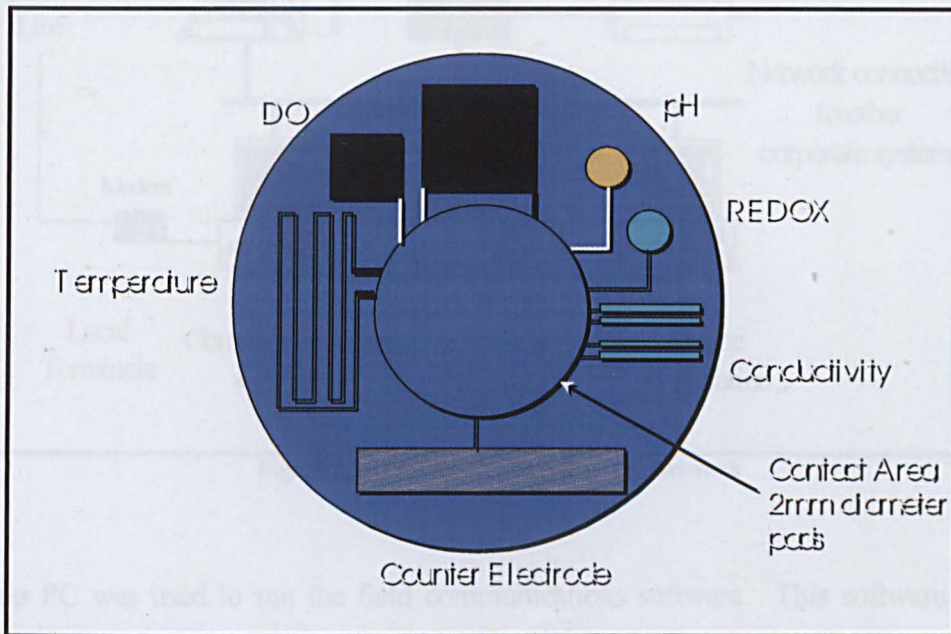


Figure 8.14 The new style thick film sensor chip

The instrumentation development is the subject of a separate project with its own comprehensive report and is therefore not discussed here.

8.4.2 Computer Hardware

Three separate, networked computers ran the online system, Figure 8.15.

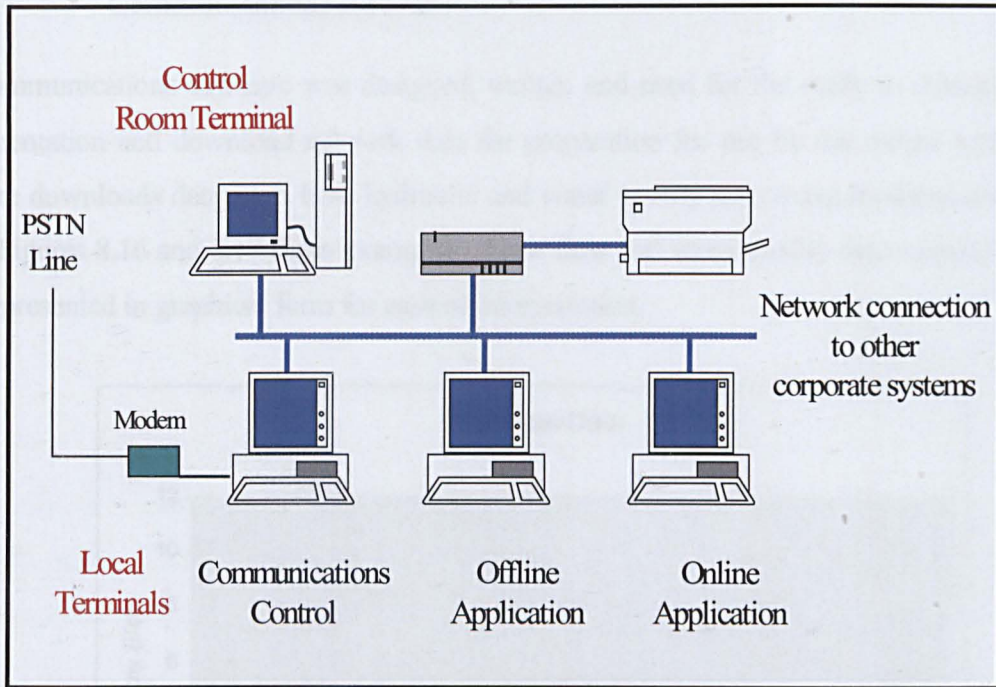


Figure 8.15 Online system hardware network

A desktop PC was used to run the field communications software. This software handled all communications with the field instruments via modem links and P.S.T.N. lines.

The main system comprised of two Professional Workstations. The first Workstation was configured to run online in a continuous cycle of taking in network data, reporting the current network condition, and predicting future network characteristics.

The second Workstation waited in standby mode until required for off-line simulations. Offline simulations are automatically initiated by an incident alarm, or manually by the need to plan work on the distribution network. In either case, the user benefits from the most up to date network information available.

8.4.2 Online Software

The online system uses three separate software modules, a communication module, a data management module and the online modelling software module.

8.4.3.1 Communications software

The communications software was designed, written and used for the study to contact the field instrumentation and download network data for preparation for use by the online system. The software downloads data from both hydraulic and water quality sites using modems and P.S.T.N lines. Figures 8.16 and 8.17 show examples of raw flow and water quality data respectively. The data is presented in graphical form for ease of interpretation.

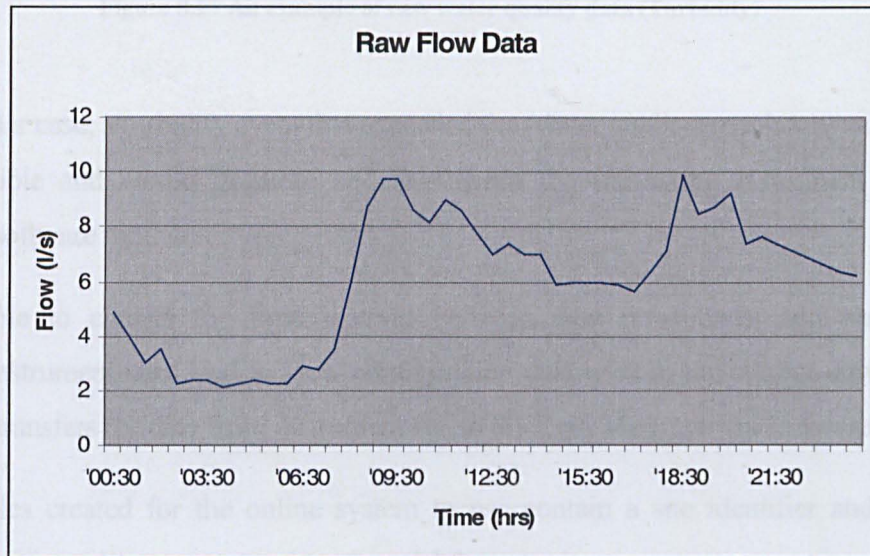


Figure 8.16 Raw flow data

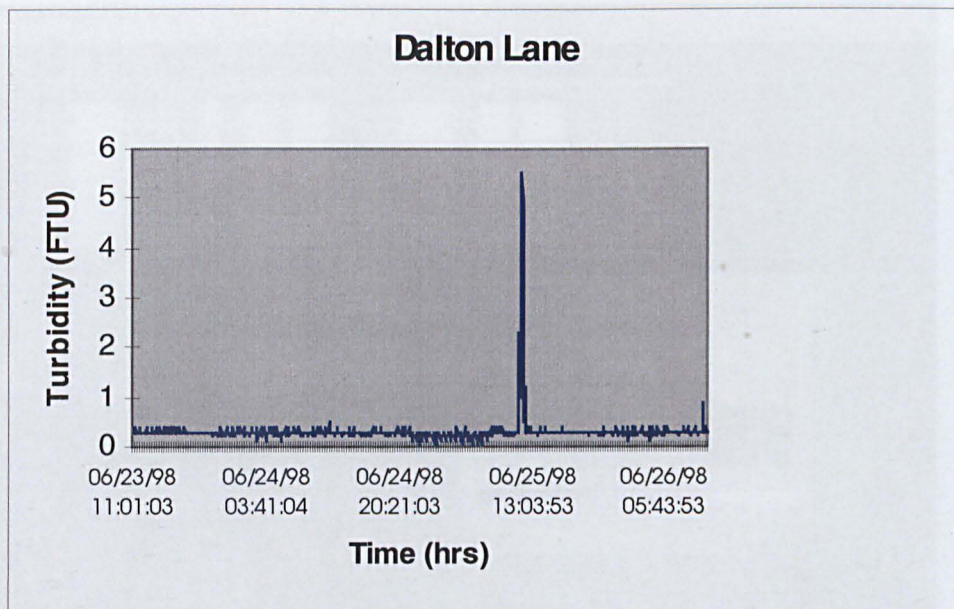


Figure 8.17 An example of raw water quality data (Turbidity)

In this particular case, a turbidity event that exceeded the (water quality) regulatory value of 4 FTU is clearly visible and would generate an alarm from the measuring instrument or the data management software module.

It was possible to change the time interval between data downloads, add new or delete unnecessary instrument sites, and to view configuration data used in the current download cycle. The software transfers the data from instrument site to the Data Management software module.

The ASCII files created for the online system to use contain a site identifier and appropriate hydraulic or water quality parameters in sequential format.

8.4.3.2 The Data Management Software

The Data Management software requests a new file from the Communications Workstation before a model simulation cycle, and converts it into a format acceptable to the modelling engine. Figure 8.18 shows the main Data Management screen.

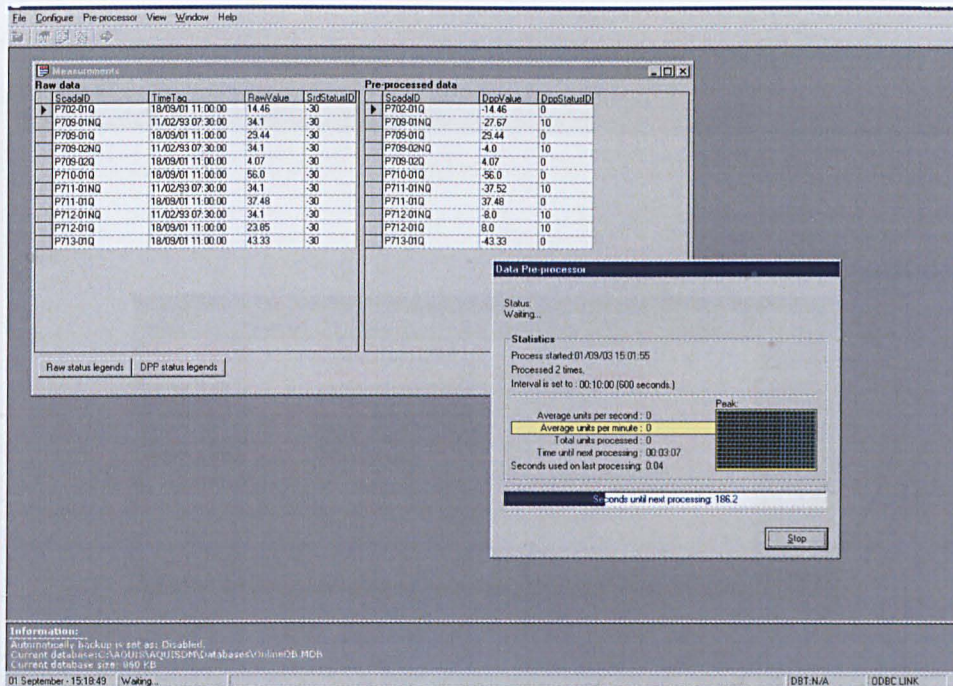


Figure 8.18 The data management module main screen

The diagram shows the pre-processed, the processed data, and the current state of the pre-processor module.

It is possible to add or delete sites from the current outstation list. Current and historic hydraulic and / or water quality data may be viewed and system paths for on-line data files may be configured. If data is in alarm condition, it displays in red in an Event Log window. Figure 8.19 shows the alarm box popped up on the main screen.

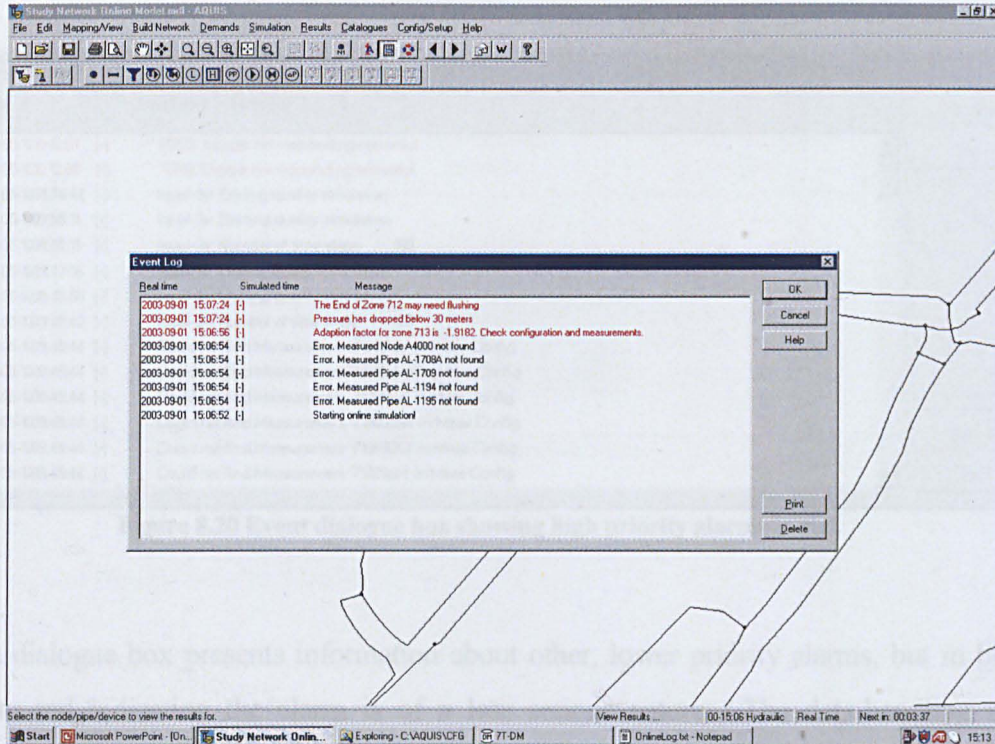


Figure 8.19 Event Log window showing current data and alarm conditions

The data software generates alarms when any measured parameter(s) fall outside user defined upper and / or lower bounds or when instruments cannot be contacted for some reason.

Empirical research was undertaken as part of this study and showed that certain substances, such as Aluminium Sulphate, will produce specific, repeatable changes in the measured water quality parameters. These changes provide a “fingerprint”, defined by Table 8.1, of the substance that is programmed into the alarm-handling algorithm.

Water quality alarms are allocated a status dependent upon how the various water quality measurements are affected. High priority alarms, such as those indicative of pollutant ingress or failure / unavailability of instrumentation, are displayed on the users online screen in an Event dialogue box and highlighted in red text. Figure 8.20 shows the Event Log dialogue box.

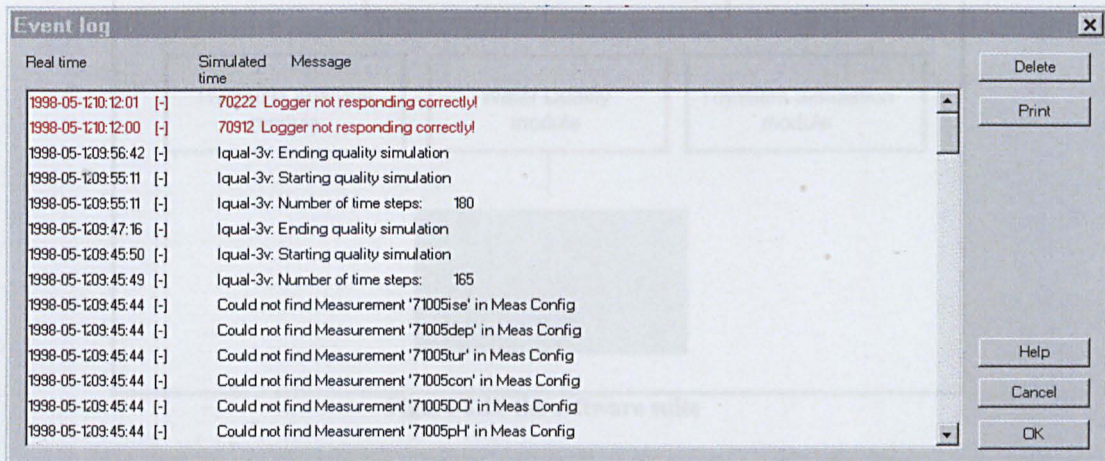


Figure 8.20 Event dialogue box showing high priority alarms in red.

The same dialogue box presents information about other, lower priority alarms, but in black as opposed to red indicating the alarm is of a less serious nature. The data-handling module processes the network data before its use by the online module. A configuration file allows the user to dictate how to deal with missing or corrupt data. It is possible to replace the missing data with a standard value or to force the system to use the last known good value.

8.4.3.3 The Simulation Software

The software has hydraulic, water quality and dynamic modules integrated into one suite shown in Figure 8.21.

The online functionality was integrated into the modelling software developed for this study as detailed above. The online module calls on all three models as and when required / configured. The system provides the user with the facilities to obtain distribution network information using the following models:

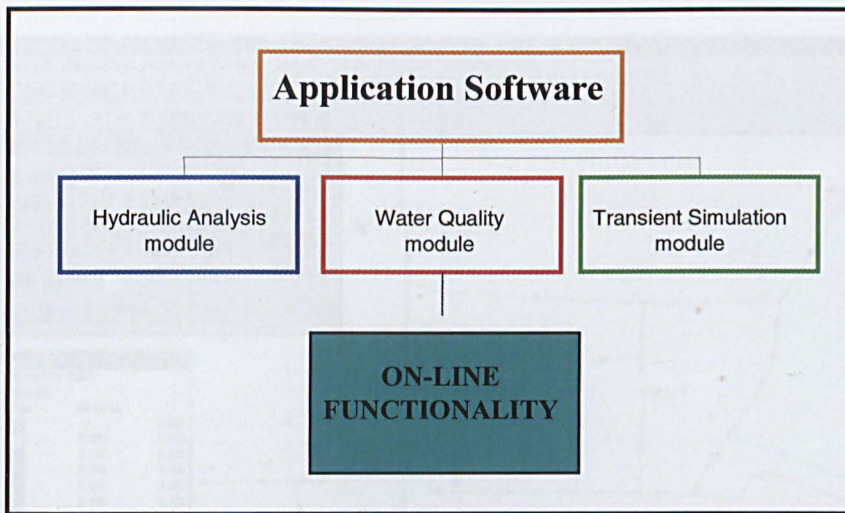


Figure 8.21 the software suite

8.4.3.3.1 The Hydraulic Model

Provides information about the hydraulic characteristics of the network including:

8.4.3.3.1.1 Flow

The reasons for wishing to monitor flows include operation of service reservoirs, detection of burst mains and unusual or illegal demands on the network. Figure 8.22 shows the users screen zoomed in on part of the network to see the detailed information.

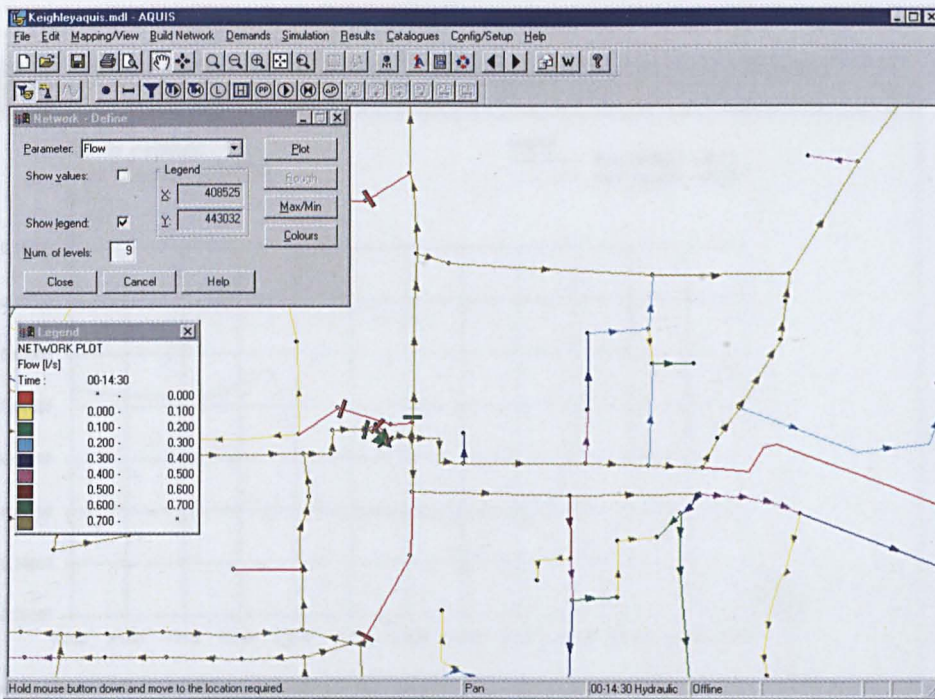


Figure 8.22 Magnitude and direction of flow

It is useful to be able to continually monitor the flows from service reservoirs. Figure 8.23 shows a user screen highlighting the reservoir, the magnitude of flow and the flow pattern from the reservoir.

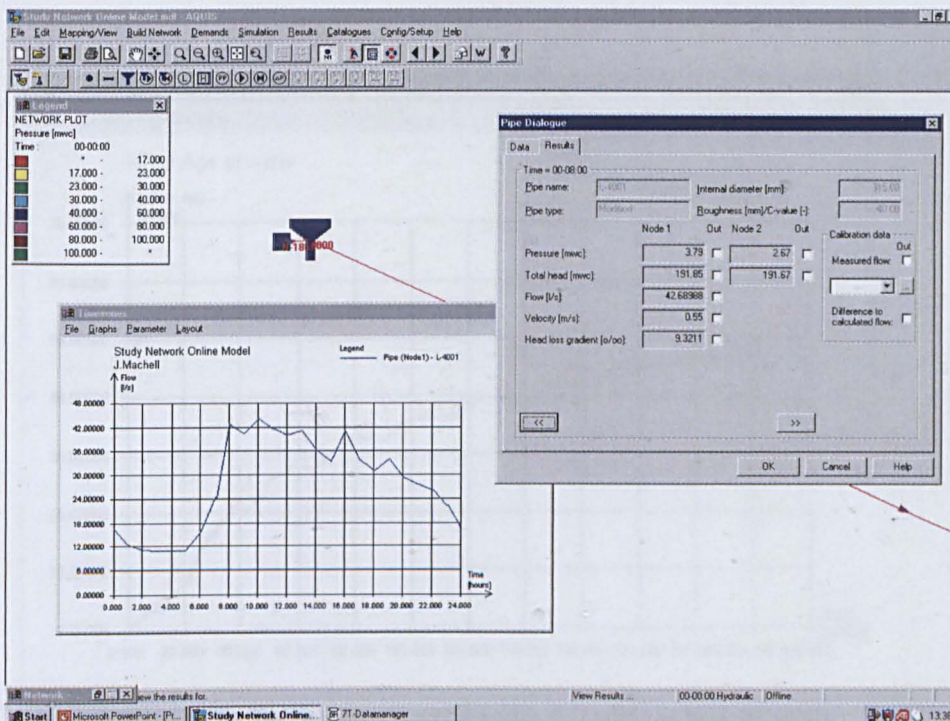


Figure 8.23 highlighting the magnitude of flow and the flow pattern from a service reservoir

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UNAVAILABLE**

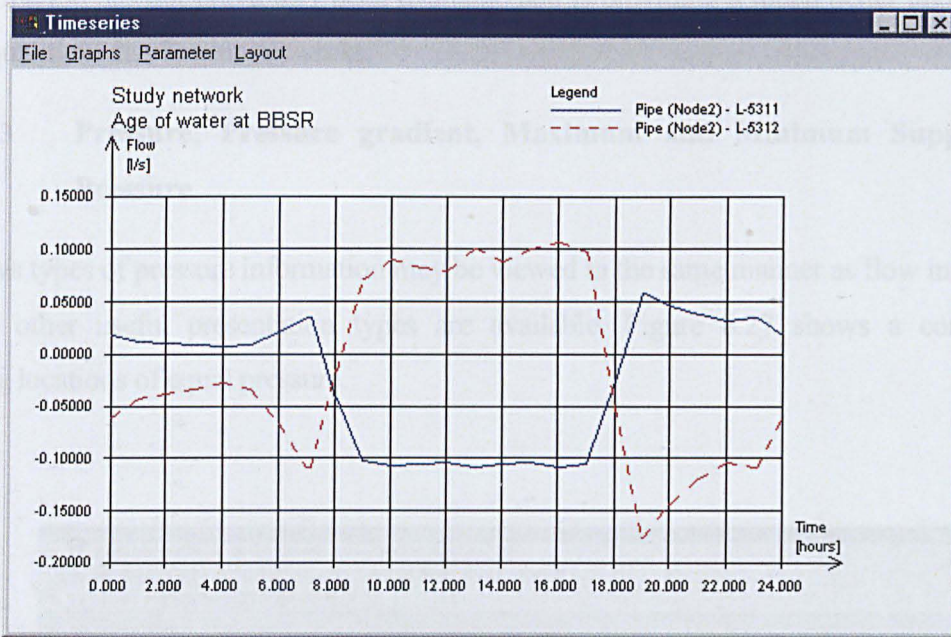


Figure 8.25 Flow time series showing magnitude and frequency of flow reversal in 2 pipes

The effects of a flow reversal on the age of water can be quickly determined. Figure 8.26 shows how the age of water in a pipe with a regular flow reversal differs to that in a pipe with unidirectional flow.

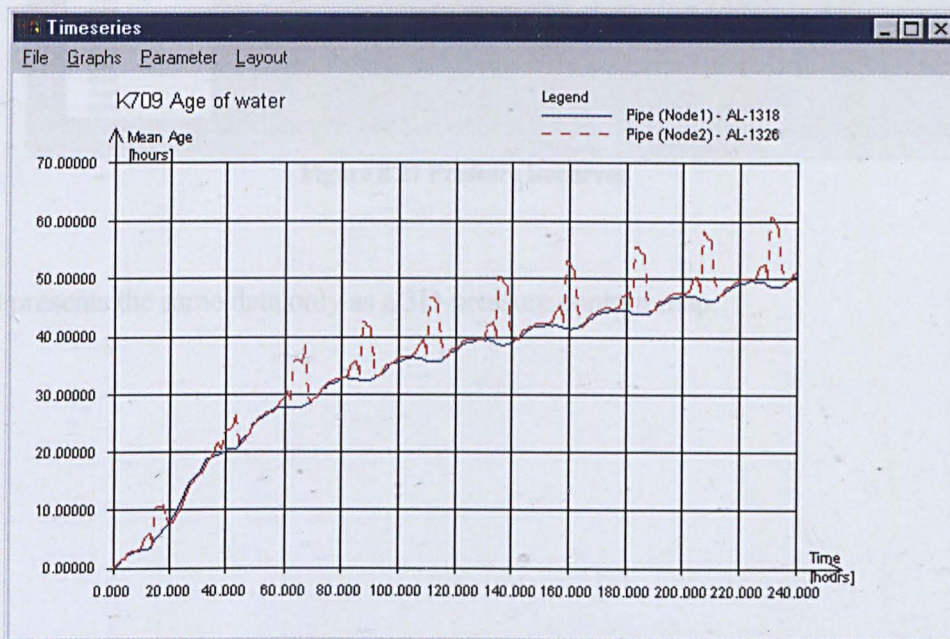


Figure 8.26 Age of water in a pipes with and without flow reversals

This information has not previously been available and is providing a much better understanding of water quality in distribution networks.

8.4.3.3.1.3 Pressure, Pressure gradient, Maximum and Minimum Supplemental Pressure

The various types of pressure information may be viewed in the same manner as flow information. However, other useful presentation types are available. Figure 8.27 shows a contour plot identifying locations of equal pressure.

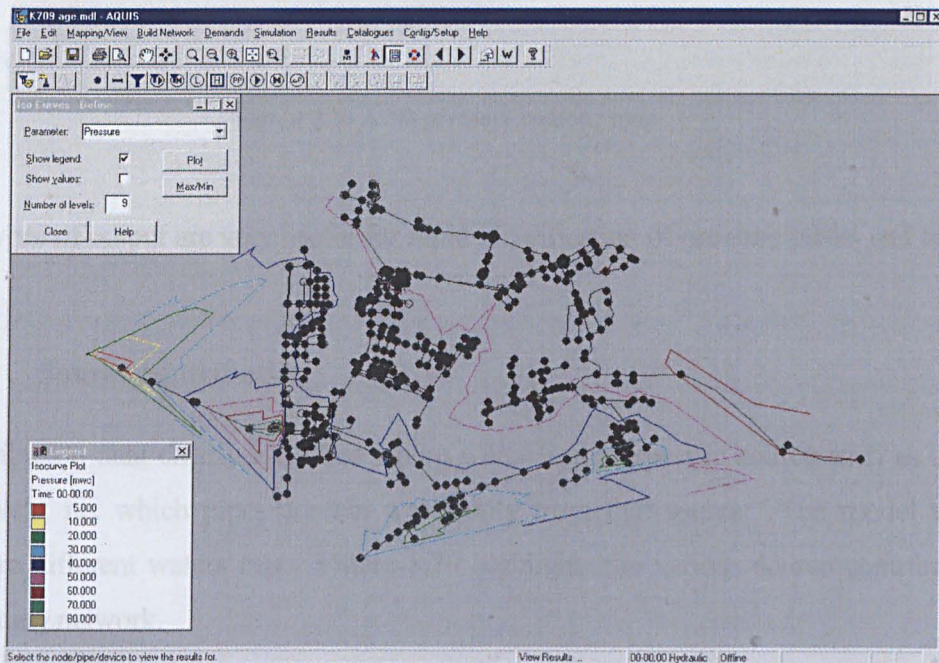


Figure 8.27 Pressure isocurves

Figure 8.28 presents the same data only as a 3D-pressure contour map.

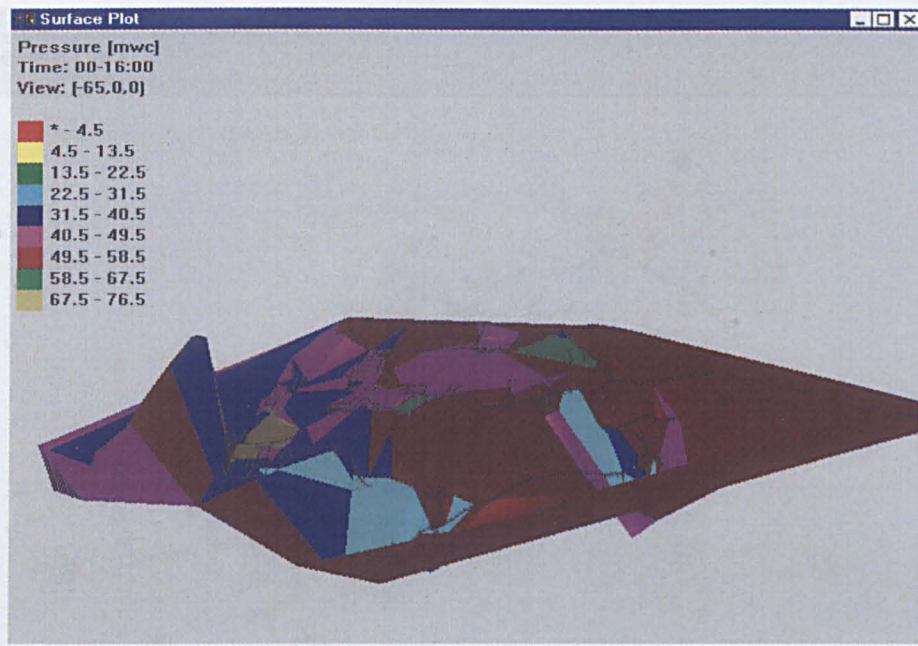


Figure 8.28 A 3D pressure contour map

Both these types of output are very useful for rapid identification of pressure peaks and troughs in a network.

8.4.3.3.1.4 Source contribution

Source contribution data clearly identifies where water from a specific source such as a service reservoir travels, i.e. which pipes contain water only from that source. The model will also identify where different waters mix. Figure 8.29 highlights the various source contributions to parts of the study network.

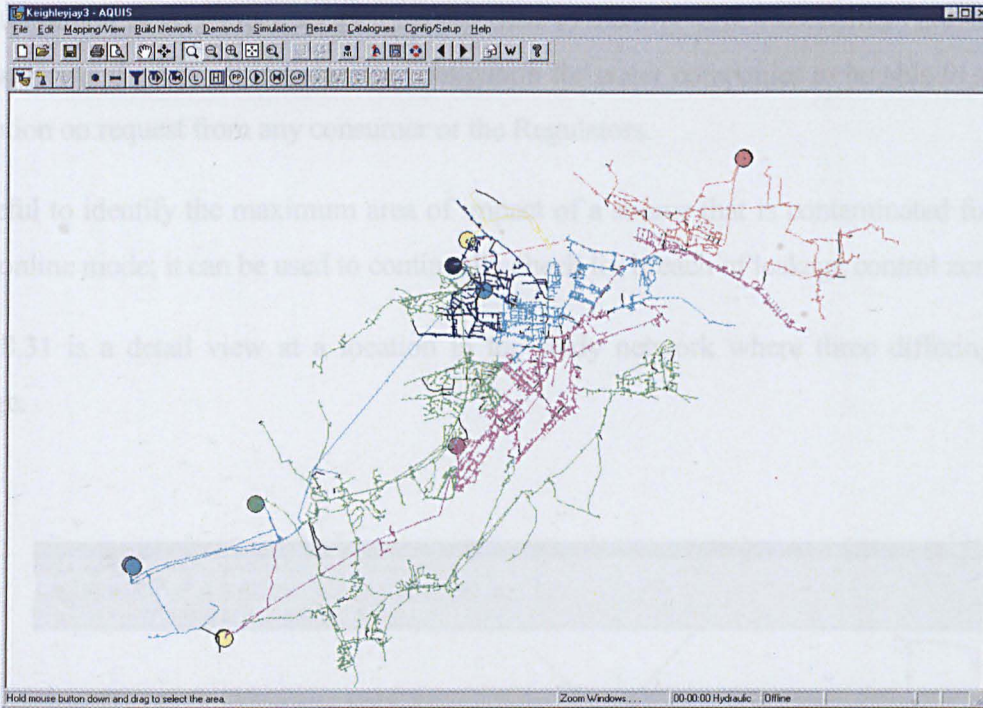


Figure 8.29 Source contributions

Figure 8.30 shows how this plot can be enhanced to highlight water from a single source.

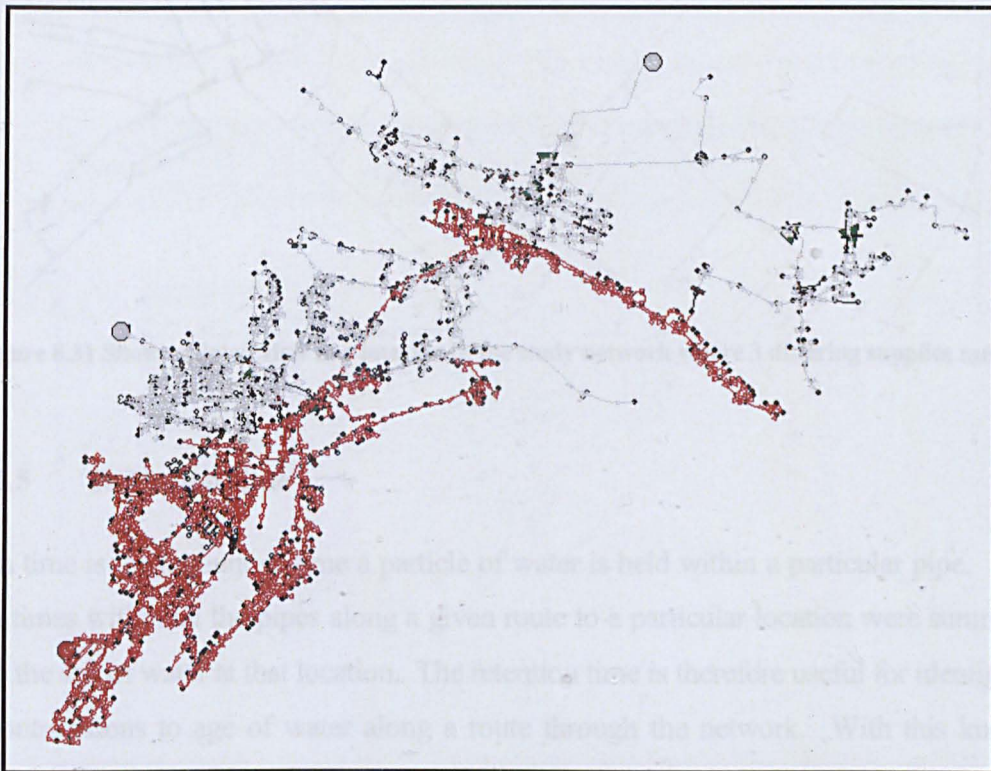


Figure 8.30 Extent of supply of a single source in a multi-sourced network

This information is very practical as it can be used to identify which consumers are supplied by which source and when. It is a statutory obligation for water companies to be able to supply this information on request from any consumer or the Regulators.

It is useful to identify the maximum area of impact of a source that is contaminated for example and, in online mode; it can be used to continually check for breach of leakage control zones.

Figure 8.31 is a detail view at a location in the study network where three differing supplies converge.

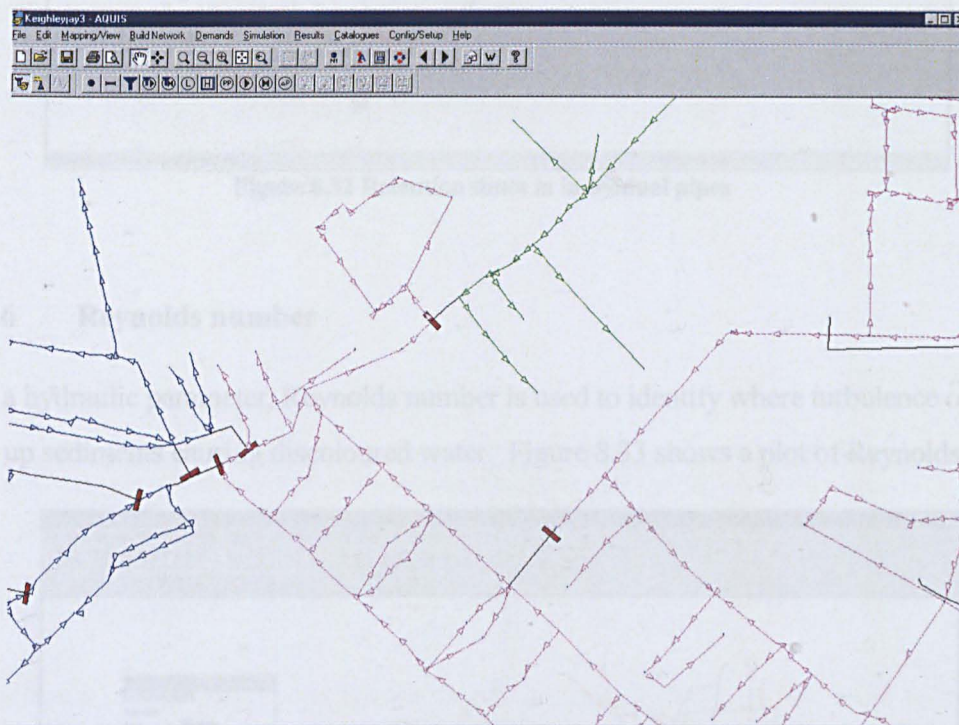


Figure 8.31 Shows a detail view at a location in the study network where 3 differing supplies meet.

8.4.3.3.1.5 Retention time

Retention time is the amount of time a particle of water is held within a particular pipe. If all the retention times within all the pipes along a given route to a particular location were summed, this would be the age of water at that location. The retention time is therefore useful for identifying the largest contributions to age of water along a route through the network. With this knowledge operational staff have an opportunity to reduce the age of water at a particular location should the connectivity of the pipe work and valves permit. Figure 8.32 presents a plot of retention times in individual pipes.

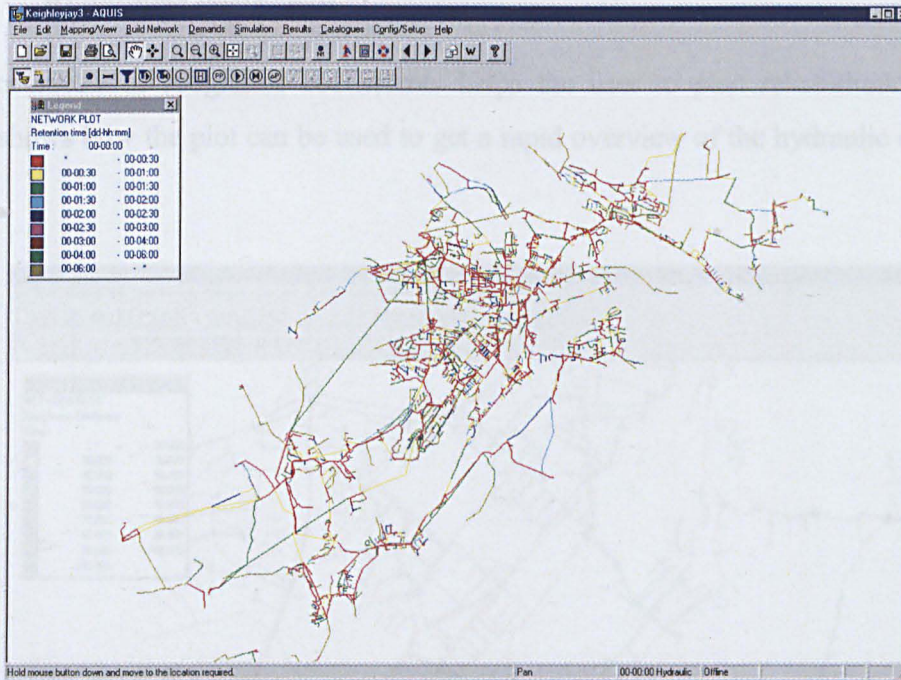


Figure 8.32 Retention times in individual pipes

8.4.3.3.1.6 Reynolds number

Although a hydraulic parameter, Reynolds number is used to identify where turbulence occurs that might stir up sediments causing discoloured water. Figure 8.33 shows a plot of Reynolds numbers

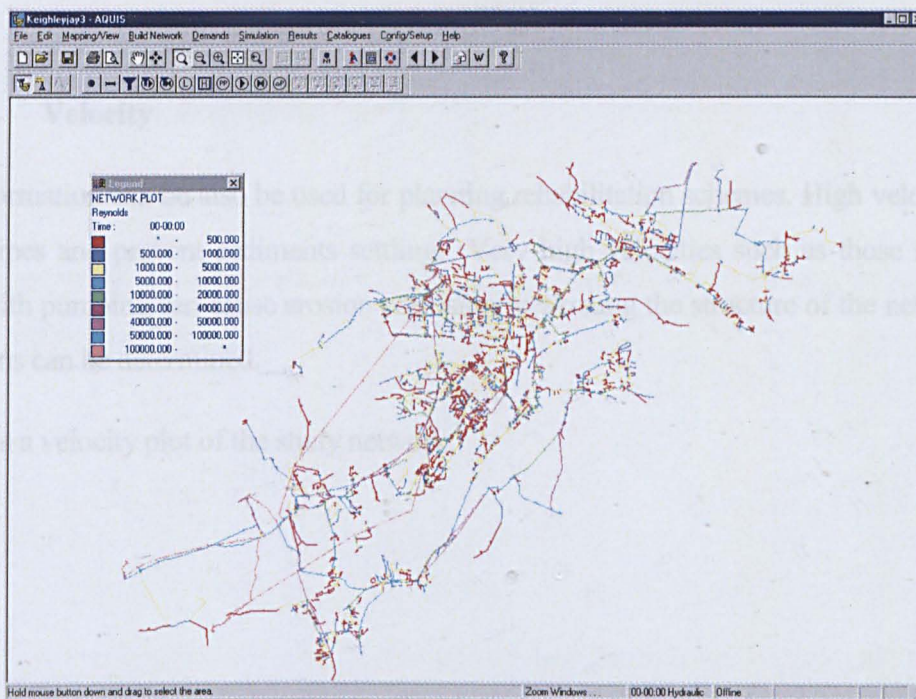


Figure 8.33 Reynolds numbers for each pipe

8.4.3.3.1.7 Roughness coefficient

Having an overview of roughness coefficients helps the user to plan rehabilitation schemes. Figure 8.34 shows how the plot can be used to get a rapid overview of the hydraulic condition of the mains.

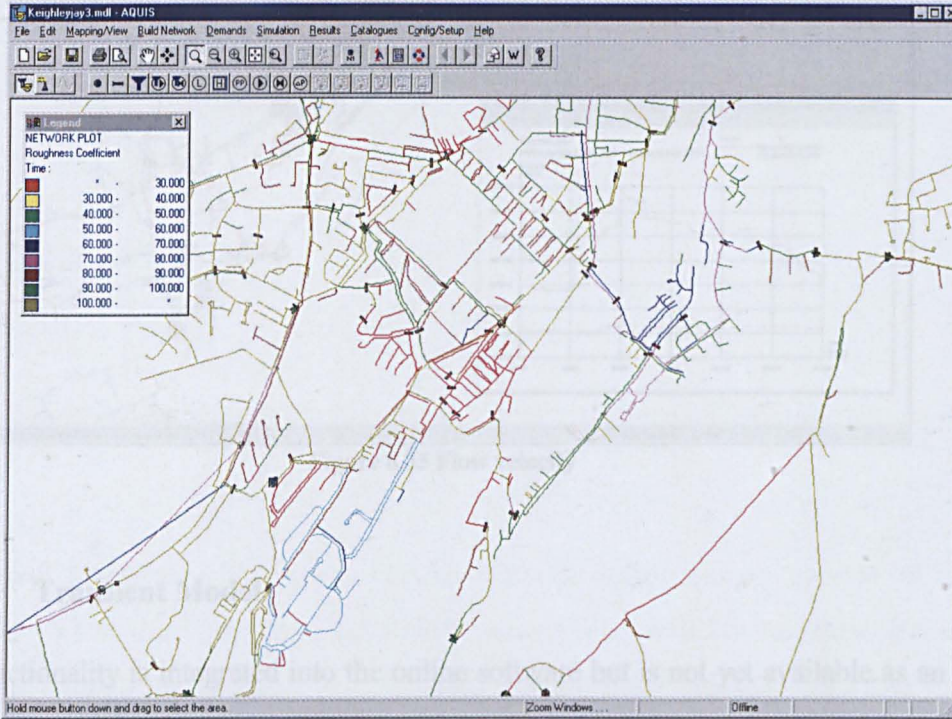


Figure 8.34 Roughness coefficients for overview of the condition of the mains

8.4.3.3.1.8 Velocity

Velocity information can also be used for planning rehabilitation schemes. High velocity flows will scour pipes and prevent sediments settling. Very high velocities such as those sometimes associated with pumping can cause erosion corrosion weakening the structure of the network, and these locations can be determined.

Figure 8.35 is a velocity plot of the study network.

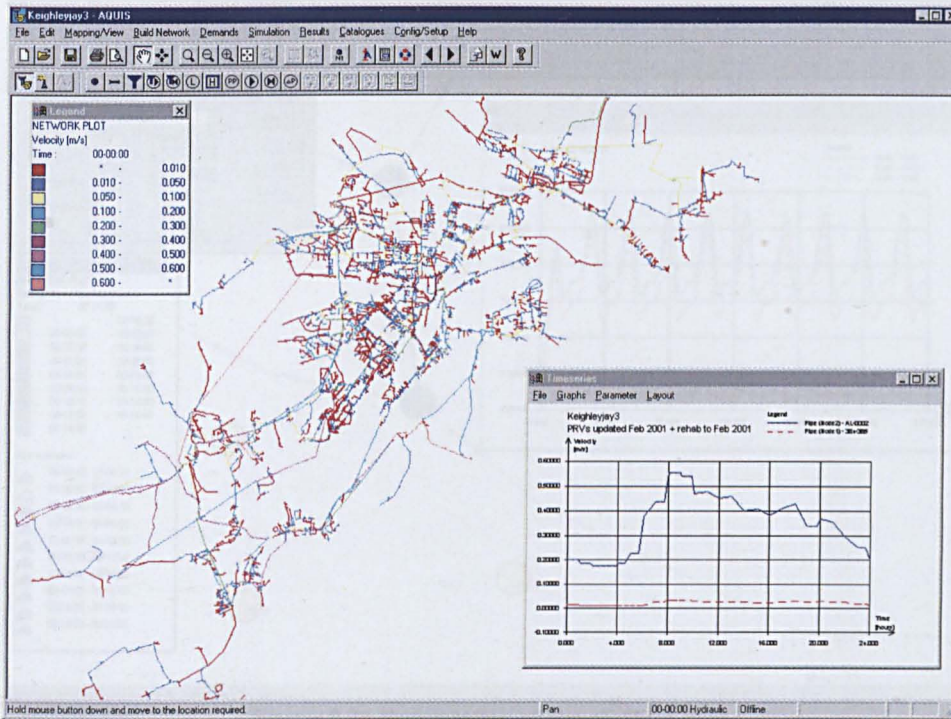


Figure 8.35 Flow velocity

8.4.3.3.2 Transient Model

Transient functionality is integrated into the online software but is not yet available as an on line function. However, it can be accessed and used for offline simulations and will be fully integrated at a future time.

8.4.3.3.3 Water Quality Model

8.4.3.3.2.1 Age of water

The mean age of water in the network can be monitored. Figure 8.36 shows how the age of water information may be presented

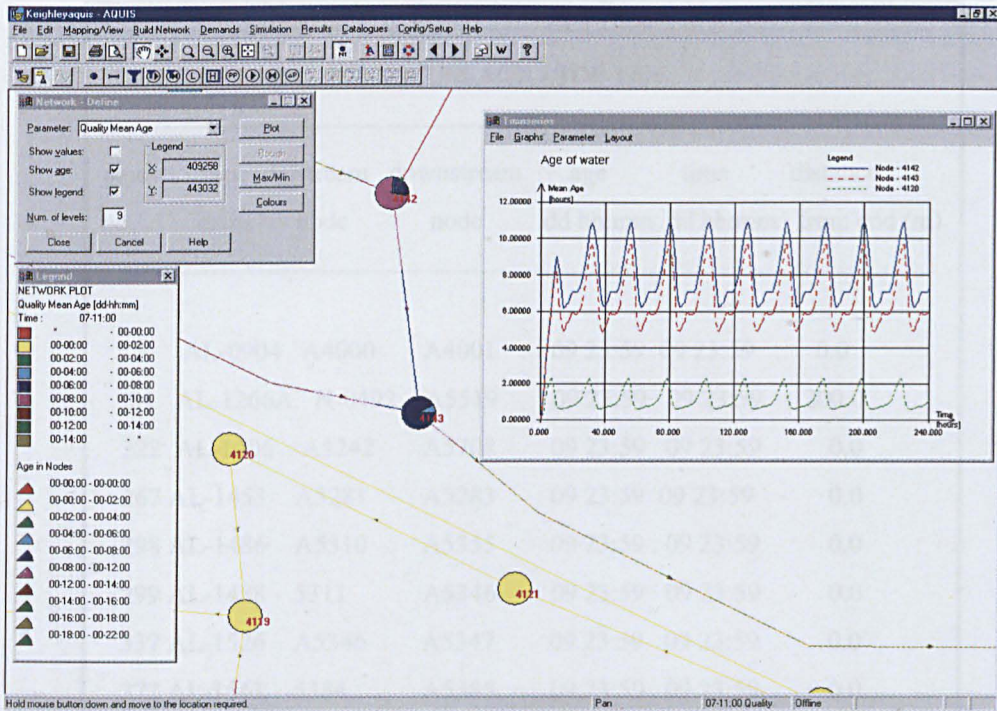


Figure 8.36 Presentation of age of water data

The background information in Figure 8.36 includes the mean age represented as different coloured pipes. Where the pipes connect at nodes, the node is enhanced to include a breakdown of the various age components presented as a pie chart. The key details the age bands into which the simulator resolved the component age fractions. Superimposed in the foreground is time series data of the age of water at three locations within the study network. Also visible is the flow direction in individual pipes represented by arrows. The relative magnitude of flow in each pipe is represented by the size of the arrows.

Maximum age information is available on a similar plot and is written to the simulation output file as a maximum age 'top ten' table. This is very useful to quickly identify problem areas within a network. Figure 8.2 is an example of a Maximum Age 'Top Ten'.

MAXIMUM AGE - TOP TEN						
pipe no.	pipe name	upstream node	downstream node	age dd hh:mm	time dd hh:mm	distance from end (m)
3	AL-0904	A4000	A4001	09 23:59	09 23:59	0.0
84	AL-1266A	N-0492	A5519	09 23:59	09 23:59	200.0
222	AL-1406	A5242	A5708	09 23:59	09 23:59	0.0
267	AL-1453	A5281	A5283	09 23:59	09 23:59	0.0
298	AL-1486	A5310	A5355	09 23:59	09 23:59	0.0
299	AL-1488	5311	A5346	09 23:59	09 23:59	0.0
337	AL-1526	A5346	A5347	09 23:59	09 23:59	0.0
372	AL-1561	5384	A5385	09 23:59	09 23:59	0.0
436	AL-1624	5447	A5448	09 23:59	09 23:59	0.0
464	AL-1655	A5478	A5479	09 23:59	09 23:59	40.0

Figure 8.37 Maximum age 'Top Ten' table

In this example, it is clear that pipe number 84 has an age of water problem. The pipe is 200 m from the end of the network, that is, before a dead end is reached in the direction of flow. At this time, the simulator reports dead ends – those pipes that are zero metres from the end of the network, but it is intended to exclude these from the output.

8.4.3.3.2.2 Conservative substance propagation

Conservative substance concentration, for example, Nitrate or Fluoride, may be tracked using the online system. Figure 8.38 shows how a conservative tracer has propagated through the network after sixteen hours.

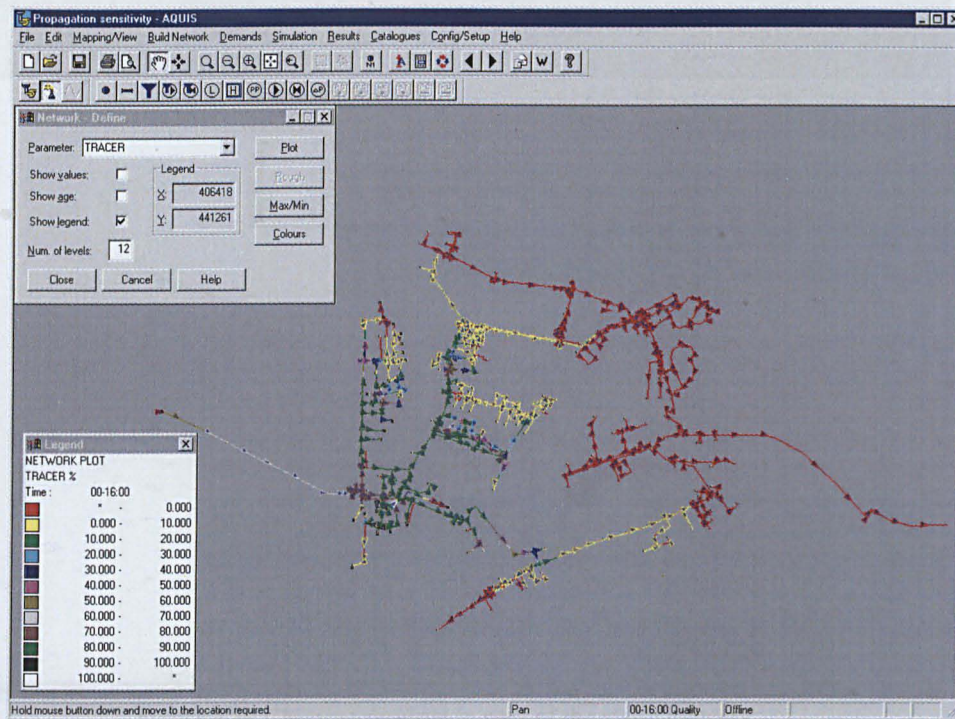


Figure 8.38 A conservative tracer propagated through the network for sixteen hours.

This functionality is the basis for the online pollution incident management detailed in section 8.9.

8.4.3.3.2.3 Non-conservative substance propagation

Non-conservative substance concentrations such as Chlorine can be simulated. Figure 8.39 is a plot of Chlorine concentration in part of the study network.

These simulations are undertaken offline. For example, the conversion of Ammonium via Nitrite to Nitrate. The stoichiometry of individual reactions may be varied and if Ammonia was to enter the network it is possible to determine how much Nitrite and Nitrate would be formed. The propagation functionality can then be utilised to determine where the Nitrogen would travel and when. The same functionality can be used to calculate Totalomethane production from organic precursors such as Colours and Chlorides. Details of this simulation can be seen in Chapter 7.

Figure 8.40 shows how a substance (Sub1) is decaying to create a new substance (Sub2). As this grows it reacts with another substance to produce Sub3 that then decays with time, as do three and non-conservative.

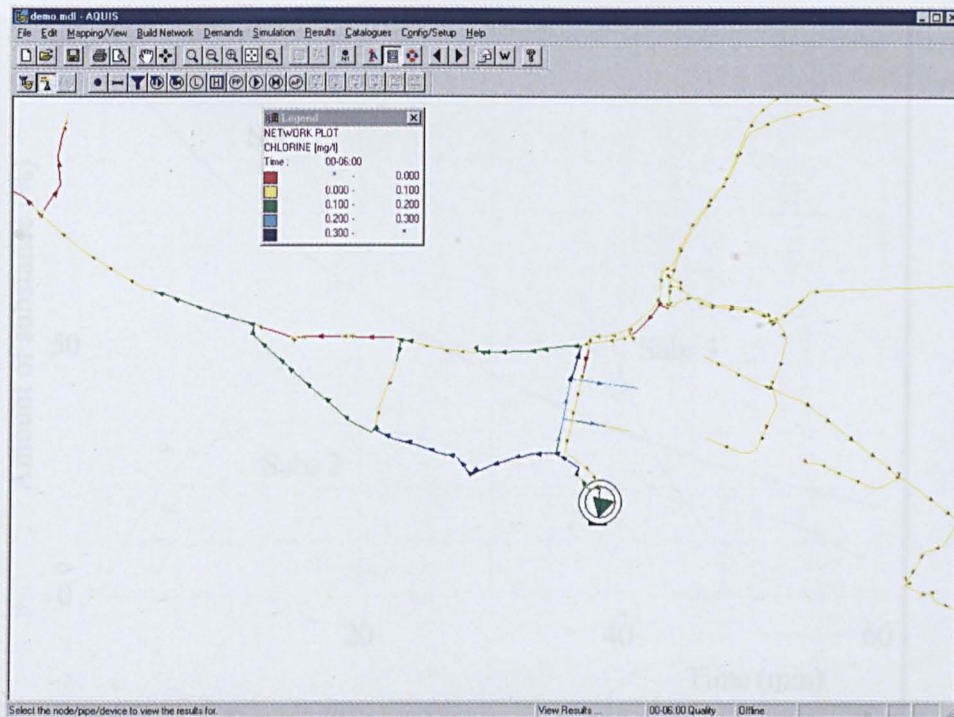


Figure 8.39 Chlorine residual in part of the study network

If a calibrated model were used in conjunction with chlorine monitoring at key locations, deviation from the normal residual in any location following a simulation would indicate a problem. This will be possible when a network has had its Chlorine demand satisfied and a continuous low level residual is being maintained.

8.4.3.3.2.4 Substance conversions

These simulations are undertaken offline. For example, the conversion of Ammonium via Nitrite to Nitrate. The stoichiometry of individual reactions may be varied and if Ammonia was to enter the network is possible to determine how much Nitrite and Nitrate would be formed. The propagation functionality can then be utilised to determine where the Nitrogen would travel and when. The same functionality can be used to calculate Trihalomethane production from organic pre-cursors such as Colour, and Chlorine. Details of this simulation can be seen in Chapter 7.

Figure 8.40 shows how a substance (Subs1) is decaying to create a new substance (Subs2). As this grows it reacts with another substance to produce Subs3 that then decays with time, as all three are non-conservative.

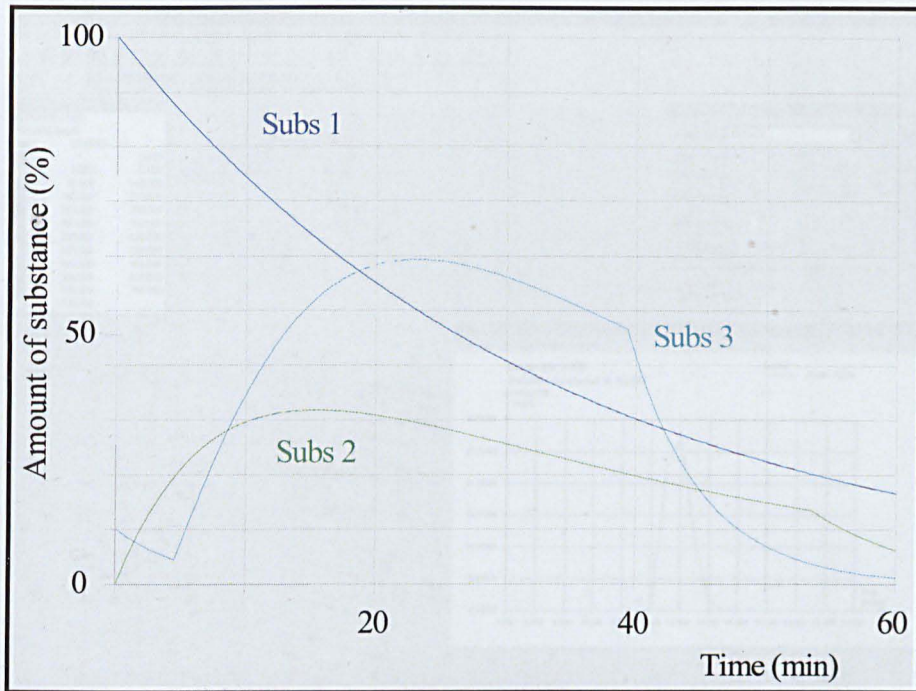


Figure 8.40 Substance conversion and decay

8.4.3.3.2.5 Diagnostic

The diagnostic model can be used to indicate the possible points of ingress of a polluting material after it has been detected in the network by instrumentation (or as a hypothetical input) and its propagation simulated by the model. The model runs the hydraulic database in reverse and the number of locations where a pollutant could have originated from is minimised. It is not yet possible to determine exactly where a pollutant would have entered but work is continuing to develop the model to do exactly that. Polluting material may not always be harmful. It would be of great benefit to be able to track down the location of the source of discolouration for example.

Figure 8.41 shows the time series of a pollutant measured at node A6280 superimposed upon the user screen highlighting the possible sources of the pollutant.

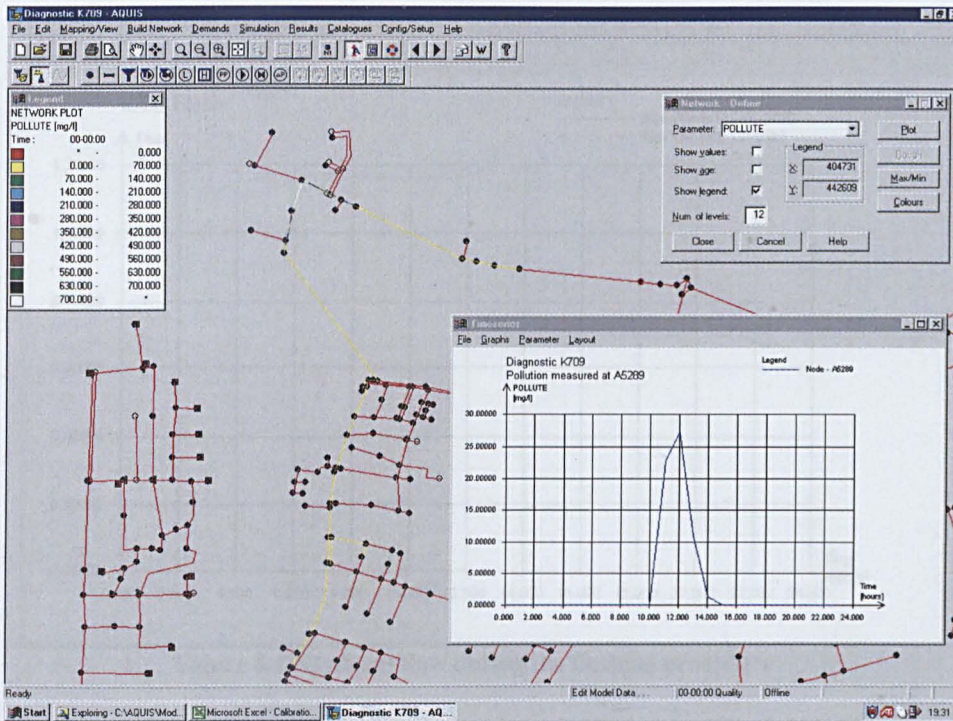


Figure 8.41 Time series of pollutant at a node and possible sources of the pollutant

8.4.3.3.2.6 Flushing

The flushing model was developed to determine which hydrants to open, in what order, to remove polluting material with least waste of water during an incident. Again, this would include discoloured or unpalatable water.

The opening of the end hydrant was simulated to show how the pollutant would be expelled from the network. Figure 8.42 represents the hydrant flow imposed for this example.

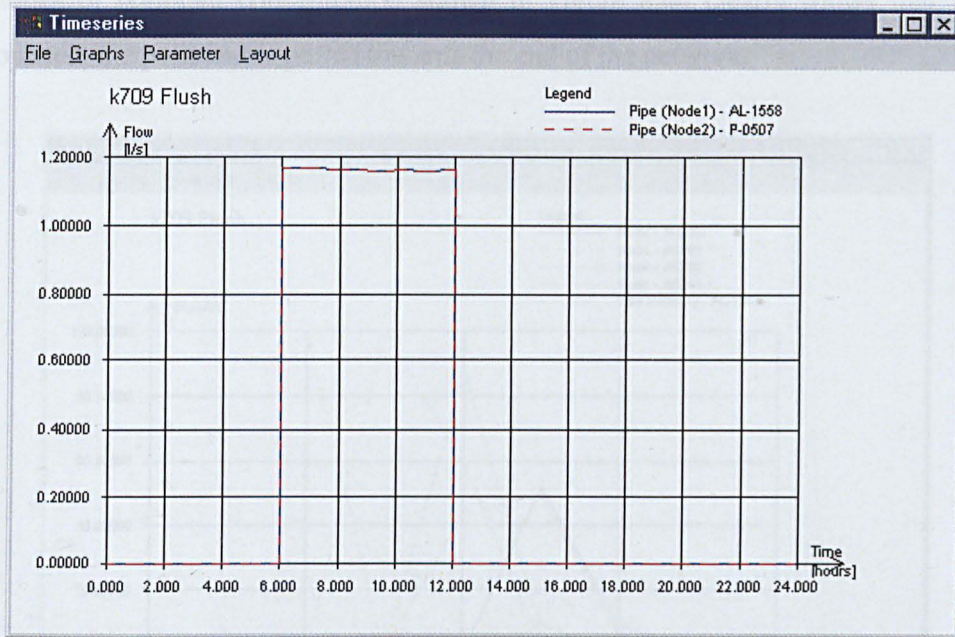


Figure 8.42 Hydrant flow during the flushing procedure

The hydrant flow can be adjusted for flow rate so that different flushing velocities can be achieved in the pipes. If there is enough pressure scouring velocities might be attained. Figure 8.43 highlights a slug of polluting material in a piece of main near the end of the network.

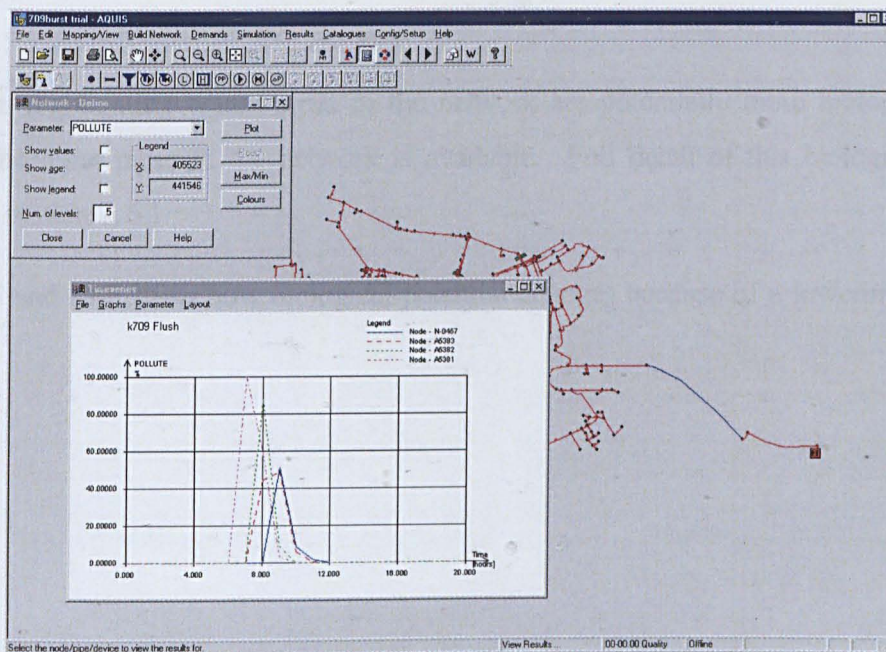


Figure 8.43 Pollution slug in a main near end of network.

The time series of pollutant concentration shown in Figure 8.44 clearly shows that when the hydrant is opened, the pollutant is pulled towards the end of the network.

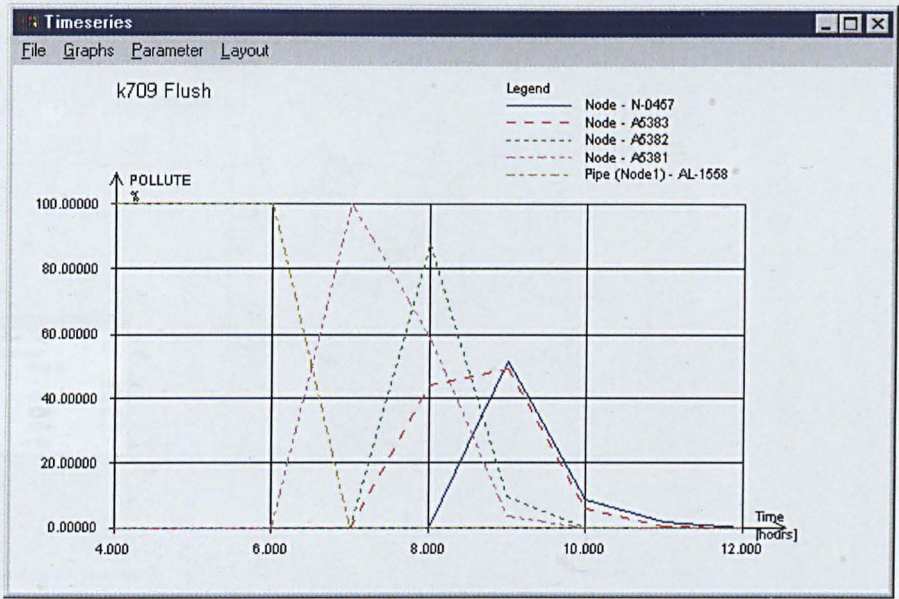


Figure 8.44 Pollutant level time series

The individual peaks represent the pollutant arriving and leaving nodes along the main towards the hydrant. Tabular output includes the flows from each hydrant (when more than one is opened) and the total volume flushed.

8.4.3.3.2.7 Biological activity

A simulation highlighting which pipes in the network are potentially more biologically active relative to the other pipes in the network is available. Full detail of this biological model is presented in section 7..6.1

Figures 8.45 and 8.46 show how biological potential changes because of a lowering of chlorine residual.

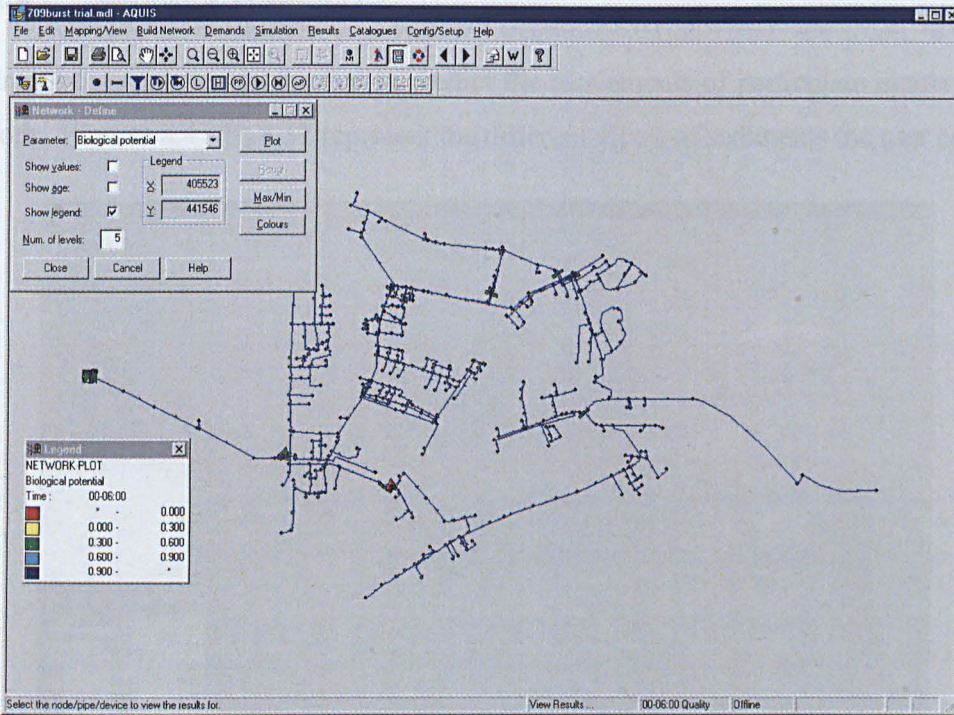


Figure 8.45 Biological potential where all pipes have same conditions.

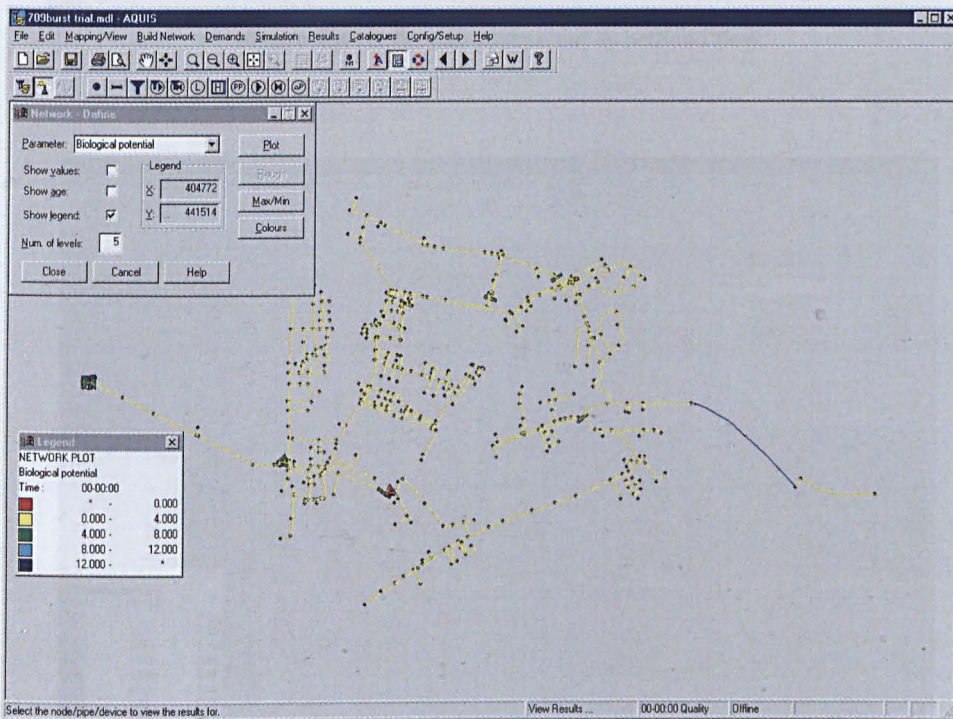


Figure 8.46 Biological potential where a single pipe has reduced chlorine residual

8.4.3.3.2.8 Sediment Transport

The sediment transport model is used to predict the movements of particulate matter throughout the network. Figures 8.47 To 8.51 represent the different views of sediments the user can obtain.

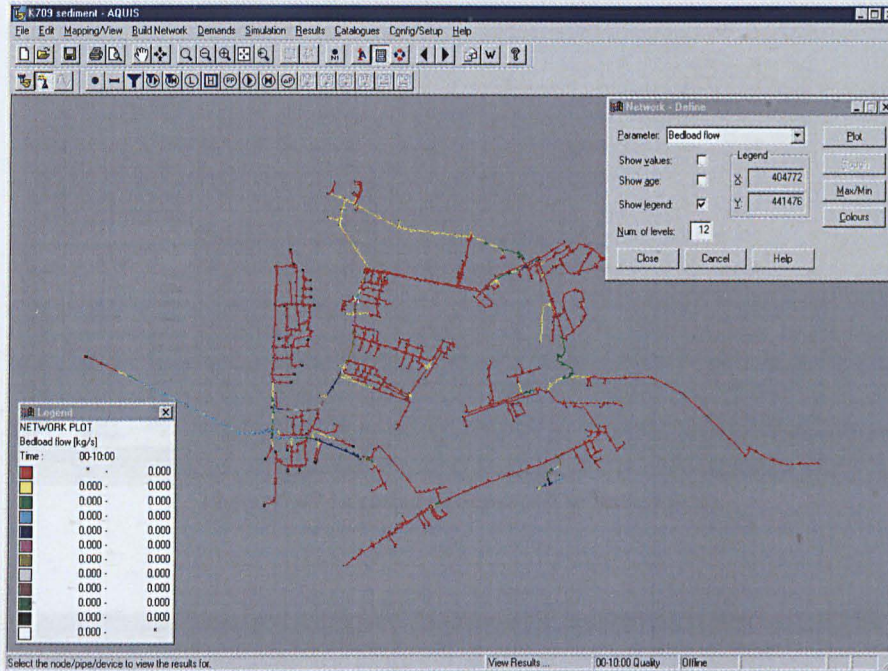


Figure 8.47 Sediment movement as bedload flow.



Figure 8.48 Sediment entrained in the bulk flow

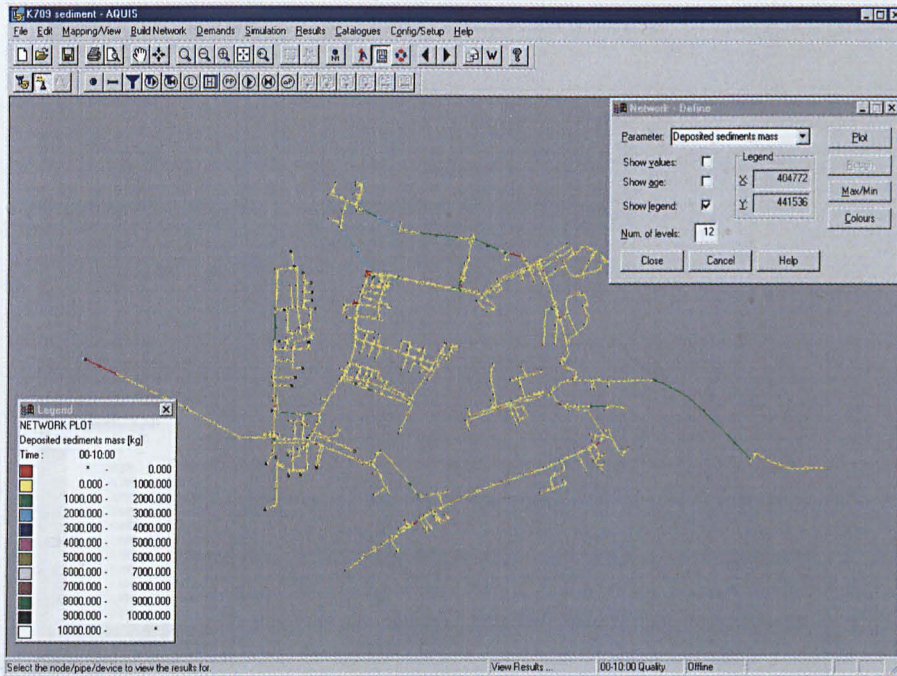


Figure 8.49 Location of deposited sediment mass



Figure 8.50 Deposited sediment fraction

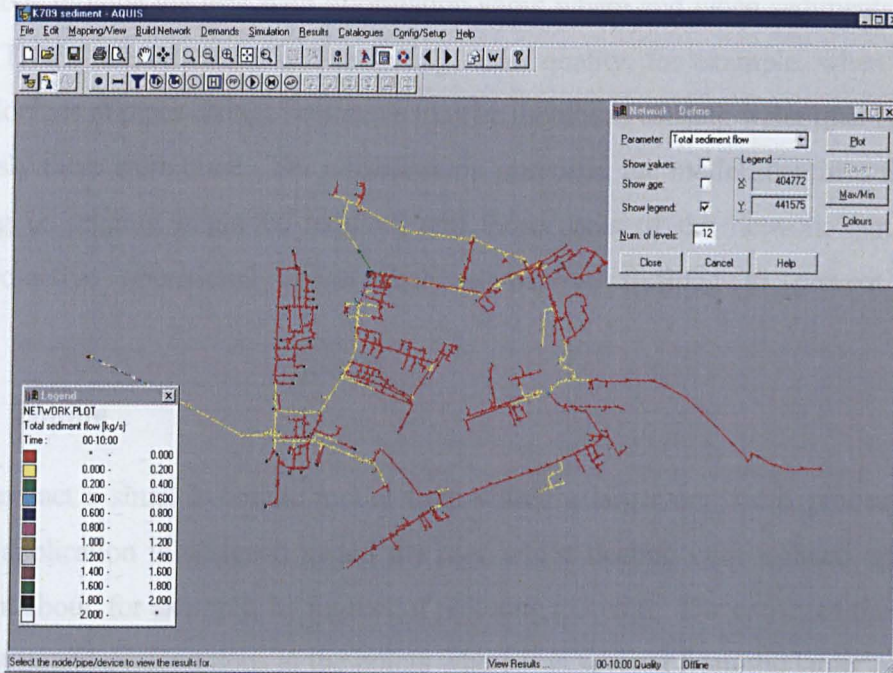


Figure 8.51 Total sediment flow

All this information is available at individual pipe level. Figure 8.52 is a time series of bedload flow in a pipe as an example.

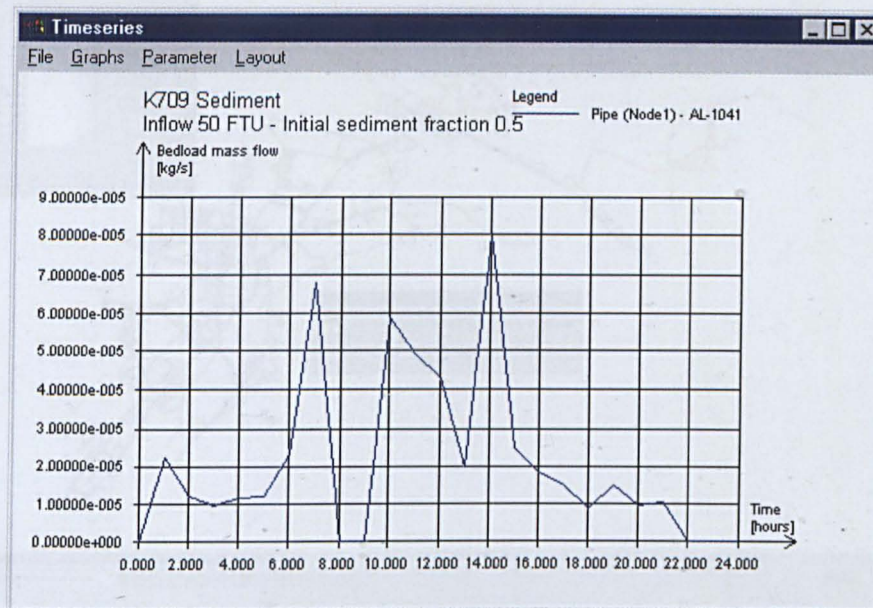


Figure 8.52 Bedload flow in a pipe

These simulations provide the user with information about where and when sediments will be in the network. This is important for understanding water quality, for example, when a valve is opened and velocities in pipes change sediments may be mobilised causing water quality problems where previously there were none. For rehabilitation purposes, the model identifies those pipes where scouring or pigging might be required and those areas of the network that may need continuous pro-active operational action such as passive flushing to prevent sediment accumulation.

8.4.3.3.2.9 Zoning

The user can extract a small hydraulic model from within a larger one - this process is called Zoning. This application is designed to aid the user whilst dealing with isolated areas of the network brought about, for example, by ingress of polluting material. The extracted model brings with it the latest boundary conditions at the points where it is severed from the larger model so it can be used for simulations immediately. Figure 8.53 shows a large model with the smaller model required highlighted in red.

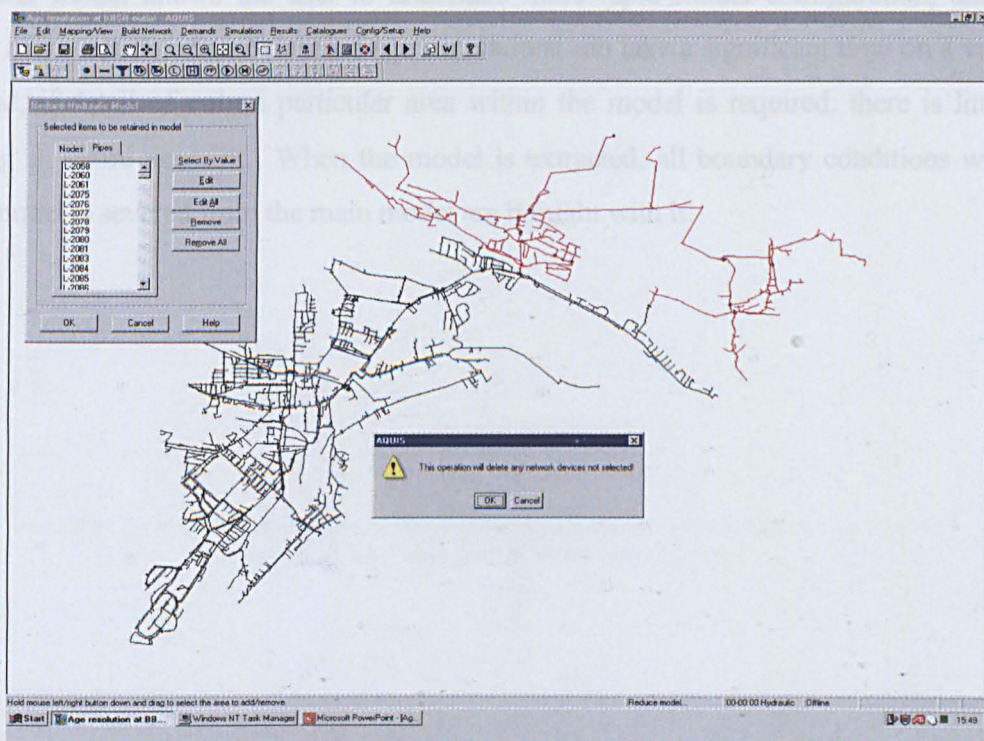


Figure 8.53 Identification of small model within a large model

Figure 8.54 is the newly extracted, smaller, model.

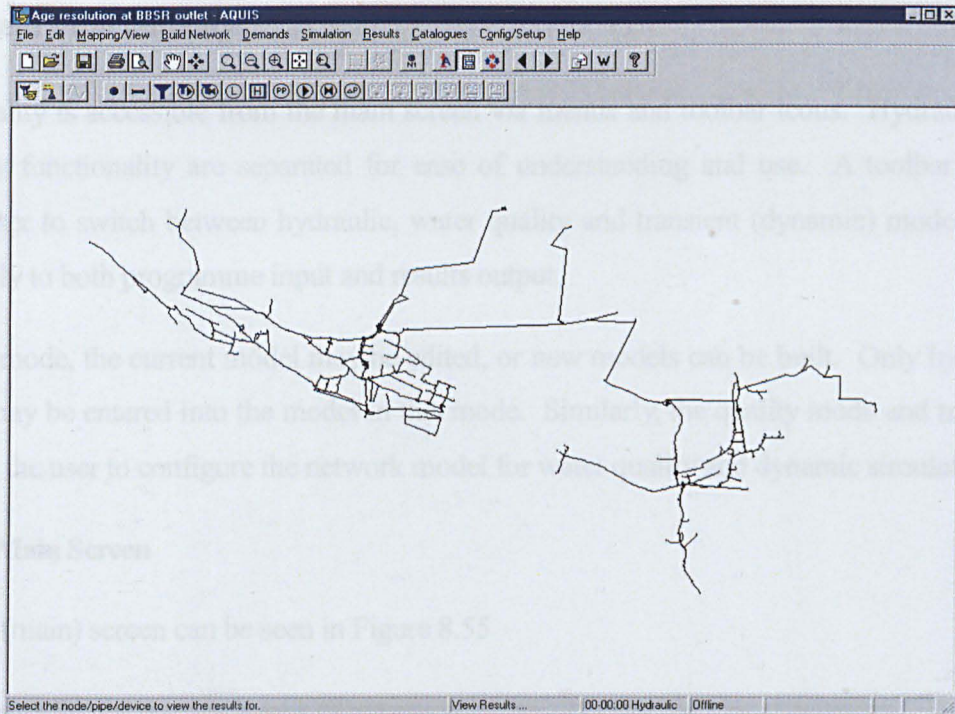


Figure 8.54 The reduced model.

The smaller model allows the user to undertake more rapid model configuration, and to run scenarios more rapidly. Some detailed age simulations can take a significant time on a very large model and, if detail of only a particular area within the model is required, there is little point simulating an entire network. When the model is extracted, all boundary conditions where the reduced model is severed from the main model are brought with it.

8.5 System Functionality

All functionality is accessible from the main screen via menus and toolbar icons. Hydraulic and water quality functionality are separated for ease of understanding and use. A toolbar button allows the user to switch between hydraulic, water quality and transient (dynamic) modes. The switches apply to both programme input and results output.

In hydraulic mode, the current model may be edited, or new models can be built. Only hydraulic parameters may be entered into the model in this mode. Similarly, the quality mode and transient modes allow the user to configure the network model for water quality and dynamic simulations.

8.5.1 The Main Screen

The opening (main) screen can be seen in Figure 8.55

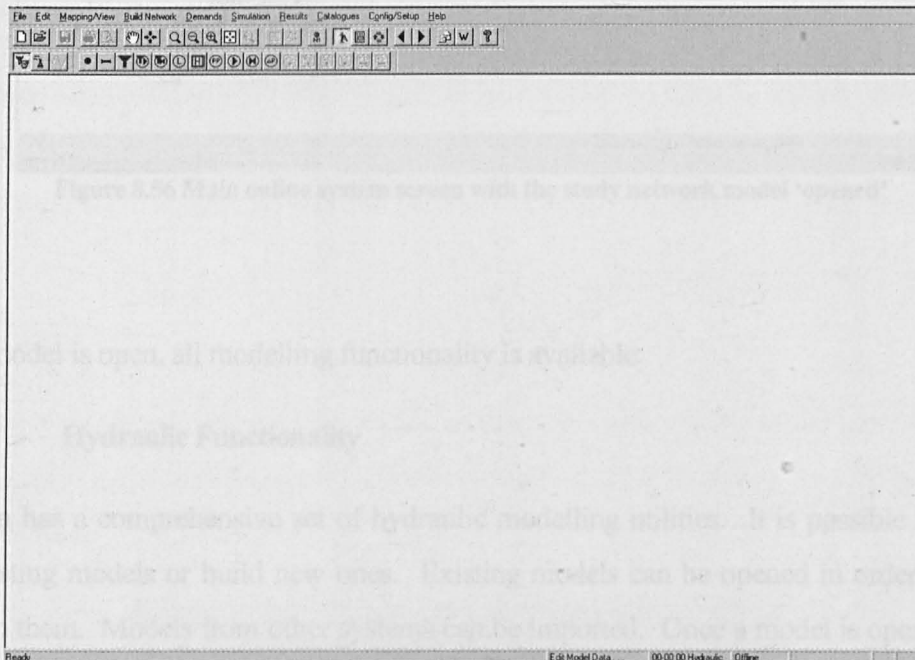


Figure 8.55 The main screen of the online system

From the opening screen, it is necessary to open an existing model or create a new model before any functionality, other than certain configuration options, is available. In Figure 8.55 it can be seen that most menu items are 'greyed out'.

Figure 8.56 shows the main screen and the availability of functionality once a model has been opened.

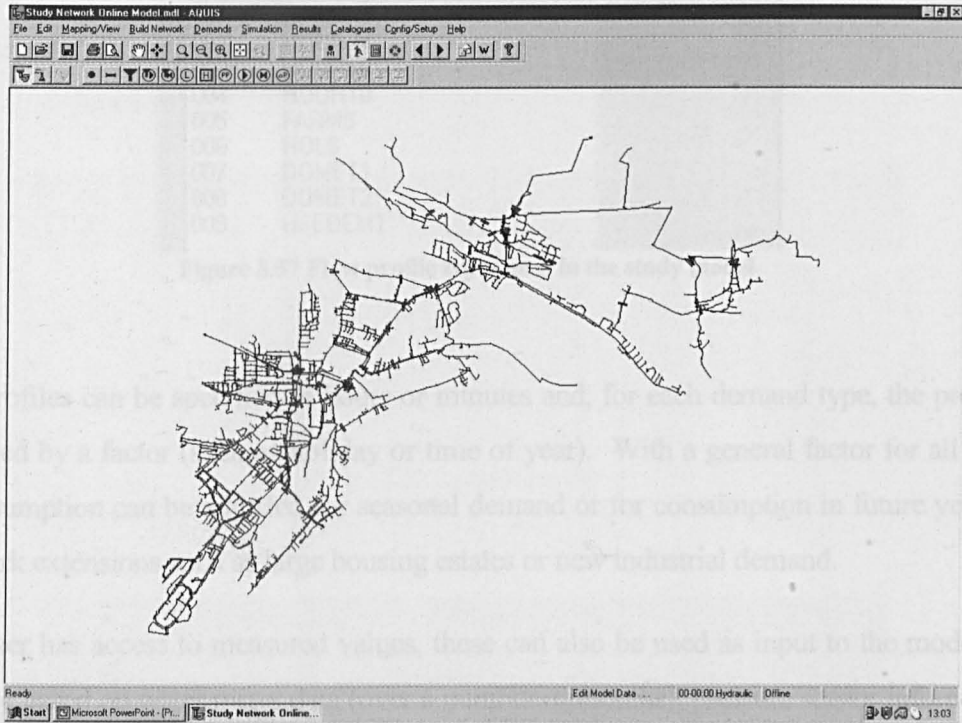


Figure 8.56 Main online system screen with the study network model 'opened'

Once the model is open, all modelling functionality is available.

8.5.2 Hydraulic Functionality

The system has a comprehensive set of hydraulic modelling utilities. It is possible to import or amend existing models or build new ones. Existing models can be opened in order to continue working on them. Models from other systems can be imported. Once a model is opened it can be edited and or configured and then used for simulations.

The hydraulic module can handle up to 200 different demand type profiles. Figure 8.57 is the flow factor dialogue box showing some of the different flow time series

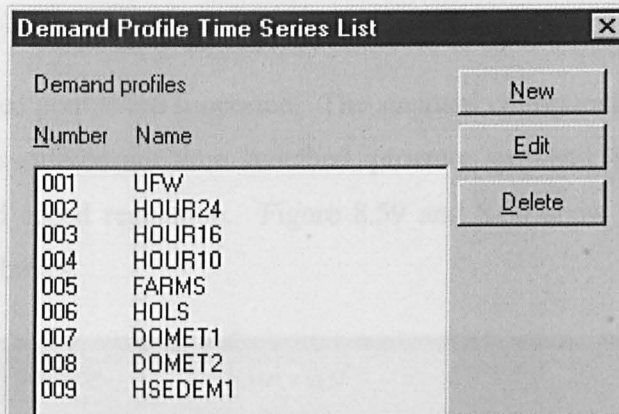


Figure 8.57 Flow profile types used in the study model

Demand profiles can be specified in hours or minutes and, for each demand type, the profile can be multiplied by a factor (such as holiday or time of year). With a general factor for all demand types, consumption can be adjusted for seasonal demand or for consumption in future years or to plan network extensions such as large housing estates or new industrial demand.

Where a user has access to measured values, these can also be used as input to the model. This input can be based on water meter data (over a number of months), pressure sensor information or water quality data. Also, time series from SCADA systems can be input directly from appropriate databases.

8.6.3 Leakage

A leak can be simulated by the model using the diameter of the 'hole' in the pipe that has burst, and the pressure-dependant leakage flow is calculated. Figure 8.58 shows the dialogue box where leaks are defined.

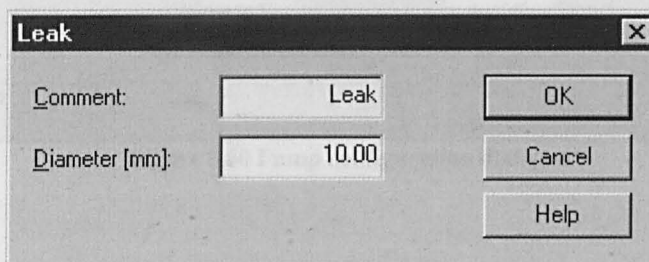


Figure 8.58 The leak dialogue box

8.6.5 Pumps

Fixed and variable speed pumps are supported. The standard configuration supports 300 pumps, and the pump controls offered are time switched, pressure switched, and level switched, and include time-controlled speed regulation. Figure 8.59 and 8.60 show the menu item and the configuration dialogue boxes.

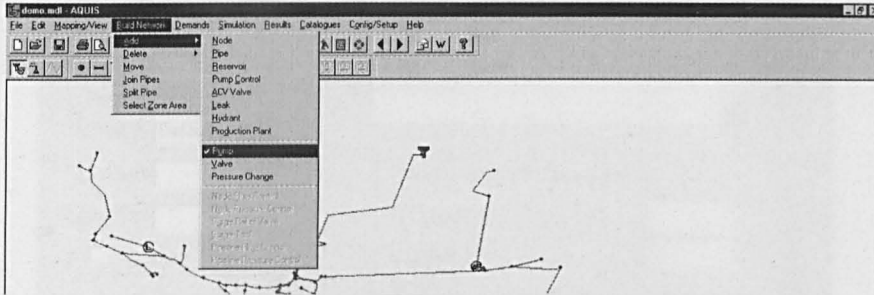


Figure 8.59 Pump menu items

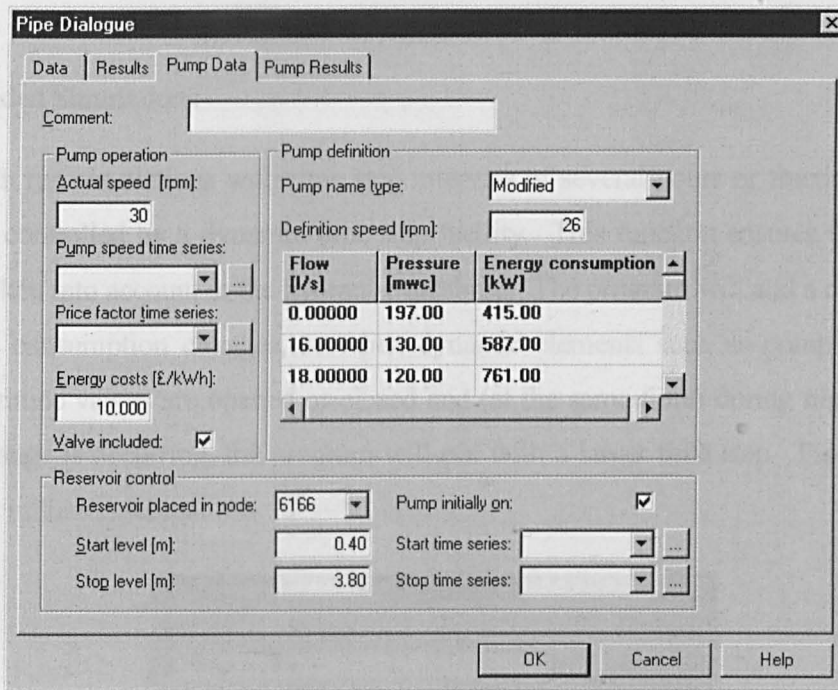


Figure 8.60 Pump configuration dialogue

8.6.5 Variable Volume Reservoirs

Variable volume reservoirs water towers can be modelled. Figure 8.61 shows how a reservoir volume / shape relationship is defined.

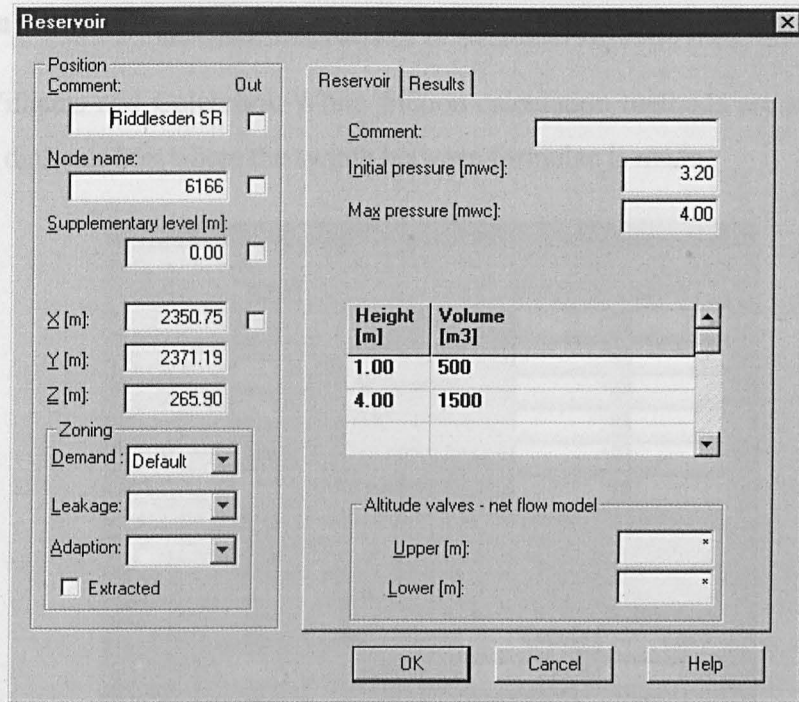


Figure 8.61 Reservoir volume / shape relationship definition

8.5.3 Extended Simulations

The model can run simulations with time step intervals of several hours or fractions of a minute automatically controlled by a dynamic time step facility. This function ensures that all dynamic changes are taken into account in the hydraulic database. The program will add a calculation point any time that consumption changes, or when dynamic elements such as pumps are started or stopped, or altitude valves are opened or closed and (at the same time) during night hours, when little or no change is occurring, the program will run with a larger time step. Figure 8.62 shows the simulation initiation screen.

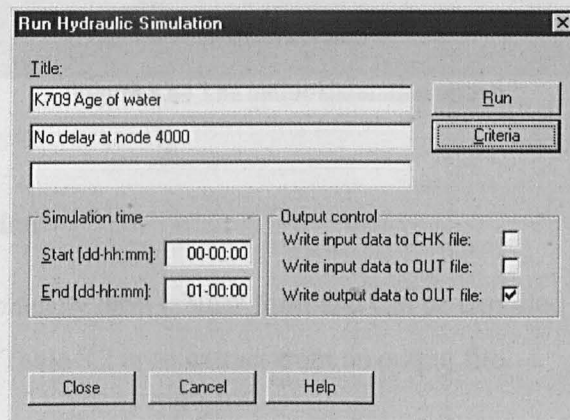


Figure 8.62 The simulation initiation screen.

8.5.4 Friction Formulas

Both Hazen-Williams and Colebrook-White friction calculation methods are supported. Figure 8.63 shows the dialogue box where the switch between formulae is made.

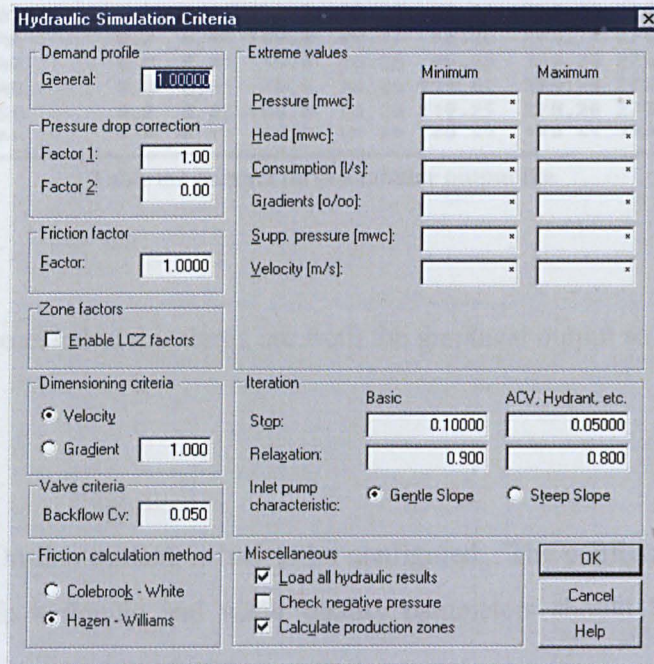


Figure 8.63 Hydraulic simulation criteria screen

The hydraulic and water quality models were configured as described in Chapter 7, however, there is an option to run online regardless of being in hydraulic or water quality mode as shown in Figure 8.64.

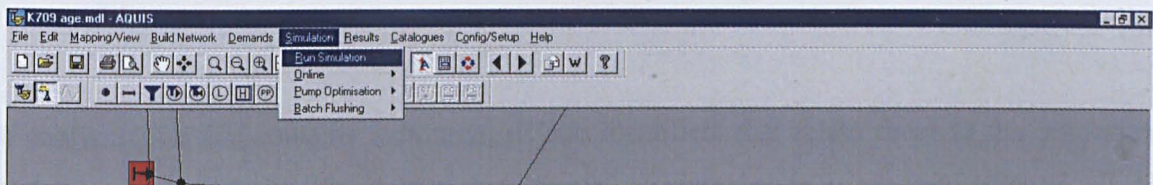


Figure 8.64 The online simulation option

8.5.5 Output Presentation

Results are presented graphically or in tabular form and can be exported in DXF format for import to CAD / GIS programs. Table 8.2 is an extract from an output file.

No	Node		Flow	Vel.	I.D.	Pressure		Head		Frict. Loss
	Ups.	Dws.				Ups.	Dws.	Ups.	Dws.	
			l/s	m/s	mm	mwc	mwc	mwc	mwc	oo/o
51	6045	6061	2.1	0.12	150.0	89.27	81.44	270.29	270.27	0.53
52	6045	6061	1.9	0.11	150.0	89.27	81.44	270.29	270.27	0.38
53	6045	6046	0.3	0.04	100.0	89.27	96.26	270.29	270.28	0.19
54	6046	6047	0.0	0.01	75.0	96.26	95.26	270.28	270.28	0.01
55	6046	6048	0.0	0.01	75.0	96.26	111.62	270.28	270.27	0.02
56	6046	6050	0.2	0.03	100.0	96.26	119.25	270.28	270.26	0.11
57	6040	6044	2.0	0.17	150.0	89.27	80.47	270.29	270.27	0.68

Table 8.2 Extract form a tabular output file

Many of the figures presented in this thesis are from the graphical output so none will be repeated here.

8.5.6 Configuration

Before operating the On-line system, it had to be configured. The configuration file, Table 8.3, tells the system which hydraulic and water quality parameters should be used as boundary conditions to drive the on-line simulations.

70201	Flo	6166	Reservoir	Flow	712	T2	C2	1	3.4	2.9	1	0	0	0	0
71201	Flo	P-2185	Pipe	Flow	712	T2	C2	1	20	-8	-1	0	0	0	0
71002	Pre	3000	Node	Pressure	712	T2	C2	1	100	92	1	0	0	0	0
70901	Pre	5000	Node	Pressure	709	T2	C2	1	60	42	1	0	0	0	0
70902	Pre	5448	Node	Pressure	709	T2	C2	1	50	32	1	0	0	0	0

Table 8.3 Extract form a configuration file

The configuration file contains measurement site identifiers that relate them to the associated downloaded form that site. In addition, it must also contain a node or pipe identifier to tell the system where the measurements are located in the model and the measurement type, for example pressure or flow. Each measured flow is associated to a Leakage Control Zone to ensure that the flow for that particular zone is attributed correctly to the various flow components.

Upper and lower measurement limits can be set for each measured parameter. If these limits are exceeded, a default value specified by the user will be used and the system will warn the user by initiating an alarm and popping it up on screen. Offset values and conversion factors for incoming data are also configured in this file.

The online software looks for and, if found, opens the data file containing hydraulic and water quality data created by the Data Management module. After data integrity and validation checks are completed, it performs simulations for current and predefined future timesteps. During the hydraulic simulation initialisation, the measured flows are adapted for each individual leakage control zone and the nodal allocation for demand scaled appropriately. During the simulation process, all input measurements and their status are on screen in a dialogue box. Figure 8.65



Figure 8.65 Operators screen showing field measurements dialogue box

8.6 Hydraulic Model Upgrade

The model upgrade process is described in Chapter 5.

8.7 Online Hydraulic Model Validation

The validation process involved comparison of measured pressure and flow data from the field-test against predicted results from the model. Result comparison from over one hundred pressure-monitoring points proved to be a time consuming task, but worthwhile, as it highlighted several network anomalies.

One of the key findings of the validation process was a suspected closed valve on the 250mm main from Albert Street feeding zone 709. A fit between predicted and measured pressures could not be achieved other than by placing a closed valve on the model along the length of main from

the bottom of Albert Street to Alice Street. Once the valve was closed in the model, an exact fit was achieved when comparing the predicted output from the model to the measured pressures gathered during the field test. The closed valve caused a resulting pressure loss of 6 mwc because the water had to travel down a smaller diameter pipe to other consumers in the zone.

In order to prove the model results were correct, loggers were placed at two monitoring points downstream of the location of the supposed closed valve. A member of the Operations staff then checked the valves on the stretch of main where the model had predicted the closed valve had to be present. The investigation confirmed that the valve was closed. When the valve was opened, the pressure increase was 6 mwc confirming the model prediction.

A second anomaly found during the validation process was the pressure-reducing valve in Albert Street. Logged data from the site showed that the pressure produced on the outlet was moving around erratically, therefore not producing the output expected in the model. Investigation showed the pressure-reducing valve to be faulty and it was replaced. Following replacement further data logging showed the output to be far more stable.

Several mains in the model that were supplied via the 315 mm trunk main in Highfield Lane in zone 713 had to be given very low C-values in order to achieve calibration. This was of concern because records had shown that scraping and lining had been undertaken in the area five years earlier so the mains should be in reasonable condition with relatively high C-values. Samples of the main were therefore taken from sites off Highfield Lane. One of the samples from a 3" cast iron main at Highfield Street showed a marked reduction of the internal diameter to less than an inch. Another sample from Belgrave Road was in much better condition clearly showing evidence of scraping and re-lining.

Finding operational anomalies of this nature gave an increased confidence in the predictive capability of the model. The validation process was completed following final amendments of the model to integrate the information obtained during the network investigations.

8.8 Hydraulically Tuning the On-line Model

8.8.1 Background

Validation of the model using historic data only proved its validity at the time the historic measurements were made. Dynamic changes in a distribution system mean that the pressure and flow characteristics change every minute of every day. To ensure the hydraulic base data used in water quality and online simulations was correct, the current network state presented by the online model was compared to field measurements over a period of two weeks.

All available measured flow and pressure data were used as boundary conditions in the online model. Checks on the validity of the hydraulic data were made over a number of days at different times. The validation information forms a large report in its own right and is available if required and not included in this thesis.

8.8.2 Results

The hydraulic results highlighted that some of the pressure readings from the water quality instruments differed by up to 5 mwc from the results predicted by the model. The reasons for this were:

The transducers in the water quality instruments were only accurate to 1% over a 100 mwc pressure range as opposed to 0.1 % in the hydraulic instrumentation.

The flow distribution method used in the model may cause differences in pressure due to allocation of flow to areas where there is actually little in reality.

All measurements were subject to fluctuations brought about by sudden changes in demand.

The pressure readings from Leakage Control Zone 709 were subject to fluctuations from the pressure-reducing valve.

8.9 Pollution Incident Management

To show how the online model may be used for proactive network management a series of studies were undertaken to provide contingency plans for pollution events.

In the late 1980s, a serious incident occurred at a water treatment plant in Southern England. A delivery of Aluminium Sulphate was erroneously tipped into a final water tank at a water treatment plant resulting in pollution of the associated distribution network and many people suffered illness as a result. This incident initiated a court case that lasted for over ten years. In the 1990s, the water industry was strongly criticised by OFWAT and others for not being prepared for this type of incident, or indeed any similar incident, and for not having contingency plans in place to consult should problems arise.

To demonstrate how contingency plans can be created with the help of the water quality model, it was used to investigate how much time would be available before consumers were affected should any part of the network become polluted. The information was used to determine how best to isolate the polluted water, how much time would be available to close appropriate valves and, where necessary, open others whilst maintaining a water supply to those users not affected.

The design of the network using the new approach was based upon zones being as independent as is practicably possible making isolation of a zone feasible without the need to shut off supplies to other associated zones. It is also possible to isolate a portion of a single zone should the topography of the network be favourable. Before this approach was applied however, the network had cascading zones that presented the problems highlighted in this example.

8.9.1 Methodology

It was not practical within the scope of this project to look at all possible pollution scenarios for the study network, so a selection of some important possibilities was investigated.

As example scenarios, the model was used to simulate the movement of a tracer substance through sections of the study network supplied by two of the service reservoirs and one of the water treatment plants. In the scenarios, the pollutant was defined as a conservative substance of known concentration. Because the input concentration of pollutant was known, the variation of the concentration of the substance with time was predicted for all pipes in the network. This information was used to highlight where water from the individual reservoirs travelled with respect to each other, at what concentrations, and where any mixing of water occurred.

Knowing which consumers would be affected, when, and at what concentration, provided valuable information enabling the impact of such events to be minimised through effective contingency planning.

Knowing where a pollutant will travel from a particular point in the network with respect to time allows the identification of key valves that can be closed in order to isolate the polluted water before it reaches the consumers. The model was used therefore to define hypothetical incidents in order to study how they develop with time, and to determine the most effective way of managing them thereby providing information from which to design contingency plans.

Functionality still being developed within the model is a flushing programme. This model allows the identification of valves in the network that should be opened to remove the polluting material most effectively and calculates how much water would be used during the flushing process. The model was used to determine how long it would take to remove the polluting material and how much water would be used during the operation for the zones fed by one of the service reservoirs.

8.9.2 Scenario 1 Bracken Bank Service Reservoir

Bracken Bank Service Reservoir is a major storage facility supplying the study network. At the time of the study, it received water directly from a water treatment plant and supplied zones 709, 710, 711, and 712. These zones were in a cascaded configuration at that time, and comprised a significant proportion of the study network providing a good example for the demonstration of this functionality.

Initially, the model was used to simulate network hydraulic and water quality characteristics over a 48-hour period to predict the extent of the effects of a polluting material entering Bracken Bank Service Reservoir at a concentration of 200 mg.l⁻¹ , over a 2-hour period beginning at midnight. The results of the simulation indicated that, in less than 24 hours, the pollutant would affect 7,700 consumers. Figure 8.66 represents the distribution of the pollutant after 2 hours.

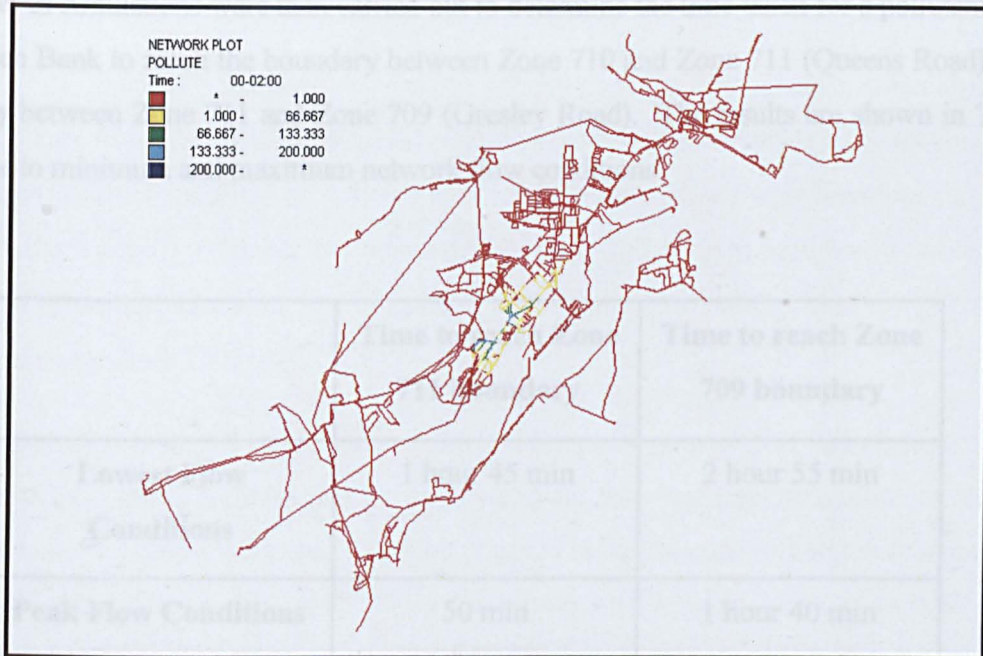


Figure 8.66 Pollutant distribution after 2 hours

Figure 8.67 shows the pollutant distribution after 12 hours.

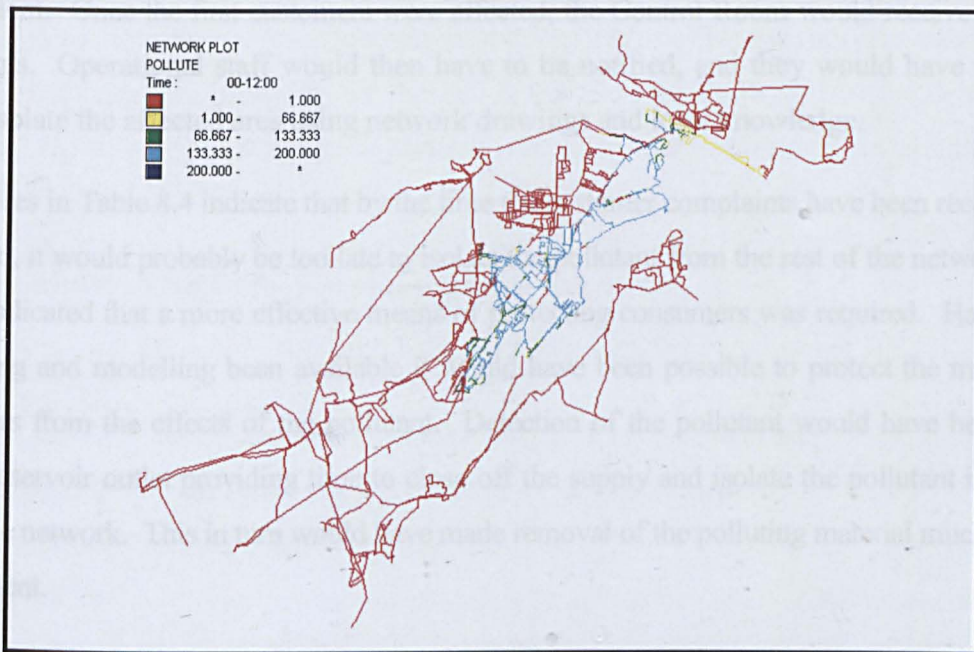


Figure 8.67 Pollutant distribution after 12 hours

A number of simulations were then carried out to determine the time taken for a pollutant entering at Bracken Bank to reach the boundary between Zone 710 and Zone 711 (Queens Road), and the boundary between Zone 711 and Zone 709 (Gresley Road). The results are shown in Table 8.4 and relate to minimum and maximum network flow conditions.

	Time to reach Zone 711 boundary	Time to reach Zone 709 boundary
Lowest Flow Conditions	1 hour 45 min	2 hour 55 min
Peak Flow Conditions	50 min	1 hour 40 min

Table 8.4 Time of travel for pollutant at low and high flow conditions.

The time required to react to an incident of this nature is composed of two elements, the time to detect that an incident has taken place, and the time to mobilise resources to take action to contain the problem. Once the first customers were affected, the Control Room would receive customer complaints. Operational staff would then have to be notified, and they would have to decide how to isolate the affected area using network drawings and local knowledge.

The Figures in Table 8.4 indicate that by the time the customer complaints have been received and processed, it would probably be too late to isolate the pollutant from the rest of the network. This clearly indicated that a more effective means of protecting consumers was required. Had on-line monitoring and modelling been available it would have been possible to protect the majority of consumers from the effects of the pollutant. Detection of the pollutant would have been at the service reservoir outlet providing time to close off the supply and isolate the pollutant in a small part of the network. This in turn would have made removal of the polluting material much simpler and efficient.

8.9.2.1 Associated hydraulic considerations

Because of the cascading nature of the leakage control zones in this example, when sluice valves between zones were closed to contain the pollutant, an alternative means of supplying the then isolated downstream zones had to be found. Hydraulic investigations were therefore carried out to determine the feasibility of using the Riddlesden to Black Hill main to back feed to zones 709 and 711 and, if necessary, use water from this main to flush the contaminant from the affected pipes, including those in zone 710.

Pressures available from the Riddlesden to Black Hill main were such that there was a potential to cause bursts by opening a connecting valve between the main and leakage control zones 709 and 711. A pressure-reducing valve was therefore required at the connection point with a downstream setting of 70 mwc if zones 709 and 711 were to be supplied from this source. However, if zone 710 were also to be supplied, then the setting would have to be increased to 105 mwc.

The flow required to supply the three leakage control zones from the Riddlesden to Black Hill main was 30 l.s^{-1} . This equated to a bulk flow of 2592 m^3 per day. The water treatment plant supplying the service reservoir had the capacity to provide the water volume but the implications of this quantity of water being taken from Riddlesden Service Reservoir had to be assessed.

Re-zoning work necessary to manage an incident in one area may have an adverse effect upon the supplies to other areas. To effectively manage the incident all these effects would have to be calculated. In this example, it would be possible to supply the extra demand from Graincliff Water Treatment Plant.

8.9.2.2 Summary of the findings for the peak flow condition

- 08.00 Pollutant leaves Bracken Bank Service Reservoir
- 08.20 Half of Zone 710 affected - 1,600 properties.
- 08.30 First complaints received.
- 08.50 Pollution reaches boundary to next zone (711) - 3000 properties affected
- 09.00 Local Distribution staff notified of problem.
- 09.40 Pollution reaches boundary to Zone 709 - 4500 properties affected.
- 09.45 Zone valve between Zone 710 and Zone 711 closed - too late.

- 10.40 Pollution reaches Zone 712 - all zones supplied by Bracken Bank now affected - 7,700 properties.

8.9.2.3 Flushing

The model was used to determine the most effective flushing regime and the length of time necessary for the pollutant concentration to fall to a pre-defined lower concentration limit. (The minimum acceptable pollutant concentration to which the model should resolve, can be set by the user). By running this scenario, it was possible to develop an efficient action plan that optimised the use of the incident management resources and minimising impact on consumers.

The model was configured to calculate of the length of time required for the pollutant to be flushed from the network to an acceptable level. This was done by identifying the locations at which water was to be flushed from the network and defining demands at the selected locations that were representative of the orifice of the hydrants or washouts to be used to flush the contaminated water from the network.

A hydraulic simulation was then run to determine the resultant network hydraulic characteristics. A quality simulation, based upon these hydraulic results, was then initiated starting with a defined concentration of the pollutant within the pipe work. The simulation stops when this concentration has been reached in all pipes.

An important point considered was that the flow from a number of open hydrants can be considerable, and could have resulted in low water levels in the service reservoirs feeding the area being flushed and low pressures within the network. The results of the hydraulic simulation were therefore carefully examined to assess the impact upon the supply in general and, when necessary, the hydrant flows were throttled in the model by reducing the defined orifice size.

This then defined the maximum flows that could be imposed on the network in order to effect flushing without causing secondary problems.

In this particular scenario, flushing was achieved by back feeding towards Bracken Bank service reservoir (assumed to be isolated) from the cross-town main connection. It was found that to flush the pollutant out 12 hours after its introduction, 10 throttled hydrants (10mm orifice diameter) were required at the flushing points.

This number of hydrants being flushed simultaneously resulted in the use of 2360 m³ of water over a 24-hour period. The impact of supplying this considerable volume of water from the Riddlesden to Black Hill trunk main had to be assessed. It was found however that the main could support the necessary flow, as it was under-utilised in its current role in the operational regime of the network.

The online model is a useful tool for assisting with the design of contingency plans. The flushing functionality identifies pipes from which where pollutant cannot be removed. New flushing locations can then be identified to remove the remaining contaminant. It can give an indication of the flows from each hydrant required to achieve flushing in a certain period, and it can indicate the volume of water required to achieve the flushing. The object of the exercise is to find out exactly which hydrants to open at what flow automatically - otherwise we might as well open them at what we feel is right and save time.

The time for the pollutant to leave the network with hydrants located at the positions in figure 6.3.5 was just under 24 hours. However, the program did identify a small number of dead end mains that would have to be individually flushed.

By closing the valves that separate individual leakage control zones, it was possible to isolate the pollutant. For example, valves at Queens Road and Gresley Road, which separate zones 710 / 711 and 711 / 709 respectively. In this scenario, it was fortunate that a single valve could be closed in order to stop the further spread of the contaminant.

8.9.3 Scenario 2 Highfield Service Reservoir.

Highfield Service Reservoir feeds by gravity directly into Zone 713 then, via a flow modulated pressure reducing valve, feeds part of zone 709. If a pollutant enters the Service Reservoir, it would be necessary to close the boundary valve between Zone 713 and Zone 709 at the point where the pressure-reducing valve was located.

Model simulations were undertaken to determine the travel time of the pollutant between the service reservoir and the boundary valve to zone 709. The results are shown in Table 8.5

	Time to reach Zone 709 boundary
Minimum Flow Condition	50 min
Peak Flow Conditions	30 min

Table 8.5 Travel time for pollutant at low and high flow rates

The results show that if the pollutant could not be contained within the times taken to reach the boundary with 709, then the whole area would be contaminated and flushing would be the only remedy. Given that the length of time before consumers are affected by the pollutant is very short in this case, particularly at peak flows, it is highly unlikely that anything could be done reactively in time. If however on-line monitoring and modelling were in operation, it is feasible that proactive management could significantly minimise the effects on consumers. This is particularly true if both inlet and outlet of the service reservoir were monitored and the pollutant was introduced from the upstream supply to the service reservoir.

8.9.4 Scenario 3 Sladen Valley Water Treatment Plant

Sladen Valley Water Treatment Plant provided a water supply to both Bracken Bank service reservoir and to White Lane service reservoir. This source was therefore critical to the supply of water for much of the study distribution network and, if a pollution incident occurred, it would be vital to prevent the contaminated water from reaching the Service Reservoirs.

The supply to Bracken Bank service reservoir from the water treatment plant was by gravity, and a small number of properties are fed directly from the trunk main connecting the two sites. A simulation predicted that the time taken for the pollutant leaving the outlet of Sladen Valley Water Treatment Plant to reach Bracken Bank service reservoir would be 2 hours and 15 minutes.

Once the pollutant was present in the whole of the transfer main, the volume of water to be flushed out was calculated as 500 m³. This was required to be flushed out from the lowest point on the main and would take 3 hours at a flushing rate of 50 l.sec⁻¹.

The supply to White Lane service reservoir was pumped, controlled by the level in the service reservoir. The travel time of water from the treatment works to the service reservoir was therefore dependant on the operation of the pumps. With the pumps operating, the time taken for water to

reach the White Lane site is approximately 1 hour 30 minutes and the quantity of water required to flush the whole main was 375 m³.

In the reconfigured network, the cascading arrangement of Zones 710, 711, 709 and 712 fed from Bracken Bank service reservoir was removed. The reservoir therefore only supplies a reduced area of zone 710 in the reconfigured network. This has implications in terms of the severity of the effects of a pollution incident at Bracken Bank. The number of properties that could be affected by such an incident is much lower than in the case of the cascading zone arrangement, but the time available to react is decreased. However, isolating the zone would not result in loss of supply to the other zones.

8.10 Summary of on-line monitoring and modelling

The development on the on-line monitoring model effectively brings together the component models into a tool that may be used for operational decision-making. This allows the management of systems to change from one of incident reactive management to one of controlled proactive management. This therefore describes the most novel and original element of the thesis.

The model will usually be applied to networks that operate normally but the major advantage is that, should an incident occur, for example major bursts, pollution incidents, unauthorised uses, pump failures, zone boundary breaches, changes in source water etc, the model may be used to provide detailed information to manage the system in near real time and thereby to minimise customer impact.

In addition the application of the diagnostic model based on the outputs of the near real time model may be used to retrospectively assess the location of the source of, for example, a pollutant or a discoloured water event.

The other major value of the model is the development of contingency plans for any hydraulic or chemical event that may be hydraulically simulated

Chapter 9 - Conclusions and Further Work

9.1 Conclusions

A review of literature identified a shortfall in the understanding of the concepts and the processes associated with water quality in distribution. The aim of this thesis was to develop such a quality model with a view to its application in near real time.

A study distribution network was selected that contained seven service reservoirs, four pumping stations, and one hundred and twenty kilometres of pipe made of a variety of materials of different ages and condition.

The network was selected because it contained all the problems associated with a typical distribution network. These included leakage control zones suffering from low and high pressure, a variety of water quality problems such as taste and odour, discolouration, biological problems, and a significant number of main bursts. The network had a mixture of domestic and industrial users having a variety of demand types and was supplied from three different water treatment plants.

The network was constructed over a long period of time, in a piecemeal fashion, with little regard to how new, local changes, would impact on the system as a whole. Little, if any, regard was taken of the effects of the schemes on surge generation or water quality.

9.1.1 Instrumentation and Monitoring

The network was monitored for flow at twenty-eight locations using ABB flow-meters and Spectralog data-loggers. As well as these network flows, a number of major industrial flows were also measured.

In the case where pressure transients were recorded, use was made of two sophisticated, high speed, Radcom Centurion instruments, logging at a frequency of 10 Hz, positioned at key locations identified using the model.

Forty-eight locations were also monitored using water quality instrumentation designed specifically for this study. Measurements of pH, conductivity, dissolved oxygen, turbidity, redox

potential and water and air temperature were taken continually over a period of a year. These measurements were taken at fifteen-minute intervals over a period of one year.

This instrumentation provided some excellent quantitative and qualitative data that was subsequently used to calibrate and verify the model and to provide near real time boundary conditions for the online application of the model.

9.1.2 Existing network problems

The network was known to have the following problems;

Low-pressure areas

High-pressure areas (some unnecessarily very high)

And to alleviate these problems intervention strategies were required. Traditionally these interventions were completed at a local level whereby only that part of the network local to, and influenced by, the intervention was modelled. No regard of how this may impact all the other parts of the supply system were taken.

9.1.3 Hydraulic analysis

Hydraulic analysis of the network was completed in two ways. First, a traditional approach was used to assess what changes to the network would be required to alleviate the problems outlined above at a local scale.

This was followed by a similar analysis, in which the entire network was simulated as a complete system.

The results of this analysis showed that a traditional approach to network scheme design was not able to accurately represent the effects of local network interventions, and could not resolve the network problems as effectively as an holistic approach. It was concluded therefore, that to accurately describe the hydraulic performance of the system for all interventions, for all pipes in the system, a fully integrated complete network model was required. The integrated approach was shown to produce a far more effective solution and, at the same time, take due cognisance of the effects of surge generating events and overall water quality.

9.1.3 Leakage

It has been shown that the new integrated produces significantly better results than the traditional approach with regard to design of pressure control for leakage management. A 23% saving was achieved using the traditional approach and this was increased to 41 % using the new approach.

9.1.4 Transient Analysis

The model included a routine to simulate pressure transients within the network. By recording pressure data at high frequency upstream and downstream of a pumping station and upstream and downstream of sluice valves, the model was shown to be able to accurately predict the shape of recorded pressure transients but not always the magnitude.

It was concluded that the latter discrepancies were due to the lack of information concerning the pipe material and its characteristics, and the moments of inertia of the pump sets. Acceptable figures for the moments of inertia were calculated by accurately measuring the transient effect of switching a pump on and off at a number of speeds and using them to configure the model.

9.1.5 Water Quality Analysis

Previously, distribution network management has taken little regard to the quality of water being transferred through the pipes. Historically it was quantity, not quality, that was of concern. More recently, water quality has become a high priority and the thesis has described the development of a comprehensive suit of water quality models to predict the propagation of conservative and non-conservative substances. The concept of the model is based on the age of the water and travel time, taking due regard of the conservative processes of dilution and dispersion, and the decay of non-conservative substances.

Tracer studies using Sodium Chloride have been used to calibrate the model and excellent agreement was obtained between measured and model predicted values.

The advantage in the use of this model is that an integrated holistic approach can now be adopted that allows not only solutions to be derived base don pressure and flow but also solutions that take into account the implications on water quality.

A sensitivity analysis was completed to assess the effect of different variables (decomposition, physical and transformation) on the performance of the model. From this analysis, it was

concluded that within the range of parameters found in practice the primary variables to influence the model were:

The decomposition (decay) rate constant but mainly a function of temperature and bulk decay

Pipe wall coefficient (a function of disinfection history)

Temperature and pressure

Bulk water volume decay is a function of the source water

The decomposition (decay) rate constant determines the slope of the decay curve. The effects have been demonstrated over five orders of magnitude thereby making the number of possible values of the decay constant almost infinite providing a high degree of model flexibility

Reactions with and / or at the pipe wall are accounted for by inclusion of a pipe wall coefficient and a molecular diffusivity component. The much larger effect of the pipe wall coefficient and the other factors swamp the small effect of the contribution from the molecular diffusivity when combined in the overall decay constant.

Temperature and pressure are both accounted for in the model and are both assumed to add proportionally to the decay rate constant. There are also factors for temperature and pressure dependency that multiply the effects of both variables making the model extremely flexible with respect to range of configuration. The magnitude of the temperature and pressure effects can therefore be set by the user.

The age model has been shown to be accurate, through calibration and testing, using tracer studies and empirical retention time calculations. The model is useful in that it can provide comprehensive age analysis for an entire distribution network.

The sensitivity of the age of water analysis can be set by the user and can range from very coarse, to extremely fine. This provides an extremely versatile tool to help with the understanding of the relationship between age of water and water quality problems such as discolouration, taste and odour or bacteriological issues.

The above effects can be added in any combined to provide extreme overall model flexibility thereby making it easier to calibrate models for different networks with differing physical, chemical and biological properties. Figure 7.98 shows the effect of the default exponential decay constant on a non-conservative substance at a temperature of 10 °C.

None of the variables in the model are fixed; they are all user definable so, as better information becomes available, any constant value can be input to the model without the need to re-code.

An original feature of the model that has been developed is the diagnostic capability, whereby it is possible to use the information on measured water quality and to link this back to the source of occurrence.

9.1.6 Online Monitoring and Modelling

The hydraulic and water quality models have been utilised in the development of an online modelling tool that describes a new and original approach to the way in which water networks may be operated and managed.

The application of the model has been demonstrated to show that it is feasible to move from a reactive to a pro-active network management philosophy.

For example, the model has been successfully used, although not described in the thesis, to detect and locate bursts as they occurred, thereby avoiding the expensive and time consuming use of manual data collection and analysis, and leakage location teams. Also, discolouration events have been tracked as they travelled through the system.

Artificial intelligence has since been used to enhance these model capabilities developed as part of this thesis. (*Mounce 2000*).

9.2 Future Work

Although the biological model is quite well developed, a significant amount of work is still required to obtain network data from which to identify the relationships between the various factors, particularly the relationship between biofilm and organisms in the planktonic phase and their overall impact on the general water quality within the network. The model identifies the pipes that are more biologically active than others are, but does not automatically trace where the active material will travel, although the propagation functionality could be used as a second step to

determine where the biological material will travel. However, even in its current form, it is a useful tool to identify where the risk of biological activity is likely to be highest.

As the data becomes available, the model can be amended to reflect the current state of the art in this area because of the flexibility built into its design.

The sediment transport model that has been developed has been set up in a flexible way to include aspects of settlement of suspended particles (precipitation) with no bed load transport, transport in suspension and by bed load movement, transport in suspension and flushing (scouring). The thesis has presented details of the bed load sub model but further work is required to obtain network data against which the model may be calibrated / validated.

The diagnostic model in its current form is very useful however; further work is required to enable the model to accurately pinpoint the location of an event such as discolouration or pollution ingress. The online monitoring and modelling system could provide the necessary data.

Clearly there is a need for much further work to assess the interactions between the biology, chemistry and the physical characteristics of both the network and the water within the network. For example, little is known about the role of quality within service reservoirs and the way in which the outputs of these interact with the distribution network.

Ultimately, it should be possible to monitor and model from source to tap providing the water companies with a much enhanced capability to shift from reactive to proactive management of entire water supply systems.