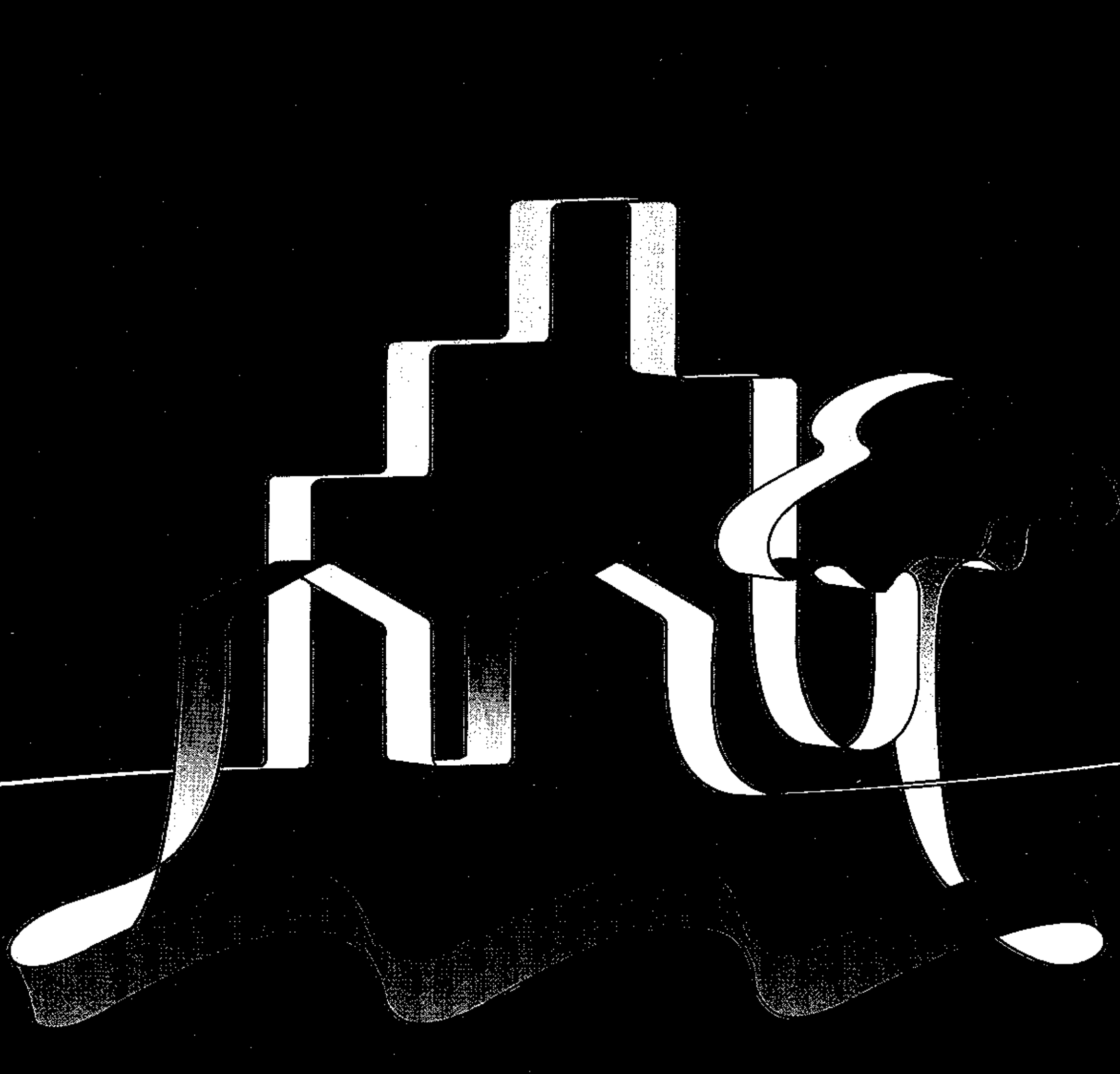
Urban Water Research Association of Australia

Predicting the Failure Performance of Individual Water Mains



Research Report No. 114

URBAN WATER RESEARCH ASSOCIATION OF AUSTRALIA (UWRAA)

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Predicting the Failure Performance of Individual Water Mains

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F O R E W O R D

This report is based on UWRAA Research Project No AM-23: 'Predicting the Failure Performance of Individual Water Mains" which was undertaken during the period July 1994 - April 1995. Organisational responsibility for the project was as follows:

Sponsoring Authority:	Melbourne Water
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Research Agency:	Royal Melbourne Institue of Technology
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The project was funded by the Urban Water Research Association of Australia, RMIT, Melbourne Water and Sydney Water.

SYNOPSIS

Water-supply mains have been examined to determine the optimum timing for the replacement of a deteriorating asset. It has been found that, with suitable filtering of raw **failure data** for a pipe asset, it is possible to make reasonable predictions of the future failure behaviour of the asset. Drawing on work in data filtering by Minetti (1995) and by Constantine and Darroch (1995) for partial data sets, models have been developed which predict the time to the next failure and then suggest whether the main should be repaired or replaced in order to minimise life-cycle cost.

The report incorporates a set of guidelines for operators of water-supply mains. These can be applied by any Authority to a main with a history of four or more failures. Authorities are expected to refine these models according to their own operating environment and specific corporate goals. However, there is a strong recommendation that a standard format be adopted for the collection of performance data so that future research projects in this area can be assured of data consistency across the country (see Appendix B).

The findings of this project will enable more effective management of water-distribution networks by ensuring that a main is replaced at the end of its economic life.

Keywords in this study are: asset management; water-supply pipe; failure; pipe replacement rules; data filtering; economic model.

Earlier the Urban Water Research Association of Australia (UWRAA) research has indicated the need to develop a reliable means to predict the failure performance of individual water mains. The project team has analysed the records of small-diameter asbestos-cement and cast-iron pipes and from the analysis developed failure and decision support models. The end product is a set of guidelines which operators can use to decide how to minimise the life-cycle cost of a main under their control.

A fundamental requirement of any water-main failure model is that it should represent the aging, or gradual deterioration, of the pipe in its operating environment. To develop a model on this basis it is necessary to remove data relating to externally imposed factors such as bad repair of a previous failure, faulty operation of the supply system, accidental damage during adjacent excavation work and intentional damage. To achieve this filtering, a process has been developed which screens the raw performance history data and leaves the failures due to inherent pipe characteristics. The latter are then employed in the failure prediction model to forecast the time to the next failure.

After extensive trials with data supplied by Melbourne Water and the Sydney Water Board, a model with an exponential form that predicts interfailure time was selected as the most appropriate. This failure model requires the full **burst** history of the main. In this regard, information on four or more failures are needed following the filtering process. If this is not available, the reader is referred to the companion UWRAA study presented in Research Report AM22 (Constantine and Darroch, 1995), which provides a failure prediction model for incomplete datasets.

The basic building block of predictive modelling is an accurate database of failure data on all mains.

The report presents results from a survey of water authorities that examined the data currently being collected in Australia. A major point coming from this survey was the obvious need for a set of standard definitions and fields for failure databases. It is strongly recommended that a standard be developed to provide consistency across the Australian water industry. A suggested format is provided in Appendix B.

The survey of water authorities also covered the currently used procedures for economic evaluation of water mains, and the replace/repair decision in particular. Greater emphasis is now being placed upon indirect costs associated with burst water mains and the total cost to the community. Using this information and applying the failure forecasting model developed in the earlier part of the project, a decision model was developed which employs the concept of **Total Failure Cost** to decide whether to repair or replace a failing main. Computer software has been written to assist authorities apply the model and this is available from the Infrastructure Research Unit, Department of Civil and Geological Engineering, RMIT.

The findings of this project are incorporated in a set of guidelines providing operators with a package of tools aimed at helping them minimise life-cycle costs. Advice on the collection and interpretation of performance data is followed by instructions on the application of filtering, the failure prediction process and the determination of an optimal replacement date.

It is suggested that authorities apply the guidelines included in this report and with local data, over a period of time, refine them to better manage their resources. The guidelines can be used immediately but they must be converted to your own environment using your own failure data.

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1.3 Perceived Benefits

The benefits of this study are expected to be:

- improved accuracy in predicting the performance of individual water mains;
- a rigorous process to identify replacement candidates;
- a heightened awareness of the need to maintain accurate and complete databases on burst mains within Water Supply Authorities.

1.4 Scope of the Report

The Research Project consists of:

- a literature search, a study of pipe-manufacturing methods and a review of relevant Australian Standards;
- filtering of data-base information supplied by Melbourne Water and the Sydney Water Board;
- development and testing of failure models;
- investigation and development of a decision model;
- preparation of a final report.

Two, two-day workshops were held in Melbourne (29, 30 September, 1994 and 9, 10 February, 1995) to assist in the study. A summary of these sessions is provided in Appendix H. This study used data from two municipalities in Melbourne and Sydney, however the techniques developed will apply equally well to data held by other authorities.

1.5 Scenario

You are the manager of a Water Distribution Branch of a Water Supply Authority.

As part of the management system of your network one of your customers (an unpaid observer) rings and advises you that a water main has burst outside their house at 10 Sprattly Street.

Your **data receiver** drags the on-screen area map that shows the address and confirms details with the caller, prints out a work-order copy for the file and simultaneously lodges the work order on the foreman's computer including a map showing valve locations for the primary shut-off block and, in a different colour, the next set of valves required in case any of the primary set is inoperable.

The display shows a graph of the previous pipe failures against time, indicates that some of the failures were due to non-structural causes and filters them from the analysis. It also shows tolerance parameters on the shape of the time/failure curve for the pipe. These parameters have been calculated allowing for the pipe material, diameter, soil conditions, climatic conditions and external loads, but the operator is unaware of this. The operator examines the graph and notes that the pipe is outside the allowable tolerances for structural breaks. Clicking onto the next screen shows the Present Value of the Total Future Cost of the main in graphical form. This graph indicated that costs to the authority will be minimised by replacing the main in two years time. The operator refers the main to the replacement priority list for managerial consideration.

The foreman duly repairs the pipe, recording both the materials used and the time taken on a **data recorder**, which also records the location of the leak to the nearest 0.1 m. Since the data recorder works under water, the foreman is most unlikely to delay the data recording until the next morning with a consequent reliance on memory, which is what frequently happens in current asset-management systems. The foreman also notes that one primary set valve was defective but that the secondary shut-off valve set functioned.

The customer database is accordingly upgraded to show the accumulated number of supply losses (both scheduled and unscheduled) to each of the clients in the primary shut-off area and the relevant part of the secondary shut-off area. A warning is issued that the authority has now exceeded the Custom contract (maximum number of unscheduled shut-offs per year) and the accounting department is advised so that the appropriate rate rebates can be applied. Maps are also drawn showing, in colour, clients who have received the maximum allowable shut-offs (without rate rebate) and for those who have one more allowable failure.

From time to time you have updated the standard estimated costs of repair from the recorded cost information, and at least annually reviewed the network status for budget and replacement priority. All the data you need are available graphically and the mains on the replacement list are also shown. The listing includes costs to your authority for rate reduction liability.

The above scenario would have seemed straight out of science fiction only fifteen years ago. Today we can accept the plausibility of the concept. This report describes research work which takes us to the threshold of developing the data-management software which will enable prototype management systems to be in place in the near future.

1.6 Outcomes from the Project

The research work carried out at RMIT was aimed at developing a **decision model** to aid managers of water-supply systems to decide whether to replace or repair individual deteriorating water-reticulation mains. Specifically, this research related to individual pipelines, not complete networks or groups of assets.

A number of computer tools have been developed to assist in this process. These include:

- an XL macro for filtering burst data;
- an XL solver application for determining the parameters of the failure model;
- a DOS-based program for the decision model.

These computer tools are demonstrated in the Case Study (Appendix I) and it is suggested that this Appendix be read before the body of the report for easier comprehension.

2 LITERATURE REVIEW

The search for literature has been conducted by scanning international electronic databases and by appeals for assistance via the Internet. Details of search techniques used for source data are given in Appendix A.

References have generally originated in the USA, Canada, the UK or Australia although there have been occasional listings from Germany and France. The following section provides a summary of the contents of the major relevant sources.

A previous search was carried out by Minetti (1993) who reported on contributing factors to failures, filtering of failure data, structural performance prediction, cost models and renewal strategies.

2.1 Factors Contributing to Failures

Water mains fail due to a combination of circumstances, which include the pipe material, the environment in which it is laid and the operating characteristics. Morris (1967) lists possible causes of water-main structural failures including soil aggressiveness, soil stability, weather conditions, bedding conditions, construction quality and land development. Cobb (1969) suggests internal corrosion, pressure surges and faulty anchorages at branches, bends and dead ends. Other authors, including Shamir and Howard (1979), extend the list by adding manufacturing flaws, traffic loading, pressure and water hammer. The importance of joints is raised by Marks et al (1987) and Damassue (1993) with the latter observing that the rapid deterioration of joints leads to leakages which can lead to structural failure. During a study of the New York water supply system the US Army Engineers (1980) found prior leakage increased the moisture content of the surrounding soil and promoted corrosion. Weather related breaks are also mentioned in the literature, ranging from the severe winter conditions in North America to the drought conditions in Australia. Clark, Eilers and Goodrich (1988) provide an analysis of break rates due to freezing ground conditions and Miller (1985) reports that soil shrink/swell has an effect on water main breaks in Sydney. This finding is also reported in Melbourne by Hatfield (1987) who provides evidence that seasonal fluctuation is significant in areas of fault activity. In a summary of pipeline failures the Water Research Centre (1994) claim they may be caused by a single catastrophic event but most often they are caused by *a continuing combination of age, overstressing and/or local adverse environmental conditions*. This source provides a useful tabulation of causes and effects of common failures and this is reproduced in table 2.1.

TABLE 2.1

CAUSES AND EFFECTS OF COMMON PIPE FAILURES

Pipe manufacturing defects	Inclusions, discontinuities or sporadic processing problems. Dimensional irregularities particularly in jointing areas.
Storage and handling	Stress deformation due to poor stacking or storage. Cuts or scratches on pipe walls or coatings. Impact cracks due to dropping or striking. UV degradation, over-weathering or contamination. Inadvertent mixing of pipe class or jointing material.
During construction	Poor laying, jointing or tapping techniques. Excessive overburden/soil slip causing distortion. Construction traffic. Groundwater flooding.
Subsequent works	*Over*: superimposed loadings, impacts or cover reduction. *Adjacent*: side slips, service pulling or moling. *Under*: loss of support or bedding.
Soil movement	Subsidence due to mining, filled land etc. Differential consolidation or geological changes. Changes in water table/soil moisture content. Extremes of climate: frost heave or clay shrinkage. Loss of anchorage/support (horizontal or vertical). Shock waves: seismic, blasting or vibration.
Soil environment	Poor original bedding/backfill. Pointloads. Migration of bedding or sidefill material. External chemical attack from natural soil. Chemical attack due to spillages. Deterioration of the joint sealing ring(s).
Impact damage	*Direct*: strike with bucket, pick or point (includes pulling). *Indirect*: point bearing in bedding or surround. Traffic loading causing fatigue or joint movement.
Temperature changes	*Expansion*: excess compression, end crushing. *Contraction*: pull out or separation of joint. *Freezing*: pipe blockages and splits.
Internal pressure	*Static*: excess testing pressure. *Dynamic*: pressure surge, water separation, vacuum collapse. Pipe tapping whilst under bending stress.
Others	Aggressive waters causing internal deterioration. Galvanic corrosion due to dissimilar metals.

2.2 Filtering of Failures

There are very few references in the literature to the filtering of failure data and those that do exist report it as a partial or informal screening process. Evins (1989) recommends the filtering of historical data relating to accidental damage and 'irrelevant' entries. Whereas Shamir and Howard (1979) merely warn against problems caused by aggregating data assumed to be homogeneous. On the other hand, anecdotal evidence suggests filtering is performed by most Authorities in an informal way based on the experience of the analyst. However, it is clear there has been no previous attempt to formalise the process in the way proposed by this process.

2.3 Structural Performance Prediction

The move by infrastructure Authorities throughout the world to manage their assets more effectively has seen an escalation in the number of technical references relating to asset condition assessment and performance. In the water-reticulation literature the focus has been on the forecasting of pipe failures, generally for network level decision making.

Walski and Pelliccia (1981) provide a description of research carried out by the US Corps of Engineers to develop quantitative rules for the replacement of a failed pipe. The study reported was conducted in Binghamton, New York State, and it was concluded that frost loads were significant for this distribution system. Further, it was found that there was a relatively low rate of change in the break rate. It was argued that this indicated that corrosion was not a severe problem for this network. The report presents a set of equations to predict the rate of breaks of pipes depending on their diameter, age, material (pit cast-iron and sand-spun cast-iron), temperature and previous break history. This approach is not suitable to individual mains.

Fitzgerald (1986) states that corrosion-related breaks increase exponentially with age in individual mains while breaks resulting from other causes are almost independent of pipe age. Tsui (1993) found that age and soil type are the main factors to be taken into account for performance prediction, but he suggests it is not enough to produce an accurate predictive model because of the interaction with other causative factors. He described the failure of water mains as a probabilistic process spread over time. He stated that condition monitoring is not warranted for the bulk of water-distribution systems and the building of statistical models is the most cost effective way to predict the behaviour of non-critical mains. Critical mains are defined as pipe sections whose potential loss is high and, as a consequence, they are generally large transmission mains. This concept has been well documented in the UWRAA Report No.33.

Tsui presents a forecasting model resulting from **regression analysis** using aggregate data:

$$R = e^{(a+bn)} \qquad \text{...(1)}$$

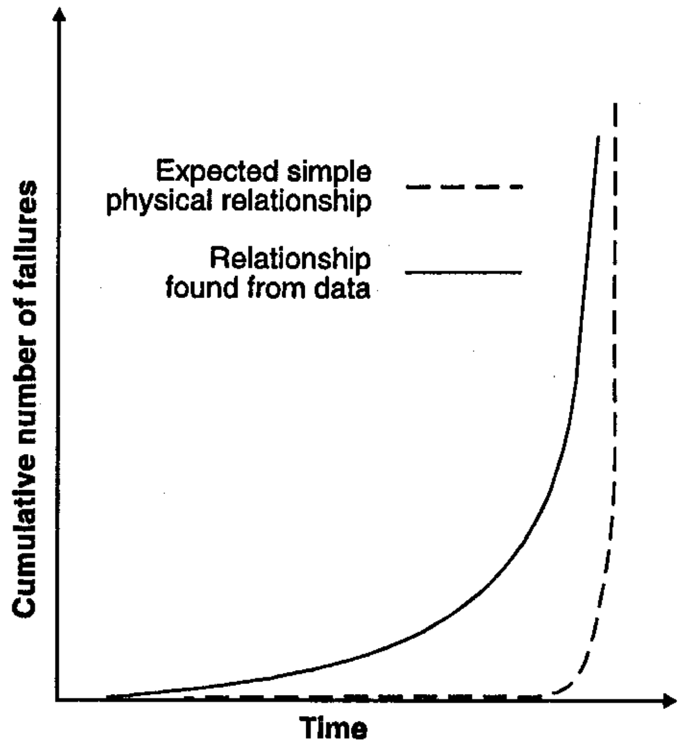
where:

R is the break rate per 100 km;
a, b are constants;
n is the age of the pipe in years.

This approach is suitable for strategic planning of networks, but the very wide variation in the behaviour of pipe sections results in it not being applicable to the management of individual mains.

Figure 2.1

Failure forecast for a group of assets using an exponential model



Shamir and Howard (1979) and Walski and Pelliccia (1982) both describe exponential models but application of these models to individual mains has not been tested. The former proposed an exponential forecasting model for water-main break rates. This model was developed at an aggregate level.

$$N(t) = N(t_0)e^{[A(t-t_0)]} \qquad \text{.......... (2)}$$

where:

N(t) is the number of breaks per 1000 feet in year *t*;
t is the current year;
t_0 is the base year when the pipe was installed;
A is the growth rate coefficient (regression analysis).

Walski and Pelliccia (1981) found that pipe type, diameter, age, occurrence of previous breaks and mean temperature in the coldest month of the year under study have a significant influence on break rates.

They employed an exponential model:

$$N(t) = C1 \times C2 \times Be^{[A(t-t_0)]} \qquad \text{(3)}$$

where:

N(t) is the break rate per year per mile;

C1 is a correction factor for previous breaks;

C2 is a correction factor for pipe size;

A and *B* are regression coefficients;

t_0 is the year of pipe installation.

The statistical validity of this model was questioned by Marks et al (1987) since there was no statistical test of significance on the correction factors.

All the above models are based on **regression analysis** of aggregate data and forecast failure rates per year per kilometre (mile). However, an individual main 200 metres in length, failing for the first time at 20 years of age and then at 25, 29, 32 and 34 years cannot be modelled by a rate approach. Rates of failure per kilometre of pipeline are easily generated from mass data but they cannot be used to describe the failure process of individual water mains.

Clark et al (1982) report a *repair frequency analysis* for distribution-system maintenance events (leaks and breaks). This study includes survival analysis, probability of maintenance events, maintenance-event equations, economic analysis of replacement and the impact on water quality of failure rate. The survival analysis produced mortality curves and these showed that *over a period of 40 years, 52.5% of the pipes ... had no maintenance events, 47.5% had at least one maintenance event, and 30% had at least two maintenance events* . The analysis indicated, for the network studied, a minority of the pipes were responsible for the majority of the maintenance events. The examination of probability of failure resulted in the conclusion that the time interval between one event and the next became increasingly shorter with age (as the number of maintenance events increases). The interval between repairs was plotted as an exponential function. The analysis assumed that no breaks occurred within the first ten years after construction since the first maintenance event did not 'usually' happen until fifteen years after the pipe had been laid. The lag between the laying and the first event and the exponential form of the gap between later events were represented by two "event equations". The economic analysis considered the factors influencing the reliability of water distribution systems and associated costs for repair and replacement. Conclusions drawn from the study include:

Large diameter pipes tend to have a longer period before the first maintenance event than do smaller diameter pipes.

In areas of industrial development the time until the first maintenance event is less than that for other types of development.

Amount of development increases repeat breaks.

However the authors were not very confident that the data used were reliable

...data from the Utilities and Planning Authorities lacked consistency and completeness.

There is no reference to the names of the Authorities supplying the data.

Survival analysis has been used by some analysts. This statistical technique gives the probability of a pipe section failing when the complete performance history for that class of assets is known. The time intervals between consecutive failures for a given class of assets can be modelled by a statistical distribution function. Then the probability of failure in the next N years for a pipe section with n historical failures to date can be calculated as the percentage of pipe sections in the class under study, with n historical breaks, which have failed within N years.

Clark et al (1982) used survival analysis to gain an insight into the behaviour of an entire network and observed that a minority of pipes are responsible for the majority of maintenance events.

Marks et al (1987) proposed a survival model based on a non-parametric multivariate model (Cox proportional hazard regression model). In this case the model was applied to the first three breaks.

Scott (1993) used survival analysis for strategic planning. He concluded that the results from the model provide a high-level strategic view or solution and are not relevant to individual pipeline issues.

However, the requirement that the complete failure history for a class of assets be available for survival analysis limits its application. When failure history is unknown until a certain point in time (left-censored data) as is the case in Sydney and Melbourne (asbestos-cement Sutton pipes), survival analysis cannot be employed.

Marks, Andreou and Jeffrey (1987) and Scott (1993) also employed survival analysis but, because data are required for the whole population, this approach is not suitable for performance prediction of individual mains.

Kettler and Goulter (1985) used data from the Cities of Philadelphia and New York in establishing a strong negative correlation between pipe size and failure rates for cast-iron pipes. This correlation, which was also found by O'Day (1982), was attributed to the increasing wall thickness and reliability of the joints in larger diameter pipes. It was also concluded that pipe material and local conditions (soil properties, depth of frost, and construction standards) had a significant influence on failure rates for this material. However, no significant variation in failure rate with pipe diameter was found for asbestos-cement pipes. The analysis showed *considerable variation over time in the changes in rates of breakage for the different modes of failure for both cast iron and asbestos cement pipe.* For cast iron, the primary type of failure was joint failure whereas for asbestos cement it was circular crack failure.

Goulter et al (1993) presents an attempt to develop a predictive model of water-main failure rates. This model is different in that it incorporates not only the temporal variation but also the spatial distribution of pipe breakages. The authors use data from the City of Winnipeg in Canada to define *the mean number of failures that have subsequent failures for specified intervals in time and space around these initial breaks.*

Non-linear regression is then used *to derive the values for coefficients of an equation that captures the changes in the mean number of subsequent failures with variation in the time and space intervals. The equation thus obtained through regression is used to further derive equations for the variation in the failure rate as a function of time or space.*

2.4 Failure Forecasting for Individual Pipelines

Few references to failure forecasting models for individual pipelines have been found in the literature.

Clark et al (1982) used a model for the time to first failure (with correlation coefficient r = 0.23) and another model for maintenance events after first failure (with correlation coefficient r = 0.47):

$$NY = a + bD - cP - dI - eRES - fLH + gT \quad \text{.....} \quad (4)$$

where:

NY is the time to first failure;
D is diameter;
P is absolute pressure;
I is the percent of pipe overlain by industrial development;
RES is the percent of pipe overlain by residential development;
LH is the length of pipe in highly corrosive soil;
T is the pipe type (1 = metallic, 0 = concrete);
a, b, c, d, e, f, g are regression coefficients.

$$N(t) = Ke^{[A(t-t_0)]} \qquad \text{.......................................(5)}$$

where:

N(t) is the number of maintenance events after the first failure;
$K = \alpha e^{\beta T} e^{\chi PRD} e^{\delta DEV} SL^{\varepsilon} SH^{\phi}$;
PRD is the pressure differential;
DEV is the percent of low or moderately corrosive land over the pipe;
SL is the surface area of pipe in low corrosive soil;
SH is surface area of pipe in high corrosive soil;
α, β, χ, δ, ε, ϕ are regression coefficients;
T is the pipe type (1 = metallic, 0 = concrete);
A is the growth rate coefficient (regression);
t is the number of years from installation;
t_0 is the number of years until first repair.

This model is applicable to individual feeder and transmission cast-iron mains.

Hatfield (1987) favoured a power equation to predict the failure of metallic pipes.

$$N(t) = at^b \qquad \text{.......................................(6)}$$

where:

N(t) is the number of repairs up to age *t*;
a, *b* are regression coefficients.

This model proved satisfactory for large water delivery mains. He concluded in his report that prediction for specific mains cannot be generated from mass data.

Evins (1989) stated that the record failures over the last five years provides the most reliable and economic method of identifying the relatively small number of non-critical mains that are likely to fail frequently over the next five years. After filtering external failures, he assumes for an individual pipe section that the same rate of failures is going to happen over the next five years. His method does address the issue of individual pipelines. However, five years is a very short term prediction and his assumption was not verified when undertaking case studies with Melbourne data.

As a conclusion, many predictive models have been developed for strategic planning and budgeting. Most models are based on regression or survival analysis of aggregate data and they forecast rates of failures per year per kilometre. They may achieve a good performance prediction for an overall system. However, these models cannot forecast the failure of individual water-reticulation mains because variables related to pipe characteristics, environmental conditions and modes of operation introduce a large variability in individual pipe section behaviour. The future performance of critical mains may be assessed by costly means such as condition monitoring but they represent a very small proportion of reticulation mains.

Evins and Warren (1988) stated that there is a poor correlation between measurements of the condition of a pipeline and measurements of the performance of that pipeline such as its burst rate. They concluded that prediction of the behaviour of water mains should be based on performance analysis rather than condition monitoring.

Furthermore, Hatfield (1987) pointed out that weather-related breaks cannot be predicted by condition monitoring and that the location where the combination of many intangible factors is likely to bring a burst is poorly correlated with pipe condition. It is more cost effective to develop statistical forecasting models, which can be applied to all pipes in the network.

The very few existing failure forecasting models for individual pipelines have been developed from the analysis of large water-delivery mains. Further research needs to be undertaken to develop models applicable to water-reticulation mains.

2.5 Cost Models

The literature on the evaluation of water supply assets includes a number of references on the assessment of pipe-line replacement/repair options and their ranking. In the most recent publications there is a trend towards the inclusion of indirect costs such as those due to environmental and social impacts. UWRAA Reports No.17 (Vass et al, 1990) and 57 (PPK, 1993) are two examples of this approach to cost analysis.

2.5.1 Repair/replace decision

In the past water Authorities had replacement policies that involved doing little or nothing until they received a significant reaction in terms of complaints from the general public to frequent supply interruptions. These policies were gradually replaced by ones that included ... *the effects of hazards to the safety of persons and property in terms of legal implications* (Cobb, 1969), ... *criteria for pipeline replacement including customer relations and traffic hazard* (Stach, 1978), ... *important subjective factors such as inconvenience caused by interruption of service and excavation in the street, loss of pressure for firefighting during a break repair, possible contamination of drinking water during repair, subsidence at the break site and icy conditions resulting from leaks reaching the road surface* (Walski and Pelliccia, 1982), ... *indirect costs associated with service disruption, firefighting capacity reduction, property damage and public health resulting from water-reticulation main breaks* (Clark, Stafford and Goodrich, 1982), ... *political and social costs* ... (Hatfield, 1987) and ... *disruption to the community* ... (Evins, 1989). Most recently UWRAA Report AM14, 1993, suggests the total cost of a break in a pipe section should include water Authority costs - *site repair costs, site restoration costs, value of lost water, loss of revenue, third-party damage (including environment), loss of public image, cost of alternative supply and other Authority services (drainage, customer service)* and community cost - *loss of amenity, cost of restarting critical operations (e.g. petrochemical plants), traffic diversion costs (for citizens), other external services (police, road Authority, Telecom) and media costs for high use roads.*

In terms of the decision making process, most writers turn to standard economic analysis techniques. Shamir and Howard (1979) employed discounted cash flow analysis with a failure rate forecasting parametric model to compare the present value of all maintenance costs with the present value of replacing the asset. The optimal timing for replacement is the time that minimises the total present cost. This is taken further by Walski and Pelliccia (1982) who developed criteria for replacement based on break rates and present value analysis. O'Day (1982) stated that the 'useful life' estimate (found to be 60 years for New York City) that is, the optimum age for replacement - is not a realistic criterion for replacement and that there is a very large variability in condition and performance between facilities built at the same time. He added that reliance on averages and generalisations prove more costly than making decisions based on specific information about facilities and called for an individual approach to the rehabilitation issue.

Evins (1989) considered the standard of service as the only objective to be met by non-critical water-reticulation mains. He assessed the disruption to the public by taking into account not only the main under study but other pipe sections nearby whose performance may affect the same customers (same shut-off block). When the level of service is not up to standard, remedial actions are investigated.

As for critical mains, a lot of effort is put into the assessment of risk of failure. Any critical main is classified into a risk category (low, medium or high) based on established guidelines. The list of critical mains is reviewed each year. Mains associated with a medium or high risk of failure are recommended for cost/benefit analysis of remedial actions.

Vass et al (1993) stated that a net present value approach is superior to others in terms of life-cycle management. Survival analysis was applied to sewer failure data and the results used in a net present value analysis for strategic planning and budgeting.

Scott (1993) developed the replacement criterion *replace after n failures* for water-reticulation networks in Melbourne but stated that it was not relevant to preventative maintenance of individual pipe sections.

Tsui (1991) stated that asset renewal should be based on cost effectiveness principles and also on service standards. He concluded that net present value techniques are suitable to calculate the optimum time for replacement.

Scott (1993) stated that the replacement of a given water-reticulation main has an effect on the failure rate of the other mains that belong to the same shut-off block. This happens because the break occurs at the weakest point of the distribution system. Nevertheless, the weakest main in a given shut-off block is most likely to have the greatest failure rate and it seems logical to replace it first. Then the other mains will need to be replaced if their failure rates become excessive.

The issue of compliance with standards of service is related to shut-off block performance. The combination of performance of several water-reticulation mains within a shut-off block determines the level of service for a group of customers. For an individual pipe section failing at a given rate, any proactive maintenance decision to assure an acceptable level of service needs to account for the failure rate of other pipe sections within the shut-off block.

As far as the cost of proactive maintenance is concerned, Champion (1990) pointed out that the lesser social impact of the new renovation methods (that is, trenchless replacement) if compared to traditional replacement techniques should be taken into account when assessing a project.

The final recommendation from the study of Walski and Pelliccia (1981) was that in general it is not economically justifiable to replace many pipes simply because of their break rates, although some replacement may be needed to provide additional carrying capacity.

2.6 Summary

Many studies have listed the circumstances that can lead to the failure of a water main. In brief, it is stated that the deterioration process depends on the pipe material, the environment in which the pipe is laid and the operating characteristics. In attempting to model this complex process most authors suggest using historical failure data. There are very few sources recommending filtering of failure data and those that do advocate only a partial or informal screening process. There is no record of previous work to formalise the process in the way proposed in this project.

Infrastructure Authorities worldwide are now required to manage their assets more effectively and this has resulted in many technical references relating solely to asset condition assessment and performance. In the water-reticulation literature the focus has been on network level decision making and predictive models have been developed for strategic planning and budgeting. Models for this purpose are based on regression or survival analysis of aggregate data and they forecast failures per kilometre per year. However, these models are not able to forecast individual failures because variables related to pipe characteristics, environmental conditions and modes of operation introduce a large variability in individual pipe section behaviour.

Where a reference does exist to failure forecasting models for individual pipes it usually relates to the analysis of large water-delivery mains. Further research needs to be undertaken to develop models applicable to water-reticulation mains.

The literature on the replace/repair decision has seen a move away from traditional economic analysis techniques or ad hoc rules of thumb towards methods flexible enough to include the requirements of customer service contracts. It remains to be seen how these will evolve as authorities move to corporatised or privatised structures.

3 FAILURE FORECASTING

3.1 Introduction

The first phase of the project was the development of a model that could be used to predict the performance of individual mains. This chapter describes the development of a failure model based on an analysis of historical data provided for municipal areas in Melbourne and Sydney. The unique feature of the modelling process is the 'filtering' or removing of failure data that does not relate to the gradual deterioration of the pipe. The filtered data is used in a model with an exponential form to predict failure patterns. The filtering process is critical in the successful forecasting of pipe performance and techniques are provided in this paper to assist Authorities to develop their own filters. This chapter concludes with an assessment of alternative predictive models and a recommendation to adopt an interfailure time exponential model.

3.2 Data Sets Employed

The data used for the research was supplied by Melbourne Water and the Sydney Water Board. In this data a pipe section is defined as a unique length of pipeline between two nodes.

3.2.1 Ringwood Municipality

The Ringwood Municipality, an area which developed rapidly during the mid to late 1960s, is situated in the eastern suburbs of Melbourne. The data, provided by Melbourne Water, contained information about failures that occurred between 1948 and 1985 in the Ringwood water-reticulation network, which consists of 1855 asbestos cement and cast-iron pipe sections having diameters of 80, 100 and 150 mm. These pipe sections represent 81% of the

Ringwood reticulation network. Table 3.1 provides a summary of the pipe diameters and materials.

TABLE 3.1

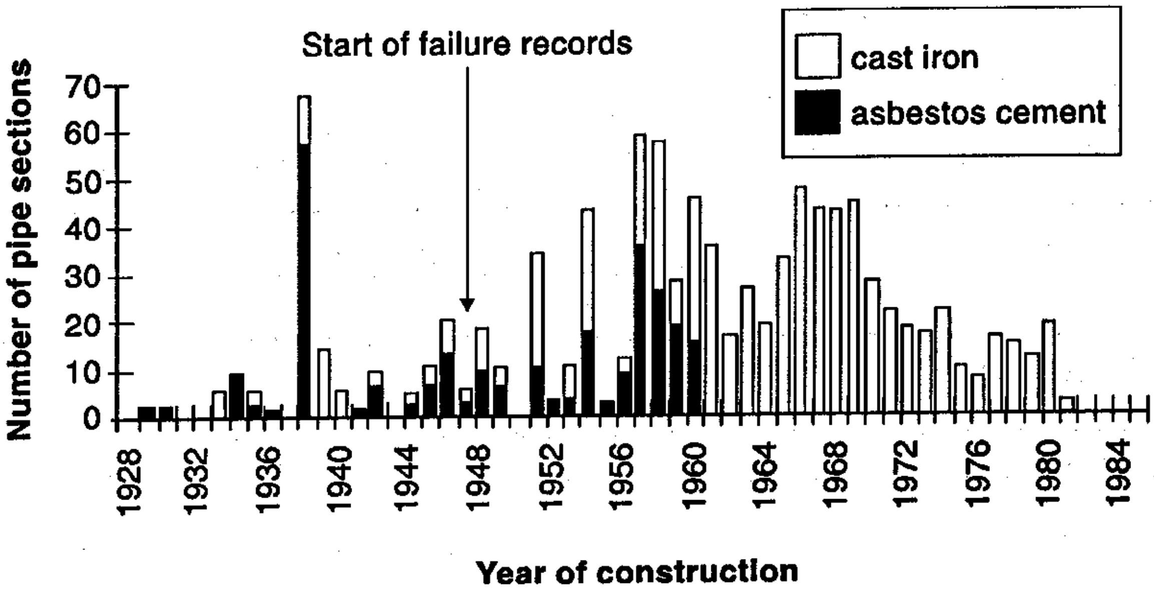
NUMBERS AND TYPES OF PIPE SECTIONS IN THE RINGWOOD DATASET

Pipe material	**Pipe diameter (mm)**			**Total**
	80	100	150	
AC Sutton	3	90	54	147
AC Mazza	0	296	49	345
AC subtotal	3	386	103	492
Cast iron	1	1064	328	1393
All pipe TOTAL	4	1450	431	1885

Table 3.1 shows that cast iron is the main pipe material in the Ringwood dataset. The first cast-iron pipe sections were laid in the 1930s. Two manufacturing techniques were used to produce asbestos-cement pipes: *Sutton* pipes before the 1940s and *Mazza* pipes after the 1940s. The construction profile showing cast iron and asbestos cement is provided in figure 3.1

Figure 3.1

Construction profile of Ringwood pipe sections



All cast-iron water-reticulation mains in Melbourne have an internal cement lining which protects the pipe against corrosion. Factory cement lining for metallic water-reticulation mains was initiated in the late 1920s and an in-situ lining program was started in the early 1960s for those cast-iron mains without lining.

Only three sections out of the 1393 cast-iron sections included in the Ringwood database were cement lined in-situ. This is due to the majority of cast-iron pipes having a construction date in the 1950s.

3.2.2 Heidelberg Municipality

The Heidelberg Municipality is located north-east of the city of Melbourne. The performance database contains information about pipe sections and failures in the Heidelberg water-reticulation system between 1948 and 1989.

The Heidelberg data set contains 2089 asbestos-cement and cast-iron pipe sections with diameters of 80, 100 or 150 mm. They represent 85% of the Heidelberg system.

Cast iron is the principal material used in the Heidelberg system. Its extensive use started in the 1950s (see table 3.2). Asbestos pipes of both types were used but the data does not distinguish the manufacturing method.

TABLE 3.2

NUMBERS AND TYPES OF PIPE SECTIONS IN THE HEIDELBERG DATASET

Pipe material	**Pipe diameter (mm)**			**Total**
	80	100	150	
Asbestos cement	14	239	115	368
Cast iron	12	1436	273	1721
All pipe TOTAL	26	1675	388	2089

The Heidelberg data set contains the following information for each pipe section:

- material;
- diameter (mm);
- year of construction;
- length (m);
- cement lining year (if applicable for cast iron);
- renewal details.

The lengths of pipe sections are widely distributed as indicated in table 3.3.

TABLE 3.3

LENGTH OF PIPE SECTIONS IN HEIDELBERG

	Length of main (m)	
	Asbestos cement	**Cast iron**
Minimum	4	1
Maximum	807	790
Arithmetic mean	141	137
Standard deviation	106	103

Of the 1784 cast-iron sections included in the Heidelberg data base, 257 have been cement lined in-situ.

3.2.3 Sunshine Municipality

The Sunshine Municipality is located north-west of the city of Melbourne. The performance database is set up in the same manner as for the Heidelberg and Ringwood Municipalities. Pipe sections are defined in the same way, i.e. as a length between two nodes, with node 0 indicating a dead-end main. The Sunshine data set contains 3707 asbestos-cement and cast-iron pipe sections whose pipe diameters are 80, 100 and 150 mm. These represent 85% of the Sunshine system, the same proportion as for Heidelberg. The remaining pipes include some of small diameter (less than or equal to 63 mm), some of large diameter (greater than or equal to 225 mm), and some other materials such as galvanised wrought iron, PVC and cement-lined ductile iron.

Cast iron is the principal pipe material. The first cast-iron pipes were laid in 1890. In the period between 1930 and 1946 more asbestos-cement pipes were laid than cast-iron pipes. Since 1980 newer materials, such as cement-lined ductile iron and PVC have been used. However, historically cast-iron pipes have been preferred in Sunshine, as shown in the table 3.4. As in Heidelberg, both Sutton and Mazza types of asbestos cement pipes have been installed. Provision has been made in the database for the manufacturing technique to be included, but in general this field has not been used.

TABLE 3.4

CONSTRUCTION HISTORY OF WATER RETICULATION PIPES IN SUNSHINE

Period	Cast-iron pipes Diameter (mm)			Asbestos-cement pipes Diameter (mm)		
	80	100	150	80	100	150
1890-92	1	3	24	-	-	-
1911	-	10	1	-	-	-
1921-30	-	147	17	-	5	3
1931-40	2	55	15	4	160	68
1941-50	-	200	15	11	120	57
1951-60	3	573	112	6	179	56
1961-70	-	607	388	-	17	7
1971-80	-	571	252	-	-	-
1981-89	-	14	16	-	-	-
Subtotal	6	2180	840	21	381	191
Total	3026			593		

Table 3.4 shows that most of the pipes have a diameter of 100 millimetre. Next most common is 150-millimetre diameter pipes and finally 80 millimetre. There are more lengths of the larger diameters (225 and 300 mm) than the 80-millimetre diameter pipes listed in this data set, but these have not been studied since they are not classified as reticulation mains.

The data set contains the following information for each pipe section:

- material;
- diameter (mm);
- year of construction, and often month and day as well;
- length (m);
- cement lining year, if applicable for cast iron;
- renewal details.

The lengths of pipe sections vary widely as indicated in table 3.5.

TABLE 3.5

LENGTHS OF PIPE SECTIONS IN THE SUNSHINE DATA SET

	Main length (m)	
	Asbestos cement	**Cast iron**
Minimum	5	1
Maximum	1298	1298
Arithmetic mean	241.7	185.0
Standard deviation	298.3	224.8

Of the 3026 cast-iron pipe sections, 108 are recorded as being cement lined in-situ.

This data set contains the same environmental details as the Heidelberg data set. The major soil type in Sunshine is Quaternary Newer Volcanics (QVN), which is a highly expansive clay. Except for three pipes, two labelled clay and one gravel, this is the only soil type used in the Sunshine database.

As for the Heidelberg data set, each record represents a single pipe failure. Where a pipe has not failed, its pipe and environmental details are entered in the database with a failure number of 0.

There are some additional fields in this database. These include material costs, and overtime and normal hours worked, which can be used to calculate the costs and labour requirements of the failure. These fields have not been completed for every failure.

There are two types of traffic loading indicated for Sunshine. These are described as local and main road types. The breakdown of pipe sections by traffic loading indicates that:

- a greater proportion of asbestos-cement pipes are laid under main roads than are cast-iron pipes;
- the larger diameter pipes are more likely to be laid under main roads.

Since only three of the 4072 pipes, of all materials and diameters, are laid in anything but soil type QVN, no analysis has been made for soil type.

TABLE 3.6

ANALYSIS OF PIPE SECTIONS BY TRAFFIC LOADING, DIAMETER AND MATERIAL

Traffic load	Asbestos cement Diameter (mm)			Cast iron Diameter (mm)		
	80	100	150	80	100	150
Local	21	365	62	6	2036	639
Main	-	114	129	-	133	202

Table 3.7 gives the same information for the pipe that have failed.

TABLE 3.7

ANALYSIS OF PIPE SECTIONS BY TRAFFIC LOADING, DIAMETER AND MATERIAL FOR PIPES THAT HAVE FAILED

Traffic load	Asbestos cement Diameter (mm)			Cast iron Diameter (mm)		
	80	100	150	80	100	150
Local	21	237	42	5	1139	134
Main	-	114	117	-	126	202

TABLE 3.8

RATIO OF FAILED PIPES TO TOTAL PIPES

Traffic load	Asbestos cement Diameter (mm)			Cast iron Diameter (mm)		
	80	100	150	80	100	150
Local	1.00	0.66	0.68	0.83	0.56	0.21
Main	-	1.00	0.91	-	0.95	1.00

For the heavier traffic loads all pipes performed equally badly, with very few pipes escaping failure. The asbestos-cement pipes had similar failure rates for the 100 mm and 150 mm sizes. For the cast-iron pipes, the 150 mm pipes with local traffic load performed significantly better than the 100 mm pipes under the same conditions. This did not apply to the main traffic load. The sample size for the 80 mm pipes was too small to make any meaningful comparisons. The 150 mm cast-iron pipe under local traffic load performed significantly better than the corresponding asbestos-cement pipe. For the 100 mm pipe under the same conditions, the cast-iron pipe performed marginally better than the asbestos-cement pipe.

3.2.4 St George District

The St George District is south-west of the city of Sydney. This data set, provided by the Sydney Water Board, contains information about failures in the St George water-reticulation system between 1991 and 1994. The reliability of the data was verified and the details of the pipe sections were added to a computerised database.

The St George data set contains 518 cast-iron pipe sections whose pipe diameter is from 90 mm to 250 mm. Cast iron is the only pipe material included in this database. The distribution of pipes by diameter is shown in table 3.9.

TABLE 3.9

ST GEORGE PIPE DATA

Pipe diameter (mm)	90	100	150	200	225	250	**Total**
Number of pipe sections	1	455	53	6	1	2	518

The data set contains the following information for each pipe section:

- number;
- material;
- diameter (mm);
- pipe joint;
- year of construction;
- length (m);
- cement lining.

Very few cast-iron water mains in the St George database are cement lined. The data set does not contain any information on the insulation of copper service pipes connected to the cast-iron mains.

TABLE 3.10

ST GEORGE PIPE SECTION LENGTHS

	Main length (m)
	Cast iron
Minimum	12
Maximum	3122
Arithmetic mean	228
Standard deviation	226

3.2.5 Other databases

Other databases of pipe failures were made available to the research group from Melbourne Water (covering the suburbs of Box Hill and Nunawading) and the Brisbane City Council, but as yet these have not been analysed. Again, details of the database structures are indicated in Appendix B.

3.2.6 Types of data available

Data associated with pipe failure (e.g. soil type, climate, pressure zone and cause of failure) are not generally available with these records. Many of these parameters are now being recorded in databases. The Melbourne Water database, with records of failures stretching back to 1948, is a uniquely valuable asset and will continue to grow in importance as it grows in size.

Data fields associated with water-supply pipe failures fall into six major categories. These are:

1. physical parameters of the pipe; such as diameter and material
2. location of the pipe; such as street, AMG co-ordinates of nodes, and offset from building lines
3. environmental parameters; such as soil, pressure, climate, number of customers and primary shut-off block,
4. pipe history parameters; such as installation dates and failure dates
5. causal parameters of failure; such as traffic load and water hammer
6. failure cost parameters; such as hours and materials.

The first three sets of parameters are essentially time invariant and can be collected in static databases. Through use of a suitable key variable it is easily possible to connect the data to an individual asset. Parameter sets 4, 5 and 6 are time variable, but can still be related to the static data with simple relational database techniques.

3.2.7 Reliability

In general, the data were found to be consistent. The small percentage of inconsistent data sets was identified and set aside for future resolution if such work is felt to be warranted. Melbourne Water, in assembling their data, had examined and rechecked them after keying-in from the original paper records. In a data set such as Sunshine with over 660,000 data items it would be unrealistic to expect no errors. In some cases there are sets of pipes with failure histories which seem statistically different from the balance of the data set. However, it is not expected that these errors would influence the thrust of the arguments of this report or the nature of the mathematical models derived.

3.3 Filtering of Data

Some authorities consider a water-main failure to be any event that results in an interruption of supply. Using this definition there is no failure if an alternative source of supply exists even though the break must be repaired. A second definition specifies any break resulting in a cost to the authority. The latter has been adopted for this project since it is more relevant to the project's objectives.

The structural failure of a water-distribution main can be caused by many different factors. Some of these are 'inherent' (see table 3.11) in that they are part of the normal operation of the pipe network and the surrounding environment. These factors result in a gradual deterioration of the pipe material and subsequent failure of the water main, i.e. it suffers a loss of structural integrity. Other factors are considered as 'imposed' (table 3.12) because they are associated with failures due to external forces on the pipe network. This type of failure cannot be prevented by proactive maintenance. Employee training, monitoring of work practices, improved coordination between public-utility authorities and supervision of building/civil engineering contractors are relevant remedial actions for problems associated with *imposed* factors.

TABLE 3.11

EXAMPLES OF INHERENT FACTORS CONTRIBUTING TO PIPE FAILURES

Factor	Example
Internal loading	Operating pressures (static, cyclic, surge), water hammer
External loading	Dead load (soil, above ground structures), live load (traffic loading, seismic activity)
Corrosion	External corrosion (soil, bi-metallic), internal corrosion (aggressive water)
Fatigue	Wear and tear, temperature-induced contraction
Bedding conditions	Quality of construction, erosion of bedding
Manufacturing defects	Quality of pipe material

TABLE 3.12

EXAMPLES OF IMPOSED FACTORS CONTRIBUTING TO PIPE FAILURE

Factor	Example
Bad repair	Poor repair under adverse conditions, inferior quality repair (workmanship)
Faulty operation	Valve fault, use of hydrants by unskilled contractors
Intentional damage	Vandalism, individual or group action for financial gain
Accidental damage	Direct impact on pipe (construction site), failure of pressure-reducing valve

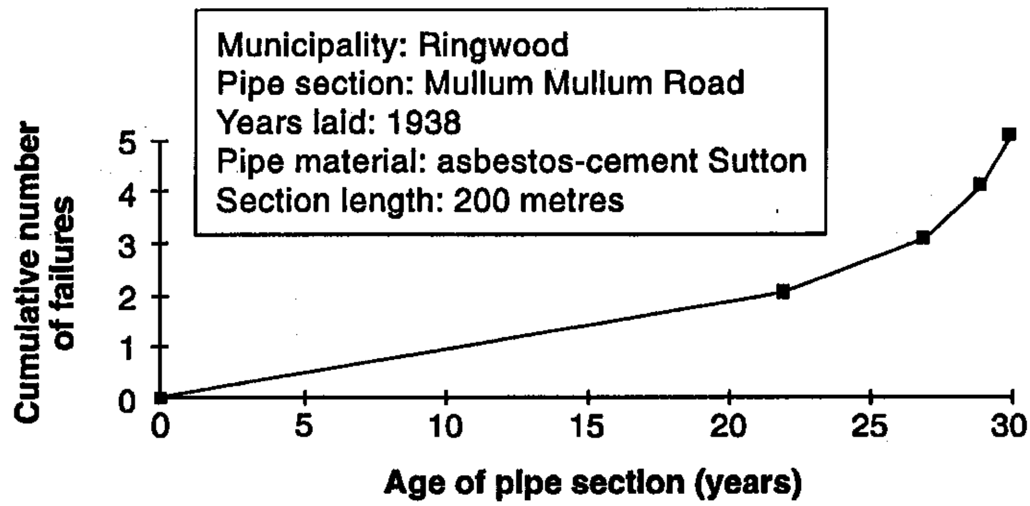
Since a model to predict the failure performance of water mains must forecast the gradual deterioration of the pipe, the data which relate to failures caused by *imposed* factors must be removed before the model is developed. These latter failures are not relevant to the issue of proactive maintenance whereas failures caused by *inherent* factors can be reduced by renewal and renovation work or changes to operational conditions. The removal of data relating to *imposed* factors is referred to as *filtering*.

3.3.1 Failure patterns

The structural behaviour of individual water mains can be analysed by plotting the cumulative number of failures per 100 m against the age of the pipe. When these curves are plotted a number of different patterns emerge. The ideal 'smooth' break pattern (figure 3.2) represents the natural continuous weakening of the pipe material. For pipes in this category there is typically a lag of around 25 years from installation to first failure. After the first failure the time intervals between subsequent failures decrease with age.

Figure 3.2

A smooth break pattern



'Irregular' or 'uneven' break patterns occur when the pipe is subjected to conditions that depart from those associated with gradual deterioration of the pipe material (figure 3.3). These plots can be described as either 'under' curves (figure 3.4) or 'over' curves (figure 3.5).

Figure 3.3

An uneven break pattern

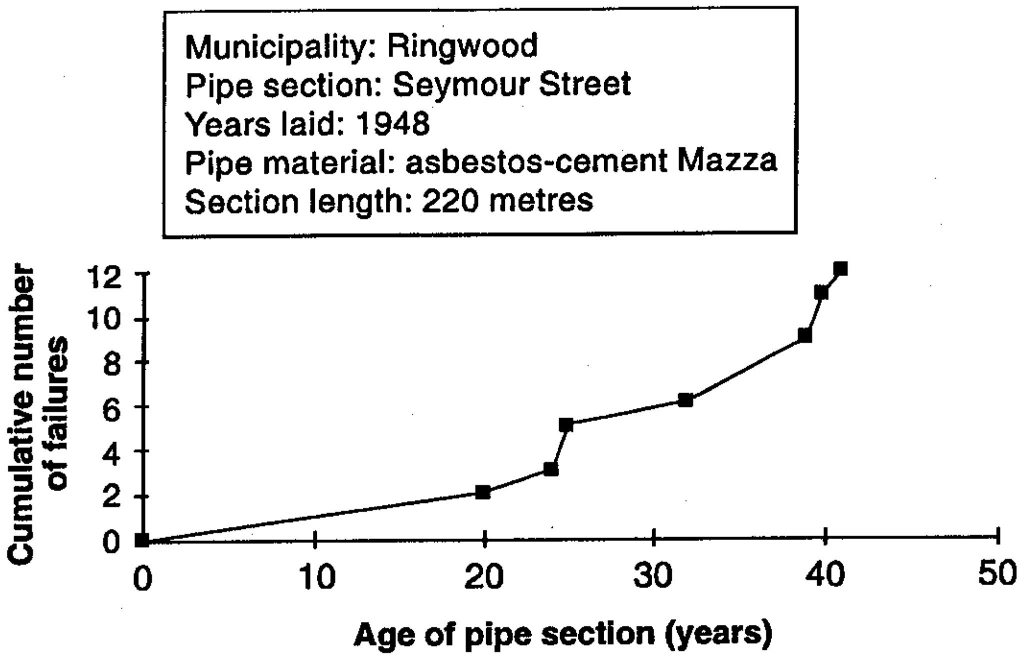


Figure 3.4

An 'under' break pattern

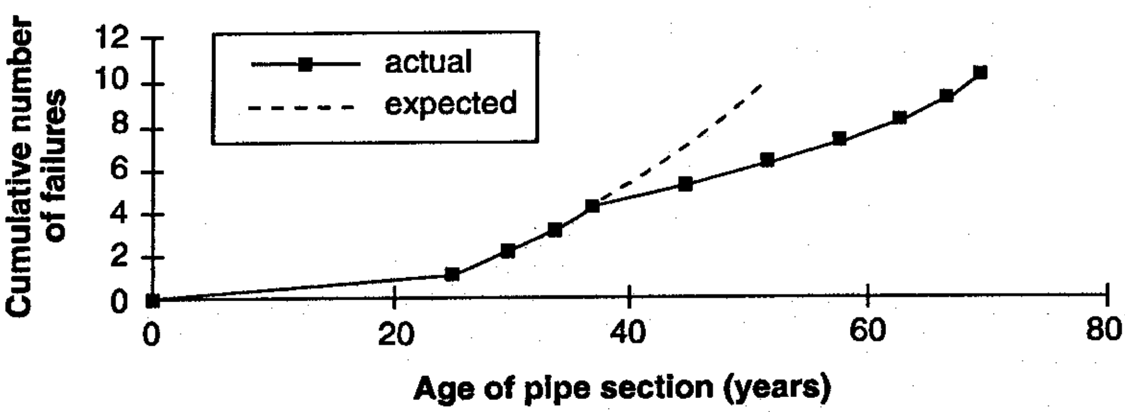
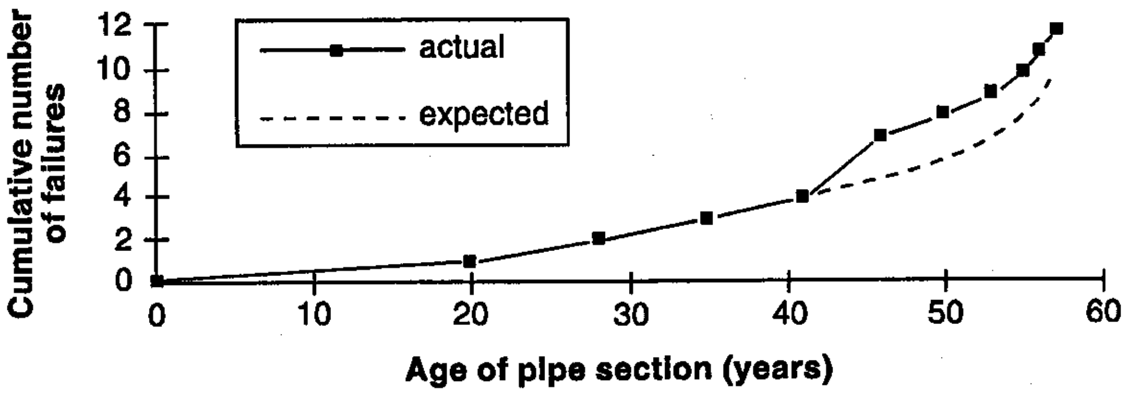


Figure 3.5

An 'over' break pattern



An under pattern will be created when proactive maintenance such as in-situ cement lining, copper service insulation and replacement of section parts have been carried out. It can also occur due to a decrease in the operating pressure. Finally, it may follow the removal of problems due to poor-quality installation or manufacturing defects. Whatever the cause, there will be a decrease in the failure rate at some point in time rather than a continuous increase. On the one hand, preventative maintenance and the creation of low pressure zones can reduce failure rates. On the other hand, poor-quality construction and manufacturing defects can result in high failure rates in the early life of the pipe. Later, the failure rates will decrease when those defects are removed. Either way, for one reason or another, the failure curve is not smooth and this must be considered when forecasting future failure rates.

The identification of these patterns is straightforward in the case of proactive actions or when a pressure decrease is registered in the database. As for installation and manufacturing defects, a good knowledge of the material behaviour and the potential age of transition is required. Poor-quality construction is identified by several breaks within the first ten years (figure 3.6). When the main has settled down in the trench this pattern of early breaks is then followed by a step (no breaks for more than ten years), which is the beginning of a new curve. Manufacturing defects can also be identified

The first break typically occurs around twenty years of age and this is followed quickly by other breaks (figure 3.7).After this phase, the manufacturing defects have been removed (by repeated repairs and replacement of a number of 'bad' pipe lengths) and a new pattern begins. The second phase is characterised by a slow and progressive increase in failure rate (a smooth pattern).

Figure 3.6

Poor-quality construction from the early part of the break curve

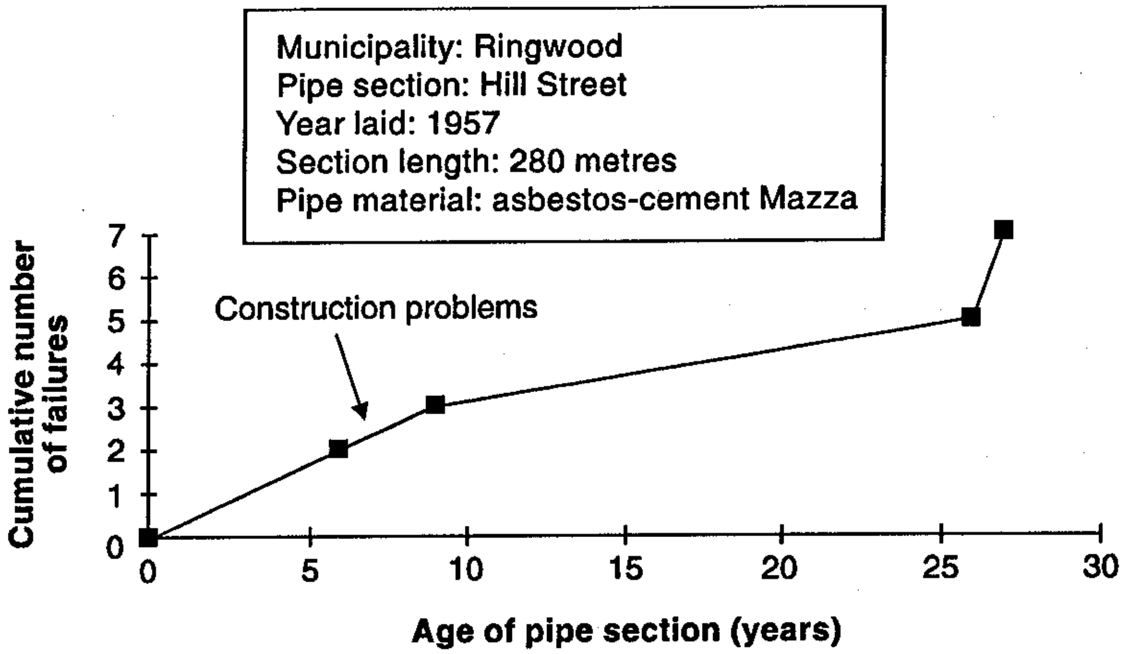
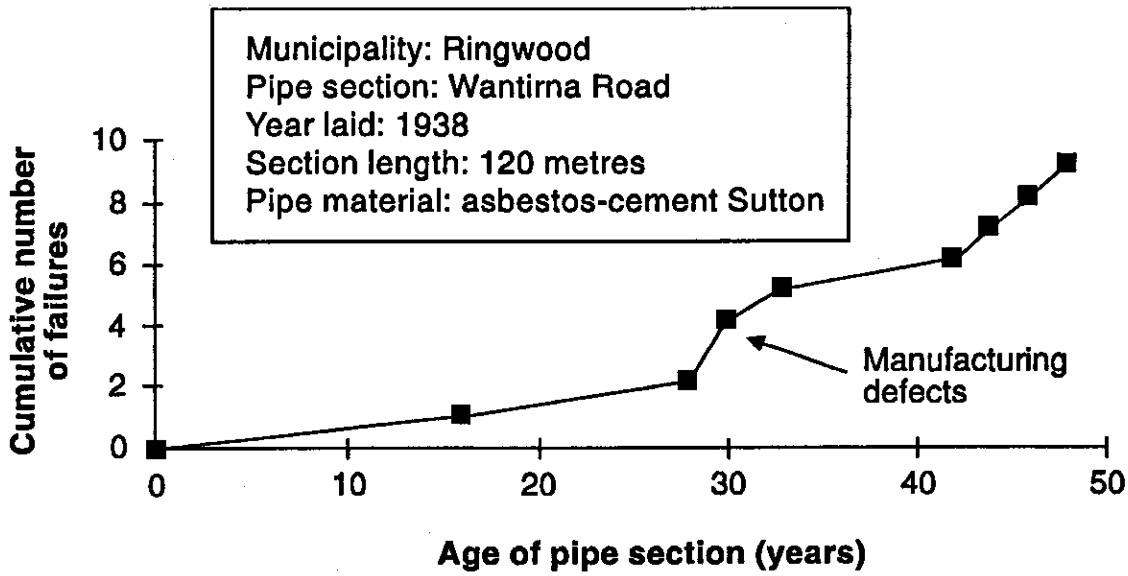


Figure 3.7

The effect of manufacturing defects



To forecast future failures, the break curve origin must be reset at the transition point so that failure predictions can be made on the basis of a continuous, smooth curve representing the age-related deterioration of the pipe material.

An over pattern results when there are two breaks in close proximity and/or within a short period of time. They may be caused by imposed factors such as bad repair, intentional damage, faulty operation or by accident. Other factors, which include a rise in pressure and weather fluctuations, can also result in an over pattern. An assessment of the time period between two consecutive breaks and their location can be used to identify over breaks.

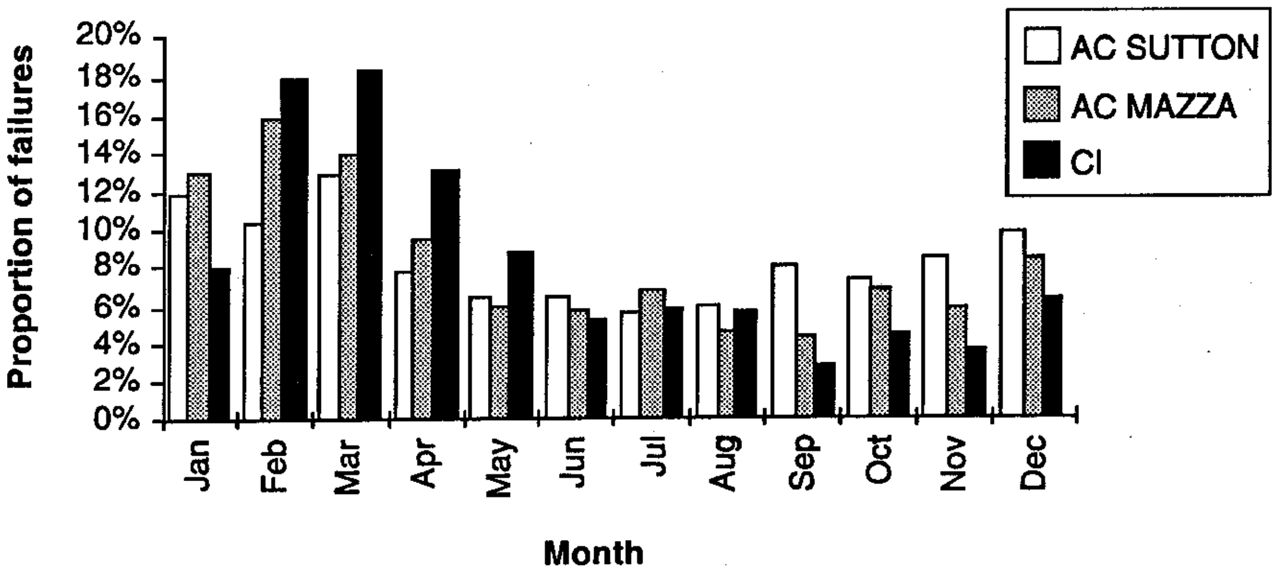
An examination of trends in failure frequency has shown that the time intervals between subsequent breaks are generally ever-decreasing. To objectively test the significance of the trend in decreasing time between breaks the arithmetic mean times (up to the sixth failure) were calculated. At the 1% significance level it was found that there was a statistical significance between the decreasing time to a subsequent failure and the increased numbers of failures. These patterns were first detected from an analysis of the Ringwood data and they have also been found in the Heidelberg database. They are applicable to both asbestos-cement and cast-iron pipes.

There are a number of benefits to be gained by studying the effect of weather on failure patterns. It is possible, for instance, to determine the part played by extreme weather conditions in an over break pattern. It is also possible to quantify the resources needed to cope with a given level of emergency. Finally, actions can be defined which should be taken during times of potentially costly weather conditions.

In Melbourne, high break rates occur during summer. They may result from fluctuations in weather conditions, which cause ground movement. The large number of failures in summer may also be caused by the increased consumption of water at that time of the year. Private and public garden watering, ablution and washing demands in summer result in the need for an increased level of supply (changes with regard to pump settings, for instance). January, February and March in Melbourne are the most critical months of the year in terms of failure numbers (figure 3.8)

Figure 3.8

Percentage of failures by month based on total number of recorded failures from 1948 to 1989 [from Scott, 1993 p 53]



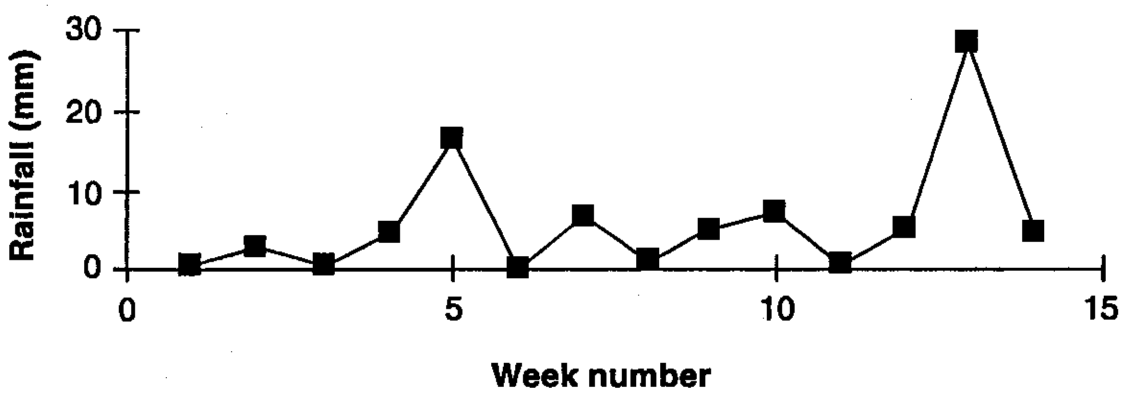
It was found that 100 mm diameter pipes are more sensitive to weather effects than 150 mm pipes. A chi-square test was applied to the proportions of breaks occurring between January and March for these two sizes. The observed difference between the two proportions (100 mm and 150 mm) was found to be significant at 1% (one degree of freedom).

Short-term weather fluctuations can cause a sudden change in the moisture content of soil. It is likely that this leads to failures due to ground movement (shrink/swell) in reactive soils and this can contribute to over break patterns. Either the time to the next break is reduced or an additional break occurs regardless of the continuous deterioration process. Rainfall data over the two weeks before the occurrence of the over break pattern being studied, can indicate this short-term variation in moisture. An analysis of rainfall data suggests weather-related over breaks result from a drying/wetting or wetting/drying pattern rather than gradual shrinkage or expansion (figure 3.9).

Critical months and years during which weather-related over breaks occurred have been assessed on the basis of high historical monthly break rates. The influence of wetting/drying or drying/wetting needs to be further examined in order to identify specific weather-related patterns.

Figure 3.9

Rainfall pattern prior to failure (wetting/drying pattern)



3.3.2 Probability of failure

The probability of failure for any consecutive pair of failures of a main can be calculated using:

$$P(n,n+1) = \frac{e^{[-L\alpha(T_2^{\beta}-T_1^{\beta})]}\left(L\alpha\left(T_2^{\beta}-T_1^{\beta}\right)\right)^2}{2} \quad \text{.......................................(1)}$$

where:

$P(n,n+1)$ is the probability of failures n and $n+1$;

L is the length of the water main (metres);

$$\alpha = e^{(c_0+c_1+c_2+c_3+c_4)} \quad \text{...(2)}$$

$c_0 = -12.047$;

T_2 is the age of failure number $n+1$ (years);

T_1 is the age of failure number n (years);

$\beta = 2.245926$;

c_1 to c_4 are given in table 3.13.

TABLE 3.13

VALUES OF c_i

c_i	Pipe dia (mm)		Pipe material			Traffic level			Soil type		
	100	150	CI	AC Mazza	AC Sutton	LT	PA	SA	QRA	SLA	SUD
c_1	0										
c_2		-0.336	0	0.586	0.887						
c_3						0	0.034	-0.010			
c_4									0	-0.972	-0.855

3.3.3 Filtering model

There are few references in the water-supply literature to data filtering. However, there are warnings by Shamir et al, (1979) and Evins (1989) that care should be taken to aggregate data that can be considered to be homogeneous with respect to the causes for breaks. These authors suggested a form of filtering based on personal judgment without providing any clear guidelines.

The following may explain the lack of documentation on filtering:

- operational staff are responsible for database reporting and they are not likely to report their own mistakes (bad repair and faulty operation);
- the definition of bad repair is a matter of opinion and needs to be clarified;
- very few databases record the type of failure and possible cause.

Hence the identification of breaks caused by imposed factors is difficult and most existing statistical forecasting models were established to use a large set of aggregate data in which the part played by imposed factors was assumed to be negligible. This approach is only valid for the strategic planning of large networks.

Filtering must take into account the stages involved in repair and the techniques and procedures used in the repair process.

The repair cycle includes:

- excavation;
- failure repair;
- restarting operation;
- site restoration.

The repair practices of Melbourne Water include:

- a clamp is bolted around the pipe for **circular fractures** or **perforations**;
- for **longitudinal fractures**: either the whole length of pipe (3.5- or 4-metre length) is replaced, which is often the case with asbestos-cement pipes where a split is difficult to repair, or the split part is cut and a section

of plastic or ductile iron pipe is inserted, which may be from 0.5- to 4-metres long. The last method is often used for split cast-iron pipes.

It should also be noted that excavation and site restoration disturb the soil and this cannot be prevented. Restarting operations must be undertaken with care to avoid water-hammer effects. Additional breaks resulting from careless recharging generally occur at the weakest points of the system, which are not necessarily at the repair location and may be some distance from the initial failure.

Bad repair

A 'bad repair' is defined as an inadequate failure repair that will necessitate an additional repair event within a short time. Assuming that a failure repair is inadequate, and taking into account Melbourne Water repair practices, it is considered that the additional failure will break out within four metres from the initial one.

Failure location is reported in the database by repair foremen. The location is given with regard to a crossing with another street, with the direction and distance from it. Distance is assessed by tape, pace or eyesight. Consequently, allowance must be made for input errors in the database. One metre, plus 10% of the distance from crossroads as reported in the database, was chosen to account for these possible errors. It is logical to make allowance for a larger input error when the location of the failure is further away from the crossroads, and therefore more difficult to assess.

As a result, the location of an additional break can be within five metres from the failure repair plus 10% of the distance from crossroads as reported in the data set under study. A bad repair is an additional failure which is imposed on the system and has nothing to do with the natural continuous deterioration of the pipe section. Consequently, the probability of that extra failure being caused by deterioration must be very low. It was decided to consider consecutive failures associated with a probability of occurrence less than 1% (calculated using a generic failure model) as candidates for filtering. The probability of two failures in a given time period is a function of many variables including the pipe age, length, material, diameter, soil type and road category. Two breaks occurring within a very short time are likely to be related to imposed factors since the probability of this happening is very low. In the case of bad repair it is important to filter data not only on the basis of *distance* between consecutive breaks but also on the basis of additional failure (low probability) because natural continuous deterioration can cause breaks within a short length of the water-reticulation main. For example, in the case of localised soil-induced corrosion.

As a conclusion, two consecutive breaks whose probability of occurrence is less than 1% and occurring within five metres from each other (plus 10%) have a high probability of being associated with bad repair.

Careless startup

After repairing a failure, the pipe needs to be recharged. A sudden *startup* is likely to cause water hammer in the system and break the pipe at its weakest point. An external failure due to faulty startup will occur on the *same day*. It has already been stated that the time intervals between subsequent failures tend to be ever-decreasing and the different break patterns observed previously lead to the conclusion that water-reticulation mains do not fail twice or more within 24 hours.

Data sets in Melbourne and Sydney do not contain the time when breaks occurred. Consequently it was decided to consider two breaks occurring within two days as related to **faulty startup**.

Intentional damage

Intentional damage is a sensitive industrial issue and its occurrence and effect on failure histories are extremely difficult to assess. Nevertheless, it was decided to consider two consecutive failures as related to intentional damage if they occurred in December (financial gain from extra overtime) or during an industrial dispute, and if they were associated with a probability of occurrence less than 0.5%. It was found that some consecutive breaks can be associated with a very low probability because they occurred early in the life of the pipe. However, early pairs of breaks may reflect an unusually poor quality of construction and not necessarily intentional damage. After fifteen years of age, poor quality of construction has almost no effect on the Ringwood data. Consequently, it was decided to add a condition to qualify for intentional damage: two consecutive breaks occurring in December or during an industrial dispute, and whose probability is less than 0.5% where the pipe age is greater than fifteen years, can qualify for intentional damage filtering.

The existence of intentional damage was questioned by some professionals from the water industry but almost all water-system managers interviewed acknowledged its existence (RMIT Workshop, February 1995). They stated that intentional damage occurs not only during the pre-Christmas period, public holidays and industrial disputes, but also on other occasions so that additional overtime (at penalty rates) can be obtained.

Some consecutive breaks can also be associated with a very low probability of occurring within a short time but may happen during *unusual weather conditions*. It was found that January, February and March are much more critical than December in terms of weather effect on failure rates.

As a result, it is unlikely that two consecutive breaks occurring in December and associated with a probability of less than 0.5%, could be related to soil shrink/swell resulting from weather fluctuations. Nevertheless, the intentional damage filtering system can be modified to bypass any consecutive breaks occurring in a critical month and critical year for which exceptional weather conditions have been reported. The analysis of monthly break rates over forty years for an entire water-reticulation network can help in setting up a list of critical months.

Faulty operation and accidental damage

Faulty operation and accidental damage are even more difficult to assess than intentional damage because they can take place at any time. Consequently this issue should be considered at the end of the filtering sequence when the test results for other external causes are found to be negative. A similar approach to that taken for intentional damage by considering two consecutive breaks associated with a probability less than 0.1% and pipe age greater than fifteen years can be regarded as related to faulty operation or accidental damage.

3.3.4 Filtering logic

The logic of filtering the data for an individual water main must initially refer to the first set of consecutive breaks - breaks 1 and 2 - and then the second set - breaks 2 and 3 - and then the third set (breaks 3 and 4) etc.

For each set of consecutive breaks (n) and (n+1), the system should first assess whether *careless startup* took place. If it did, the break (n+1) is tagged and the filtering system considers the next set of breaks. If careless startup did not take place the probability of the occurrence of the (n) and (n+1) break is calculated. If that probability is less than 1%, the system should assess whether *bad repair* took place - otherwise the next set of breaks is considered.

When the system tests positive for bad repair it tags the break (n+1). Otherwise, it considers the next set of breaks if the break (n) has already been tagged and if not, it tests for *intentional damage*. Whether the break (n) has already been tagged or not is important because a very low probability of two consecutive breaks (n) and (n+1) may be found when the break (n) is an external failure. The breaks (n) and (n+1) may have occurred within a very short time and are therefore associated with a low probability but in fact, the external failure (n) should not have happened and the very low probability of the set (n) and (n+1) should not lead us to suspect the break (n+1) of being an intentional damage, faulty operation or accidental damage failure.

When the system tests positive for intentional damage it tags the break (n) or (n+1) and then goes to the next set of breaks. Otherwise, it is tests for *faulty operation or accidental damage*.

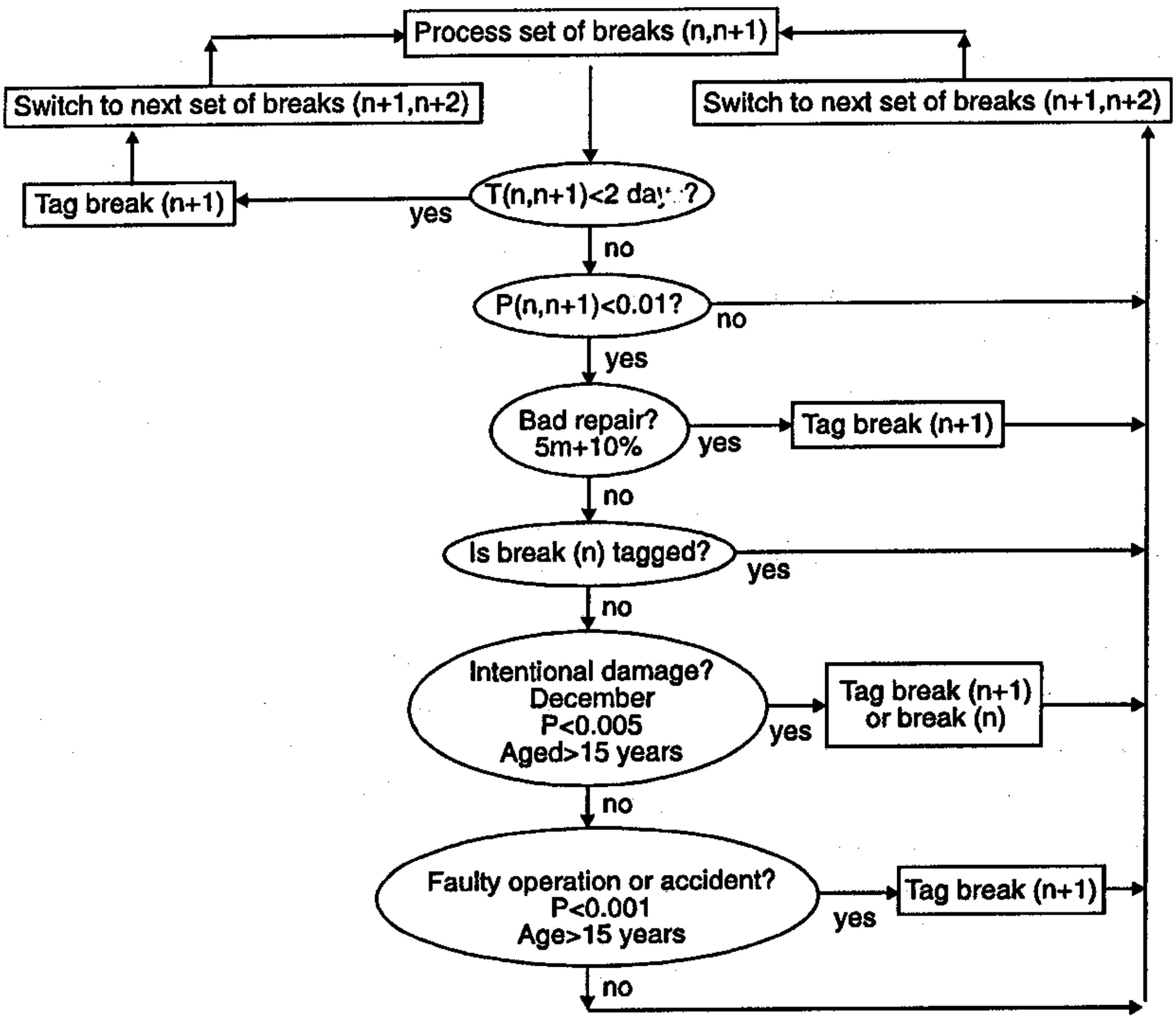
When the system tests for positive faulty operation or accidental damage it tags the break (n+1) and then goes to the next set of breaks. Otherwise it goes directly to the next set of breaks.

When the last historical set of breaks for the pipe section under study has been analysed by the filtering system, all failures tagged as careless start-up, bad repair, intentional damage and faulty operation or accidental damage are removed from the data set.

The filtering logic system is summarised in figure 3.10.

Figure 3.10

Filtering logic system for an individual pipe section



Definition of symbols, abbreviations and parameters used in figure 3.10:

n	failure number (where n = 1,2,3,4...);
n+1	the next failure;
(n,n+1)	a consecutive failure set;
T(n,n+1)	the time between consecutive failures;
P(n,n+1)	probability of failures (n) and (n+1).

Note: Filtering logic is available as an XL macro from the Infrastructure Research Unit, Department of Civil and Geological Engineering, RMIT.

3.3.5 Justification of filtering logic

Careless startup

When the system is recharged after a main break has been repaired, careless startup can cause additional breaks at the weakest point of the asset. An additional failure at the weakest point of the system might be regarded as an early inherent break which would have taken place anyway sooner or later in the life of the pipe section. The possibility of moving forward the occurrence date of start-up related breaks in historical databases so that continuous deterioration of the pipe section is obtained, was considered. Assuming that break (n+1) is related to careless startup, it is possible to relocate the date of failure midway between the failure date of break (n) and the failure date of break (n+2).

A trial on Ringwood data was undertaken and the effect of modifying the failure dates of identified startup related breaks on the uneven profile of failure curves was assessed. It was found that the modification of the failure date of start-up related breaks did not remove over-break patterns in most cases. The reason is that if the break (n+1) has been associated with careless start-up, the time period between the break (n) and the break (n+2) is generally not long enough for the revised failure date of break (n+1) to have any effect on the curve profile. As a result, it was decided to simply remove start-up related breaks from the data set.

As discussed previously, it is very likely that two breaks within two days were caused by careless startup or bad repair. The possible contributing factors to two breaks within three days were questioned with regard to careless startup. This issue was raised in a personal communication by Tsui (1995). He pointed out that some water-reticulation main breaks in Sydney can be repaired a couple of days later if the leak is not major. The alternative of considering three days in the filtering system was tested on Ringwood. It was found that the number of startup related breaks identified by the filtering system had increased and that most of those additional failures were in fact coming from the categories bad repair and faulty operation or accidental damage. This finding is not surprising because it is very difficult in some cases to assess whether the repair was poor or the startup was inadequate. Furthermore, careless recharging can be regarded as a form of faulty operation.

It was decided to consider only two days in the filtering system because:

- using two days is more conservative in the sense that the conditions to qualify for bad repair and faulty operation are stricter than those for careless start-up;
- failures tagged on behalf of careless start-up, bad repair and faulty operation or accidental damage are all removed anyway.

Bad repair

Five metres plus 15% of the distance from crossroads as reported in the database was initially used for bad repair, but 15% was found to be excessive from the analysis of Ringwood data. Trials were carried out using less than 15% and as a result it was decided to consider two breaks within five metres plus 10% of the distance reported in the data set as the criterion for bad repair.

Another condition for a set of failures to qualify for bad repair is to present a probability of occurrence less than 1%. The low probability just reflects the fact that an external failure is not part of the inherent degradation of the pipe material and is consequently unexpected. Reducing that maximum probability was not possible because numerous consecutive failures occurring in Ringwood with probabilities between 0.5% and 1% qualify for bad repair and display over-break patterns in their failure curve profiles. Since the performance of the filtering system is based on the effective removal of over-break patterns, it was decided to keep 1% in the test for bad repair.

Intentional damage

With regard to intentional damage, industrial disputes have not been recorded by Melbourne Water and only the possibility of intentional damage during Christmas periods was studied.

The filtering system was applied to Ringwood data with any failure identified as intentional damage if it occurred in December and was associated *with a probability of occurring of less than 1% and having a pipe age greater than fifteen years*. The arithmetic mean date of the failures tagged with intentional damage is 15 December, and the distribution of those failures before and after the 15 December is similar. Repeating the same analysis after taking out the drought years (1972, 1981 and 1982) did not make any difference. As a result, it was decided to consider any day in December as part of the Christmas period. Then the maximum probability to qualify for intentional damage was assessed. Using 1% initially, the filtering system was applied to Ringwood data. The arithmetic mean probability of the failures filtered by the system as intentional damage was 0.2% with a maximum of 0.7% and a standard deviation of 0.2%. Bearing in mind that there is absolutely no evidence of intentional damage it was decided to use a maximum probability of 0.5% in the filtering system.

Faulty operation and accidental damage

Many sets of consecutive breaks with a probability less than 1% are not filtered by the system as careless startup, bad repair or intentional damage. The analysis of the distribution of those failures with regard to pipe material, diameter, soil type and month of occurrence resulted in no meaningful conclusions.

It was expected that most of those breaks should be related to soil shrink and swell. In fact, no trend was observed. There are on average 2.38 more breaks in January, February or March than there are in any other month of the year based on the entire Ringwood data set. For the failures not filtered (in spite of a probability less than 1%), there are only 2.24 more breaks in January, February or March.

Some of those failures are related to poor quality of construction (where the pipe age is less than fifteen years) and the rest contribute to over-break patterns in failure curves. As a result, it was decided to consider failures associated with pipe age greater than fifteen years and probability less than 0.1% as candidates for faulty operation or accidental damage. This low maximum probability reflects the fact that, similarly to intentional damage, there is no evidence of faulty operation or accidental damage recorded in the data set. In some cases it may be appropriate to not consider intentional damage, faulty operation or accidental damage in the filtering system.

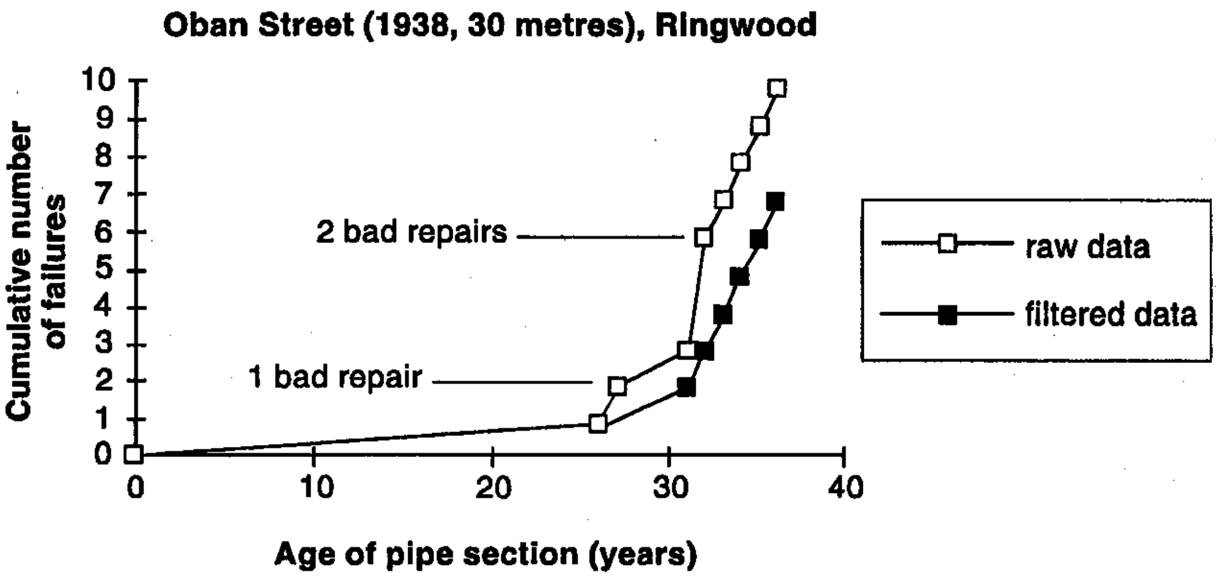
3.3.6 Performance of the filtering system

The performance of the filtering system was assessed on the basis of the effective removal of over-break patterns. Ringwood data were used to create and calibrate this filtering system. The data from the St Georges District and from the Heidelberg Municipality were used to confirm and verify the effectiveness of the system.

A comparison between raw failure curves for the Ringwood data and filtered failure curves clearly indicates that the filtering system is removing over-break patterns or reducing their impact (figure 3.11). Filtering does not produce perfectly smooth failure curves in all cases but the profile of filtered failure curves is much smoother than before. In order to remove all over-break patterns, more historical breaks would need to be filtered and consequently it was found that the filtering system is rather conservative. However, it is expected that many water-supply network managers will be reluctant to filter a large number of historical failures especially when the effect of weather fluctuations, faulty operation or accidental damage is uncertain. As a conclusion to these trials it was considered that the performance of the filtering system on Ringwood data was acceptable.

Figure 3.11

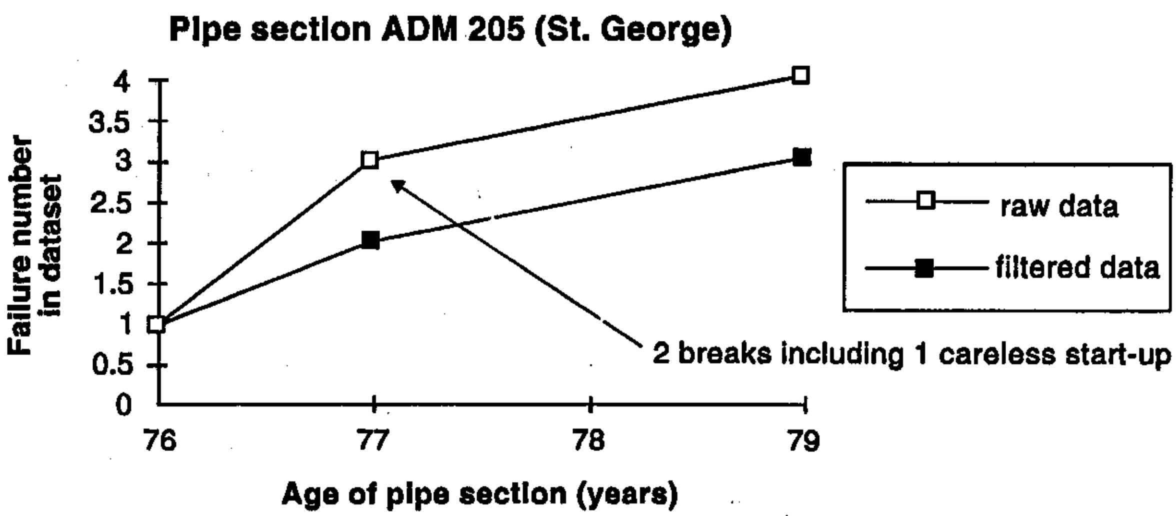
Raw and filtered data curves for Ringwood data



The comparison between raw failure curves of the St George District and filtered failure curves gave the same results and conclusions as for Ringwood data (figure 3.12). The small size of this data set and the small number of recorded failures per main make the analysis of Heidelberg data necessary in order to confirm the results of filtering on Ringwood and St George.

Figure 3.12

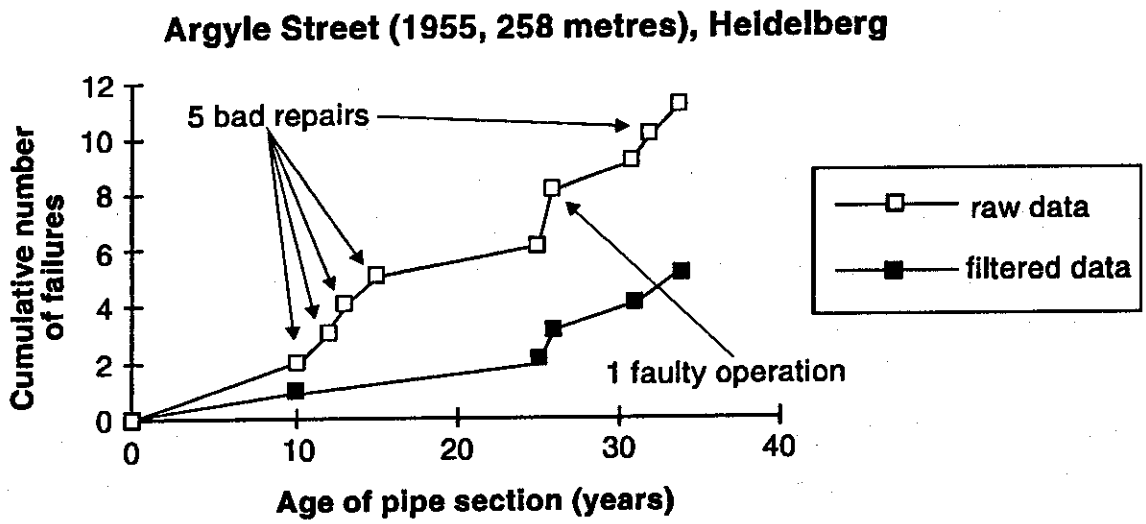
Raw and filtered data curves for St George data



The results of the application of the filtering system to Heidelberg data are very similar to those found for Ringwood (figure 3.11). As a conclusion, it was decided to adopt the filtering system as presented in figure 3.13.

Figure 3.13

Raw and filtered data curves for Heidelberg data



3.3.7 Summary of breaks filtered

The percentages of historical failures removed by the filtering system in the three data sets under study are presented in table 3.14.

TABLE 3.14

SUMMARY OF BREAKS FILTERED

District	CS	BR	ID	AD	TFB	P<1%
Ringwood	43	70	24	78	215	203
% of total breaks	2.8	4.6	1.6	5.1	14.2	13.4
Heidelberg	44	159	48	131	379	318
% of total breaks	2.0	7.4	2.1	5.1	17.7	14.9
Saint Georges	19	17	6	26	68	33
% of total breaks	2.9	2.6	0.9	4.0	10.4	5.1

where:

CS	=	careless startup;
BR	=	bad repair;
ID	=	intentional damage;
AD	=	faulty operation or accidental damage;
TFB	=	total filtered breaks.

The total number of breaks filtered in each database is between ten percent and eighteen percent. A large number of breaks occurring in Heidelberg have been associated with bad repair and faulty operation. This difference is probably due to variations in the quality of workmanship between municipalities.

The most important external factor in terms of numbers of breaks filtered is faulty operation or accident. Bad repair comes next and then careless start-up. The least important factor is intentional damage. The total number of breaks filtered in each database is reasonable and this contributes to the credibility of the filtering system. It can be noticed that the number of breaks whose probability of occurring is less than 1% and which have not been removed by the filtering system can be almost as high as the total number of breaks filtered. This indicates that reducing the maximum probability for intentional damage from 1% to 0.5% and for faulty operation or accidental damage from 1% to 0.1 % was necessary in order to filter a reasonable number of historical failures.

3.3.8 Application of filtering

The failure models discussed in the next section are based on an assumption of gradual deterioration of the pipe due to corrosion and wear and tear. Events which do not fit this general pattern need to be removed to allow the fitted model to predict failures due to deterioration. This is the purpose of the data filtering systems which have been developed and discussed in this section. However care must be taken when applying these filters, as discussed below.

Filtering models developed for a particular area or type of pipe may not be transferable to pipes in other areas. It was found that the probability-type filter, developed on the Ringwood pipe data, overfiltered the breaks for the asbestos-cement pipes in the Sunshine database. This was because the expansive clays in Sunshine appear to have caused rapid deterioration in the Sutton AC pipes hence structural failures occurred earlier and with a greater frequency than for the Ringwood pipes. Later in this section we discuss techniques which can be used to validate or develop filters.

After filtering the data for a particular pipe, the filtered-out data points should be examined as the pipe may have a history of double breaks, which would be removed by the filtering process. If this is found to be the case some action should be taken. One possibility is to record the pipe as one that requires special care when repaired to avoid a second break. A second possibility is to treat each break that is predicted by the model as a double break and build the associated cost into the economic forecasts. *In any case, the data points filtered out should not be removed altogether from the database and discarded as they may be required for future analysis.*

If the number of failures on a particular pipeline is large, say more than twenty, the removal of a few data points by filtering may not significantly alter the predictive performance of the model. In such a case, after comparing the models for the filtered and unfiltered data, the operator could prefer to use the unfiltered data.

3.3.9 A method for validating and developing filtering systems

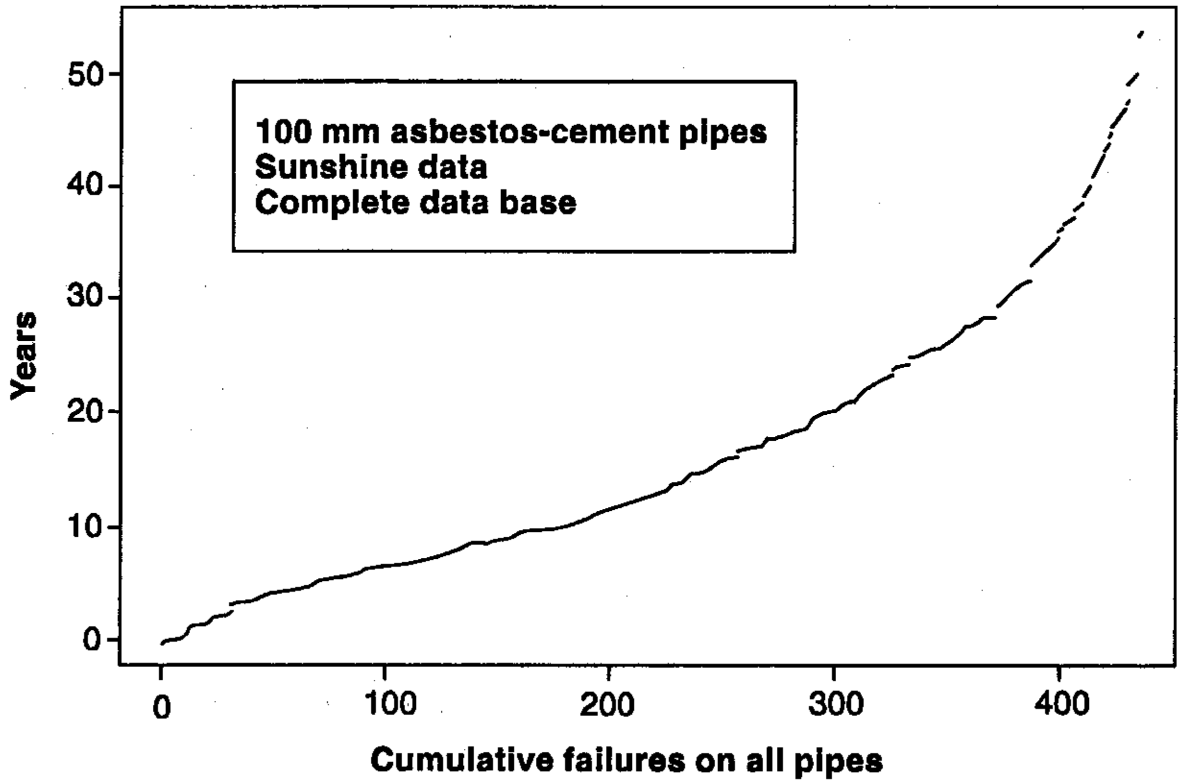
In this section a simple graphical technique is presented for determining the time before which a first failure is due to faulty materials or workmanship, or a second or further failure is due to a bad repair or disruption to the pipeline caused by the initial burst. This can be used to check if an existing filtering system can be applied to a different area, or type of pipe, or to find some initial parameters for building a new filtering system.

This technique is illustrated by using the failure data for the 100-mm diameter asbestos-cement pipes in Sunshine. The technique has been applied to other pipes in Sunshine and to the areas of Nunawading and Ringwood and has been found to work in a similar fashion for these areas, though giving different time periods.

The time from construction to first failure was calculated for each main with at least one failure. These failure times were then ranked in ascending order and plotted against their rank. The plot is shown on figure 3.14. As can be seen, the curve is reasonably smooth and the slope is increasing.

Figure 3.14

Cumulative failure time vs pipe age



It is often found that taking logarithms of this type of curve will show unseen features of the data. Figure 3.15 is a plot of the logarithms of the failure times against their ranks. Natural logarithms have been used but logarithms to base ten or any other convenient number would do as well.

Figure 3.15

Cumulative failure time vs log pipe age

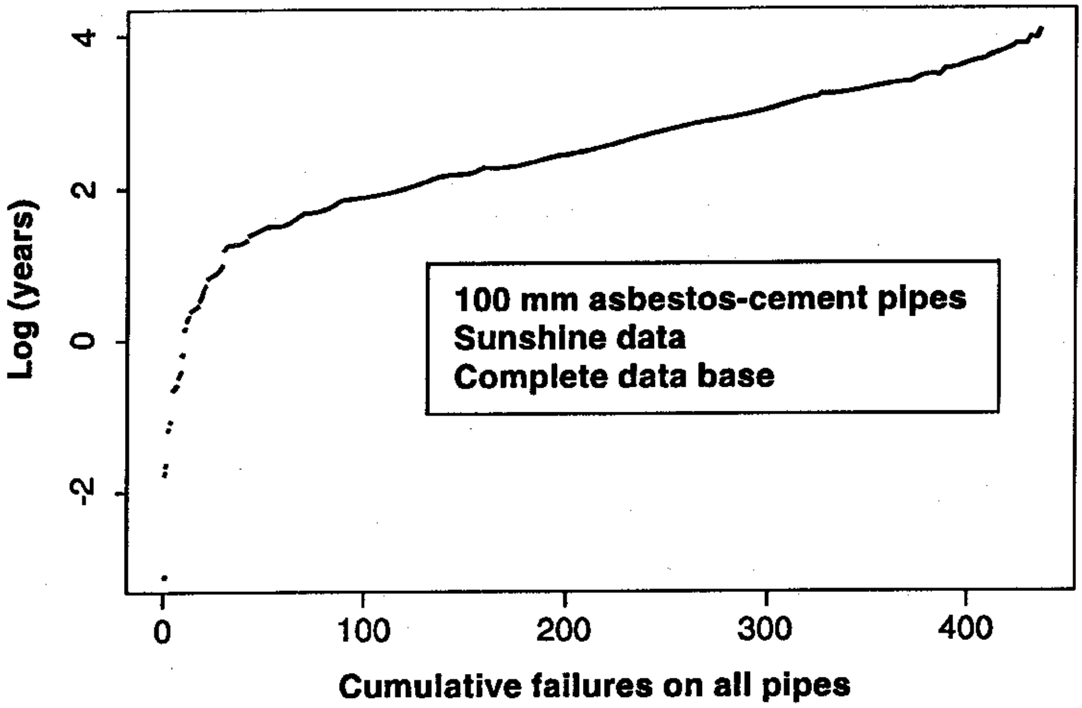


Figure 3.15 shows two distinct behaviours: a rapidly rising section up to about *log(year) = 1.2* and a more slowly rising section after this point. This time period corresponds to the first three years of the life of the pipe and the change in behaviour could be attributed to the change from the failures being caused by poor construction to that of failures caused by aging of the pipeline. A similar procedure was carried out using the time between the first and second failures on a pipe. (In order to take logarithms, failures that occurred on the same day had to be omitted.) The results of this procedure is shown in figures 3.16 and 3.17.

Figure 3.16

Cumulative failure time between first and second failures vs pipe age

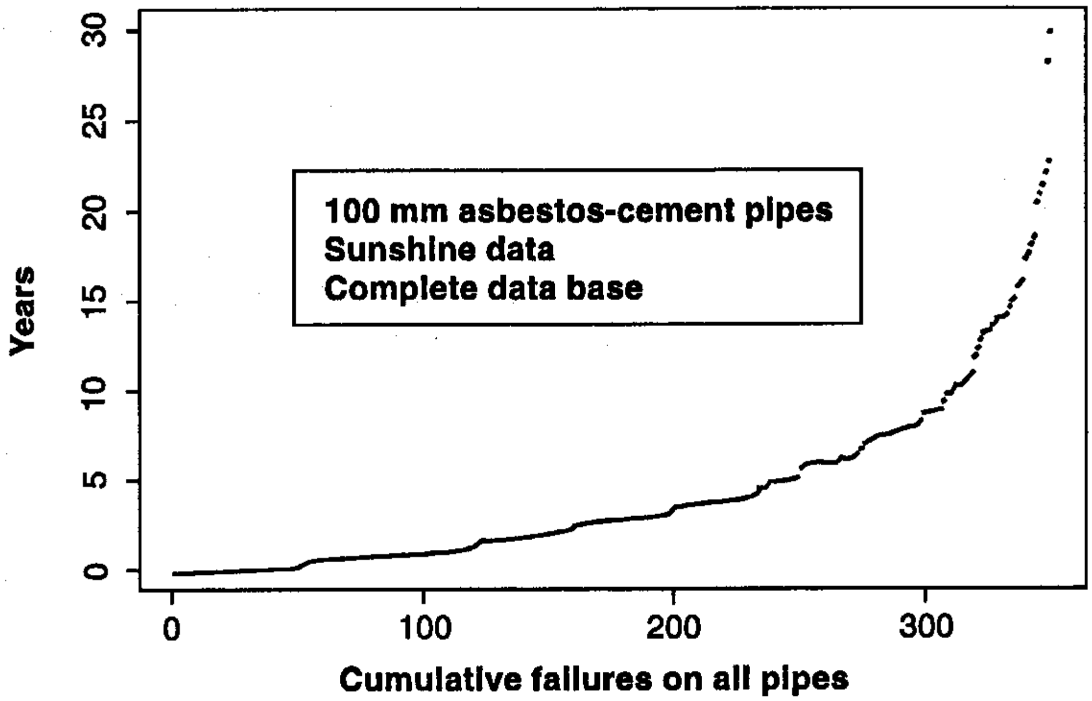
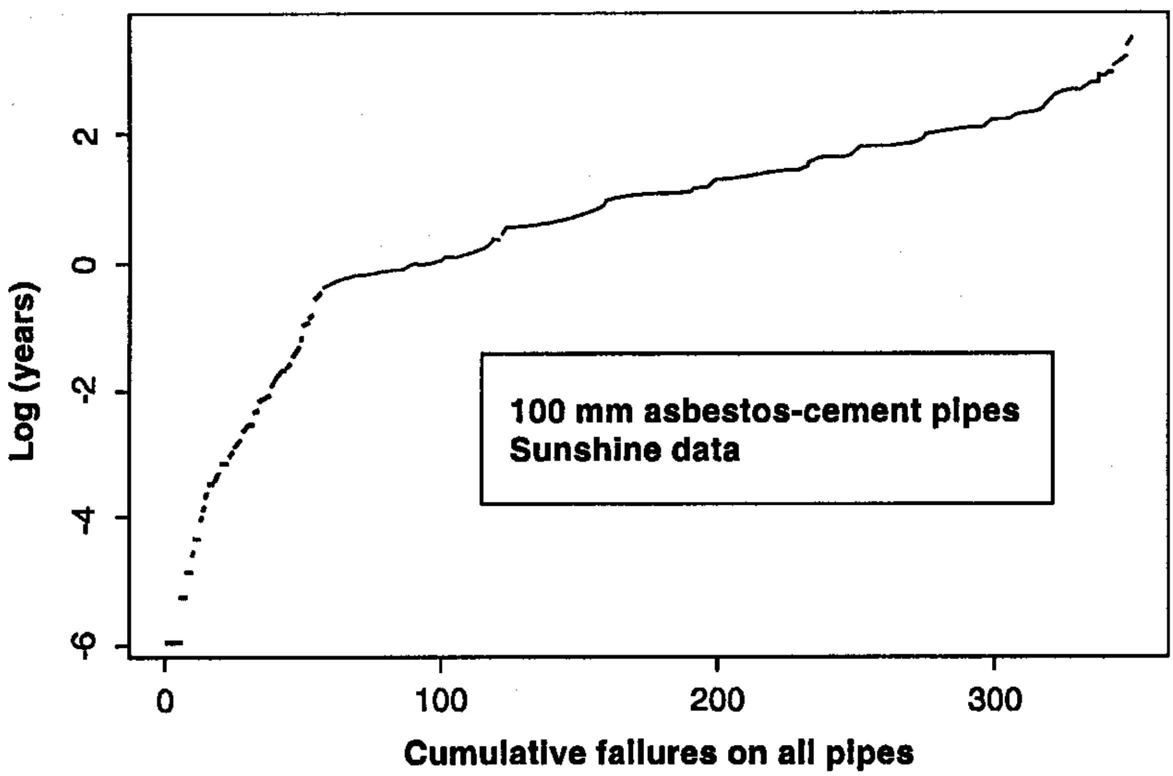


Figure 3.17

Cumulative failure time between first and second failures vs log pipe age



Again, a distinct change in behaviour can be seen on the logarithmic graph. The change point here occurs at about six months and could be attributed to the effects of bad repair or disruption to the pipe and bedding caused by the first burst. These figures could then be used in a filtering system that:

- disregarded any breaks that occurred in the first three years as being due to construction faults;
- queried any pair of breaks that occurred within a six-month period and suggested deleting the second.

The original filtering system developed on the Ringwood data queried any two breaks occurring within a twelve-month period. This analysis suggested that for the AC pipes in Sunshine this criterion was too severe and the filter logic was modified for this class of pipe.

3.4 Failure Prediction Models

An effective predictive model for inherent structural failures must use filtered data. However, it is neither realistic nor necessary to establish different models for different categories of inherent failures. Most water-reticulation mains are subject to the full range of inherent breaks and, to minimise life-cycle costs, it is more important to predict the occurrence of a failure rather than its cause. This chapter examines a number of models derived from filtered data but begins by considering the use of unfiltered failures.

3.4.1 Failure histories

In an attempt to highlight factors which may have a strong influence on failures of individual mains, unfiltered data were examined. Details of this analysis are provided in Appendix I. The physical failure model investigation involved plotting pipe failure dates against the number of failures for each individual length of pipe. Eight basic curves resulted from this analysis.

For a given pipe length, curves with steeper slopes have fewer failures and are therefore performing better. These curves were developed using data from the Sunshine database and it was expected that the different curve shapes would relate to specific spatial factors such as town planning zone, soil type, traffic loading or, possibly, topography. In order to test this theory a geographic information system was established to assess the spatial distribution of the curve types. Appendix G provides details of this system and some sample output. Although the findings from this part of the project are inclusive at the level of analysis possible and in the time available, the approach has some advantages. Authorities using a geographic information system may be able to identify specific factors from a more extensive search of spatial data. It is likely that this approach will be extremely useful in identifying critical assets which may require unique maintenance strategies. However, it may also allow authorities with incomplete data sets to detect specific problems which can be rectified quickly to control spiralling costs associated with increasing numbers of repairs in particular parts of the network. It is inevitable that geographic information systems will play a greater part in asset management in the future and authorities will recognise its power when they gain experience in using it not simply as a spatial data system but having it integrated with other systems that record asset details.

3.4.2 Predictive models with filtered data

In this section various models are considered for predicting failures on individual pipes. Different models are recommended for different purposes. Two types of model are considered; one based on the *exponential function* and one based on a *power function*. Both of these model types give failure rates which increase with time. Details of the equations, fitting methods and performance are given in later sections but are summarised below.

For a direct prediction of the number of failures since construction, the power model performed better than the exponential. Both models can be fitted using ordinary *least-squares techniques*. The power model tended to underestimate the numbers of failures. If this is a problem in the application the exponential model, which tended to overestimate the number of failures, should be used. Both the models have the disadvantage that they can give an estimate of the number of failures at some time in the future that is smaller than the number that has actually occurred at the present date or, conversely, suggest that a greater number has occurred than the history indicates. This problem can be overcome by using one of the *interfailure time models* discussed later.

To predict the time to a given number of failures from the construction date (the failure time) the exponential model should be used, as it has better fit and smaller prediction errors than the power model. However, if underestimation is the major concern, the power model is less likely to underestimate the failure time than the exponential model. The same problems arise with the prediction of failure times as for the prediction of failure numbers above. Again both these models can be fitted using ordinary least squares.

To avoid the problems of the predictions not matching the historical data, we can predict instead, *the time to the next failure*. If the number of failures is required within, say ten years, it is necessary to predict successive failures until the sum of the interfailure times exceeds ten years. The exponential model, on average, has been shown to fit the data better and give smaller prediction errors. However, a non-linear least squares procedure, preferably with the facility to add constraints, is required to fit the model. These procedures are becoming available in standard computer packages, e.g. *Solver* in Excel5. If similar computing facilities are not available it is recommended that the restricted power, square root, model be used. It can be fitted using ordinary least squares and its performance was reasonable compared with the exponential model.

3.4.3 Comparison of models

As pipes age it is expected that failures become more frequent due to the effect of corrosion and wear and tear on the pipe. For the purposes of this study the only models considered are those which have this increasing failure rate. The models discussed were compared by examining their performance on data sets provided by Melbourne Water.

The power model is given by:

$$n_i = \alpha t_i^{\beta} e_i \qquad \text{...(3)}$$

where:

n_i is the number of failures during the time t;
t_i is the time since construction;
α and β are regression coefficients to be estimated for each pipe;
e_i is a random error term.

The exponential model is given by:

$$n_i = \alpha \exp(\beta t_i) e_i \qquad \text{...(4)}$$

where the symbols are as defined above.

A third type of model was also used to calculate probabilities for one of the data filtering procedures. In this model the number of failures in a given time since construction was assumed to be a random variable having a **Poisson distribution.** The mean of this distribution depends on the time since construction. It is given by αt^{β}.

Since the mean is not constant, as in the usual Poisson model, this is known as a non-homogeneous Poisson process. When predicting the number of failures on a single pipe, the non-homogeneous model and the least-squares power model give the same results even though the theoretical approaches are different.

The models are used in three ways. These are:

- to predict the time to the next failure, in which case the number of failures is the independent variable;
- to predict the number of failures that can be expected to occur in a given time interval, when time becomes the independent variable;
- to predict the time between successive failures, in which the failure numbers become the independent variables.

Each of these uses requires a different approach for estimating the regression coefficients and the research indicates that no one model is best for all types of prediction.

Two criteria were used to assess the performance of the models. The criteria were fitted to all pipes with five or more failures. Firstly, the models were fitted to the whole failure history of each pipe and the standard deviations of the actual data points from the fitted models were calculated. This gives a measure of the goodness-of-fit of the model. Secondly, the models were fitted to the failure history with the last failure deleted, then this model was used to predict the final data point. This gives a measure of the accuracy of the model's predictions. In the applications of these models the second criterion of predictive accuracy was expected to be the more important.

3.4.4 Prediction of failure numbers

As stated above, the non-homogeneous Poisson model gives the same results as the power model for this application, so it will not be considered further here.

In order to fit the remaining models using ordinary least-squares techniques the equations must be made linear in the coefficients α and β. This is done by first taking logarithms of both equations. (Here and elsewhere the word logarithm and symbol log refer to natural logarithms, unless otherwise stated.) This gives the linearised models:

$$\log(n_i) = \alpha_1 + \beta\log(t_i) + \log(e_i) \qquad \text{...(5)}$$

for the power model, and:

$$\log(n_i) = \alpha_1 + \beta t_i + \log(e_i) \qquad \text{...(6)}$$

where:

$\alpha_1 = \log(\alpha)$.

Before calculating the standard deviations of the errors about the estimated models and the prediction errors, the models were converted back into their non-linear form. Table 3.15 gives a summary of results from the fitting procedure.

TABLE 3.15

PERFORMANCE OF MODELS FOR PREDICTING FAILURE NUMBERS

Model	**Number of pipes**	***First quartile***	***Median***	***Third quartile***
		RINGWOOD DATA - ALL PIPES		
		Standard deviation of errors		
Exponential	83	3.14	6.122	12.17
Power	84	1.08	1.632	3.277
		Prediction errors		
Exponential	83	-9.283	-3.892	-0.1686
Power	84	1.268	1.921	2.736
		SUNSHINE DATA - 100 mm DIAMETER AC PIPES		
		Standard deviation of errors		
Exponential	246	4.74	7.324	13.15
Power	247	1.225	1.606	2.089
		Prediction errors		
Exponential	246	-9.703	-4.622	-1.701
Power	247	1.496	2.142	2.812
		SUNSHINE DATA - 100 mm DIAMETER AC PIPES		
		Standard deviation of errors		
Exponential	234	4.072	7.333	12.42
Power	235	1.081	1.320	1.695
		Prediction errors		
Exponential	234	-12.10	-5.399	-1.363
Power	235	1.679	2.039	2.580

Note: Models are assessed on their prediction of the most recent failure using a data set with the last failure excluded.

It can be seen from table 3.15 that the power model has smaller standard deviations and smaller prediction errors. It is also readily seen that the power model consistently underestimates the number of failures, while the exponential model consistently overestimates. Because the model will be used primarily for prediction it is recommended that the power form be used.

3.4.5 Prediction of failure times from construction

As in the prediction of failure numbers the models must be rearranged and linearised for estimation. This gives the equations:

$$\log(t_i) = \alpha_2 + \beta_2 \log(n_i) + \log(e_i') \quad \text{.......................................(7)}$$

for the power model, where:

$$\alpha_2 = \log(\alpha)/\beta$$
$$\beta_2 = 1/\beta$$
$$e' = 1/\beta e$$

and for the exponential model:

$$t_i = \alpha_2 + \beta_2 \log(n_i) + \log(e_i') \quad \text{.......................................(8)}$$

where the symbols are as defined above.

As before, the models are compared on the standard deviations of the errors and the prediction errors.

TABLE 3.16

PERFORMANCE OF MODELS FOR PREDICTING FAILURE TIMES

Model	Number of pipes	*First quartile*	*Median*	*Third quartile*
RINGWOOD DATA - ALL PIPES				
		Standard deviation of errors		
Exponential	82	1.16	1.757	3.692
Power	84	3.459	6.061	11.04
		Prediction errors		
Exponential	81	0.6291	1.266	2.156
Power	84	-8.386	-4.518	-1.446
SUNSHINE DATA - 100 mm DIAMETER AC PIPES				
		Standard deviation of errors		
Exponential	246	1.663	3.166	5.906
Power	246	2.523	3.987	6.539
		Prediction errors		
Exponential	246	0.8179	1.665	2.764
Power	246	-9.834	-4.675	-2.104
SUNSHINE DATA - 100 mm DIAMETER AC PIPES				
		Standard deviation of errors		
Exponential	234	1.121	1.848	3.141
Power	235	3.286	4.616	6.209
		Prediction errors		
Exponential	234	0.6358	1.334	1.932
Power	235	-9.472	-6.418	-4.176

Note: Models are assessed on their prediction of the most recent failure using a data set with the last failure excluded.

In this case the exponential model performs better using both the standard deviation and prediction errors criteria. The exponential model tends to underestimate the time to failure while the power model overestimates.

3.4.6 Prediction of times between successive failures

The simple power and exponential models above have the disadvantage that they can predict a failure time that is smaller than the previous failure time or predict fewer failures than have actually occurred. To overcome this *the times between successive failures* can be modelled. This avoids the problems of predicting a next failure time which is earlier than the present date but results in a more complicated model which requires iterative methods to estimate the coefficients.

The model proposed by Minetti (1994) uses as the underlying model the exponential relationship. Rearranging this model to find the interfailure times gives:

$$T_n = 12\alpha \log\left(1 + \frac{100}{100n + \beta L}\right) \quad \text{.........(9)}$$

where:

T_n is the time interval between breaks n and $n+1$;
L is the length of section;
α, β are regression coefficients;
n is the failure number.

If the power model is used a non-linear relationship results:

$$T_n = \left(\frac{100(n+1)}{\alpha L}\right)^{1/\beta} - \frac{100n}{\alpha L^{1/\beta}} \quad \text{.........(10)}$$

This model has to be estimated using non-linear techniques. Because of its form, the most popular of the standard algorithms for this technique, the Gauss-Newton algorithm, fails because the gradient matrix is singular. An alternative power model, in which the value of the power $\beta > 0$ is recommended to overcome this difficulty. Studies by Constantine and Darroch (CSIRO) indicate that estimating over the whole data set gives a value of β close to 2. This suggests that the failure rate increases linearly with time. Assuming that β is exactly 2 the following model can be used:

$$T_n = \alpha + \beta\left(n^{1/2} + (n+1)^{1/2}\right) \qquad \text{.......................................(11)}$$

where the symbols are as defined above. This model can be fitted using the standard least-squares methods.

Fitting the interfailure times directly gives the earlier interfailure times a greater weight in the prediction than the recent past. The earlier interfailure times are generally many times larger than the later failure times. To avoid this problem it is recommended that the fitting procedure minimises the sum of squares between the estimated cumulative failure times and the failure dates: that is, the sum of squares is minimised:

$$TSS = \sum\left(fd_i - \left[cd + T_1 + \text{.........} + T_i\right]\right)^2 \qquad \text{....................................(12)}$$

where:

fd_i is the ith failure date;
cd is the construction date.

Alternatively the failure date can be adjusted by subtracting the construction date: that is, by calling the construction date time zero and then using the cumulative estimated failure times, giving:

$$TSS = \sum\left(ft_i - [T_1 + \text{......} + T_i]\right)^2 \qquad \text{.......................................(13)}$$

where:

ft_i is the *i*th failure time since construction.

This procedure cannot be applied to the square-root model without changing it from a single-variable model to one containing two variables since the fitted model becomes:

$$cft_n = \alpha n + \sum \beta\left(i^{1/2} + (i+1)^{1/2}\right) \qquad \text{.......................................(14)}$$

where:

cft_n is the cumulative fail time.

However, since this model contains two variables and no intercept, the least-squares equations can be easily derived and are given in the case study (Appendix I).

The interfailure time model for the exponential case and the power model with unknown β are non-linear in the coefficients. This means that the ordinary least squares equations cannot be used to estimate the model. Instead some iterative procedure is required. These procedures are available in statistical software packages and are available in some spreadsheet programs, such as the add-in *Solver* in Excel5. Since these are iterative procedures they require an initial solution or guess for the coefficients α and β. Problems may be encountered when finding the solution for the exponential type model because of the presence of the β inside the logarithmic expression. If the algorithm choses a value of β between - 100/L and zero this will result in a divide by zero error or a logarithm of a negative number. Unless the software used has special error recovery procedures this will cause termination of the procedure with a failure message. In Excel5's *Solver* it is possible to place restrictions on the values of the coefficients. However since the two constraints required, $\beta > 0$ and $\beta < -100/L$, are incompatible it is necessary either to run the algorithm twice, once with each constraint, or use an integer linear programming formulation. (See for example Taha, Operations Research - An Introduction, Collier-Macmillan, 1971, Section 8.1.2.) Because of the inefficiencies introduced by integer programming it is generally quicker to run the optimisation procedure twice. Alternatively, since the failure rate is assumed to be increasing, β can be restricted to values greater than zero only, which is the procedure adopted here when fitting models.

TABLE 3.17

PERFORMANCE OF MODELS FOR PREDICTING TIMES BETWEEN SUCCESSIVE FAILURES

Model	**Number of pipes**	***First quartile***	***Median***	***Third quartile***
		RINGWOOD DATA - ALL PIPES		
		Standard deviation of errors		
Exponential	82	1.576	3.124	4.639
Power	84	1.326	2.383	3.788
		Prediction errors		
Exponential	81	-0.546	0.4941	2.096
Power	84	-1.832	0.2363	2.857
		SUNSHINE DATA - 100 mm DIAMETER AC PIPES		
		Standard deviation of errors		
Exponential	57	1.467	2.236	2.655
Power	246	1.361	2.015	2.837
		Prediction errors		
Exponential	42	0.181	0.783	2.873
Power	246	-1.282	0.6649	3.177
		SUNSHINE DATA - 100 mm DIAMETER AC PIPES		
		Standard deviation of errors		
Exponential	52	1.404	2.181	2.892
Power	235	1.521	2.034	3.126
		Prediction errors		
Exponential	54	-0.979	-0.133	1.402
Power	235	-2.039	0.2807	2.488

Notes: Models are assessed on their prediction of the most recent failure using a data set with the last failure excluded.

Fewer pipes were fitted using the exponential model than the power model as the requirement that β be greater than zero meant that model for each pipe had to be fitted individually rather than automatically using a computer program.

As can be seen in table 3.17, the exponential model generally performs better than the power model hence it would be preferred provided the software is available for fitting the model. However, if the facility is not available then the power model can be fitted using the standard least-squares method with some loss of predicative accuracy.

3.4.7 Modelling using incomplete data

It is possible to build prediction models similar to the ones discussed previously in this section by replacing the number of failures (n) with n+k where k is the number of known failures. The first recorded failure is denoted as one with n being the number of unknown earlier failures. Thus n is an extra parameter to be estimated, giving a total of three for all the models that have been considered. Such a model should, therefore , not be fitted to a pipe with fewer than four known failures.

The addition of n as a parameter means that all the models are non-linear in the parameters, so non-linear fitting techniques must be used. These are available in all statistical packages and many modern spreadsheets. Most of these use the Gauss-Newton algorithm, which is easily programmable. Many texts on numerical analysis contain subroutines in various languages.

Since all the models are now non-linear the only model to be discussed here will be the model for times between successive failures, though similar comments apply to the other failure models.

The exponential model now becomes:

$$T_{n+k} = 12\alpha \log\left(1 + \frac{100}{100[n+k] + \beta L}\right) \qquad (15)$$

where:

T_{n+k} is the time between breaks $n+k$ and $n+k+1$ (months);
L is the length of the section;
α, β are regression coefficients.

For the square-root power model the equation becomes:

$$T_{n+k} = \alpha + \beta\left([n+k]^{1/2} + [n+k+1]^{1/2}\right) \qquad (16)$$

As for the exponential model, a negative β, which often occurs during the iteration process, will cause the algorithm to fail due to the negative argument in the logarithm. To avoid this a constraint must be able to be placed on β. This is not possible on all non-linear least-squares programs. It is possible, however, to do this with Excel5's *Solver* add-in. It is also possible to constrain n (the number of previous failures) to be a non-negative integer, which gives a more realistic model. Alternatively, as there are only a small number of possible n values to consider, it is possible to try various values of n and choose the one that minimises the total squared error. An example of this procedure is given in the case study (Appendix I).

The exponential model will generally give better predictions and is hence preferred. If the operator is working with software that does not allow constraints to be added the power model can be used. There will be some reduction in predictive accuracy. It should be remembered that, similar to the complete data model, better predictions result from a model that is fitted by using cumulative failure times rather than individual interfailure times.

3.5 Summary

After a description of the data used, this section establishes a definition of the term *deteriorating physical condition*, which is used in the development of a failure prediction model. The assumption made is that a pipe's failure history, if it is represented by *inherent* failures only, can be used to predict its future behaviour. It is further assumed that the plot of cumulative number of failures against time results in a smooth curve. Hence any failure point that departs significantly from the smooth curve should be considered as emanating from a cause that does not relate to the inherent physical condition of the pipe. This failure, caused by an *imposed* factor, must be removed before a failure prediction model is finalised. The removal of failures due to imposed factors is performed by a process known as *filtering*, which detects causes such as faulty operation, bad repair, accidental damage and intentional damage. Trials on data from Melbourne and Sydney resulted in between 10% and 18% of all breaks being filtered. However, Authorities should develop their own filtering process after gaining a good understanding of their own operating environment and the part played in it by imposed factors in pipe failure. A method is supplied to validate the filtering process. Finally, the filtered data is used in a predictive model to forecast the time between successive failures. An evaluation of different models concluded that the exponential model forecasting interfailure times was preferred on the basis of its predictive ability.

Note: An Excel *Solver* application for the Failure Model is available from the Infrastructure research Unit, Department of Civil and Geological Engineering, RMIT.

4 DECISION MODELS

4.1 What are the Objectives of a Decision Model?

In the management of water mains, a water-supply authority is confronted with various different, although related, issues.

Firstly, there are several direct impacts that must be evaluated in responding to a burst main.

- Should the main be placed on a list for short-term replacement?
- Is the burst likely to have a significant impact on the authority, either financially or strategically?
- Does the burst indicate that the management policy for this asset and possibly other similar assets needs review?
- What financial impost is likely from this main with regard to the short-term and long-term future?

Secondly, there are indirect impacts which the asset manager needs to assess.

- Are some customers suffering an excessive number of supply interruptions?
- Is water quality being adversely affected by the aging mains?
- Are public health issues likely to arise?
- To what extent are leaks and valve malfunctions impacting on the efficient operations of the water authority?

Ideally a decision model should have the capability of assisting the authority to deal with all these issues.

4.1.1 Short-term and medium-term planning

The model should be able to act as a decision tool in determining whether a water main that has required many maintenance events should be replaced before the next failure or should continue to be repaired. The model should assist in preparing a list of water mains needing imminent replacement and in developing a suggested priority order for these mains as well as providing cost estimates for input into the annual budget. It should, in addition, provide estimated costs likely to be incurred by the water authority for the maintenance and replacement of water mains on rolling periods of three to five years.

4.1.2 Long-term planning

The model should be able to provide estimates of the expected long-term costs likely to be faced by the water-supply authority in maintaining the water-supply system. This will assist with long-term strategic planning related to managing the water-supply infrastructure. Also, if applicable, it should be able to direct the authority towards the achievement of long-term social objectives, which may form part of its charter.In Research Report No.17, published by the Urban Water Research Association of Australia (UWRAA), an asset-management system was developed for pipeline assets. This report concentrated on sewer mains rather than water-supply mains. Nevertheless, the overall structure of an economic decision model and how it could be integrated into an asset-management system, as discussed in section five of Report No.17, is certainly relevant to this study.

4.2 Decision Criteria

There are many decision criteria that may be relevant to the management of water-mains assets. The particular decision model used should be able to generate a response based on the criteria which the particular water authority decides are important. It is clear from a survey of water authorities in

Australia that there is a wide range of opinion as to which criteria should be used in determining asset-management policy for water mains. The details of the survey are discussed in section 4.

The survey respondents indicated that they considered the following criteria to be important in evaluating water mains:

- *minimising direct monetary costs of water-mains maintenance or replacement.* Direct monetary costs include the cost of repairing mains failures, the cost of replacing mains, site reinstatement costs, provision of alternative supply, clean-up costs, the value of lost water, and third-party damage directly attributable to mains failure;
- *minimising direct plus indirect monetary costs associated with water-mains maintenance or replacement.* Indirect monetary costs include losses to business because of water-supply disruption, losses to business from traffic disruption, and costs of repairs to or relocation of other services affected by the failure of a main;
- *minimising direct plus indirect monetary costs plus non-monetary costs.* Non-monetary costs associated with the failure of a water main include disruption to non-business consumers resulting from loss of supply, traffic disruption, loss of prestige to the water-supply authority because of unplanned supply disruptions, environmental impacts and safety hazards created;
- *consideration of water-supply service issues such as* water quality, inadequate pressure and leakage;
- *incorporation of customer service contracts into the decision process.*

In recent times, as water authorities have been moving towards a corporatised or privatised structure, many authorities have introduced proposals to guarantee an upper limit to the number of annual unplanned supply interruptions to their customers. Depending on the level of any penalty for non-compliance with this 'contract', it may be that this criterion will come to play a major role in determining mains asset-management policy. Unless the decision model is sufficiently flexible to include this criterion, the planning directions of the authority may be in error.

4.3 Decision Models Used by Australian Water Authorities

A survey of Australian water authorities was undertaken in March 1995. Its objective was to discover the types of decision models that are currently used, as well as to determine their perceived requirements of these models. Details of the questions asked, and the responses made, are included in Appendix E.

Conclusions drawn from the survey responses included:

- Most authorities considered that the management of water mains represented a significant cost to their organisation. In general, the only authorities who were not concerned with this issue owned relatively young assets in which failures were infrequent.
- The greater number of authorities did not use a formal economic model to aid their water-mains management decisions and tended to adopt a reactive approach. However, the larger authorities were generally using some formal model. Many of the authorities that were not using any formal model were inhibited from doing so because they did not have an electronic database of mains assets and maintenance history available. Some authorities in this category stated that they were currently in the process of establishing such a database.
- Of those authorities which were using a formal economic decision model, nearly all used a **Discounted Net Present Value** approach. One authority used the **5% Rule** (explained in section 4.4.1).
- There was a range of opinion as to the type of indirect and non-monetary costs, which should be included in a decision model. Most seemed to believe that indirect monetary costs should be included, while opinion was fairly evenly divided as to whether or not non-monetary costs should be included in the model.
- There was a wide range of other factors that the water authorities considered were important in the decision process. The most commonly mentioned of these was 'customer service'.

4.4 Selecting a Suitable Decision Model - Evaluation of Some Models

There are a number of decision models available, some of which are in use by different water authorities in Australia. The suitability of these models to meet the objectives discussed above is considered in some detail. As the scope of this report is to address issues relating to the management of individual water mains, the features of each of the models will be considered with this emphasis, although the strategic planning value of each of the models will also be mentioned. The models for consideration were chosen because they are currently in use or are perceived as being of value by the Australian water authorities in their responses to the March 1995 questionnaire.

The decision models evaluated are:

- the 5% Rule;
- the Net Present Value Method;
- the Criticality Model;
- multi-objective analysis;
- a model to incorporate customer service contracts.

4.4.1 The 5% Rule

The 5% Rule is simply stated as: *repair when repair cost per annum is less than 5% of capital replacement cost*

This approach has a long history of use by Melbourne Water. It is simple to apply and is very useful in assisting with the repair/replace decision. Its disadvantages are that it is reactive and it is not amenable to long-range forecasting and budgetary planning. A comparison of likely outcomes compared with the Net Present Value model is shown in section 4.7.2.

4.4.2 Net Present Value Models

The general approach in Net Present Value (NPV) models is to:

- identify a range of alternatives to be considered;
- identify the benefits and costs which flow from each alternative;
- discount the flow of benefits and costs to current-day values by using an appropriate discount rate;
- select the alternative which has the maximum net present value.

Most users of this approach would also require the net present value to be greater than zero, although this requirement may be waived for projects which enhance the general social well-being of the community. In applying this model to the analysis of water mains, it is usual to consider costs only, as benefits are assumed to be identical in each alternative.

The Simple Net Present Value Model

Using this methodology, it is therefore possible to develop a Simple Net Present Value Model modified to assist with the replace/repair decision. The decision criterion for this model is to ***list a main for replacement at a time when the total discounted cost of all predicted future repairs to the main will exceed the current replacement cost of the main.***

In applications of this methodology difficulties always arise in determining the scope of the economic impact of the project alternatives, the costs that should be included, the monetary value to be assigned to those costs, and the appropriate discount rate to apply. It is common to include only direct monetary costs in the analysis, ignoring both indirect monetary costs and non-monetary costs. If these latter costs are significant, then the analysis is likely to be of little value.

When applying this rule to the management of water mains, an additional difficulty arises in determining the appropriate analysis period. In normal circumstances it makes little difference if there is uncertainty in the value of the analysis period. This is because, at conventional discount rates, the discounted values of costs incurred beyond 30 years into the future are almost negligible, provided the flow of cost events is fairly regular with time. When costs associated with repairs to water mains are considered, however, this situation does not occur because of the exponential nature of the increase of mains structural failures with age, which is generally the norm for failing mains. This tends to balance the effect of discounting the future costs. As a result, the discounted value of the costs of future repairs is sensitive to the length of the analysis period adopted.

The General Net Present Value Model - Total Future Cost

A preferred approach is to consider the life history of the water main. If the main suffers a burst then it could be repaired with no further action taken. Alternatively, it could be listed for replacement when funds are available. After the main is repaired, it can be expected to fail again some time in the future, at which time this same situation must again be faced.

If the alternatives to be analysed are either to repair with no further action or to replace in the near future, then the costs which should be considered are:

- the discounted cost of predicted repairs of the current main until replacement;
- the discounted cost of the installation of the future replacement main;
- the discounted cost of predicted repairs for the new main until the end of the analysis period.

The Total Future Cost is the sum of these three cost components.

Since no action is likely unless it is in response to a burst, then it is probably best to analyse the Total Future Cost in terms of mains bursts rather than years. The decision can be determined based on a range of scenarios.

- What is the Total Future Cost if replacement is made before the next failure?

- What is the Total Future Cost if replacement is made after the next failure?
- What is the Total Future Cost if replacement is made after two failures (or any other number of failures) into the future?

Adopting this methodology leads to a General Net Present Value Model. The decision criterion for this model is to ***list a main for replacement at a time when the discounted value of Total Future Cost is a minimum.***

The advantage of the General NPV Model over the Simple NPV Model is that it is possible to determine the optimal replacement time for the main without having to estimate its economic life. In addition, the General NPV Model will allow the user to predict the total future maintenance cost for the main.

The application of a General NPV Model to the analysis of individual water mains was adopted by Clark, Stafford and Goodrich (1982) in which they developed a theoretical model incorporating these concepts and applied the model to the analysis of mains owned by two utilities in the USA. They used the exponential burst predictive model of Shamir and Howard in their economic model to arrive at an optimal replacement time for a main. Their model did not include the cost of repairing future bursts in the replacement main although this issue was considered separately.

Walski and Pelliccia (1981) devised a similar model to determine optimal replacement times for water mains. Their model allowed for direct repair costs only and assumed an exponential growth rate of future failures. It did not allow for any future failures in the replacement main. Using general cost data, the model was applied to analyse water mains in the city of Binghampton in New York State, USA. A list of mains was identified that the model recommended for imminent replacement.

Walski (1987) also adopted an approach similar to that in the General NPV Model to determine optimal replacement times for water mains. His model included allowance for leakages and replacement of damaged valves. It assumed a power law for estimating future bursts but also did not include an allowance for any costs attributable to the new main.

4.4.3 The Criticality Model

The use of the **Criticality** concept in the analysis of water mains was the subject of Research Report No.57 - Identification of Critical Water Supply Assets published by the Urban Water Research Association of Australia. In this report the authors state:

> *(the) Criticality Model is based on loss potential, which combines both severity (or consequences of failure) and failure risk (likelihood of failure) to give a Criticality value or ranking for any asset. The units of Criticality are probable cost/year.*

The severity component includes all direct and indirect monetary costs as well as non-monetary impacts, which have been assigned a value based on a shadow pricing approach.

Application of this Criticality concept would allow a water authority to produce a ranked list of mains assets in order of potential financial impact on the organisation.

Although such a list is obviously important for strategic planning in the organisation, it would not in itself be directly helpful with the replace/repair decision. Further, just because an asset has a high Criticality value it does not necessarily mean that it should be the target of investment. The deciding criterion should be marginal Criticality - that is, investment should be made in the assets that will suffer the maximum reduction in Criticality as a result of this investment.

Sydney Water has implemented a decision process based on the Criticality concept (Tsui,1995 B) with a view to identifying high-risk assets so that effort can be focused on controlling risk to the organisation. Investment is made on the assets which will provide the greatest reduction in Criticality for each dollar of investment. Risks considered are financial, safety, environmental and customer service.

Sydney Water has found it desirable to adopt two significant variations from Research Report No.57. Firstly, this authority has redefined Criticality to mean severity of failure only with the probability of failure being treated separately. Secondly, each different type of risk, although quantified, is treated separately. Risks are not added together as in Research Report No.57. This risk assessment process is applied to high Criticality assets only. Decisions on the maintenance of low Criticality assets are made using a Net Present Value method.

4.4.4 Multi-objective analysis

In situations where there are several objectives that the analyst is attempting to optimise, multi-objective analysis has been used with some success, particularly so when some of the objectives are not readily expressed in monetary terms. The principles of the multi-objective analysis technique, together with many case studies, is described in a book by Goicoechea, Hansen and Duckstein, Multi-objective Analysis with Engineering and Business Applications, John Wiley and Sons (1982).

In principle, the technique is well suited to the water-main asset-management problem where the objectives can be identified, as described above, into direct- and indirect-cost monetary components affecting mains maintenance plus non-monetary components of customer service contracts, water pressure, water quality, social, political and environmental impacts and public perceptions of the performance of the water authority.

Although the non-monetary objectives are treated independently of the monetary criteria, it is still a requirement that they must be quantified in some way in order for the analysis to proceed. Obviously a fair degree of effort would be required to quantify these non-monetary objectives if, indeed, it were possible at all. Consequently, it is probably realistic to attempt to apply this technique to major assets only.

The approach adopted by Sydney Water, as outlined above, in dealing with assets which are associated with high-potential losses to the organisation, is a type of multi-objective analysis in so far as each of the objectives of financial risk, safety risk, environmental risk and customer-service risk is treated separately with the aim of producing an optimal solution, which minimises total risk exposure for the organisation.

The Engineering and Water Supply Department of South Australia (EWS) uses an **expert system** to assist the analyst in assigning a points score to the various monetary and non-monetary objectives relating to water-mains investment strategy. The analysis does not adopt a multi-objective approach, however, as all the points are then combined together to produce a single points value from which the optimal solution is sought.

4.4.5 A model to deal with customer-service contracts

Many water authorities in Australia have now adopted some form of customer-service contract, in which the authority guarantees an upper limit to unplanned supply interruptions during each supply period designated. For those authorities with aging assets the likely impact of this contract is uncertain. It is quite possible in some situations that the need to meet this objective may over-shadow any other objective including financial ones. In principle, it would not be difficult to construct a decision model that embraced the requirements of this objective, provided a database of water mains, together with a database of burst histories was available. A major problem to be addressed before such a model could be created is the need to be able to identify shut-off blocks and relate each main to its appropriate shut-off block(s). Hence, as well as the problems of burst prediction associated with the other decision models discussed, the ability to construct a model incorporating a customer-service criterion depends upon a database of mains assets including complete shut-off block data.

A customer service criterion could be included in each of the decision models discussed above. Hence this model is best regarded as a variation of the previous four. Nevertheless, a model which incorporates a customer-service criterion differs from the other models in the range of management responses available to the water authority if the performance criterion is not met. In the other models, the alternatives evaluated after repairing the burst may be:

- take no further action;
- list the main for possible replacement.

In a model which includes customer service considerations the alternatives being evaluated after repairing the burst may be:

- take no further action but include the costs of any possible customer redress in the analysis;
- include additional shut-off valve(s) in the distribution network to reduce the impact of failure of adjacent mains upon a particular customer;
- list the main for possible replacement.

In order to include these criteria in the decision model, it is important that the financial impact of failure to meet the customer-service criterion can be quantified.

4.4.6 Summary of the evaluation of decision models

Each of the decision models discussed has some advantages and disadvantages. The 5% Rule is simple to apply but is reactive and of little value for long-term strategic planning. The Net Present Value methods, particularly where a Total Future Cost model is adopted, can be used to aid both the short-term and long-term decision making process. However, Net Present Value Models do have difficulty dealing with non-monetary cost components. The Criticality approach is very helpful to assist strategic planning particularly for assets that threaten the authority with major risk exposure, but it would be extremely time consuming and impractical for most water authorities to use this decision tool to aid the management policy for low Criticality assets. Multi-objective analysis is conceptually sound but is probably impractical for dealing with any but major assets. Finally, customer-service objectives can be included in each of these models provided the asset databases are suitably developed and the costs associated with customer redress can be clearly defined.

A summary comparing each of the models is shown in table 4.1.

TABLE 4.1 COMPARISON OF DECISION MODELS

Method/model	Decision criterion	Advantages	Limitations
The 5% Rule	Replace a main when the expected annual maintenance cost exceeds 5% of the replacement cost.	• Simple.	• Reactive. • Limited use for long-term planning.
Simple Net Present Value	Replace a main now if the discounted value of expected costs of future bursts exceeds the current replacement cost.	• Easily understood. • Widely used. • Useful for the repair/replace decision.	• Analysis period adopted is critical especially under regime of increasing failures. • Indirect and intangible costs difficult to value.
General Net Present Value	Replace a main at a time when the sum of discounted value of expected costs of future bursts plus the discounted cost of replacement is a minimum.	• Easily understood. • Useful for the repair/replace decision. • Useful for long term planning.	• Indirect and intangible costs difficult to value.
Criticality (as in UWRAA Report No.57)	Invest in mains which will result in the maximum reduction in Criticality.	• Useful for managing mains with a high loss potential to the water authority. • Can readily include a wide range of impacts.	• Too complex for medium and low Criticality mains. • Indirect and intangible effects difficult to value. • Criticality calculation sensitive to errors in estimating the probability of occurrence of impact.
Multi-objective analysis	Optimise an objective function consisting of the sum of individual independent objectives.	• Can readily include a wide range of impacts. • Requires analyst to quantify or rank all impacts. • Requires authority to quantify a trade-off between different objectives.	• Complex. • Not generally well-known. • Difficulties in quantifying or ranking impacts.
Customer-service	Initiate appropriate management response when there are more than a specified number of unplanned supply interruptions for any customer.	• Not a new model, but is merely an additional restraint to apply to the four models above. • Important issue to most water authorities.	• Needs data base to support full shut-off block data.. • May be difficult to quantify customer redress. • More complex management response required than for other models.

After consideration of the advantages and disadvantages of each of the models discussed, it is recommended therefore that, after high-Criticality water mains have been identified and treated separately, a General Net Present Value Model based on the Total Future Cost concept should be adopted as the decision model to use for other water mains. These mains generally constitute the bulk of the water mains controlled by water authorities in Australia.

This method is recommended because:

- it is well understood and widely used;
- it is appropriate to aid both short-term decisions relating to an individual asset and long-term strategic planning of the organisation;
- it is very flexible and allows each organisation to consider different cost elements in its decision process.

This last feature is important as the survey of Australian water authorities shows that there is a wide variation of opinion as to which costs should be included in the analysis.

4.5 Implementing the Total Future Cost Model

It was suggested above that the General Net Present Value Model incorporating the Total Future Cost concept was the most appealing economic model. The reasons for this choice were:

- for the flexibility it offered in being able to readily incorporate a wide range of different costs;
- the ease with which it can be understood;
- its ready adaptability to both the short-term repair/replace decision and also to long-term financial planning.

4.5.1 General description of the model

The Total Future Cost is the sum of three cost components:

- the discounted cost of predicted repairs of the current main until replacement;
- the discounted cost of the installation of the future replacement main;
- the discounted cost of predicted repairs for the new main until the end of the analysis period.

The analysis is designed to advise on the ***alternatives*** to either:

- repair the main with no further action, or
- to replace it in the near future.

The decision criterion for this model is *to list a main for replacement at a time when the discounted value of Total Future Cost is a minimum*.

Inputs required for the analysis of water mains are:

- the discount rate which the water authority considers appropriate for this type of infrastructure;
- the repair cost, which consists of the sum of all the costs which the water authority considers should be attributed to the repair of the individual water main concerned, including direct and indirect monetary costs as well as the value assigned to any relevant intangible costs;
- the water-main replacement costs;
- the correlation coefficients, a and b (obtained from statistical analysis), which are used to describe the predicted burst behaviour of the existing main;
- the current main estimated burst number that has most recently occurred;
- an estimate of the maintenance category of the replacement main.

The output of the analysis consists of a series of present values each representing the Total Future Cost if replacement were made at each predicted future burst over a 50-years analysis period as well as the time when each burst is predicted to occur. The optimal replacement time is when the Total Future Cost is a minimum.

Under a particular combination of input parameters, the analysis may indicate that the optimal replacement time is at the current burst. In this situation, the main should be placed on a list of similar mains. In addition to providing this advice to the short-term problem of whether to repair or replace the main, the program also provides the expected present value of the Total Future Cost at each future burst and thus may also be used for strategic planning and budgeting.

4.5.2 Estimating the parameters - repair cost estimates

Some of the costs which a water authority may consider to be appropriate to include in the decision model were listed in section 4.2. A reasonable valuation of the direct financial costs associated with the failure of an individual water main would probably be available from the water authority's records. However, a valuation of the indirect financial and intangible costs would be much more difficult to achieve and would involve, in the latter case, adopting shadow pricing techniques in which a market value is assigned to non-market items such as environmental impacts. Substantial research, beyond the scope of this current project, would generally be required to obtain these valuations. The method used in the case study (see Appendix I) is to test the sensitivity of the output of the decision model to a range of indirect financial and intangible costs.

Research Report No.17, published by the Urban Water Research Association of Australia, gave consideration to the inclusion of indirect and intangible costs resulting from failures of sewer pipelines. The approach adopted, as outlined in Appendix D of Research Report No.17, was to apply a weighting factor, computed from consideration of indirect and intangible costs, to the direct repair costs.

4.5.3 Estimating the parameters - water-main replacement cost

The replacement cost of the main is the sum of all the current costs incurred by the water authority if the segment of the main under analysis is replaced. These costs would include all direct and indirect monetary costs plus the value of intangible costs, if any were considered appropriate to main replacement.

4.5.4 Estimating the parameters - failure predictions

As with any predictive model, its validity relies on an ability to estimate future events; in this case the future burst behaviour of the existing main as well as the future burst behaviour of the replacement main. Predicting the future burst behaviour of the current main is the subject of section 3.3 of this report and, provided some burst history is available, it is possible to make a reasonable estimate of future bursts. Extensive analysis of a range of pipe burst databases has indicated that an exponential curve gives the best estimate of predicted burst behaviour. The exponential relationship has been modified to produce a time-to-next-break relationship.

For the replacement main, and in situations where a burst history is not available for the current main, it is necessary to make an estimate of the future failure characteristics of the main. The method adopted in the analysis described in this report is to assume four different maintenance categories.

4.5.5 Water mains maintenance categories

In this report, water mains have been categorised into four groups defined by their burst history. These are:

- category 1, which contains low-maintenance mains (those defined as having no more than two bursts per 100 metres of their length in their first forty years);

- category 2, which contains medium-maintenance mains (those defined as having more than two but no more than eight bursts per 100 metres of their length in their first forty years);
- category 3, which contains high-maintenance mains (those defined as having more than eight but no more than fifteen bursts per 100 metres of their length in their first forty years);
- category 4, which contains very high maintenance mains (those defined as having more than fifteen bursts per 100 metres of their length in their first forty years).

An analysis of water-main burst data, obtained from Melbourne Water and covering four municipalities in Melbourne, was undertaken in order to examine the proportion of water mains that fall into each of these categories. Failures in mains constructed between 1948 and 1988 were examined. The year 1948 was chosen as the commencing year for the analysis as this was the first year in which complete burst data was recorded on the databases. Bursts in mains less than forty years old were proportioned in a linear fashion to estimate the bursts at forty years. Should the burst numbers in a main rise in an exponential fashion with time, then this approach may underestimate the number of bursts likely after forty years and therefore place the main in a lower category than is appropriate. The results of the analysis must therefore be interpreted carefully. Nevertheless, it is clear from examination of table 4.2, that by far the greatest proportion of water mains fall into the lowest maintenance category: the least proportion being 74.6% in Sunshine and the highest being 90.8% in Nunawading.

TABLE 4.2

PERCENTAGE OF MELBOURNE WATER MAINS CONSTRUCTED BETWEEN 1948 AND 1988 IN DEFINED MAINTENANCE CATEGORIES

Municipality	Category 1	Category 2	Category 3	Category 4
Nunawading	90.8%	8.1%	0.8%	0.3%
Box Hill	89.7%	8.0%	1.6%	0.7%
Heidelberg	78.9%	17.8%	2.4%	0.9%
Sunshine	74.6%	20.4%	4.1%	0.9%

4.5.6 Estimating the parameters - the replacement main

The analyst must decide into which category to assign the replacement main. Generally the analysis is not very sensitive to this decision provided category 1 or 2 is chosen. This is because the main being replaced is certain to have a significant failure history and therefore it is most likely that the new main will have much better maintenance characteristics. Hence the predicted growth rate of failures with a newer, better quality main will be much lower than for the current main since it will be starting on a lower base and growing at a lesser rate. In addition, if the main being replaced had a poor maintenance history, it would be sound management practice to ensure that the environmental and/or constructional problems that led to this situation were eliminated for the replacement main. These expectations, when combined with the discounting effect of future failures, tend to make the analysis fairly insensitive to the category assumption for the replacement main for most cases.

4.6 Computer Software Supporting the General Net Present Value Model

In order to facilitate determination of the optimum replacement time, computer software has been developed. This program is available from the Infrastructure Research Unit, Department of Civil and Geological Engineering at the Royal Melbourne Institute of Technology. The mathematical algorithms supporting the program, together with information on running the program are contained in Appendix F.

4.6.1 Program input

Inputs required for the use of this software are described in section 4.5.1 for the General Net Present Value Model. The inputs are:

- the discount rate;
- the repair cost consisting of the sum of all the costs that the water authority considers should be attributed to the repair of the individual water main concerned, including direct and indirect monetary costs as well as the value assigned to any relevant intangible costs;
- the replacement costs of the main;
- the correlation coefficients, α and β (obtained from statistical analysis), which are used to describe the predicted burst behaviour of the existing main;
- the current main estimated burst number that has most recently occurred;
- an estimate of the maintenance category of the replacement main.

If insufficient failure history is available for the main to estimate the correlation coefficients, the program offers the user the option of estimating the predicted burst behaviour by assigning the water main to a maintenance category described in section 4.5.4 for replacement mains.

4.6.2 Program output

The output of the program consists of a series of present values representing Total Future Costs if replacement were made at each predicted future burst over a 50-year analysis period as well as the time when each burst is predicted to occur. The optimal replacement time is when the Total Future Cost is a minimum. In addition to the numerical values, a graphic display is also available should the analyst wish to obtain a quick overview of the output.

4.7 Mains Asset-Management Policy - a Case Study

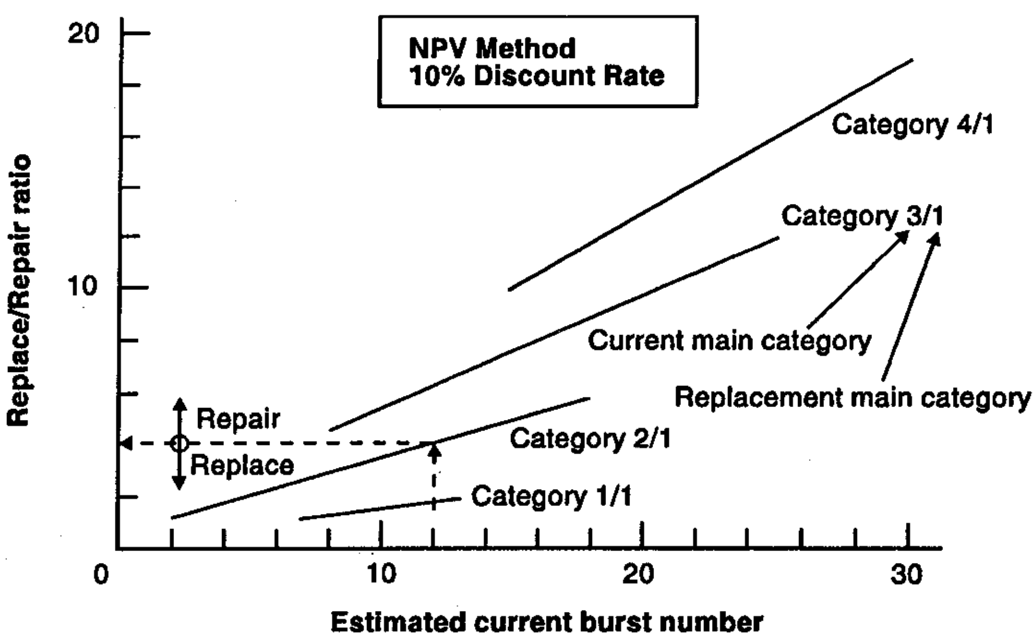
The computer program was used to examine situations which indicated that the optimal replacement time was at the current burst. In order to be able to incorporate a wide range of water-main repair and replacement costs into the analysis, a parameter designated the **replace/repair** ratio was introduced. This ratio consists of the replacement cost of the main, see section 4.5.3, divided by the total repair costs (section 4.5.2). The discount rate was taken as 10% and the mains maintenance characteristics for each of the four defined categories were considered.

4.7.1 The General Net Present Value Model

At each future burst number combined with each different maintenance category, there was a critical value of the **replace/repair** ratio which indicated that the minimum present value of Total Future Costs occurred at the present burst number. If the actual replace/repair ratio was above this critical value, then this model recommends that the burst main should be repaired. If the actual ratio was below the critical ratio, then the model advises replacement.

Figure 4.1

Plot giving critical replace/repair cost ratio for recommended imminent replacement of water mains of specified maintenance category and known burst history



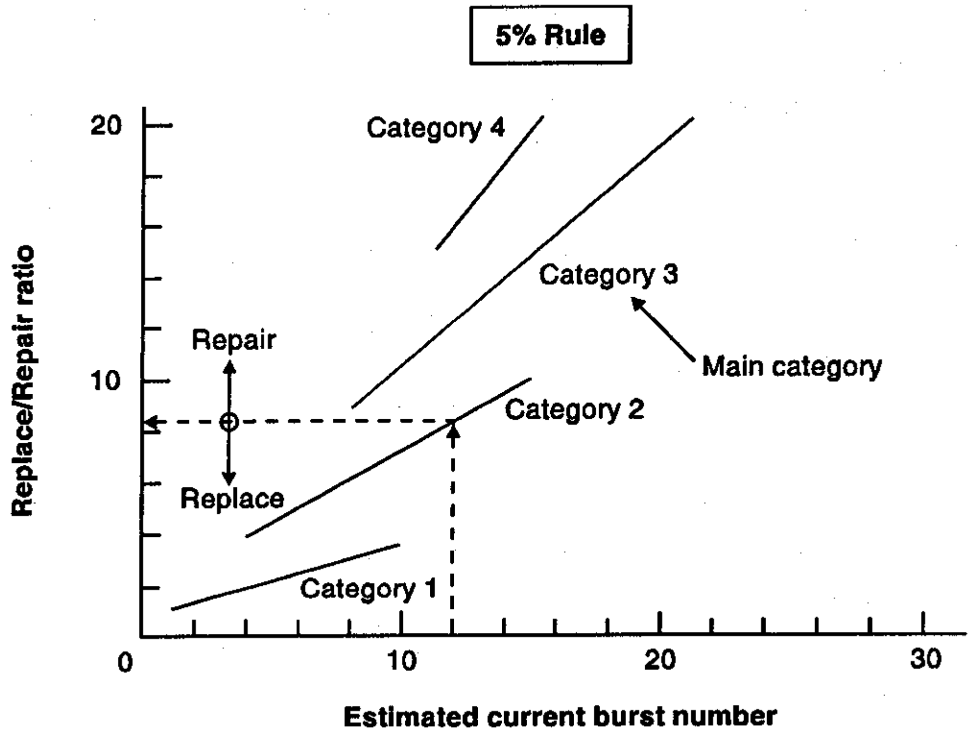
The results of the analysis shown in figure 4.1 are relevant to a discount rate of 10%. The example depicted in the figure indicates that if the current burst number is 12 (horizontal axis), the current main maintenance category is 2 and the anticipated new main maintenance category is 1 (i.e. select the graph labelled 2/1), then the main should be listed for replacement if the replace/repair ratio is less than 3.8 (intersection of the current burst number for the selected graph with the vertical axis), otherwise it should continue to be repaired.

4.7.2 The 5% Rule

The 5% Rule was represented for each of the four mains maintenance categories to compare the analysis with the General Net Present Value (NPV) Model. The results are shown in figure 4.2. Except for maintenance category 1 mains, each of the other categories had a significant vertical shift upwards when compared with the curves derived using the General NPV Model. This indicated that the 5% Rule would suggest mains replacement in many more cases than would the General NPV Model. Thus, for a main with the same estimated current burst number and main maintenance category as in the example above, the 5% Rule indicates replacement if the replace/repair ratio is less than 8.3 compared with 3.8 from the General NPV Model. That is, of these mains, those which have a repair cost between 12% and 26% of their replacement cost would be repaired under the General NPV Model and listed for replacement under the 5% Rule.

Figure 4.2

The 5% Rule replace/repair decision



4.7.3 The likelihood of a recommendation of imminent main replacement

Table 4.3 shows values abstracted from a report prepared by Scott in 1993. This report gives direct mains repair and replacement costs incurred by Melbourne Water in each of its four regions. The values shown in the table represent average replace/repair ratios for 100-metre long mains in the four regions in Melbourne in 1990.

TABLE 4.3

REPLACE/REPAIR RATIOS FOR 100-METRE WATER MAINS IN MELBOURNE IN 1990

Diameter of main	North region	East region	West region	South region
100 mm	19.4	16.2	16.2	13.4
150 mm	23.7	19.8	19.8	16.4

If this table is read in conjunction with figure 4.1, which shows replace/repair ratios for different mains categories in which the immediate replacement of the main would be recommended under the General Net Present Value Model, then it is clear that it would be a vary rare circumstance in which replacement would be advised by this model. In fact, for the cost structure shown in table 4.3, only short, very high maintenance mains (category 4) in the Southern Region are likely to be candidates for imminent replacement.

This conclusion is supported by Clark and Goodrich (1989) who modelled water-mains burst statistics in Cincinnati and New Haven in the USA. They concluded that *justification of replacement based on breakage or repair record alone will be difficult. Even with the most dramatic breakage rate, the direct economics of repair barely justify replacement.* They later add that other factors such as loss of pipe capacity and damage risks associated with the burst should be considered.

The conclusion may be entirely different, however, if costs other than direct repair costs were relevant for the main being examined. For instance if there were significant road reinstatement costs, third-party damage or supply disruption costs that the water authority wished to include in the decision process, then the replace/repair ratio may be substantially lower than the values shown in this table. The advantage of using the General Net Present Value Model is that each case can be analysed separately and a rational decision made with the water authority having the opportunity to include only the costs it considers to be important when determining the future management policy for the individual water main being assessed.

4.8 Summary

In this section the objectives of a decision model have been examined. It was suggested that the model should be able to provide the organisation with advice on both short-term and medium-term planning as well as assist with long-term strategic planning. A range of decision criteria which the model could be expected to embrace were discussed. These criteria were:

- minimising direct monetary costs of water-main maintenance or replacement;
- minimising direct plus indirect monetary costs associated with water-main maintenance or replacement
- minimising direct plus indirect monetary costs plus non-monetary costs;
- addressing strategic considerations of the organisation; inclusion of consideration of water-supply service issues;
- incorporation of customer-service contracts into the decision process.

A survey of economic decision models used by Australian water authorities was examined. Possible suitable decision models were then evaluated. These models were:

- the 5% Rule;
- the Net Present Value Method;
- the Criticality Model;
- multi-objective analysis;
- a model to incorporate customer-service contracts.

After evaluation, a recommendation was made that a variation of the General Net Present Value Model be used to assist in implementing maintenance policy for water mains which did not offer a high-loss potential to the organisation. This model was explained and computer software to aid in its implementation was developed.

An analysis of the implications on water-mains asset-management policy was made using output from the computer model and the likelihood of a recommendation for imminent replacement was considered. As well, there was a comparison made of the likely outcome of the General Net Present Value Model when compared with that from the 5% Rule.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The Australian water-supply industry has an urgent need for a reliable procedure to forecast the performance of individual water mains because of the worsening maintenance load as significant parts of supply networks approach the end of their economic lives. The drive to make service authorities more commercial has also pressured the industry into examining the timing of the decision to replace an asset. This study has generated a set of guidelines for the collection and interpretation of performance data, including the prediction of future performance and a decision tool to help operators judge whether to repair or replace a main. These guidelines can be applied universally to the minimisation of life-cycle costs for individual water-supply mains.

Infrastructure data often exists as hard-copy plans, in contract documents, in the memories of key staff members and as miscellaneous records in a variety of locations. Being able to predict the performance of a particular water-supply main requires all this data, and more, to be gathered together in a comprehensive database. A survey conducted as part of this project indicated that authorities are at different stages in the development of reliable databases. Some have 'complete' datasets, but maybe too much data, some of which is suspect. Others are considering how they can start a useful record which will form the foundation of an effective asset-management system. Since data collection is very expensive, the ideal system will use all records and not require any additional data. This study proposes a format for the collection of data on the performance of water-reticulation mains. The data listed in Appendix B fulfils the requirements of the models described in this report.

There are many reasons why water-reticulation mains fail. This study has identified causal factors and classified them as either 'inherent' (internal to the pipe system) or 'imposed' (externally applied). It is argued that the inherent factors represent the gradual deterioration of the pipe. Failure data linked with these factors should form the basis of a performance prediction model. A filtering process has been developed to separate the inappropriate imposed failures from the inherent failures. Authorities can create their own algorithms based on the filtering logic provided in this report.

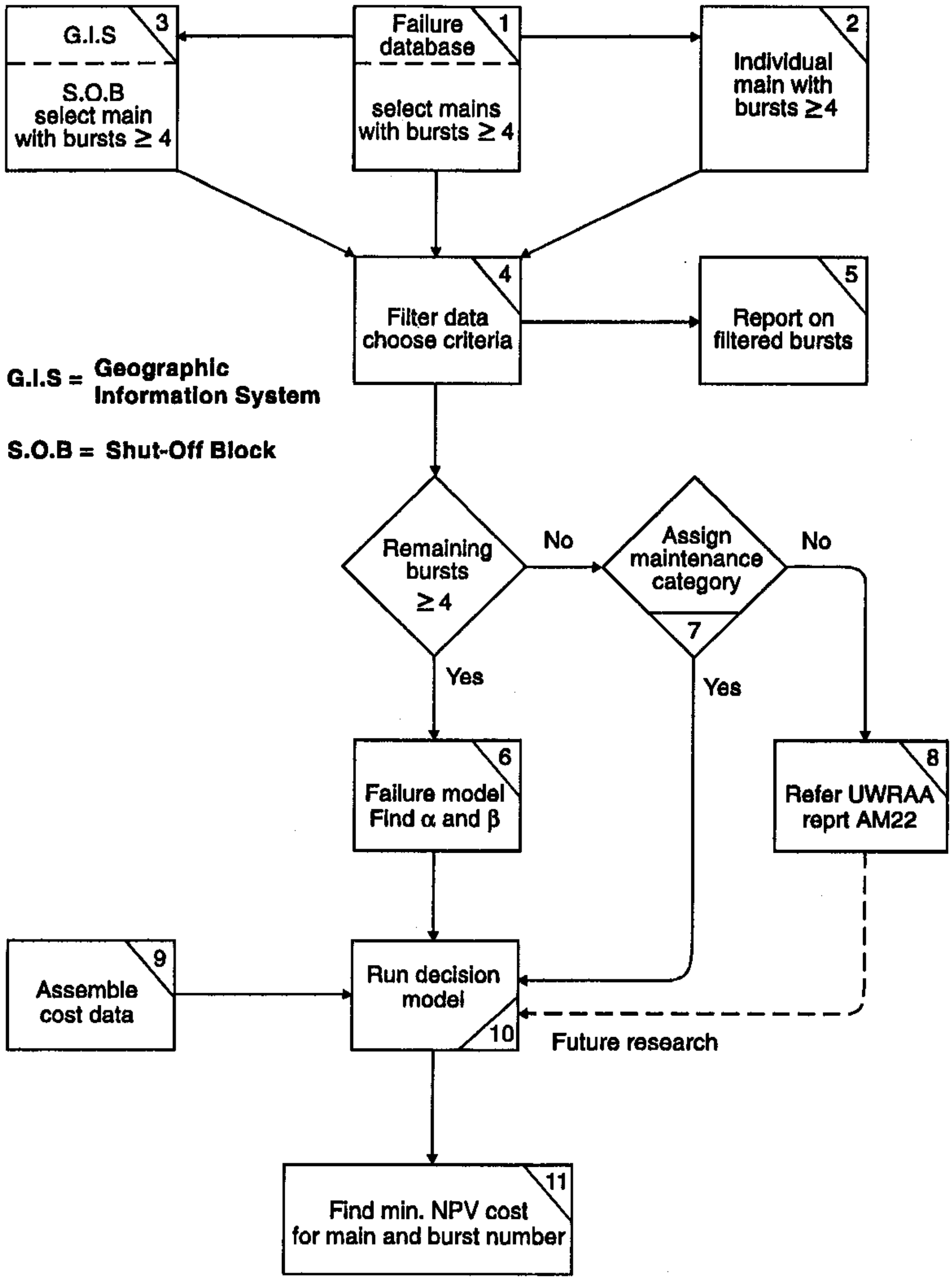
The structural performance of an individual water main can then be represented by its filtered failure record. However, since failures are usually spread over a long period of time, a model based on the failure rate (per km per year) is not suitable. After examining a number of alternatives, it was concluded that the most appropriate approach is to model the time interval between subsequent breaks.

All infrastructure assets have finite economic lives beyond which they are no longer the lowest cost options to provide specific services. A water-reticulation main is part of the physical infrastructure and must be replaced at the end of its economic life. The pressure is on authority managers to minimise life-cycle costs and the timing of replacements is a critical part of asset management. In determining the replacement date for a water main an authority must consider both the direct and indirect costs associated with continuing to repair an aging asset as well as the costs of replacement and future maintenance over a given analysis period. This is not a trivial process, especially for large networks. However, this report provides authorities with a decision tool that is characterised by its ease and flexibility of application. A key feature of the Total Failure Cost model is that an authority need only include the costs it considers important for the main being assessed. This project has involved a detailed study of a number of comprehensive water-reticulation databases with the aim of producing guidelines for the management of individual mains. A set of guidelines has been developed based on the 'complete' datasets available for analysis. However, it is recognised that many authorities will have incomplete records and their situation is also covered in the guidelines. These guidelines are presented below. Authorities with little or no data are referred to the companion UWRAA report by Constantine and Darroch (1995).

5.2 Guidelines for Applying Models to Individual Water Mains

Figure 5.1 is a logic diagram that illustrates the guidelines for applying models to individual water mains.

Figure 5.1 - Guidelines for applying models to individual water mains



Referring to figure 5.1:

1. This is the starting point if you wish to establish lists of mains for future repairs and replacements. (Generate an Excel Workbook file to include all pipes with four or more bursts.)

2. Another starting point is when you have identified an individual main with four or more bursts. Typically a manager is faced with making a decision on the future of an individual main that has just failed. (Generate an Excel Workbook file for this main.)

3. Another starting point could be a geographic information system (GIS) plot, which can locate each burst. A series of bursts in a small area (spatial problem) can be readily identified. The effect of these bursts on various shut-off blocks (SOB) can be studied. (Mains can be selected and an Excel Workbook file generated.)

4. The failure data is filtered. The criteria used in this filtering process can be altered to suit the prevailing circumstances.

5. By using a list of filtered bursts, prepare a report covering such topics as faulty operation, poor repair, accidental damage and intentional damage.

6. If four or more bursts remain after filtering, then run the failure prediction model. (You can use the Excel *Solver* to find α and β.)

7. If fewer than four bursts remain after filtering, the option exists to assign a maintenance category to the main (see section 4). If this is not desirable proceed to box 8.

8. Although beyond the scope of this report, it would be possible to integrate UWRAA Project No.22 with the decision model for mains with fewer than four failures and for which it is not possible to estimate a maintenance category.

9. Assemble cost data for input to the decision model. Select an appropriate discount rate, as well as repair and replacement costs for the main. Also estimate the maintenance category of the new main (normally category 1).

10. Run the decision model using the software available.

11. The output from the decision model gives the estimated NPV of the main at all future burst numbers. When the minimum NPV cost occurs at the latest burst the main is placed on a replacement list for further analysis (customer service, shut-off block etc.). When the minimum NPV cost occurs at a later burst, estimate the costs of future maintenance or justify the need for a proactive maintenance program.

5.3 Recommendations

In addition, there are a number of specific recommendations which result from this report.

1. The industry should adopt the database format presented in Appendix B as the minimum required to forecast performance and to determine when replacement should occur. An industry-wide standard will have significant advantages in terms of the implementation of refined performance/decision models.

2. It is very important that filtered data should not be rejected but should be used to initiate remedial programs, e.g. staff training in operational procedures, workshops on industrial relations and courses on supervision of contractors. In this way, failures caused by 'imposed' factors will be reduced.

3. The asset manager should use the performance forecasting model provided in the above guidelines or the generic model presented in the AM 22 report (Constantine and Darroch, 1995), whichever is applicable to their own circumstances.

4. The asset manager should use the Total Failure Cost model detailed in the guidelines to determine replacement strategies across the supply network. This will lead in turn to annual repair programs and maintenance budgets.

5. Asset managers should consider linking their own database to a geographic information system so that risk-management strategies can be implemented effectively.

The study has generated the above set of guidelines and the general recommendation is that this be used by the industry to improve repair/replace decision making process for individual mains. Each authority will apply and refine the guidelines differently according to their own operating environment and their corporate goals.

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APPENDIX A: REFERENCE SOURCES

The search for reference sources commenced with a literature search carried out in 1993 by Rodolphe Minetti, a post-graduate student at RMIT. With the assistance of the RMIT Library staff, this investigation was widened and the databases listed below were searched using the following **keywords**:

- corrosion;
- decision making;
- deterioration;
- economic aspects;
- maintenance;
- mathematical models;
- model studies;
- optimisation;
- planning;
- probability of failure;
- predictive models;
- resource allocation.

Databases

Dialog - Water Resources Abstracts,
US Department of the Interior, Geological Survey.

This database commenced in 1968 and contains material collected by over fifty water-research centres and institutions in the USA.

Water Net, American Water Works Association.

This database, commenced in 1971 and contains an index of premier publications relating to water and wastewater. Coverage includes North America, Australia, South Africa and some European publications.

Fluidex - UK database with links to the Dialog database.

This database commenced in 1973 and covers technical journals, books, conferences and standards from the UK and Europe as well as some UK patents.

Aqualine, Water Research Centre, UK.

This database commenced in 1974 with data from 1960 onwards. It is worldwide in its coverage of water, wastewater and the aquatic enviroment amongst other topics.

The RMIT library holds some CD-ROMs, which were searched also.

- Applied Science and Technical Index, H.W.Wilson and Co Publications,.
- Engineering and Applied Science, Australian Engineering Database - Engine - Institution of Engineers, Australia.

The internet Newsletter SCI.ENGR.CIVIL was used to locate workers in the same field.

APPENDIX B: SURVEY OF DATABASE STRUCTURES

Data field[1]	BCC[2]	CHW[3]	CHCC[4]	EWS[5]	MW[6]	OEW[7]	RSC[8]	RCC[9]	SWB[10]	TCC[11]
ID	Y	Y		Y	Y		Y			Y
Municipality	Y*		N/A		Y	N/A		N/A		N/A
Street name	Y		Y		Y		Y	Y	Y	Y
Street title					Y		Y	Y		
Failure date	Y	Y	Y	Y	Y	Y	Y	Y		Y
Pipe size	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
Pipe material	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
Construction year	Y			Y	Y					Y
Distance to cross street	Y	Y			Y		Y		Y	
Direction to cross street	Y	Y			Y		Y		Y	
Cross street name	Y	Y			Y		Y		Y	
Cross street title					Y		Y			

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1 The fields and initial order used for these fields is based on those of the Ringwood database. Other Authorities have used different names for what is obviously the same category of data.
2 Brisbane City Council
3 Central Highlands Water
4 Coffs Harbour City Council
5 Engineering and Water Supply Department
6 Melbourne Water (all Melbourne municipalities, including Ringwood)
7 Oxley Electricty and Water
8 Redland Shire Council
9 Rous County Council
10 Sydney Water Board
11 Toowoomba City Council

* suburb

Data field	BCC	CHW	CHCC	EWS	MW	OEW	RSC	RCC	SWB	TCC
Pipe lining type					Y		Y			
Manufacturer or method					Y					
Cement lining date					Y					
Soil type	Y		Y		Y		Y			
Town planning zone					Y					
Number of properties affected		Y			Y				Y	
Pipe status: abandoned or active					Y					
Pipe end code: node 1					Y					
Pipe end code: node 2					Y					
Distance between nodes					Y					
Primary shut-off block					Y					
Contour level					Y					
Asset length				Y	Y					
Insulation date					Y					
Size of repair pipe		Y			Y			Y		
Type of repair pipe		Y			Y			Y		
Repair date			Y		Y			Y		
Road category					Y		Y			

Data field	BCC	CHW	CHCC	EWS	MW	OEW	RSC	RCC	SWB	TCC
Repair pipe length		Y			Y					
Age at failure					Y					
Time to next failure					Y					
Dummy field (future use)					Y					
Age to date					Y					
Censoring variable					Y					
Total repair time	Y	Y			Y		Y	Y	Y	
Overtime repair time					Y					
Ordinary-time repair time					Y					
Number of men					Y			Y		
Total repair cost					Y					
Labour cost					Y					
Materials cost					Y					
Failure number for asset			Y		Y			Y		
Antecedent rainfall index					Y					
Top water level applying to pipe					Y					
Static pressure					Y					
Renewal age					Y					

Data field	BCC	CHW	CHCC	EWS	MW	OEW	RSC	RCC	SWB	TCC
Time since previous failure					Y					
Reason for (cause of) failure	Y			Y	Y		Y	Y	Y	Y
Joint type					Y		Y	Y	Y	
Property address	Y	Y					Y	Y		
Burst location (e.g. nature strip)	Y	Y					Y		Y	
Pipe depth	Y	Y	Y				Y		Y	
Burst type (e.g. broken back)	Y	Y					Y		Y	
False alarm		Y								
Repair method	Y							Y		Y
Repair detail: split collar		Y							Y	
Repair detail: section replaced		Y					Y			
Other fittings used		Y					Y	Y		
Opening length		Y								
Opening width		Y								
Property damage	Y	Y					Y		Y	
Statement of property damage		Y							Y	

Data field	BCC	CHW	CHCC	EWS	MW	OEW	RSC	RCC	SWB	TCC
Time action commenced		Y								
Time water was off		Y							Y	
Date water was off		Y								
Time water was restored		Y								
Date water was restored		Y								
Other details		Y					Y	Y		
Area			Y				Y			
Water table			Y							
Reason for exhumation			Y							
Type of protection			Y				Y			
Visual description			Y							
Testing (were samples taken?)			Y				Y		Y	
Type of failure				Y		Y	Y		Y	
Location of failure along pipe	Y			Y						
Condition of main	Y			Y			Y		Y	
Leakage rate							Y		Y	
Pipe bedding							Y			
Type of development (e.g. residential)	Y								Y	

Data field	BCC	CHW	CHCC	EWS	MW	OEW	RSC	RCC	SWB	TCC
Traffic disruption level	Y									
Asset external condition	Y									
Internal condition	Y								Y	
External coating condition	Y									
Proximity and detail of service near failure point	Y									
Damage to other Authority assets	Y									
Visual inspection report							Y			
Weather conditions									Y	
Terrain									Y	
Water-charged soil									Y	
Corrosion[12]					Y				Y	

As can be seen from the table, many of the data items collected by the various Authorities are similar in intent, but not necessarily similar in the detail of collection or recording. It is easy to visualise that differences in interpretation of causes of failure would exist not only between Authorities, but also within Authorities. Further work seems warranted to define an Australian Standard for collection and storage of data to simplify the processes of interpreting and comparing data.

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[12] Melbourne Water has a separate and extensive database on pipe corrosion.

Databases for Burst Mains - Standard Format

It is recommended that water authorities should collect and maintain in an electronic format the following data:

- asset number;
- pipe material;
- method of manufacture;
- diameter;
- length;
- joint type;
- construction year;
- preventative maintenance history (e.g. cement lining, insulation, renewal)
- soil type;
- traffic level;
- failure date;
- failure number;
- failure cause;
- failure location (distance and direction from nearest crossroads)
- static pressure;
- customer details.

It is also recommended to keep records of industrial disputes and major bushfires.

APPENDIX C: PIPE MANUFACTURING DATA

Relevance

It is highly plausible that the life of any asset is a function of its material and manufacturing standard and process. For the pipes in the Melbourne Water database the following processes have been identified. It is likely that the historical data will be relevant to other authorities in Australia given the small number of manufacturers associated with the water-supply industry.

Australian Standards

Although individual standards were not examined in detail, the following data was extracted by McCausland (1994) from the Australian Standards Database. The list is current as at November 1994

ITEM	DESCRIPTION	CURRENT STANDARD	SUPERSEDED STANDARD
Polymer pipes		1559-1988	1463-1988
	Cross-linked polyethylene (PE-X) pipe for hot- and cold-water applications	2492-1994	2492-1981
	Polybutylene pipe systems: polybutylene (PB) pipe extrusion compounds	2642.1-1994	2642.1-1989
	Polybutylene pipe systems: polybutylene (PB) pipe for hot- and cold-water applications	2642.2-1994	2642.2-1989
	Propylene copolymer compounds for the manufacture of pressure pipe and fittings	2949-1987	
	Propylene copolymer pressure pipe	2950-1988	
	Acrylonitrile butadiene styrene (ABS) pipes and fittings for pressure applications: pipes	3518.1-1988	
	Acrylonitrile butadiene styrene (ABS) pipes and fittings for pressure applications: solvent cement fittings	3518.2-1988	

	Glass filament reinforced thermosetting plastics (GRP) pipes - polyester based - water supply, sewerage and drainage applications	3571-1989	
	PE pipes, pressure applications	4130-1993 (Interim - expires May 95)	
	PE pipe compounds	4131-1993 (Interim - expires May 95)	
Cement based pipes	Fibre-reinforced concrete pipes and fittings	4139-1993	1392-1974
Metal pipes	Arc welded steel pipes and fittings for water and waste water	1579-1993	1579-1973
	Ductile-iron pressure pipes and fittings	2280-1991	A125-1971
	Grey-iron pressure pipes and fittings	2544-1982 (under revision)	2280-1988
	Polyethylene/aluminium and cross-linked polyethylene/aluminium macro-composite pipe systems for pressure applications	4176-1994	
Cement lined pipes	Cement mortar lining of steel pipes and fittings	1281-1993	1281-1981
	Cement mortar lining of pipelines in situ	1515-1994	1516-1980
	Precast concrete pipes (pressure and non-pressure)	4058-1992	
Cement lined pipe joints	Elastomeric seals for waterworks purposes	1646-1992	A139-1972
Polymer pipe fittings	Fittings for use with polyethylene pipe: mechanical jointing fittings	1460.1-1989	1460-1973
	Fittings for use with polyethylene pipes: electrofusion fittings	1460.2-1989	
	Mechanical jointing fittings for use with cross-linked polyethylene (PE-X) pipe for hot- and cold-water applications	2537-1994	2537-1982
	Polybutylene pipe systems: mechanical jointing fittings for use with polybutylene (PB) pipes for hot- and cold-water applications	2642.3-1994	2642.3-1983
	Acrylonitrile butadiene styrene (ABS) pipes and fittings for pressure applications: pipes	3518.1-1988	
Metal pipe fittings	Flanges for pipes, valves and fittings	2129-1994	2129-1982

	Ductile-iron pressure pipes and fittings	2280-1991	2280-1988
	Grey-iron pressure pipes and fittings	2544-1982	
	Metallic flanges for waterworks purposes	4087-1993	
	Guide to the cathodic protection of metals: pipes, cables and ducts	2832.1-1985	
	Guide to the cathodic protection of metals: compact buried structures	2832.2-1985	
Cement-lined pipe joints	Elastomeric seals for waterworks purposes	1646-1992	1646-1987 A139-1972

Ferrous based pipes

Most of the data in this section was extracted from Newman (1994) and Scott (1993).

Up to about 1970, ferrous-based pipes were all in the form of cast iron. All cast-iron pipes (CI pipes) up to 1921 were cast horizontally in sand moulds. From 1921 Melbourne Water started using imported centrifically cast pipes and in 1929 these were made locally. All cast-iron pipes were normally coated externally with a tar-based dip. Internal cement lining was used in pipe manufacture from 1926. Cement-lining programs were widely used 'in situ' to line the earlier unlined cast-iron pipes. Cement lining contactors were employed in the late 1930s and also in the period 1962 to 1985.

Where pipes were lined, the following parameters from AS 1281 applied:

Pipe diameter (mm)	Lining thickness (mm)	Tolerance (mm)
100 to 200	8	+6, -0
200 to 500	10	+6, -0
500 to 750	12	+6, -0
750 to 1000	16	+6, -0
1000 to 1400	20	+6, -0
1400 to 1800	25	+6, -0

Cast-iron pipes were slowly superseded by ductile-iron pipes, which had significantly thinner walls and were thus cheaper to manufacture. Melbourne Water commenced using ductile-iron pipes in 1981. These pipes are not old enough to have developed a failure history which would allow comparison with the cast-iron pipes.

Plastic Pipes

Most of the data in this section was extracted from Domma (1994).

Melbourne Water have three established contracts for supply of UPVC water pipes in 100 mm and 150 mm diameters.

The pipes comply with AS 2977.1 and are manufactured to match the external sizes of cast-iron pipes to enable use of the existing standard fittings.

Asbestos-Cement Pipes

Pipeline Material - History of AC pipes in Australia

In discussing pipeline performance it is recognised that any pipeline material, once placed in the ground, is subjected to the corrosion/aging process and is also subjected to varying internal and external loadings.

For years now both the Water Industry and manufacturers have sought to balance the capital cost of the pipeline against its performance.

The history of asbestos (AC) pipe making in Australia illustrates this point.

These pipes were first made in Italy in 1913 and were called Eternit pipes after the firm which made them (Societa Anonima Eternit Pietra Artificale of Genoa). The search for an alternative pipe to the then universal cast-iron pipe saw the Melbourne and Metropolitan Board of Works import some Eternit pipes in 1928 for test purposes. The Commissioners were concerned about the rising cost of cast-iron pipes, which were then about $30/ton (15 pounds).

James Hardie was manufacturing AC sheets in Melbourne, Sydney, Perth and Auckland for general building work and was interested in making AC pipes. Hardie considered the royalties for the Eternit process to be excessive and Mr Sutton of Hardie commenced research into pipe manufacture. Pipes were made by taking green AC sheets (moist and pliable)and rolling them around a collapsible steel mandrill. A calico sheet was then wrapped around the pipe and pressure applied by winding a steel cable over the calico material. When the pipe had 'set' it was released from the mandrill and cable and cured in water for 14 days followed by air curing for 14 days. The pipe ends were machined to suit gibault joints and were generally known as Sutton pipes. Some authorities refer to them as 'crinkly pipes' because the steel cable produced circular ribs along the barrel of the pipe.

The Melbourne Board conducted many trials using the new pipes and three factors became immediately evident.

1. More labour was required to trim the floor of the trench for AC pipes due to the lower beam strength when compared with CI pipes. This disadvantage was offset somewhat by the AC pipes being much lighter than the CI pipes.

2. Although Hardie and Eternit recommended that ferrules could be tapped directly into AC pipes, in practice they proved to be fairly insecure. Ferrules were then placed in a four bolted CI clip.

3. The internal diameters of AC pipe were reduced to enable standard CI fittings to be used with the new pipe (4" was reduced to 3.5" ID, 6" was reduced to 5.5" ID, 9" was reduced to 8" ID) and pipes were joined using standard CI gibault joints.

The Melbourne Board approved Fibrolite pipe as an alternative to CI pipe in 1931 and this placed great pressure on the Melbourne foundries to reduce the price of CI pipes. They reduced prices on 4" (100 mm) diameter pipes from $30/ton to $20/ton but fibrolite pipes were less than half the cost of CI pipes - about 11 cents per foot compared with 30 cents per foot for CI pipes. The figure at the end of this appendix illustrates the changes in prices of CI pipes.

This occurred at a time when the Melbourne Board was asking for CI pipes to be cement lined thus adding again to the cost pressure. At that time there were nine foundries in Melbourne capable of making CI pipes. There were many letters to the Editor in the *Age* and several deputations to the Commissioners of the Board defending the status quo.

The usage of AC pipe increased steadily until, in 1935, the worthy Commissioners and their engineers became alarmed at the failure rate of the new AC pipes. A chemical investigation revealed that the soft Melbourne water was leaching out the cement from the pipe wall. The Public Works Department (PWD) in Western Australia reported a high failure rate of AC pipes in Perth and on the gold fields. In this case the cause was a sulphate attack on the free lime in the pipe. The conversion of lime to calcium sulphate resulted in a 220 percent swelling and delamination in the pipe wall. In both cases some improvement in performance resulted from coating the pipe interior with a cut-back bitumen.

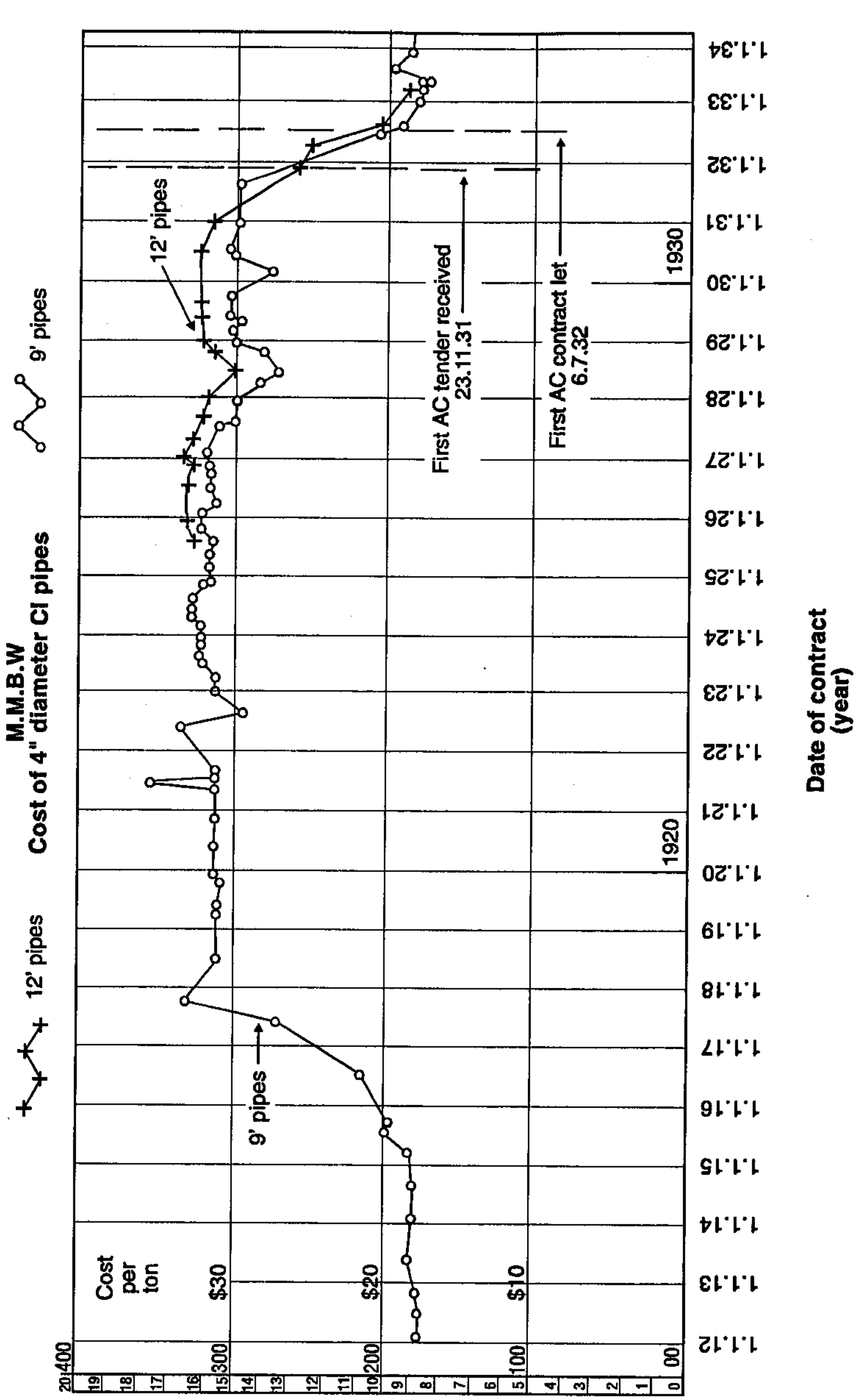
In 1938 an improved process (Mazza) was introduced by Hardie where the pipe wall was compacted using longitudinal pressure rollers. In 1954 the pipes were cured in an autoclave (pressure steam curing). This process overcame the problem of sulphate attack as the free lime was combined with silica, which was added to the mix as a finely ground powder. In the 1930s a push-in joint with a round rubber ring was developed and in 1962 a new supertite rubber ring joint was introduced and approved by the NSW Public Works Department. Pipes with this joint were subsequently made in sizes up to 750 mm diameter.

The arrival of PVC and polyethelene pipes, plus public pressure against asbestos products, resulted in James Hardie ceasing AC pipe manufacturing in Australia in 1987.

References

Carrol B., A Very Good Business: One Hundred Years of James Hardie Industries Ltd 1888 - 1988, Imprint - Sydney: James Hardie Industries, 1987.

Scott R.J., Reticulation Water Mains 1857 - 1990, Internal Melbourne Water Report.


M.M.B.W
Cost of 4" diameter CI pipes
12' pipes
9' pipes
Cost per ton
$30
$20
$10
00
100
200
300
400
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
1.1.12
1.1.13
1.1.14
1.1.15
1.1.16
1.1.17
1.1.18
1.1.19
1.1.20
1.1.21
1.1.22
1.1.23
1.1.24
1.1.25
1.1.26
1.1.27
1.1.28
1.1.29
1.1.30
1.1.31
1.1.32
1.1.33
1.1.34
1920
1930
First AC tender received
23.11.31
First AC contract let
6.7.32
Date of contract
(year)

APPENDIX D: STATISTICAL TECHNIQUES

Several statistical techniques were used to analyse the data sets in this investigation. These include exploratory data analysis techniques in the development of the filtering system, and linear and non-linear-least squares techniques used in the failure prediction models. The work of Darroch and Constantine (1995) in which a non-homogeneous Poisson model is fitted using maximum likelihood techniques is referred to also.

Filtering of Data

In the body of the report recommendations are made on setting up and validating filtering systems. These are basically graphical techniques where the failures are plotted and changes in failure patterns with time are noted. If these changes can be attributed to failures caused by faulty construction or the effects of a previous repair, the times at which these changes occurred can then be used in a filtering system.

For example, to assess the affect of poor construction or materials the time to first failure from construction of all pipes of a particular type was found. The failures were sorted into ascending order and the time to failure was plotted against the cumulative failure number to give a failure curve. The logarithm of the failure times was taken to flatten the curve. The plots that resulted generally appeared piecewise linear. The initial straight line had a comparatively steep slope, indicating a high initial failure rate due to construction faults. The second straight line had a gentler slope, indicating the slower failure rate caused by corrosion and wear. The time where these lines met could be used in a filtering system to filter out the failures caused by faulty construction. A similar procedure was used with second and higher failures to determine a filtering time for failures caused by bad repair.

The use of logarithmic transformations to flatten or linearise a curve is a standard technique used in exploratory data analysis. It is particularly useful when the underlying process is governed by some exponential or power function. For more detailed explanations, or details of other transformations, a good reference is Exploratory Data Analysis by J.W.Tukey, Addison-Welsey, 1976 U.S.A.

Failure Prediction Models

With the exception of the exponential interfailure time model (Minetti 1995), all the models were fitted using ordinary single-variable least-squares techniques. To fit the model

$$y = a + bx \qquad \text{...(1)}$$

the following well-known equations were used:

$$\hat{b} = \frac{n\sum x_i y_i - \sum x_i \sum y_i}{n\sum x_i^2 - \left(\sum x_i\right)^2}$$

$$\hat{a} = \frac{\left(\sum y_i - \hat{b}\sum x_i\right)}{n} \qquad \text{...(2)}$$

where n is the number of observations (failures) on which the model is based. In the table D1 the quantities corresponding to a, b, x and y are given for each of the failure models discussed in section 3.

TABLE AD1

MODEL PARAMETERS

Predictor	**Model type**	***x***	***y***	***a***	***b***
Failure numbers	exponential	times	log(fail)	$\log(\alpha)$	β
	power	log(times)	log(fail)	$\log(\alpha)$	β
Failure times	exponential	log(fail)	times	$\log(\alpha)/\beta$	$1/\beta$
	power	log(fail)	log(times)	$\log(\alpha)/\beta$	$1/\beta$
Inter-failure times	square root	sqrt(fail)+sqrt(fail+1)	inter-failure times	α	β

where **fail** means failure number and **times** means failure time.

Before fitting any models, the data should be filtered to remove double breaks and other exceptional events which would adversely effect the predictive ability of the model. These models can then be fitted using the formulas above, or the standard least-squares regression function in a statistical or spreadsheet program, provided the appropriate transformations have been made to the data. Most introductory statistics text books give details on fitting lines of best fit using least squares. For a more detailed approach two references are <u>An Introduction to Linear Regression Analysis</u> by Montogomery and Peck, Wiley, U.S.A. 1982 and <u>Applied Regression Analysis</u> by Draper and Smith, Wiley, U.S.A. 1966, although there are many other good books on the subject.

The analysis was carried out using the standard least-squares function in *Splus*, an advanced statistical package. An additional program was written in *Splus* to search the data bases for pipes with five or more failures, estimate the model for these pipes, and calculate the standard deviations of the residuals to give a measure of the goodness of fit. The program then deleted the last observation, recalculated the model, predicted the next failure based on this model, and compared its prediction with the deleted observation to estimate the prediction error. The results of this analysis are given in section 3.

The final model, predicting interfailure times using an exponential equation, requires a different fitting technique since the model is not linear in the parameters, and cannot be made so by any legitimate algebraic manipulation. The technique used was non-linear least-squares. That is, the sum of the squared residuals of the model was minimised as in ordinary least-squares, but in this case no closed form formula is available for the α and β values. Instead, the values must be found by using an iterative procedure - picking starting values then changing them until the minimum value is found. The Gauss-Newton algorithm is the most popular for solving this type of problem and is the one used in the *Splus* non-linear least-squares function and the Excel5 *Solver* add-in. The problem of non-linear least-squares and the Gauss-Newton algorithm are discussed in most text books on numerical analysis; one reference is Numerical Analysis - A Comprehensive Introduction by H.R. Schwartz, Wiley, U.S.A. 1989. Draper and Smith also give an introduction to non-linear least-squares for the regression problem.

For this model the analysis was carried out using the Excel5 spreadsheet program with the *Solver* add-in. The model used:

$$T = 12\alpha \log(1 + \frac{100}{100n+\beta})$$

....(3)

contains a log term. For negative values of β the argument of the log term can become negative, which causes a non-recoverable error in the fitting program. Although the final values of β are not negative, for an increasing failure rate, the iterative algorithm will often choose intermediate values which are negative, causing the program to terminate. Because of this difficulty it was necessary to choose a fitting procedure which allowed us to force the value of β to be strictly positive. The *Solver* add-in allowed a constraint of the form $\beta >= \delta$ to be applied. A value of δ of 10 to the power -9 was used. The starting values of the parameters used were $\alpha = 0.50$ and $\beta = 0.10$. The same procedure was used for calculating standard deviations and prediction errors as for the linear case, but because of the limitations of the spreadsheet the procedure could not be automated, so each pipe had to be individually found and the model fitted. The results of this analysis is given in table 2.5.3. The model parameters for every pipe in the data sets were not calculated because of the time this would have taken.

Incomplete data

The fitting of models with incomplete data was discussed. This involves the estimation of the number of previously unrecorded failures as an extra parameter. This makes all the models already discussed non-linear. This technique replaces n, the total number of failures, by n+k, where k is the number of known failures and n is the unknown number of failures that occurred before records were kept. The models can be easily fitted by minimising the sum of the squared residuals: a non-linear least-squares routine. This facility is available in all statistical packages and many of the recent spreadsheets. Some of these allow constraints to be placed on the number of unrecorded failures to ensure it is a non-negative integer, but the optimisation can be carried out without this constraint, with the value changed to the closest that is practical at the operator's discretion. For fitting the exponential interfailure time model, the same problems occur with negative values of β as were discussed in the complete data model.

Non-Homogeneous Poisson Model

One of the data filtering systems proposed in section 3 is based on the probability of two events occurring within a certain time. These probabilities are calculated using a model developed by Darroch and Constantine (1995). An outline of the procedure is given below and further details can be found in their report, which is the companion volume to this one. The book, Statistical Analysis of Reliability Data by Crowder, Kimber, Smith and Sweeting, Chapman and Hall, Britain, 1991 gives a discussion of the application of non-homogeneous Poisson models to repairable systems.

The following statistical data is based on work by Dr Graham Constantine of the CSIRO Division of Mathematics and Statistics. Dr Constantine is developing techniques for dealing with incomplete data sets.

The likelihood of failure at time *T* is given by:

$$L = \prod_{n_i}^{p} e^{l_i \left[\frac{\tau_i}{\theta_i}\right]^{\beta} \frac{l_i \left[\frac{\tau_i}{\theta_i}\right]^{\beta n_i}}{n_i\,!}} \left\{ n_i\,! \prod_{j=1}^{n_i} \beta e^{-\beta} s_{ij} \right\}$$

.....................................(4)

where:

L is the likelihood of failure at time T;
l_i is the length of asset i;
p is the number of assets;
n_i is the number of failures for the ith asset;
$s_{ij} = \log_e \left(\frac{T_i}{t_{ij}} \right)$;
τ_i is the age of the ith asset;
t_{ij} is the time after construction of the jth failure of asset i;
β is a regression coefficients;
α is a vector of regression coefficients;
$\theta_i = e^{\alpha x_i}$;
$N = \sum_{i=1}^{p} n_i$;
x_i is the vector of covariates of the previous equation, with
$x_{i1} = 1$

Also,

$$LL = \log L$$

$$LL = \left\{ -\sum_i l_i \left[\frac{\tau_i}{\theta_i} \right]^{\beta} + \beta \sum_i^{p} \left(\log T_i - \log \theta_i \right) \right\} \left\{ -\beta \sum_i^{p_i} \sum_{j=1}^{n_i} s_{ij} + N \log \beta \right\}$$

....................(5)

The coefficient β can be estimated from the second term of equation (5) by:

$$\frac{\partial \mathrm{LL}}{\partial \beta} = -\sum \sum s_{ij} + \frac{N}{\beta} = 0$$

....(6)

$$\Rightarrow \beta = \frac{N}{\sum_i \sum_j s_{ij}}.$$

$$\frac{\partial l}{\partial \alpha} = -\sum l_i \tau_i^{\beta} e^{-\alpha x_i} \left(-x_i \right) - \beta \sum n_i x_i = 0$$

....(7)

The Gauss-Newton method was used to solve equation (7) for α. For full maximum likelihood, equation (5) was differentiated with respect to β and equated to zero.

$$-\sum l_i \left(\tau_i e^{-x_i} \right)^{\beta} \log \frac{\tau_i}{\theta_i} + \sum n_i \left(\log \tau_i - \log \theta_i \right) - \sum_i \sum_j s_{ij} + \frac{N}{\beta} = 0$$

.....................(8)

By choosing an initial value of β (as described above) and estimating the first iteration of α to get α^1, this value can be substituted in equation (7) to find β^2. Then estimating a second iteration of α, α^2, and by using equation (8) a second iteration of β, β^2, can be found. This process is continued until convergence is reached.

This model can also be fitted to an individual pipeline provided it has experienced at least two failures. However, with small data sets (n_i<10) the results are not reliable. The values for α and β can be found directly in this case from equations (9) and (10) where the terms are as previously defined.

$$\beta = \frac{n_i}{\sum_{j=1}^{n_i} s_{ij}}$$

....(9)

$$e^{-\alpha} = \frac{l_i \tau_i^{\beta}}{\beta n_i}$$

....(10)

APPENDIX E:

SURVEY OF DECISION MODELS USED BY AUSTRALIAN WATER AUTHORITIES - MARCH 1995

ORGANISATION	ACT Electricity and Water	Barwon Water
State	ACT	Victoria
Significance of maintenance/replacement costs	Approximately 30% of total hydraulic operations maintenance.	Important.
Basis of the repair or replace decision	Informal, but moving to formal mode.	Informal, reactive.
Is preferred decision making model the one used? If not, what constraints apply?	NPV model including primary, secondary and consequential damage preferred.	Not preferred. Adopted because of lack of data.
Level at which replacement decision is made	Region.	Originated by urban maintenance group.
Characteristics of formal mode		
Reliable database, period of existence of database	Database exists. Currently being updated to include records back to 1972.	Electronic maintenance database established in 1988.
Failure prediction model		None.
Cost elements included	Direct and indirect costs.	Direct costs.
Are third party costs important?	Include insurance costs.	Negligible compared with total costs.
Should third party costs be included?	Should be included.	Include traffic disruption, lost production, loss of supply.
Are intangible costs included? Comment on methods of evaluation.	Include cost of disruption per household based on GDP per capita/hour.	Not included.
Discount rate used	Weighted average cost of capital.	4% discount used generally on projects. Higher rate would be used if Authority was privatised.
Other issues	Risk exposure to Authority; customer service; Political pressure.	

ORGANISATION	Bundaberg City Council	Caloundra City Council
State	QLD	QLD
Significance of maintenance/replacement costs	Moderate.	$350,000 pa.
Basis of the repair.or replace decision	Informal.	Informal.
Nature of formal model if used	Respond to failures only.	
Is preferred decision making model the one used? If not, what constraints apply?	Financial.	Lack of technical support staff.
Level at which replacement decision is made	Regional.	Water Supply Foreman.
Characteristics of formal model		
Reliable database, period of existence of database	No.	No data base.
Failure prediction model		
Cost elements included		
Are third party costs important?		
Should third party costs be included?		
Are intangible costs included? Comment on methods of evaluation		
Discount rate used		
Other issues		

ORGANISATION	**Central Tablelands County Council**	**City West Water Ltd**
State	Victoria	Victoria
Significance of maintenance/replacement costs	Second to water treatment costs.	Very significant.
Basis of the repair or replace decision	Informal, reactive.	Formal.
Nature of formal model if used		NPV model. Includes direct costs only. Twenty-five years economic life.
Is preferred decision making model the one used? If not, what constraints apply?	Not preferred. Adopted because of lack of data on pipe performance.	Preferred.
Level at which replacement decision is made	Originated by engineering staff.	Central, no regional structure.
Characteristics of formal model		
Reliable database, period of existence of database	Electronic data base is being established.	Database from 1948.
Failure prediction model		None.
Cost elements included		Direct costs.
Are third party costs important?	Not included in decision process.	May be high, e.g. reinstatement costs and clean-up after flooding.
Should third party costs be included?	Not included.	Should be included in some circumstances.
Are intangible costs included? Comment on methods of evaluation		Not for households, but possibly for industrial customers.
Discount rate used.		Company policy not yet established.
Other issues	Capacity of consumers to pay; real increase in level of service.	Customer service contracts with a maximum of three disruptions per year; customer satisfaction.

ORGANISATION	Coliban Water	Hobart City Council
State	Victoria	Tasmania
Significance of maintenance/replacement costs	Not high in proportion to capital budget.	$800,000 pa on asset base of $100/m.
Basis of repair/replacement decision	Formal.	Informal.
Nature of formal model if used	Discounted NPV.	Reactive.
Is preferred decision making model the one used? If not, what constraints apply?	Preferred.	Database recently established.
Level at which replacement decision is made	Centrally.	Central.
Characteristics of formal model		
Reliable database, period of existence of data base	Information available, electronic database under development.	Database recently established.
Failure prediction model		None.
Cost elements included		Direct costs plus overheads.
Are third party costs important?	Not formally included.	Consequential damage should be included.
Should third party costs be included?		Yes, unsure of response.
Are intangible costs included? Comment on methods of evaluation		Not included, but should be.
Discount rate used	Based on perceived return required by government.	
Other issues	Criticality; safety; co-ordination with other authorities.	

ORGANISATION	Kiewa Murray Water Authority	Pawa Darwin
State	Victoria	NT
Significance of maintenance/replacement costs	$100,000 pa with 11,880 assessments.	Maintenance costs low. Oldest pipes in system 50 years old.
Basis of repair/replacement decision	Informal.	Informal, reactive.
Nature of formal model if used		
Is preferred decision making model the one used? If not, what constraints apply?	Preferred.	
Level at which replacement decision is made	Works Officer/Manager operations.	Regional.
Characteristics of formal model		
Reliable database, period of existence of database	Yes; asset register three years, breaks 20+ years.	Electronic database is being established.
Failure prediction model		
Cost elements included		
Are third party costs important?		
Should third party costs be included?		
Are intangible costs included? Comment on methods of evaluation	Unscheduled interruption.	
Discount rate used		
Other issues	Customer service; reliability and continuity of service; liaison with roads construction; adequate fire-fighting supply; upgrading because of development.	

ORGANISATION	Shoalhaven City Council	South Eastern Water Ltd
State	NSW	Victoria
Significance of maintenance/replacement costs	Low compared with asset value.	
Basis of the repair or replace decision	Informal.	Formal.
Nature of formal model if used	Reticulation pipeline replaced after 3 breaks per year.	Replace when annual maintenance costs exceeds 5% of renewal costs and/or >3 unplanned interruptions per year.
Is preferred decision making model the one used? If not, what constraints apply?		Preferred.
Level at which replacement decision is made	Centrally.	Region.
Characteristics of formal model		
Reliable database, period of existence of database	Yes, 10 years.	Database available back to 1942.
Failure prediction model	None.	WRAP model.
Cost elements included		Direct and Indirect costs.
Are third party costs important?		Consequential damage and road reinstatement may be major components.
Should third party costs be included?	Yes.	
Are intangible costs included? Comment on methods of evaluation	Disruption caused by flooding.	Not included.
Discount rate used		8.70%
Other issues		Replacement confined to AC pipes laid about 1950; non AC pipes are not breaking due to material failure.

ORGANISATION	Sydney Water	Sydney Water - Illawarra Region
State	NSW	NSW
Significance of maintenance/replacement costs	Average $25 million pa over last 3 years.	Ranges from $1 million pa to $9 million pa. Annual budget $30-40 million.
Basis of the repair or replace decision	Formal.	Formal.
Nature of formal model if used	NPV model with internal and external costs.	NPV model.
Is preferred decision making model the one used?	Preferred.	Preferred.
Level at which replacement decision is made	Central.	Region.
Reliable database, period of existence of database	Reasonably reliable database.	Database for 10 years.
Failure prediction model	As in UWRAA Report No.57.	Nil.
Cost elements included	Direct and indirect.	Direct and indirect.
Are third party costs important?	Community and traffic disruption included separately. Third party damage included as internal cost.	Consequential damage included. May be a major component.
Should third party costs be included?	Should be included.	Are included. Would probably be dismissed if authority was private body.
Are intangible costs included? Comment on methods of evaluation		Customers eligible for 10% rebate for each shutdown in excess one hour (unplanned) or 6 hrs (planned) per quarter.
Discount rate used		Treasury sets 7% rate. Test for sensitivity to 4% and 10% rates.
Other issues	Customer contracts.	

ORGANISATION	Yarra Valley Water Ltd.	Your authority (own use)
State	Victoria	
Significance of maintenance/replacement costs	$3.125 million pa; 2,500 bursts per year.	
Basis of the repair or replace decision	Formal.	
Nature of formal model if used	NPV model. Includes direct costs only. 25 years economic life.	
Is preferred decision making model the one used? If not, what constraints apply?	Preferred, although aggressive customers may jump queue.	
Level at which replacement decision is made	Region.	
Characteristics of formal model		
Reliable database, period of existence of database	Database from 1948.	
Failure prediction model	Compute current burst rate and assume a 5% growth over the next 25 years.	
Cost elements included	Direct costs.	
Are third party costs important?	Not included in decision process.	
Should third party costs be included?	Covered by insurance.	
Are intangible costs included? Comment on methods of evaluation	Not included.	
Discount rate used	Company policy not yet established.	
Other issues	Customer service contract of a maximum of 3 disruptions per year; repair/replacement costs are tax deductible - this should be allowed for in the analysis.	

APPENDIX F: DECISION ANALYSIS COMPUTER SOFTWARE

Introduction

Computer software was developed to assist in implementing the Total Future Cost decision model. The algorithms supporting the software, the requirements of the input and the nature of the output are explained in this appendix.

The computer program computes the Total Future Cost of the water main by summing the discounted cost of the three elements comprising the life cost of the main.

These elements are:

- the present value of the repairs to the current main
- the present value of the future replacement of the main, and
- the present value of the repairs to the new main.

In order to calculate the present value of these maintenance events, it is necessary to predict the time of their occurrence. A statistical approach as detailed in section 3 was used. The **inter-failure time** model was adopted. In situations when insufficient data are available for the current main to determine the statistical parameters needed, the main may be assigned to a maintenance category as defined below. The time of occurrence of maintenance events for the replacement main can only be estimated after this main has been assigned to a maintenance category.

Symbols used:

TFC Total Future Cost

PVO_n Present value of repairs to the current main up to burst n

PVI Present value of installing a new main at time Tn from the present time

PVN_b Present value of repair to b bursts of the new main after replacement

R Average main repair cost

N Cost of replacing the main, or part thereof, under consideration

i Discount rate used by the water authority for investment analysis

x General burst number in a main

n Burst number at replacement of the current main

a Current burst number of the current main

b Number of bursts predicted in the replacement main during the analysis period

T_x Time to busrt number x from the present time

T_n Time to burst number n from the present time

T_b Time to burst number b in the replacement main from the time of replacement until the end of the analysis period

T_t Duration of the analysis period

L Length of the section of the main under analysis (meters)

α, β Regression coefficients based on a least squares fit of burst history to that section of the main being analysed

Failure Prediction

The Statistical Model

The model used to estimate failure predictions in both the current main and the replacement main is the inter-failure time model explained in section 3.

This model can be represented by the relationship:

$$T_n = 12\alpha \log\left(1 + \frac{100}{100n + \beta L}\right).$$

Maintenance Categories

In situations when a full maintenance history is not available for the current main and for the new replacement main, the regression coefficients α and β may be estimated by assigning a maintenance category to the main being analysed. The burst history of the main - and values of α and β adopted, which are consistent with each maintenance category - are given in table AF.1.

TABLE AF.1

REGRESSION COEFFICIENTS FOR DEFINED WATER-MAINS MAINTENANCE CATEGORIES

Maintenance category	Failures per 100 m in the first 40 years of life	α	β
1	0 to <2	6	2.4
2	2 to <8	3	2.4
3	8 to <15	2	2.4
4	>15	1.4	2.4

Financial Calculations

<u>Repairs to the Current Main</u>

The total time from the current burst number a to burst number n is given by:

$$T_n = \sum_{x=a}^{n-1} 12\alpha \log\left(1 + \frac{100}{100x + \beta L}\right)$$

Hence, if replacement were to be made after burst n, and allowing for the repair of the current burst, the present value of all the repairs up to and including burst n is given by:

$$PVO_n = R + R\sum_{T_x=T_a+1}^{T_n} \frac{1}{(1+i)^{T_x}}$$

Installation of the New Main

The present value for installation of the new main after burst n is given by:

$$PVI = \frac{N}{(1+i)^{T_n}}$$

Repairs to the Replacement Main

The present value for repairs to the new main after replacement will be similar to that for the current main except that bursts will be counted from burst number 1 and continue until the end of the analysis period. In addition, there is no need to allow for a repair at the time of replacement of current main.

It is therefore first necessary to establish the number of bursts of the new main (b) which are predicted within the analysis period. This can be determined by finding the largest value of b which satisfies the relationship:

$$T_b + T_n \le T_t$$

where:

$$T_b = \sum_{x=0}^{b-1} 12\alpha \log\left(1 + \frac{100}{100x + \beta L}\right).$$

Note that α and β are the regression coefficients applying to the new main.

Hence, the present value of the repairs of b bursts to the new main is given by:

$$PVN_b = R\sum_{T_x=T_t}^{T_b}\frac{1}{(1+i)^{T_x}}.$$

Total Future Cost

The present value of the life cost of the main is given by summing the three cost components above:

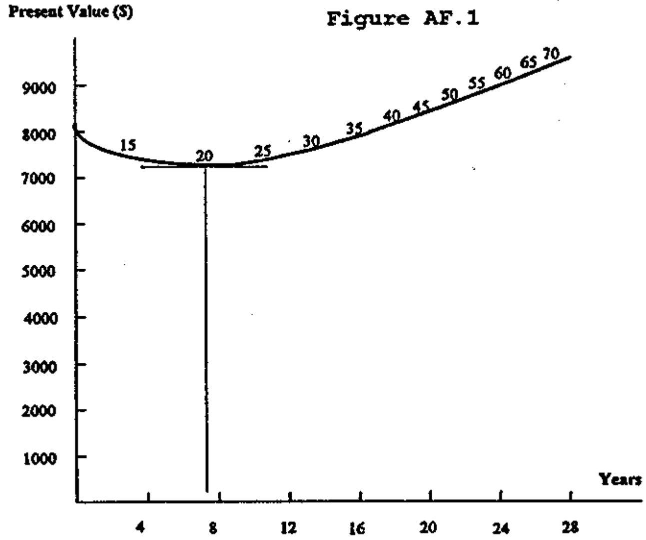
$$TFC = PVO_n + PVI + PVN_b.$$

Details of Computer Software Operation

The computer software computes the Total Future Cost value for the life of the main as described above. The DOS-based executable file is named 'main.exe'.

The program has been designed for easy data input and also to allow the analyst to move freely between the input screen and the output screen so that the sensitivity of the output to changes of input data can readily be examined, see Figure AF.1 below.

Figure AF.1



Present Value of Total Future Cost for Water Main Replacement at Year Shown
(Burst Number Shown on Curve)

Input

Values for the following parameters are required before execution can commence:

- discount rate in %;
- main length in metres;
- the replacement cost of the main;
- the repair cost of the main;
- details of the current main;
    1. the estimated current burst number of the current main;
    2. α and β, the regression coefficients used to predict future failures (or an estimate of the main's maintenance category);
- estimated maintenance category of the replacement main.

Note: The α and β regression coefficients, where provided, will assume priority over the estimated maintenance category.

The cost of replacing the main is the cost applying to that part of the main being replaced. If partial replacement is made, then the failure history analysis and failure predictions relevant to only that part of the main being replaced should be used.

The repair costs should include all costs, direct or indirect, tangible or intangible, which the water authority considers to be important in the analysis of the individual main concerned.

Output

The program provides an analysis for a period of up to 50 years from the present, with a limit of 80 bursts. A set of Total Future Cost values is provided assuming replacement at each future predicted burst number. In addition, the predicted time of each of the future bursts is shown. The optimal replacement time will then be at the burst number corresponding to the minimum value of Total Future Cost. The output of the analysis will therefore indicate the optimum replacement time for the main as well as the estimated costs required to maintain the main. A graphic allowing the analyst to quickly determine the value and time of occurrence of the minimum Total Future Cost is also available with the program. An example of this graphic, generated for case study 1 in Appendix I, is shown.

APPENDIX G: GIS MODELLING OF PIPE SYSTEMS

A geographic information system (GIS) provides the ability to store, manage, analyse and display large volumes of pipe and parcel-based data. During the course of this project a prototype was developed to demonstrate the potential of a GIS to effectively provide solutions in the modelling of pipe systems. The development of the prototype consisted of a number of stages.

Stage 1 GIS Database Development

The spatial data used in the prototype consisted of parcel boundaries and pipe locations. This data, in DXF format, was imported into Arc/Info^{TM}. Substantial processing was required to convert DXF data into the topologically structured GIS database. The processing used several Arc Macro Language (AML) programs consisting of a number of automated data processing steps. In addition, manual editing was also required to complete the database creation process.

Once the spatial database processing was completed, attribute data contained in both the DXF files and dBase files (pipe failure data) were linked to the appropriate spatial object. This attribute data consisted of node numbers, valve information and pipe characteristics. The database development stage led to the development of a number of algorithms that will be used in later database creation exercises.

Stage 2 Analysis of Pipe Failures

The prototype enables pipe failure data to be analysed by:

- selecting a pipe on the display screen and displaying the attribute data of the pipe
- displaying pipes based on a set of attribute criteria. This provides the capability to group pipe data together on a spatial as well as an aspatial set of criteria.

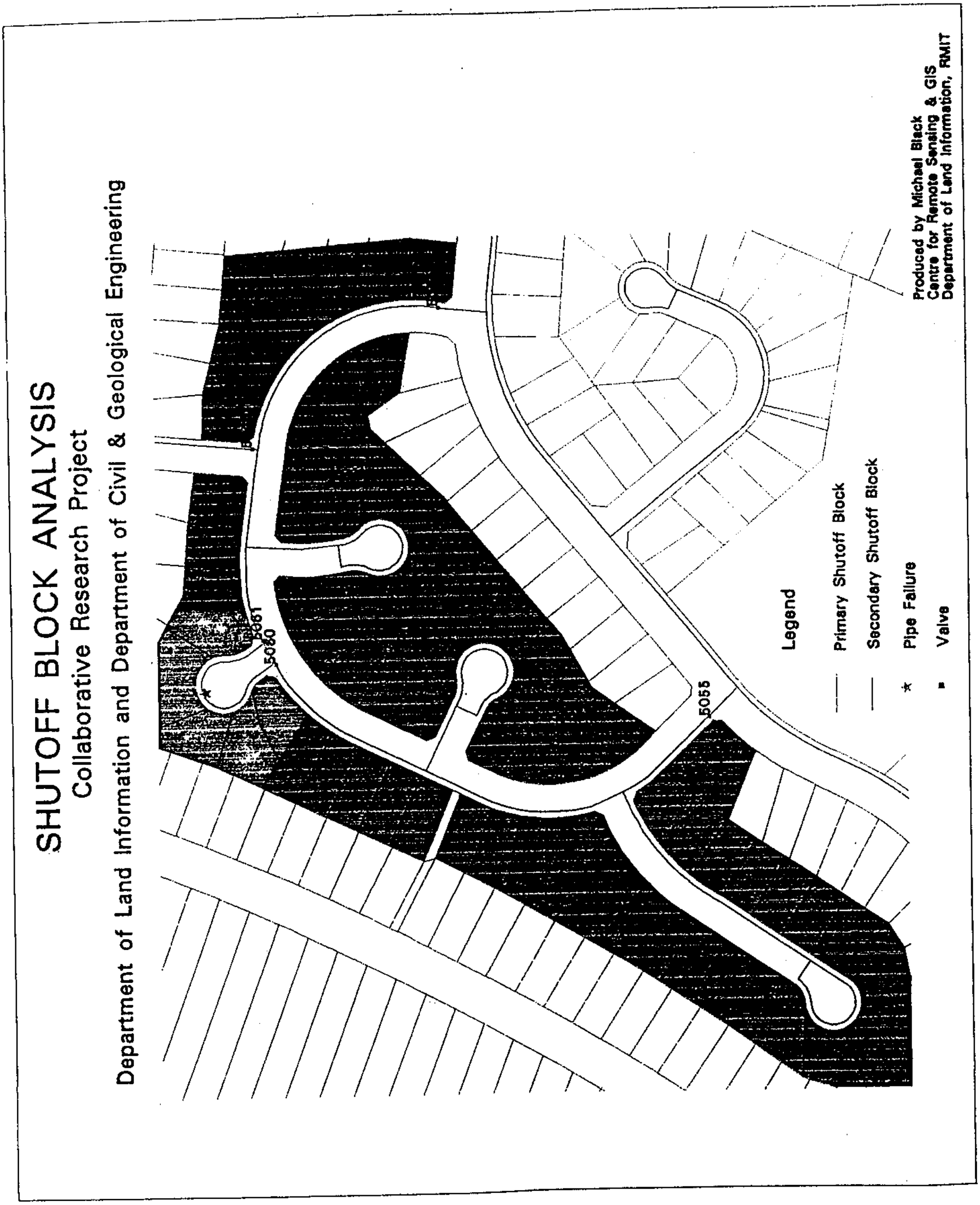
Stage 3 Pipe Shut-Off Block Identification

The prototype identifies valves to be shut off in response to a pipe failure. The failed pipe is identified by a street number and address. Once the pipe is selected, the GIS searches back along the pipe network until it finds the valves defining the primary shut-off block. With identification of the shut-off block the affected parcels are then identified and highlighted. The present development phase of the prototype identifies the primary and secondary shut-off blocks.

Further Developments

It is intended to develop the shut-off block algorithm further to incorporate tertiary and higher order shut-off block analysis. This development, together with other applications of the GIS to infrastructure management, will be undertaken through further collaboration between the Department of Civil and Geological Engineering (RMIT) and the Department of Land Information (RMIT).

Figure AG.1



SHUTOFF BLOCK ANALYSIS
Collaborative Research Project
Department of Land Information and Department of Civil & Geological Engineering
5061
5060
5055
Legend
Primary Shutoff Block
Secondary Shutoff Block
Pipe Failure
Valve
Produced by Michael Black
Centre for Remote Sensing & GIS
Department of Land Information, RMIT

APPENDIX H: MELBOURNE WORKSHOPS - MANAGEMENT OF WATER RETICULATION NETWORKS

RMIT Conference, 9th and 10th February,1995

Workshop Delegates:

Allen,	Rod	Land Information, RMIT
Andre,	Mike	Coliban Water
Caligan,	D	Central Highlands Water
Carr,	John	EWS Adelaide
Changsiri,	Achara	Tubemakers
Chant,	Ralph	Coliban Water
Comerford,	Laurie	Civil and Geologic Eng, RMIT
Constantine,	Graham	CSIRO, Adelaide
Ferguson,	Phillip	Tubemakers
Giddings,	Barry	Yarra Valley Water, Melbourne
Gilbert,	Leon	Coliban Water
Gombos,	Joe	South East Water, Melbourne
Hennessy,	Greg	Melbourne Information Technology Services (MITS)
Holt,	Ivan	Civil and Geologic Eng, RMIT
Huguenin,	Barry	Thor Plastics
Kildea,	Daniel	Statistics and Operations Research, RMIT
Mavin,	Ken	Civil and Geologic Eng, RMIT
McBride,	Rex	City West Water, Melbourne
McClellan,	Duncan	Civil Eng Student, RMIT
Minetti,	Rodolphe	PhD Student, RMIT

Mitchell,	Heather	Statistics and Operations Research, RMIT
Muir,	Sandy	Gutteridge, Haskens and Davey (Melbourne)
O'Hara,	Glen	EWS, Adelaide
Parameswaran,	Suthiasn	Yarra Valley Water, Melbourne
Richards,	Brad	Barwon Region Water
Schofield,	Bill	Albury City
Scott,	Randall	Melbourne Water
Sim,	Stephen	Barwon Region Water
Smith,	Peter	Statistics and Operations Research, RMIT
Stahmer,	Mike	Vinidex
Stewart,	Doug	South East Water, Melbourne
Taylor,	Ed	South East Water, Melbourne
Thomason,	Martin	South East Water, Melbourne
Tsui,	Ed	Sydney Water Board
Ware,	John	Mid Goulburn Region Water
Wilson,	David	Civil and Geologic Eng, RMIT

UWRAA Water Renewal Decision Process Research Projects Conference
- Melbourne Water Head Office, 29,30 September,1994

Conference Delegates:

Comerford,	Laurie	RMIT
Constantine,	Graham	CSIRO,Adelaide
Giddings,	Barry	Yarra Valley Water, Melbourne
Gombos,	Joe	South East Water, Melbourne
Mavin,	Ken	RMIT
McLay,	Gary	Melbourne Water
Minetti,	Rodolphe	Post Graduate, RMIT
Mitchell,	Heather	RMIT
Parameswaran,	Suthiasn	Yarra Valley Water, Melbourne
Scott,	Randall	Melbourne Water

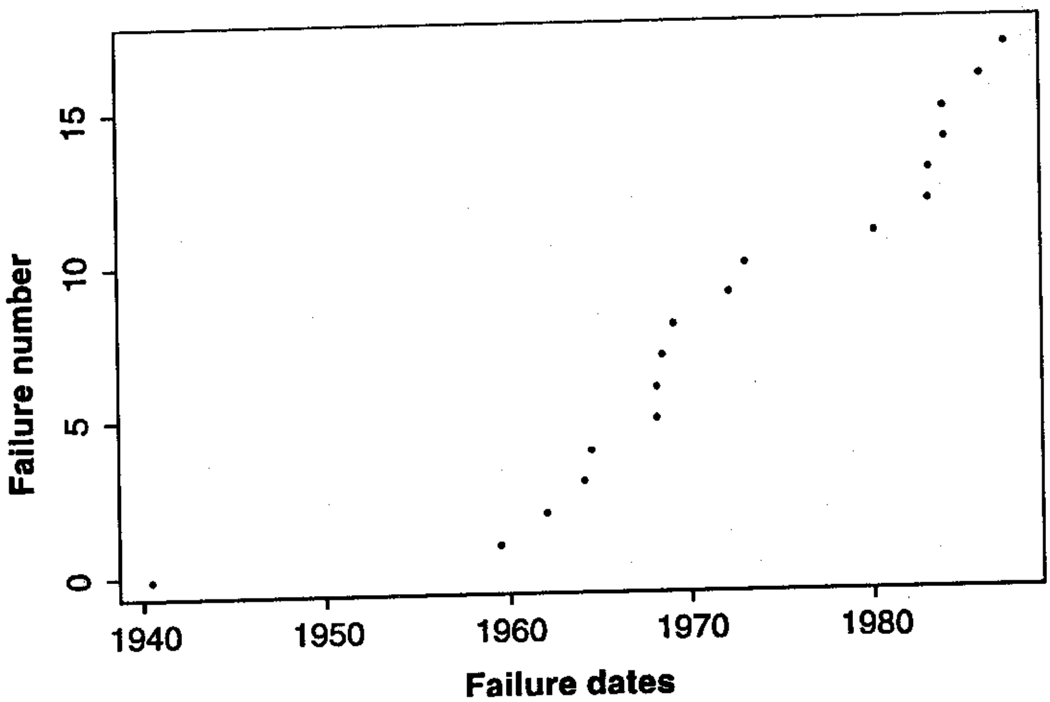
APPENDIX I: CASE STUDY

The statistical techniques of this report were applied to data from a water main in Ringwood to demonstrate their operation. The main, in William Street, is a 100-millimetre diameter asbestos-cement pipe of the Mazza type. It is 320 metres long and was laid in 1940 in the Silurian Andersons Creek formation, which is massive siltstone interbedded with thin sandstones. It serves 145 residential lots and has a recorded failure history of 17 failures: the first occurring in 1959 and the last recorded in the database available to this study, in 1987.

Figure AI.1 shows the failure history of the pipe.

Figure AI.1

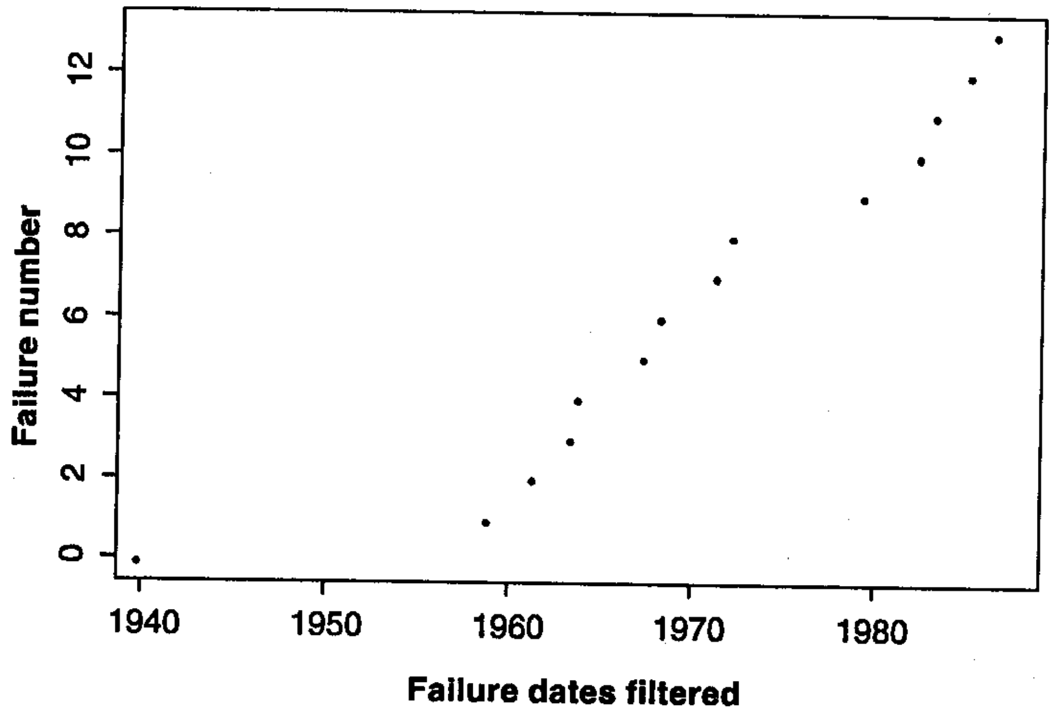
Failure history of a pipe



The probability-based filtering model described in section 3 was used to filter the data. This process removed four failures. Two of the filtered data points were removed because they occurred within two days of a previous failure, and it was considered that these secondary failures were caused as a consequence of the first. The other two failures occurred within four months and were at a distance of less than 10% of the main length from a previous failure. These are believed to occur because of a faulty repair or start up of the main after the previous failure. Figure AI.2 shows the filtered failure data.

Figure AI.2

Filtered failure data



The filtered curve is smoother than the unfiltered and approximately one quarter of the data has been removed by the filtering process. It should be remembered that this main has a tendency towards double breaks, which need to be taken into account in repair practices and economic assessments.

An exponential-type model for the time between successive failures was fitted to the data. This process is demonstrated also for incomplete data. To do this the data set was shortened by removing all failures occurring before 1970 (though it is assumed that the construction date is known). The analysis used the Excel5 *Solver* add-in, though similar facilities exist in other packages and spreadsheets.

Working With a Complete Data Set

The first step in the analysis process is to convert the failure dates stored in the database into a form that the software requires.

The next requirement is to set starting values for the regression coefficients, α and β. Starting values of $\alpha = 0.5$ and $\beta = 0.1$ were found to converge fairly quickly. Since β is a logarithmic term it should be constrained to a value greater than zero. A very small positive value can be set as a lower bound for β, say 10^{-9}.

Having set the regression coefficients the interfailure times can be calculated for each failure. This is done using the formula:

$$\text{interfailure time} = 12\alpha \ln\left(1 + \frac{100}{100[\text{failure number} - 1] + \beta}\right)$$

The numbers 12 and 100 are scale parameters and could be omitted if desired.

Next the estimated failure dates are calculated. For the first failure, this is the first estimated interfailure time plus the construction date. In some of the Melbourne Water data sets only the construction year was available, in which case the pipe was assumed to be constructed on the 30th June giving a maximum error of six months. For subsequent failures the estimated failure date is the previous estimated failure time plus the present interfailure time.

The total sum of squares for error is calculated next.

From the William Street Main data the estimated values of α as β were 1.340 and 0.280 respectively. This gave an error standard deviation of 1.31 years and a cumulative prediction error on the last failure of 2.76 years. The prediction error on the last interfailure time was 1.32 months.

To illustrate the different values which the coefficients can take the same procedure was applied to a second main. The main selected was a 100-millimetre diameter Mazza asbestos pipe in Clayton Street Sunshine. This main is 96 metres long and serves 43 residential lots. The soil type is Quaternary newer volcanic. The main was constructed in 1953, and has had a total of seven failures: its first recorded failure in 1968 and its last recorded in 1985. The filtering procedure did not remove any failures. By applying the same fitting methods, the estimated value of α was 0.706 and β was 0.311. This gave an error standard deviation of 3.10 years and a cumulative prediction error on the last failure of 2.64 years. The prediction error on the last interfailure time was 6.07 months.

Working With Incomplete Data

In addition to starting values for α and β, a starting value for the number of failures that occurred before records were kept (N) needs to be made. This value must be a non-negative integer.

The estimated interfailure times can be calculated for each failure by using the formula:

$$\text{interfailure time} = 12\alpha \ln\left(1 + \frac{100}{100[\text{failure number} + N - 1] + \beta}\right)$$

This is the same formula as used for a complete data set with the exception that N is added to account for the failures before records were kept, and the variable failure number is now the number of the failure in question since records have been kept.

Before applying this procedure to the William Street main, all failures occurring before 1970 were deleted. This left seven filtered failures on which to do the estimation. The estimated values of α and β were 0.728 and 0.011 respectively. N was estimated to be 0 instead of the actual six. This gave an error standard deviation of 1.566 years and a cumulative prediction error on the last failure of 0.22 years. The prediction error on the last interfailure time was -0.34 months. The difference between the estimated and true values of N, together with the improvement in the accuracy of prediction, suggests that the behaviour of the water main was different in the early life of the main.

Square Root Interfailure Time Model

As suggested previously, if constraints on the parameters cannot be included in the fitting procedure, then the square-root power-type model can be used. No special computer software is required to do this: it could be carried out on a hand held calculator.

The x values for the least squares equation are calculated using the equations:

$$x_{1,n} = n$$

and

$$x_{2,n} = \sum_{i=1}^{n} \left\{ \sqrt{\text{failure number}_i - 1} + \sqrt{\text{failure number}_i} \right\}$$

Use the ordinary least-squares equations (or the function supplied by the computer program you are using) to e
stimate the values of α and β in the equation:

$$\text{cft} = \alpha N + \beta \sum \left\{ \sqrt{\text{failure number} - 1} + \sqrt{\text{failure number}} \right\}$$

where cft is cumulative failure time.

Alternatively, the formulae required are:

$$sum = \sum x_{1,i}^2 \sum x_{2,i}^2 - \frac{1}{n} \sum x_{1,n}^2 x_{2,n}^2$$
$$\alpha = \frac{\sum x_{1,i} y_i \sum x_{2,i}^2 - \sum x_{1,i} x_{2,i} \sum x_{2,i} y_i}{sum}$$
$$\beta = \frac{\sum x_{2,i} y_i \sum x_{1,i}^2 - \sum x_{1,i} x_{2,i} \sum x_{2,i} y_i}{sum}$$

where y is the cumulative failure time.

For the William Street Main, using this procedure, the value of α was 9.264 and β was -1.220. This gave an error standard deviation of 4.557 years and a cumulative prediction error on the last failure of 2.18 years. So, as expected, its predictions are not as good as the exponential model.

Square Root Interfailure Time Model with Incomplete Data

When the this model is adapted to suit incomplete data it becomes non-linear in the parameters and so requires an iterative non-linear least-squares algorithm for estimation. It is also possible for the algorithm to fail because of negative values in the square root if the value of N falls below zero. However, because N is an integer and unlikely to be large it is possible to trial various values of N, starting with zero, and increasing until the minimum error sum of squares, or error standard deviation depending on the program, has been found.

To calculate the x values for the least-squares equation use:

$$x_{1,n} = \text{failure number} + N$$

and

$$x_{2,n} = \sum_{i=1}^{n} \left\{ \sqrt{\text{failure number}_i + N - 1} + \sqrt{\text{failure number}_i + N} \right\}$$

Again, this is the same as the formula for the complete dat set except for the addition of the N term to account for the failures before records were kept, and, once again, the variable failure number is now the number of the failure in question since records have been kept.

By using the ordinary least-squares equations, the function supplied by the computer program, or the equations given for the complete data the values of α and β can be estimated in the equation:

$$\text{cft} = \alpha N + \beta \sum \left\{ \sqrt{\text{failure number} + N - 1} + \sqrt{\text{failure number} + N} \right\}$$

for N = 0,1,... until the minimum sum of squares has been found.

For William Street, the data was shortened as before by deleting all failure data before 1970. This gave the results of the trial in table AI.1.

TABLE AI.1

WILLIAM STREET TRIAL RESULTS

N	α	β	Std Error	Pred Error
0	26.33	-5.814	5.196	3.88
1	19.76	-3.801	2.523	1.98
2	19.76	-3.807	1.507	0.82
3	12.29	-1.824	1.254	0.09
4	10.02	-1.306	1.295	-0.38

The minimum has been reached with N = 3 failures before records were kept. As for the exponential models, the incomplete data model gives better prediction than that requiring a complete data set.

Economic Analysis

Case study 1: **Details of main location**

William St, Ringwood, Melbourne.

Pipe description:	100-millimetre diameter AC
Length:	320 m

Estimated replacement cost: $70000

Schedule of Estimated Repair Costs

Direct Monetary Costs

Direct cost of repair:	$1500
Site reinstatement:	$1000
Clean-up:	$ 500
Third-party damage:	$ 500
TOTAL:	$3500

Indirect Monetary Costs

Losses to business from supply disruption (table AI.2)	= Nil
Losses to business from traffic disruption	= Nil
Other	= Nil

Intangible Costs

Disruption to non-commercial customers	= Nil
Diminution of prestige of water authority (table AI.3)	= Nil
Environmental impacts	= Nil

Total Repair Cost: $3500

Main Maintenance Characteristics (after data filtering)

Current Burst Number: 13

Correlation coefficients from statistical analysis:

$\alpha = 1.340$

$\beta = 0.280$

Estimated maintenance category of replacement main: 1

Output from computer program 'main.exe':

TABLE AI.2 OUTPUT FROM COMPUTER PROGRAM 'MAIN.EXE' FOR VARIOUS REPAIR COST SCENARIOS FOR CASE STUDY 1

Scenario	Discount Rate	Direct Monetary Costs	Indirect Monetary plus Intangible Costs	Total Repair Cost	Present Value Total Future Cost (min value)	Estimated optimum replacement time from the present (years)	Estimated burst number at optimum replacement time
1	5%	$3500	nil	$3500	$78800	4	17
2	10%	$3500	nil	$3500	$55300	14	33
3	10%	$3500	$2000	$5500	$72200	7	21
4	10%	$3500	$4000	$7500	$82500	2	15
5	10%	$3500	$4300	$7800	$86400	0	13

Scenario 1 and 2 both explored the situation where only the direct monetary costs of the predicted repairs are included. The minimum present value of total future cost in scenario 2, evaluated using a 10% discount rate, is predicted to occur at burst number 33 - that is, well into the future. Therefore, under this situation it is clearly preferable to continue to repair future bursts rather than consider replacement of the main. However, if the appropriate discount rate were 5%, as shown in scenario 1, the situation changes substantially. The minimum present value of total future cost is predicted to occur in 4 years from the present time. In this situation, it would therefore be worth considering replacing the main in the near future.

Scenarios 3, 4 and 5 all used a 10% discount rate and are aimed at assessing the impact of indirect and intangible costs on the decision. Scenario 5 shows that if these costs reach a level of $4300 per repair, then the optimal decision would suggest imminent replacement of the main.

Case study 2 Details of main location

Clayton St, Sunshine, Melbourne

Pipe description:	100-millimetre diameter AC
Length:	96 m

Estimated replacement cost: $22000

<u>Schedule of Estimated Repair Costs</u>

Direct Monetary Costs

Direct cost of repair:	$1500
Site reinstatement:	$500
Clean-up:	$500
Third-party damage:	Nil
TOTAL:	$2500

Indirect Monetary Costs

Losses to business from supply disruption	= Nil
Losses to business from traffic disruption	= Nil
Other	= Nil

Intangible Costs

Disruption to non-commercial customers		= Nil
Diminution of prestige of water authority		= Nil
Environmental impacts		= Nil
Total Repair Cost:	$2500	

Main maintenance characteristics (after data filtering)

Current burst number: 7

Correlation coefficients from statistical analysis:

$\alpha = 0.706$

$\beta = 0.311$

Estimated maintenance category of replacement main: 1

Output from computer program 'main.exe': see table AI.3

TABLE AI.3 OUTPUT FROM COMPUTER PROGRAM 'MAIN.EXE' FOR VARIOUS REPAIR COST SCENARIOS FOR CASE STUDY 2

Scenario	Discount Rate	Direct Monetary Costs	Indirect Monetary plus Intangible Costs	Total Repair Cost	Present Value Total Future Cost (min value)	Estimated optimum replacement time from the present (years)	Estimated burst number at optimum replacement time
1	5%	$2500	nil	$2500	$25500	0	7
2	10%	$2500	nil	$2500	$24700	0	7

As table AI.3 shows, the outputs from scenarios 1 and 2, adopting a 5% and 10% discount rate respectively, indicate that the main should be considered for imminent replacement. This recommendation would apply even though the indirect monetary costs and the intangible costs associated with a burst main are negligible.

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