Capital Cost Optimisation of Pumping and Reservoir System Design

B Barta • N Rowse

Report to the Water Research Commission
by the
Water Systems Research Group
University of the Witwatersrand

WRC Report No 757/2/98

Capital Cost Optimisation of Pumping and Reservoir System Design

by

B BARTA and N ROWSE

Report to the Water Research Commission

by the

Water Systems Research Group
University of the Witwatersrand
Private Bag 3
WITS
2050

Head of Department
and Project Leader: Prof D Stephenson

Report No 757/2/98
ISBN 1 86845 453 3
ISBN SET 1 86845 454 1

NB. THIS REPORT SUPPORTED BY SOFTWARE CALLED "RSDO"

The software emanating from this study called "RSDO"-Reservoir System Design Optimisation is available only from the WRC internet site, http://www.wrc.org.za. Should you not have access to internet facilities or encounter problems in downloading the software, kindly inform us by email or fax, and a copy of the software will be forwarded to you:

Water Research Commission
PO Box 824
PRETORIA, South Africa
0001
Tel : (012) 331-2565
Fax : (012) 331-2565
email: wrcit@wrc.org.za

EXECUTIVE SUMMARY

Background of the research project

Motivation for this research project arose from high pumping costs experienced by the Rand Water and other water supply authorities in South Africa. This component of water supply costs was therefore identified as of vital importance for research in optimisation of urban water supply systems. The Water Research Commission granted two years of research funding for this project in 1996 and 1997 financial years.

This research project is a two phase project with two separate but related subjects compiled in reports as follows:

(a) Pumping rules phase report and software program. Investigates a dynamic computer simulation model which is then used to optimise the rules for pumping operation used by the operators of reservoir storage systems with respect to demand patterns, pumping costs and storage volumes. The research in this phase culminated in the computer program Reservoir System Pumping Optimisation (RSPO).

(b) Design parameters phase report and software program. This phase of the research project investigates and evaluates existing design parameters relating to storage capacity for balancing draw-off and pumping and emergency storage in the light of the costing model and the design parameters calculated from it. The model determined and proposed in this report is able to assess rapidly various options for a water supply system with regard to service or distribution water storage and conveyance. The research culminated in the computer program Reservoir System Design Optimisation (RSDO).

The prime objective of this research project has been set at reconciling the water supply optimisation techniques by means of computer facilities to enable Rand Water and other water supply authorities to keep up with new research developments and demands amongst their consumers. During the project execution, various approaches in optimising of urban water supply systems were investigated and suitable optimisation techniques and variations in design criteria are proposed for application.

The findings of the research compiled in this report are based primarily on the background of the large Rand Water supply system considered to be a second tier bulk water supply authority. The Rand Water bulk water supply system is operated and managed to supply the demands for potable water generated primarily by the third tier water supply authorities, that is, mainly local authorities and large industries.

The total cost of a water supply system consists of the costs of pumps, pipes, storage, operation and maintenance and energy to run the system. The size of the pipe is a most important factor in the whole system. The pump size is largely dependent on the size of the pipeline and the operational policy of the balancing storage. The balancing storage is primarily dependent on consumer demand for water.

The problem has been solved by developing a comprehensive econo-mathematical model of a water supply system incorporating capital and energy cost functions to determine the minimum total cost of the system.

Water Supply Optimisation Problem Statement

Almost without exception, water supply systems providing either raw or purified water to the urban areas are facing the likelihood of problems related to optimal sizing of the system's specific component or pumping rate. However, most water supply systems are pumping systems depending heavily on electrical energy inputs. With increases in energy cost and costs of interest on capital funding for new projects, even marginal savings in overall project cost by adequate optimising can allow for more people to be supplied with water.

The major problem of the South African water supply industry is demand for water supplies to the new urban developments. This process materialises either by introducing new water supply systems or by extending existing supply infrastructure subsequently in need of optimisation process based on the changes introduced to an existing system. The capital cost optimisation problem solutions are generally sought to the following water supply scenarios:

(a) need for a new or additional water supply due to growth in urban, agricultural and energy water demands;

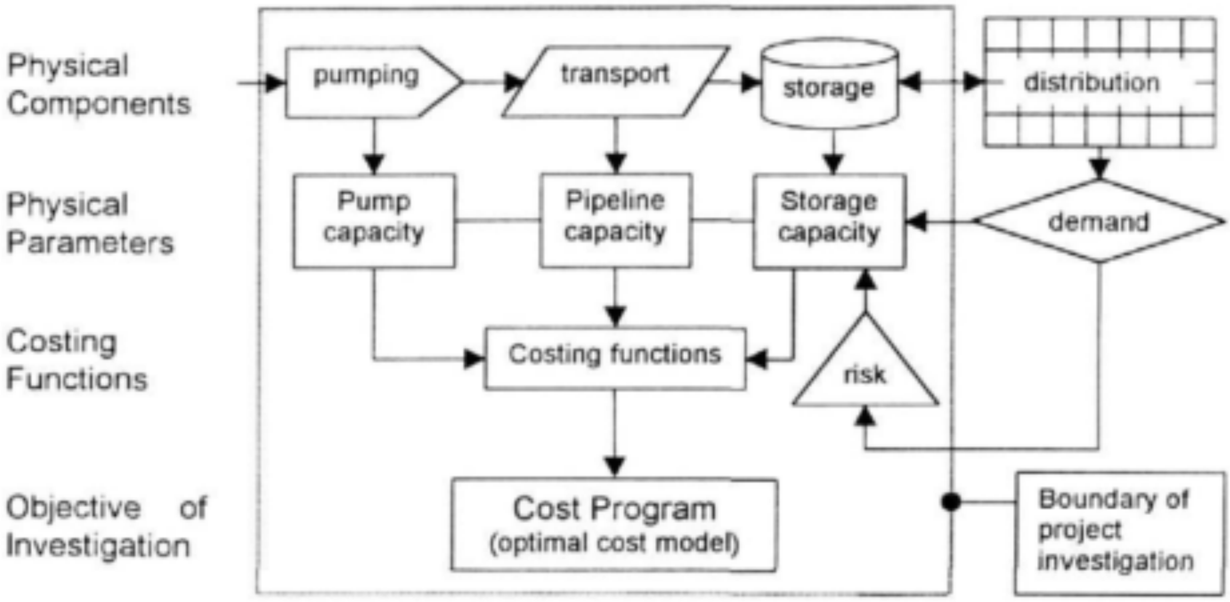
(b) subsequently there will be a need for an optimal design of a new or enhanced water supply system;

(c) consequently there will be a need for an optimum sizing and timing of water supply system components.

The approach in capital cost optimisation of these scenarios adopted by this research project is based on the principle as shown below:

The objective function of this problem is sought by means of capital cost optimisation model which is to minimise the overall costs of the water transport, storage and pumping energy requirements for a trunk main water supply. The assessment of a feasible solution is mathematically formulated and exercised by means of computer spreadsheets, developed originally for use on the Microsoft Excel application, and subsequently rewritten as a stand-alone application.

The program simulates a water supply system for a given operation period and determines the optimum pipe diameter, balancing storage volume for various pumping operating factors for a pre-determined water demand profile. The conceptual problem statement for this project is illustrated below:



The required inputs to the program are of a technical nature including the pre-determined supply area demand patterns, pumping and operating rates, pipeline and balancing storage sizes. The financial and economic indicators are related to the chosen planning or capacity horizons. The cost functions for various water supply system components are required to be determined as well as the interest and inflation rates which are important inputs to the model.

The risk of failure or reliability of the investigated water supply system are defined as the probability that the system will provide demanded flow rates at required pressure heads. The reliability parameters are related to the probability distribution of failure and repair times for pipelines and pumps and can be tested by using a spreadsheet model developed by the Water Systems Research Group (WSRG) for incorporation in the emergency storage of service reservoir.

Overview of current water supply design standards

The prime limitations of current guidelines for sizing of storage reservoirs and trunk mains (i.e. feeders) as identified are as follows:

(a) the size, character or unique features of the supply area are only partially reflected in the guidelines by adopting a single value of the AADD for a supply area.

(b) the pre-specified feeder pipe capacity into the storage reservoir does not allow for the optimum cost combination of supply pipe and storage size.

(c) the probability of water supply system failure due to the operational risks is not determined by the current guidelines (i.e. the inherent factors implied by the guidelines are not known).

(d) the reliability of supply concept inherent to the optimal design of a water supply by means of capital cost optimisation can ensure a reasonable trade off between the costs and efficiency contrary to the applications of standard guidelines.

(e) the current design standards do not allow for the consumers or a financing party to decide if they are prepared to absorb the financial consequences with regard to various levels of supply reliability to be based on the economical optimum storage volumes for emergency and particularly fire-fighting purposes.

Development of Capital Cost Optimisation Program

Consumer demand parameters - For the purposes of appropriate inputs into the capital optimisation program, the weekly demand profiles obtained from monitoring of experimental supply areas within the Rand Water limits of supply were categorised into residential, commercial and industrial water use categories. From the data available, the demand patterns were derived for each specific day of the week and type of water use category. The different daily demands quite explicitly indicated working or social habits within monitored supply areas.

It was tested that the instantaneous (hourly), daily and monthly peak demands can be determined from the monitored supply flows as the product of the Annual Average Daily Demand (AADD). It was also established that the corresponding peak factor would be a function of the number of consumers residing within a specific supply area (e.g. low or high income). The sequence in determination of design demand for a specific supply area adopted in the development of capital cost optimisation program is as follows:

Supply area design demand (kl/d) = Residential (kl/stand/d) + commercial (kl/ha/d) + industrial component (kl/ha/d)

The residential demand can be divided into the high, medium, low and very low income group demand components. In the real-life setting, a large portion of such information would be derived most likely from the municipal structure development plan or determined by the Town Planning Department. In many instances, the original proposals have to be regularly revised to accommodate changes in demand for water and in such instances the capital cost optimisation program can be highly useful.

The key problem affecting the development of water supply systems is forecasting of consumers' demands ahead in time. The long-term demand forecasts (10 to 30 years ahead) are usually estimated by means of population and per capita water demand growth rates. The short-term (i.e. hourly, daily, weekly or monthly) forecasts of water demand for the purposes of water supply optimisation (e.g. minimum cost pumping) are commonly based on the statistical parameters of historical data.

In the demand forecasting process two independent demand components are important to observe. These are: deterministic demand component - indicating commonly the regular repetitive trends (e.g. long-term trends, seasonal trends or habitual trends), and stochastic demand component (i.e. random events which are difficult to predict in their recurrence and magnitude, as for example, fire, pipe breakage's and outages).

The prerequisite in demand forecasting is to determine the specific quantifiable features of typical demands for a specific geographical area (or system zone) for which demands are to be designed or monitored and forecasted.

Pipeline optimisation parameters - The problem of pipeline main optimisation refers to the relationship between pipeline diameter, flow velocities, head losses and pipeline costs. On the other hand, the overall costs of water pumping supply can be minimised by selecting the optimum pipeline diameter reservoir storage volume and pumping rate. The pumping main diameter is selected by minimising the total capital plus operating costs brought to a common time basis.

The flow velocity at optimum pumping main diameter varies from 0,6 to 2,0 metres per second, depending on flow and working pressure. By doubling the diameter of the pipeline and other factors being constant (i.e. pumping heads) the pipeline capacity increases nearly six-fold in comparison to much smaller increase in cost of a pipe.

The water supply authorities are finding it economic to size the large pipelines for a service life of between 10 to 30 years. It should be realised that the location of a booster facility on a pipeline can reduce significantly the pressures and pipeline wall thickness may be minimised. The cost of a booster station has to be competitive to the gains from cost reduction on the trunk main pipeline.

A range of pipe diameters will produce different friction heads for a particular pumping rate. The cost of pumping is directly related to the internal pipe friction, making the pipe diameter an important component in determining the pumping cost. The Darcy formula for friction is used in determining the optimal diameter. The Darcy headloss coefficients are usually provided by the pipe manufacturer.

The majority of the trunk main systems are designed to satisfy and regulate major demand fluctuations (i.e. average and peak demands) generated within a supply area. The initial supply regime for a trunk main is essentially determined with a provision for future likely developments within the designated supply area. Note that for a geographically large area, considerable energy is used in overcoming the natural topography. The initial costs related to the pipe system are far greater in proportion to the operational and maintenance costs, so therefore the design stage has the greatest impact on the ultimate project cost.

Pumping optimisation parameters - The main purpose of the pumping operation is to transfer energy to the through-flow of water by effecting a step-increase in head or pressure. In water supply applications, pumps are typically running continuously and in most cases the required step-increase in pumping head is substantial.

The most advanced approach to the selection of pumping installation should be based not only on the parameters of pumping head and quantity of liquid to be pumped, but on more broader assessments of intermediate or transient conditions of the whole pumping system.

An advanced method in pump installation selection is called the total integrated approach to pump selection. Such an approach requires access to sophisticated financial packages, production control software and computer aided design software. To date, most pumping installations are selected manually from catalogues based on very little information available.

A typical "user friendly" program for the designer of pumping installation is the AQUATEC program available in South Africa from the University of the Witwatersrand. The program is written for an IBM compatible PC with DOS version 3.1 or higher, 640 KB RAM and a hard drive. The program will interface with a monochrome or colour monitor.

Optimal pumping operation - This aim has been fulfilled with the completion of the computer program *Reservoir System Pumping Optimisation* (RSPO), which allows historical demand data to be imported and analysed, a stochastic model of that data to be fitted, and optimum operation policies to be calculated.

RSPO analyses the available historical data by fitting either a linear or exponential trend, whichever gives the best fit. This trend is then removed from the data, and annual, weekly and daily periodic trends are fitted. To model the serial autocorrelations of the data, a linear stochastic model is fitted.

The simulation option of RSPO allows the user to calculate the total running costs of a given reservoir system using either historical, or synthetically generated, demand data. Both the cost functions and the operating policy used in the simulation can be specified by the user. This allows the user to test the effect of different cost and operation policies on the system, over any period the user chooses. The synthetic demand data generated using the simulation option has been shown to have a high statistical similarity to the historical data modelled.

RSPO also allows the user to calculate optimum operating policies for the system using either the Downhill Simplex method or the Dynamic Programming method of optimisation. RSPO is an integral part of this research project available to the user of RSDO.

Balancing storage optimisation - The storage reservoir is a key component in both the gravity and pumping water supply systems. The current design standards applied widely for determination of reservoir storage and trunk main sizes subscribe to a unitary application of the safety or time coefficients and annual average daily demand (AADD) of a supply area. The standard approach allows very little incentive for a designer to adopt the capital cost optimisation procedure in the design of a water supply system.

The sizing of balancing storage in bulk supply reservoirs depends primarily on he pumped system operation policy adopted by the supply authority. Such policy may be based on the following:

(a) *Constant pumping* - becoming rather obsolete for most modern pumped water supply systems.

(b) *Reduction in energy* - fluctuating water demands are met through adjusting the operating rate or number of operating pumps.

(c) *Supply system reliability* - the variation in water demands is correlated with the non-supply events which might occur within the supply system.

By present standard measures and the unitary electrical energy tariffs offered to consumers (e.g. Rand Water) by ESKOM (i.e. no differentiation is made between the day and night tariffs) the water supply utilities in South Africa do not necessarily define their bulk reservoir storage operational policies according to either (ii) or (iii) approaches as described above. At present the only stimulator in policy determination is considered to be a peak tariff imposed on users during "peak periods" (i.e. a period of excessive demand for power sold under higher tariff by a supplier).

Rand Water, on which the research focused, operates at any given time some 50 reservoirs ranging in size between 6 Ml and 600 Ml. With a total balancing storage of about 3900 Ml, this largest South African water supply authority has to cope with a wide range of operational requirements from its customers. It is, however, very difficult to argue with this policy approach as Rand Water supplies a vast number of customers with tremendous diversity in water demands and also with varying preferential priorities. To make it worse, some consumers do not comply with Rand Water's regulations regarding particularly distribution storage operations (i.e. the reservoir storage's of third-tier consumers). This has been observed by the researchers, e.g. on Rand Water's dedicated trunk supply line of the Bloemendal/Wildebeestfontein pumping sub-system.

The computer program Reservoir System Pumping Optimisation (RSPO) proposed in Phase A of this project dealt with this problem although it allows for simulation of such situation it is based strictly on the historical records. Should there be a general improvement in consumer demand patterns, the model will have to be recalculated to accommodate such changes in the pumping policy of Rand Water for a given supply area.

Storage installed in service or distribution reservoirs - It has to be realised that the research investigation process had to take into consideration a variation to the number of storage components installed in the service reservoirs versus the distribution reservoirs (i.e. reservoirs installed mainly by the third-tier water supply authorities). General design standards observed in determining of storage volumes of either reservoir are primarily based on the number of hours that are required to compensate for consumers' variable water demands, possible interruptions in water supply on account of the system's components failure and operational requirements.

In view of standard regulations for designing of distribution reservoirs, the development process of capital cost optimisation program had to incorporate also a provision for the fire-fighting storage component,. This makes the program useful to most designers sizing the balancing reservoirs. A fair amount of research time had to be devoted to this aspect of design parameters investigation.

As illustrated in the RSDO User Manual, all the reservoir's storage components instead of a demand storage (i.e. estimated by means of consumers income group composition) have to be determined separately. A designer has an option either to adopt standard design approach or to follow the methods in determination of storage components proposed by this report. Particular attention has been given to the methods in determining emergency storage. See Chapter 7.3.2 and relevant examples.

Storage component:	Current design parameters:	Proposed variations:
Operational freeboard	1.5 hours AADD	As current design approach
Emergency storage (i.e. pipeline breaks, etc.	Typically: 12 hours of the peak day	Apply spreadsheet of WSRG or other methods identified
Demand storage (i.e. to balance the variable flow rates)	Typically: 12 hours of the peak day for industrial area, 6 hours for all other areas	Apply RSDO to suit new or enhanced supply area
Fire-fighting storage	Varies according to fire-fighting regulations adopted	As current design approach
Bottom storage	1.0 hour AADD	As current design approach

Risk and reliability in water supply - The issue of risk in water supply system with balancing storage reservoirs is treated in the capital cost optimisation program as a combination of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequence of the occurrence. The risk reduction by the supply authorities through preventively spent capital is assumed to be inherent in a system's operation by the extent of annual maintenance expenditure.

The capital optimisation model developed under this research work into the program of Reservoir System Design Optimisation (RSDO Version 1.01) follows to a large extent the supply system reliability policy approach by means of estimating the emergency storage component with a separate spreadsheet model developed by the WSRG called EM_TRI.XLS.

Costing functions for optimisation - The designing process of a new or enhanced water supply system is related primarily to determination of the lowest initial cost, lowest running cost and the longest working life of the whole system and its components.

It is common that the costing during the conceptual design of a water supply system often excludes the costs for operation and maintenance. The initial development cost is usually taken as a function of the design capacity only. The shortage of information on costing functions for individual components of the supply system is usually the major reasoning for such simplification. This should not be accepted in optimising of the supply system as it may lead to an erroneous decision being taken.

The cost functions for various key components of a water supply system are usually determined from the sets of cost data represented by the best curve fitting. Such costing information can be obtained from analysis of developed or projects presently being constructed. The functions can have linear, quadratic or exponential form and can be expressed as a function of unit design capacity, diameter, pumping rate, etc.

Traditionally, the construction unit costs are retained in confidentiality by the construction specialists primarily for tendering purposes in a highly competitive market. The designer will usually adopt the most current or past escalated market costs, providing that these will be made available to him by the relevant specialist, into the optimisation procedure commonly without determining the cost sensitivity characteristics. Generally there is little costing data available through public domain channels for optimisation purposes of water supply.

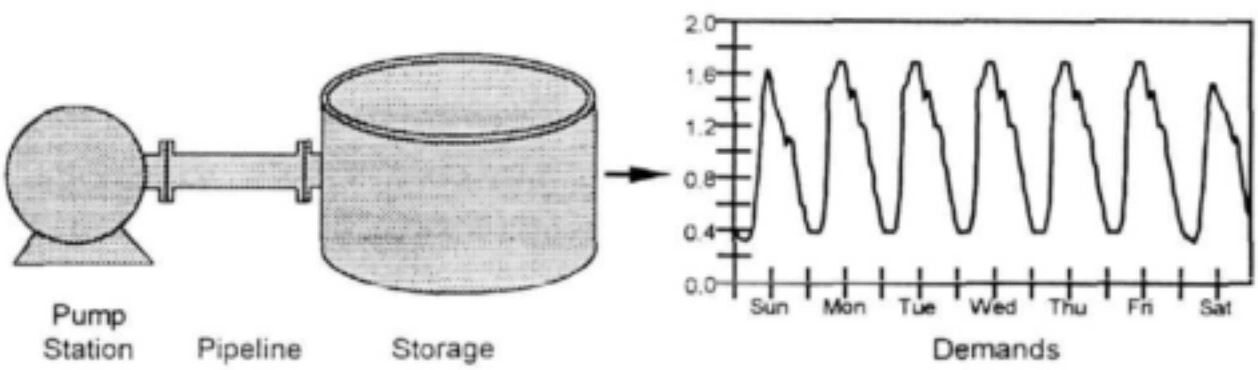
The prime source of costing information for this project were the Department of Water Affairs and Forestry, Rand Water and several consulting firms situated in Gauteng. The SA Federation of Civil Contractors made available sizeable extracts from their costing data base on civil water-related projects constructed between 1993 and 1996.All historical costs used in costing analysis were escalated to the 1996 design time base. The Construction Year Deflector Index (YDI) has been adopted in the escalation procedure.

Reservoir System Design Optimisation - or RSDO for short, is a tool developed for designers of service and distribution reservoir systems. It allows the designer to calculate optimum capacities for the pumping, storage and pipeline components of distribution and service reservoir systems.

Reservoir systems consist of a number of components which each require capital costs to install or running cost to operate or both. Optimising a reservoir system means finding the combination of capacities for each of the components that results in the lowest lifetime cost for the system.

The cost of constructing the pump station and the pipeline are a function of the maximum possible pumping rate. The size of the reservoir, and therefore its installation cost is a function of the relationship between the maximum pumping rate and the maximum demand rate. If the pumping rate can meet any demand rate, then no storage would be required. If the maximum pumping rate is only equal to the average demand then a maximum amount of storage would be required. The size of the pipeline affects both the capital and the running costs. A large diameter pipeline would be expensive to install, but would have low frictional resistance. This means that the running costs for pumping would be lower. A smaller diameter pipeline would be cheaper to install, but would cause greater friction, and therefore higher running costs.

The optimum capacities for each of the components depends on the relative costs involved and on the nature of the demand that must be met. Reservoir System Design Optimisation enables a designer to calculate these optimum capacities simply and quickly.



Other ways to optimise water supply

In addition to optimisation capability of the capital cost optimisation program (i.e. RSDO), there are also other ways available to the designer particularly in enhancing of existing supply system. These would be mainly in line with annual energy savings in water supply pumping or in adopting more relevant electricity tariffs. ESKOM has developed a long term tariff plan which offers to the large consumers of electricity cost effective solutions (e.g. standard rate tariff kVA demand versus the kW demand). The relevant examples are given in the report.

ACKNOWLEDGEMENTS

We wish to express our gratitude to the following persons for their contribution to the research work as reported in this document.

This project was guided by a steering committee constituted as follows:

Mr J N Bhagwan	W R C
Mr H C Chapman	W R C
Dr T C Erasmus	W R C
Mr J J van der Walt	Magalies Water
Prof. E A Moss	University of the Witwatersrand
Mr I Buchan	Umgeni Water
Mr G Hough	Bloem Water
Mr J Frame	City Council, Cape Town
Mr A C Fritz	W R C
Mr C Coetzer	D W A F
Mr G Constantinides	Rand Water

Other persons who contributed to the research work included:

Mr R Barry	Rand Water (Operations)
Mr M Lines	Rand Water (Operations)
Mr R C Thomson	Rand Water (Planning)
Mr D A Raymer	City Council, Port Elizabeth
Mr G A Willemse	Western Transvaal Regional Water Company
Mr A G Cooper	City Council, Kimberley
Mr D P Theron	City Council, Kimberley
Mr A J Dippenaar	Goudveld Water
Mr L E van Oudtshoorn	Bloem Water
Mr M Mol	City Council, Cape Town

CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF EXAMPLES

1. INTRODUCTION

1.1. General

The optimal development of a water supply system infrastructure depends heavily on a good analysis of the trade-offs between the demand for water and optimal sizing of key components of a water supply system. The optimal sizing of the water supply system is a problem of combined technical and costing constraints. Developing a comprehensive econo-mathematical model of a water supply system incorporating capital, operation and maintenance can solve this problem, and energy cost functions to determine the minimum total cost of the system. The timing of the water supply development problem depends fundamentally on reasonably accurate assessments and predictions of consumer demands.

The total cost of a water supply system consists of the costs of pumps, pipes, storage, operation and maintenance and energy to run the system. The size of the pipe is a most important factor in the whole system. The pump size is largely dependent on the size of the pipeline and the operational policy of the balancing storage. The balancing storage is primarily dependent on consumer demands for water. Typically the total reservoir storage consists of demand (or balancing), emergency, operational and fire-fighting storage's. There is a small proportion of the total storage allocated for operational purposes of the reservoir. This storage is to compensate for operational level errors and maintenance. The purpose of water supply system storage reservoirs is either operational or as a distribution reserve.

The selection procedure of optimal pipe diameter, balancing storage and pumping rates outlines the magnitude of the optimisation problem. According to Stephenson (1995) to optimise the total cost and operation costs of water supply, the pumping or gravity main diameter must be selected in conjunction with optimal operating (i.e. mainly energy) cost which decreases with increasing pipeline diameter. The optimisation procedure must also attend to the pumping rates with regard to optimal balancing storage and relative pipeline capacity. With increasing balancing storage, the delivery pipeline operation factor (i.e. ratio of average pumping rate to peak) increases and pipeline cost can be minimised. For any supply system there exists optimum sizes of pipeline, pumps and balancing storage.

There are a number of problems with this ideal situation. One is to find the "average" demand. When a new reservoir is being designed, it is not possible to measure the water demand being drawn from that reservoir. Even if there is an existing reservoir, the average demand measured will not be quite the same as for the new reservoir. This average demand also varies continually as the number of users grows and the types of user changes. The average demand also changes from season to season, and differs from weekdays to weekends. All these factors mean that there is no accurate way to know what the average demand will be by the time the reservoir is built, and certainly not 5 or 10 years into the future.

The prime objective of this research project has been set at reconciling the water supply optimisation techniques by means of computer facilities to enable the Rand Water and other water supply authorities to keep up with new research developments and demands amongst their consumers. The research project investigated various approaches in optimising of urban water supply systems and suitable optimisation techniques and variation in design criteria are proposed for application.

The findings of research compiled in this report are based primarily on the background of the large Rand Water supply system considered to be a second tier bulk water supply authority. The Rand Water bulk water supply system is operated and managed to supply the demands for potable water generated primarily by the third tier water supply authorities, that is, mainly local authorities and large industries.

1.2. Motivation for the research project

In 1995, pumping costs represented over 16 per cent of Rand Water's expenditure. Therefore optimisation of this component has been identified as of vital importance. However new developments in sizing of daily, fire and emergency storage's of service reservoirs and new developments in consumer demand assessment by means of telemetry and logging devices have highlighted needs for more adequate design techniques for water supply development.

The design of a supply system by the optimisation models allows for a reduction in operating costs and will result in lower water tariffs. The cost of water also affects the cost of almost everything produced, so a decrease in the cost of water can result in a general decrease in the cost of living for all consumers.

Stallard (1980) focused on Rand Water's global demand pattern and tried to optimise the pumping at the Vereeniging, Zuikerbosch and Zuurbekom pump stations. The pump stations and reservoir located in the supply subsystems were excluded from his analysis. The recent developments in planning, design, construction and operation of water supply systems highlighted a need for the rapid and reliable optimisation techniques based on the refined cost inputs and computer-based programs. Taking into consideration ever increasing demand for water mainly within the urban areas and development associated with bringing water over long distances, the water supply capital cost optimisation program is seen to be a highly suitable tool for planners and designers of water supply.

1.3. Methodology in project execution

The approach to development of the capital cost optimisation program is based on the principle of optimisation as illustrated below:

The objective function of capital cost optimisation model is to minimise the overall costs of the water transport, storage and pumping energy requirements for a trunk main water supply. The assessment of a feasible solution is mathematically formulated and exercised by means of computer spreadsheets, developed for use on the Microsoft Excel application.

The program simulates a water supply system for a given operation period and determines the optimum pipe diameter, balancing storage volume for various pumping operating factors for a pre-determined water demand profile.

The required inputs to the program are of the technical nature including the pre-determined supply area demand patterns, pumping and operating rates, pipeline and balancing storage sizes. The financial and economic indicators are related to the chosen planning or capacity horizons. The cost functions for various water supply system components are required to be determined as well as the interest and inflation rates, which are important inputs to the model.

The reliability of the investigated water supply system is defined as the probability that the system will provide demanded flow rates at required pressure heads. The reliability parameters are related to the probability distribution of failure and repair times for pipelines and pumps and are incorporated in the emergency storage of service reservoir.

1.4. Research project structure

This project is a two-phase project with two separate but related subjects compiled in reports as follows:

(a) Pumping rules phase report and software program: Investigates a dynamic computer simulation model which is then used to optimise the rules for pumping operation used by the operators of reservoir storage systems with respect to demand patterns, pumping costs and storage volumes. The research in this phase culminated in the computer program Reservoir System Pumping Optimisation (RSPO).

(b) Design parameters phase report and software program: Investigates and evaluates existing design parameters relating to storage capacity for balancing draw-off and pumping and emergency storage in the light of the costing model and the design parameters calculated from it. The model determined and proposed in this report is able to assess rapidly various options for a water supply system with regard to service or distribution water storage and conveyance facilities. The research culminated in the computer program Reservoir System Design Optimisation (RSDO).

2. LITERATURE AND FIELD SURVEYS

2.1. State-of-the-art in water supply optimisation

In the process of the early development of this project, an extensive literature survey was conducted for the purpose of assessing state-of-the-art in water supply optimisation locally and internationally. Some fifty-eight technical papers and numerous textbooks were identified to be relevant to the subject of water supply optimisation. The literature articles studied or consulted are listed in the Bibliography attached to this report.

From the outset of the literature survey, it became clearly obvious that the subject of the distribution network optimisation (i.e. water distribution to the end user) has been researched more extensively in comparison to other aspects of the water supply system optimisation. The distribution network optimisation using various methods and techniques is well attended internationally as well as locally. Several computer software packages are available internationally. The most popular techniques adopted in distribution network optimisation are based on the dynamic programming method adopting, for example, a heuristic procedure for the initial diameter estimation.

It became apparent further from the literature survey that the less attended areas of research in water supply are as follows:

i) optimisation of the source storage in conjunction with the optimal water treatment capacity;
ii) water supply system optimisation/storage in conjunction with pumping capacity;
iii) optimal size of large trunk mains in conjunction with pumping capacity;
iv) optimum design of a pumping system with daily variable inflow - mainly in sewer systems;
v) implications of optimal design on the operation of supply system.

This project deals primarily with the problem of optimisation indicated in (ii) and (iii) above. The local expertise from which this project drew considerably is documented by Stallard (1980), Basson (1985), Stephenson (1989) and Loubser (1991). More specific research in optimisation of reservoir fire-demand components and the Monte Carlo reservoir storage simulation have been compiled by van Zyl and Haarhof (1995) and Nel and Haarhof (1996) respectively.

Internationally, particularly in the developed countries, the research in optimisation of water supply systems has been attended to, inter alia by Featherstone (1983), Wagner and Shamir (1987), Coulbeck (1988), Martin (1990), Jowitt and Xu (1993) and more recently Ostefeld and Shamir (1996) and Nitivattananon et al (1996). These authors used primarily linear or dynamic programming methods to solve various optimisation problems.

2.2. Field surveys in support of the project

The survey of local and international literature indicated that the water authorities responsible for water supply evolved their flow measuring methods mainly according to available technology in the flow metering market. Recording of the flow data within the water supply system is a complex task. The data has to be collected on an ongoing basis or well in advance of any physical or institutional change made to the system operation.

The need for adequate data logging within the water supply system is highlighted by the objectives to identify the weaknesses within the supply system or to determine changes in water consumption patterns. The actual logging procedures refers to the data logging of intermittent flow patterns, peak factor analysis and/or commissioning of pressure control.

The locations for the loggers are typically chosen at the storage reservoirs or pumping stations in the trunk main system. The locations of flow metering devices in the distribution networks are usually established at the fire hydrants, water meter manholes, commercial buildings or even at private residences.

All available local research relevant to urban water supply system optimisation has been studied and where possible the field data incorporated into this project. The results of the field logging research programme conducted by Turner and Manson (1995) on fourteen varied urban areas situated within the Rand Water limits of supply have been adopted into this project. The field logging programme has been extended under this project to another eight urban users to determine water demand profiles and peak demands of primarily third tier users (i.e. municipalities or mines) connected to a highly dedicated Rand Water pumped supply subsystem situated in the Far East Rand supply zone. (See Appendix A for the details on the Bloemendal/Wildebeesfontein pumping subsystem.) The programme was exercised mutually between the Rand Water and Water Systems Research Group at the University of the Witwatersrand. The Meinecke and SALM loggers were used for one week flow monitoring on delivery pipelines sized between 100 and 600 mm in diameter.

In addition to the field logging programme conducted for purposes of determining water demand characteristics within the Rand Water limits of supply, a post-mail survey of another sixteen water boards and six foremost large municipal water supply systems was carried out under this project.

The objective of this survey was to get familiar and to obtain data on the critical aspects for sizing of the urban water supply systems of smaller capacities than the Rand Water. The survey yielded inter alia information on the physical components and various policies (e.g. reservoir operation policy) of varied water supply systems with design capacities between 30 Ml/d and 3000 Ml/d.

Important information obtained from this survey is related to the composition of the water consumers within varied supply systems from around the country. Such information is considered to improve planning and design of sizing the composition of future supply areas with regard to the purpose of water supply. The details are illustrated in Table 1.

Table 2.1: Type of supply according to various water supply systems

Supply system	Capacity (Ml/day)	Sector share of water supply (%)				
		Domestic/ commercial	Industrial	Mining	Agriculture/ stock-watering	Other
1	28	58	7	-	35	-
2	70	85	15	-	-	-
3	160	75	20	5	-	-
4	250	36	64	-	-	-
5	280	80	20	-	-	-
6	480	33	14	53	-	-

The data on the relationship between design capacities and energy requirements for running the medium to large water supply systems were also obtained. The survey also yielded information on the electricity tariffs and trunk main losses for various supply systems situated in various regions of South Africa.

The gathering and collation of specific information, for example the pipeline failures, outages, refurbishment of pipelines, costing aspects for various water supply system components, were satisfied by means of questionnaires via fax surveys and several personal communication sessions were arranged at relevant information sources (e.g. SA Federation of Civil Contractors, etc). Selected relevant information is attached to this report in Appendix A.

3. WATER SUPPLY OPTIMIZATION PROBLEM STATEMENT

3.1. General

Almost without exception, most water supply systems providing either raw or purified water to the urban areas are facing the likelihood of problem related to optimal sizing of the system's specific component or pumping rate. However most water supply systems are pumping systems depending heavily on the electrical energy inputs.

With the increases of energy cost and costs of interest on capital funding for new projects even marginal savings in overall project cost by adequate optimising can allow for more people to be supplied with water.

The next major problem facing many existing urban water supply systems is demand for water supplies to the new urban developments. This process materialises either by introducing new water supply systems or by extending of the existing supply infrastructure subsequently in need of optimisation process based on the changes introduced to existing system.

The solution to the optimisation problem is in most cases motivated by the following reasoning:

(a) need for a new or additional water supply due to growth in urban, agricultural and energy water demands;

(b) subsequently there will be a need for an optimal design of new or enhanced water supply system;

(c) consequently there will be a need for the optimum timing and sizing of water supply system components.

3.2. Economic design of a new water supply system

The design of a new water supply system refers usually to two situations:

i) independent new water supply system (i.e. most common in the developing countries);

ii) adjacent new water supply system to already existing system.

The planner and designer, particularly in the developing country, face in most cases the constraints with regard to initially installed capacity of the whole or particular components of supply system. The future demands for water are highly uncertain and usually not more than speculation for anticipated 30 to 50 years life-span of designed supply system. The stage development is usually considered to avoid an initial large surplus of installed capacity. Designer's options are:

(a) a few large increments in capacity (advantage of economy of scale);

(b) many small ones (less initial capital outlay).

The decisions are to be based on a comparison of the economy scale against the economy of finance (i.e. unit size for minimum combined cost of construction and finance or unit size for minimum capital cost). The final design might not be in some cases influenced by optimal criteria only, if the security and reliability of the supply system are more relevant aspect (e.g. redundancy of pipelines, minimum volumes in balancing reservoirs, stand-by equipment, etc.).

Traditionally, the optimal design of a new water supply system addresses the water supply optimisation process in two not mutually exclusive ways. The first refers to minimum total capital investments of the new system. The second is to minimise the annual operation and maintenance costs. If a more advanced approach is used, the optimisation process also includes minimum risk and maximum reliability analysis.

3.3. Enhancement of existing water supply system

The enhancement design of existing supply system addresses usually the upgrading of the system's operational or performance deficiencies. The optimisation through enhancement design can include:

i) replacement of existing pipeline or parallel laying of new pipelines;

ii) cross-connections between existing and/or new portions of supply system;

iii) construction of new reservoirs or enlargement of existing reservoirs;

iv) upgrading of existing pumping stations and/or addition of new pumping or booster station.

Regardless of what method or technique is to be used, the designer faces major uncertainties with regard to the consumer demand patterns and geometrical as well as physical data of existing infrastructure, the parameters to be investigated are commonly as follows:

i) peak demands due to changes within the limits of supply;

ii) actual lengths of pipelines;

iii) effective diameters and pipe roughness;

iv) valve settings;

v) leakage rates due to pipe breakage's or pump outages.

The conventional methods of design limit the designer in determining fast results for various scenarios. If the computer model is used in a proposed design, various alternatives can be rapidly tested. Such a design requires that the synchronised measurements of pressures and flow rates together with reservoir levels. The field work should ideally be done at various places of existing supply system. The information on the leakage's and probability of outages to occur in enhanced supply system are essential for the designer in well evaluated improvements, extensions or upgrading for a given supply system.

The enhancement of existing water supply systems also refers to the revision of adopted guidelines in policies for sizing of various components of the water supply. The most relevant examples are the typical urban water supply guidelines for sizing storage reservoirs and trunk mains.

3.4. Constraints limiting optimal design

The designer's limitations for optimal design of a water supply system can be summarised as follows:

i) shortage of capital available for development;

ii) minimum and maximum pressures at critical points of the supply system;

iii) minimum and maximum velocities in the pipelines;

iv) commercial sizes and operation ranges of various components (e.g. pipes, valves, pumps, etc.);

v) inadequate design guidelines;

vi) risk and reliability requirements;

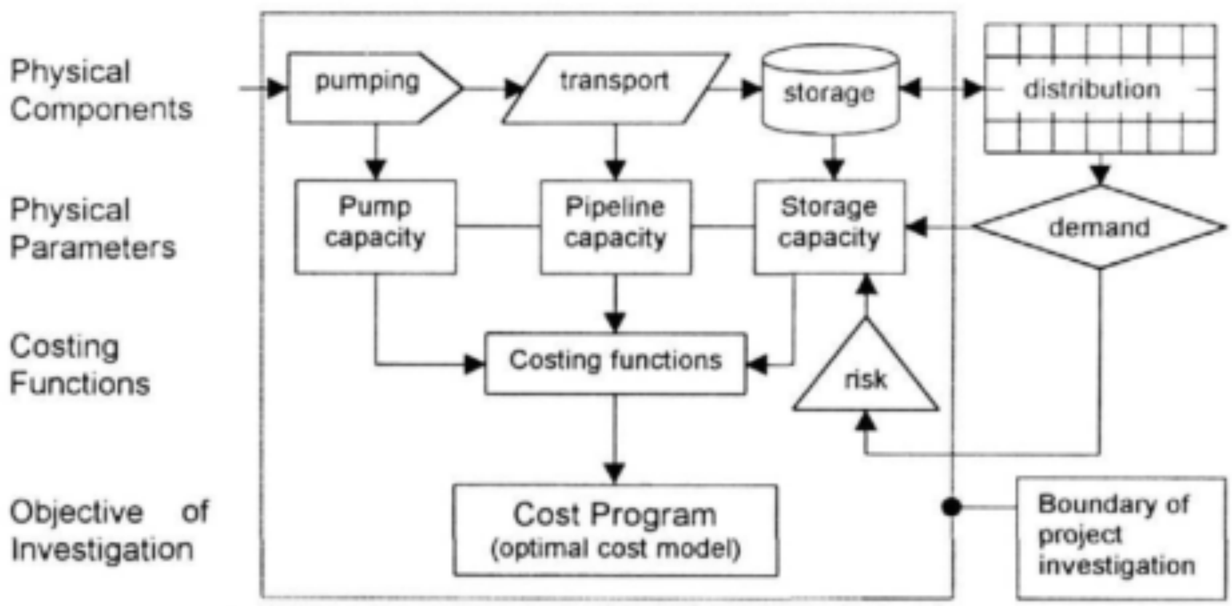
vii) permitted values of water quality parameters;

viii) protection of existing infrastructure and natural features.

The designer's limitations primarily dealt with in this project are indicated in paragraphs (iii) to (v) above and to a lesser extent in paragraph (vi).

3.5. Optimisation problem statement for this project

The conceptual problem statement for this project is illustrated below:



As illustrated above, this research project investigates primarily the water supply system comprising the pump station, pumped trunk main and storage reservoir. Such a supply system is dependent on the water demands generated by various water users situated within existing distribution networks. The combined demand profiles are used in the optimisation process for designing the new or extended supply system. Essentially three major water supply components are optimised to determine the global optimum capital cost of a new supply system. The investment appraisal method used in this investigation is based on life-cycle costing by means of the present value payment discounting technique. The costing function values for the pump costs, pumping operation, pipeline and storage costs are essential to this optimisation model. The major product of this investigation is the capital cost optimisation program software useful for the purposes of the conceptual design of a pumped water supply system.

4. DESIGN DEMAND FOR LARGE WATER SUPPLY SYSTEM

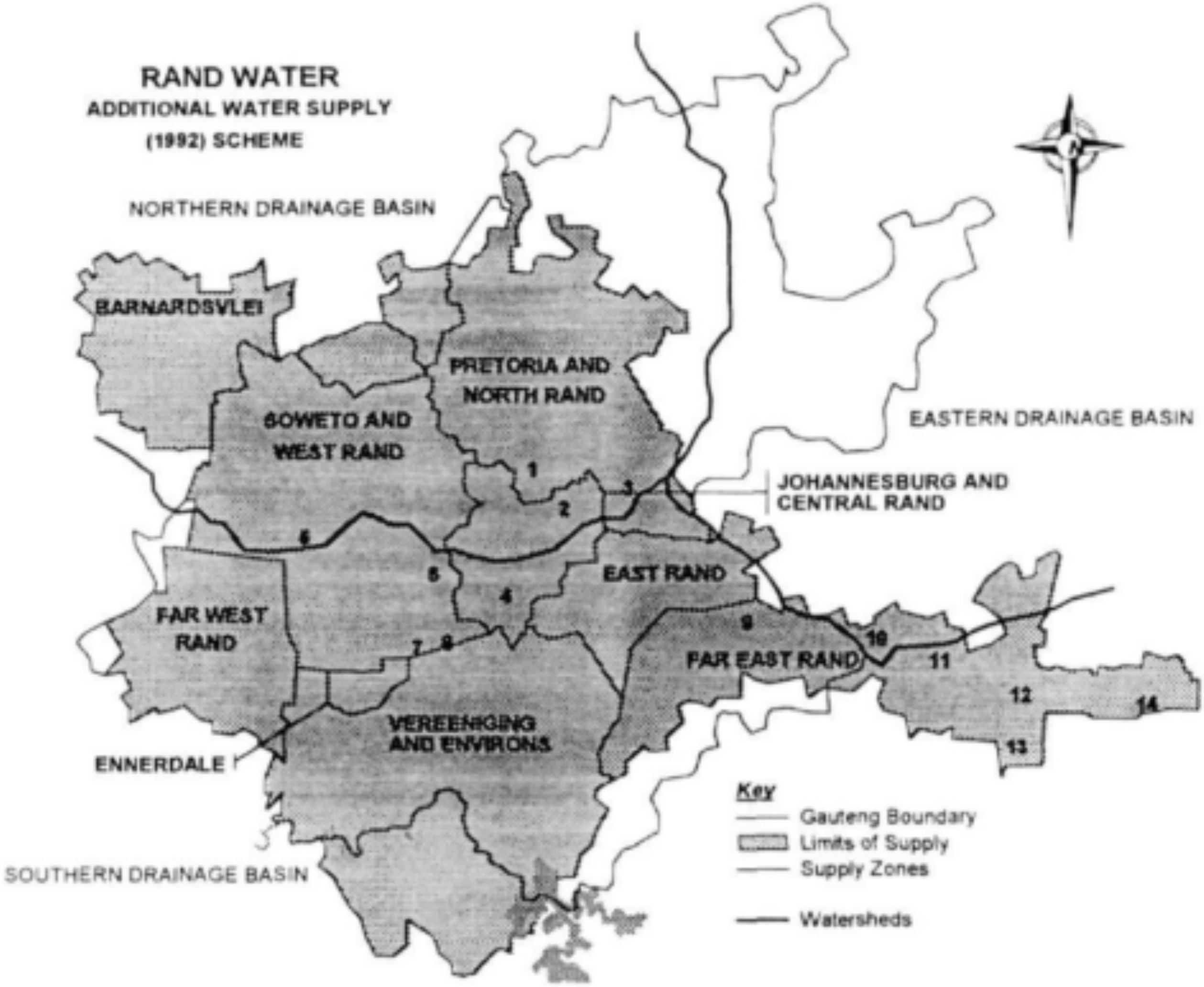
4.1. Monitoring of demand data for optimisation input

The critical operation period of a water supply system can be well observed from the hourly, daily or weekly demand profiles. This period also corresponds with the minimum system's pressure. It appears from extensive monitoring of consumption patterns in the South African urban areas situated in the large supply systems that critical period for water supply is typically between 06h00 and 20h00. The type of water user or urban area are important indicators of the demand pattern and the critical operation period.

The research conducted by Turner and Manson (1995) under a programme funded by the Water Research Commission on the subject of urban water consumption patterns, generated information from fourteen experimental urban areas situated with the Rand Water limits of supply.

Since then the logging and data gathering programme has been extended by the Water Systems Research Group at the University of the Witwatersrand under this project to another eight urban users within a highly dedicated Rand Water's pumped supply subsystem with storage and the demand profiles being obtained from predominantly urban and industrial users. The users monitored were primarily the third-tier water supply authorities (i.e. municipalities) or individual large mines.

The sequences of data logging of intermittent flow patterns in the supply areas as indicated in Figure 4.1 endorsed with a high degree of certainty that water consumption patterns vary significantly from one urban location to another. The research results indicate that densely populated, low income areas have a lower per capita consumption that high income areas, which often have low population densities. Further to these findings the effect of levelling out the consumption per unit area to about 10kl/ha/day, with a maximum of about 20kl/ha/day in high density, high income areas such as cluster complexes, fall into this category.



Reference:

1.	Sunninghill	8.	Aeroton
2.	Alexandra	9.	Bloemendal
3.	Rabie Ridge	10.	Devon
4.	Johannesburg CBD	11.	Leandra
5.	Azaadville	12.	Evander
6.	Witpoortjie	13.	Secunda
7.	Noordgesig	14.	Bethal

Figure 4.1: Monitored areas within Rand Water limits of supply

The demand patterns obtained from the field monitoring of the urban water meters sites by flow data loggers were downloaded onto the PC database and analysed for the critical operation period, demand profile and peak demand factors. Figure 4.2 illustrates combined week and week-end demand profiles of predominantly industrial area situated in the Rand Water supply area.

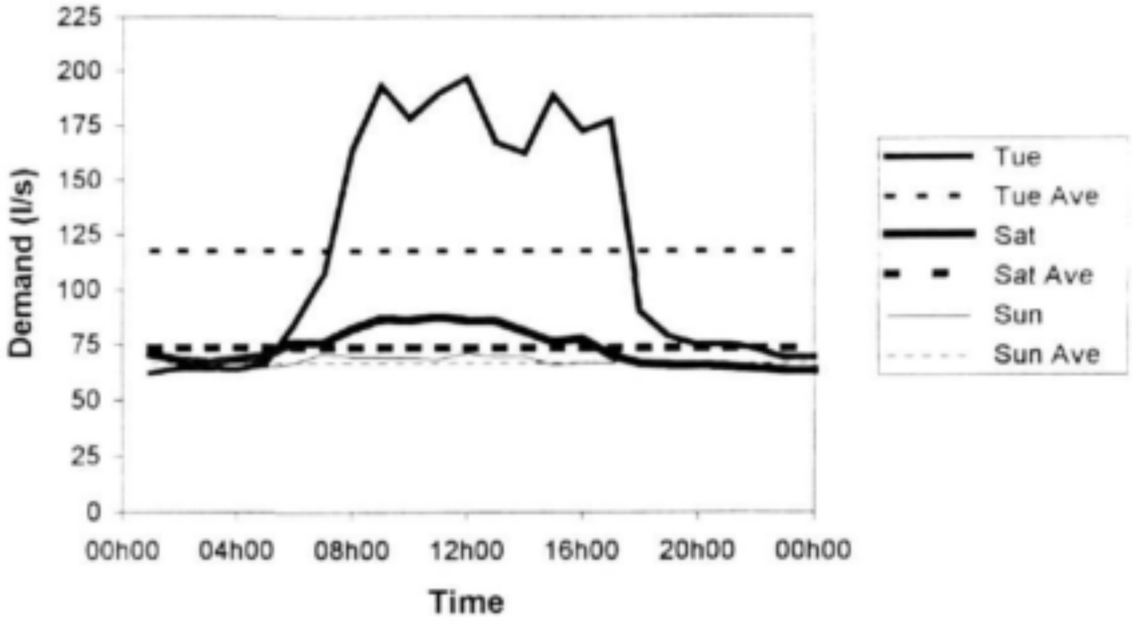


Figure 4.2: Example of demand for industrial area situated in a pumped system with storage

The typical demand profiles for a gravity and pumped system with storage are illustrated in Figure 4.3. The gravity supply system with an average flow of 40 l/sec represents the medium density distribution area. The pumped supply system represents a much more complex supply system. The profile demonstrates the effect of recorded level changes in the service reservoirs. Both demand profiles indicate the peaks within the supply system. However the pumped system profile has peaks rather concealed.

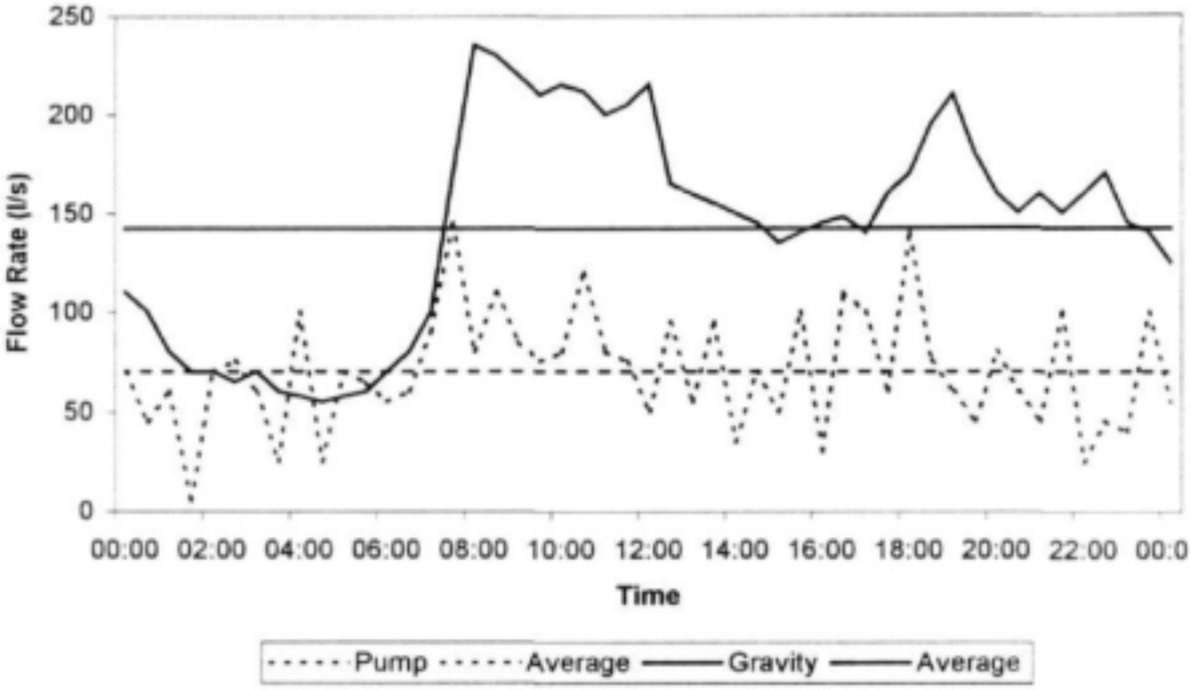


Figure 4.3: Typical demand profiles for gravity and pumped supply system

Note that the demand profile for the pumped supply system indicates a highly irregular pumping system. The bulk suppliers (e.g. Rand Water) are experiencing serious operational setbacks with some third-tier consumers drawing water from their service reservoirs under highly irregular demand.

4.2. Normalised demand profiles and peak demands

The demand patterns obtained from the field monitoring are normalised by dividing the flows by the mean flow for the area. To determine the variation in demand, each water use category is allocated a normalised (i.e. average value = 1) demand profile. This allows the patterns to be compared, without concern for the overall magnitude of the demand. The typical normalised demand patterns can be combined to produce the urban demand pattern graphs suitable for any given supply area combination. Once the demand profiles for a particular area has been decided upon the optimum balancing storage requirements can be determined and the system optimised. The typical weekly normalised demand profile for a low cost housing area situated in the Rand Water supply area is shown in Fig 4.4.

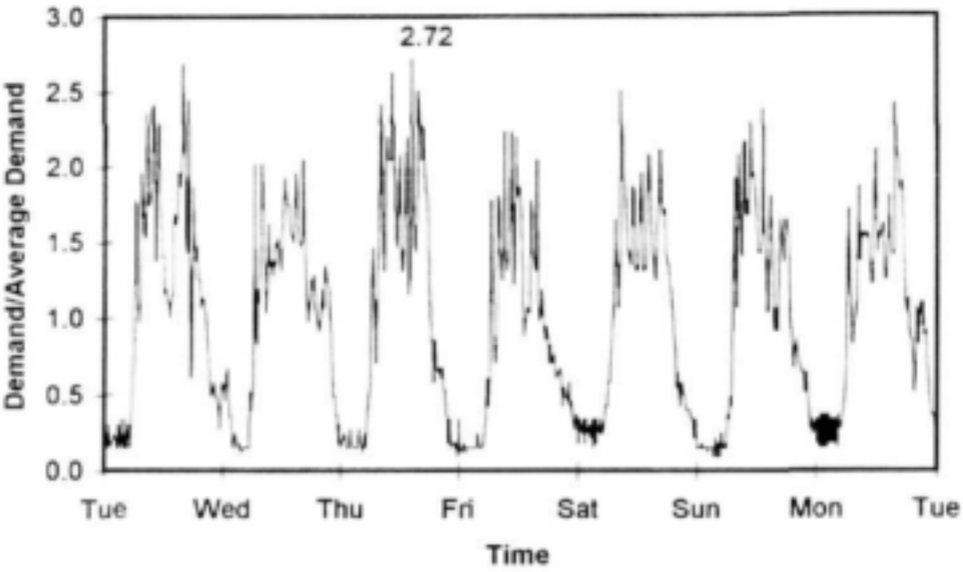


Figure 4.4: Typical weekly normalised demand profile for a low cost housing area

The peak factor analysis of data logging results, particularly from the Bloemendal/Wildebeestfontein trunk link on the Far East Rand, correlated well with the information available on peak factors from the same urban region. Table 4.1 illustrates the peak factor range for various density urban areas.

Table 4.1: Peak factors for various urban areas situated within the large supply system

Urban area	Average demand (l/s)	Peak month factor	Peak week factor	Peak day factor	Peak hour factor
Low density residential	<10 10-50 50-200 >200	1.35 1.30 1.25 1.20	1.70 1.65 1.60 1.50	2.30 2.20 2.00 1.80	5.50 4.50 3.90 3.30
Medium density residential	<10 10-50 50-200 >200	1.30 1.25 1.20 1.15	1.60 1.55 1.45 1.40	2.30 2.00 1.80 1.70	4.60 4.00 3.30 2.90
Industrial/ commercial	<10 10-50 50-200 >200	1.25 1.20 1.15 1.15	1.50 1.45 1.40 1.40	2.00 1.80 1.75 1.70	3.40 3.00 2.80 2.60

N.B.: After Vorster *et al.* (1995) Report to the East Rand Services Council

Most useful information from the field monitoring programme is usually the 24-hours demand profile from which the period of peak demand corresponding to minimum system pressure may be determined. It is important to realise that as demand patterns may differ very significantly from supply area to supply area, the peak factors vary also considerably.

4.3. Design demand for supply system optimisation

For the purposes of appropriate input into the cost optimisation program, the weekly demand profiles obtained from research monitoring of experimental supply areas within the Rand Water limits of supply were categorised into residential, commercial and industrial water use categories. From the data available, the demand patterns can be derived for any specific day of the week and type of water use category. The different daily demands quite explicitly indicate working or social habits within monitored supply areas.

The instantaneous (hourly), daily and monthly peak demands can be determined from the monitored supply flows as the product of the Annual Average Daily Demand (AADD). The corresponding peak factor is a function of the number of consumers residing within a specific supply area (e.g. low or high income) as shown in Table 4.2. The design demand for a specific supply area is then determined as follows:

Supply area design demand (kl/d) =

Residential (kl/stand/d) × Number of Stands +

commercial (kl/ha/d) × Number of hectares +

industrial component (kl/ha/d) × Number of hectares;

In the real-life setting, a large portion of such information would be derived most likely from the municipal structure development plan or determined by the Town Planning Department. In many instances, the original proposals have to be regularly revised to accommodate changes in demand for water. Also the supply zones for distribution of water have to be determined to address demand sizes.

To convert the supply area or zone components, the reticulation design standards can be used, as for example the "Blue Book" (Guidelines for the Provision of Township Services in Residential Townships by the CSIR) or the "Red Book" (Guidelines for the Provision of Engineering Services and Amenities in Residential Township Development also by the CSIR) can be used. The common mean values used in the Rand Water limits of supply are indicated in Table 4.2.

Table 4.2: Design mean values according to various land use categories

	Mean value	
Land use	(kl/ha/day)	(l/cap/day)
Residential:		
(i) low density	8	340
(ii) medium density	11	85
(iii) high density	20	
Commercial	8	-
Industrial	20	-

Alternatively the water demand for the area to be supplied can be determined according to the method and parameters below:

DEMAND = RES + IND + REC

Residential use (RES) is largely a function of population and standard of living. The standard of living (size of stand, use of appliances, etc.) is reflected in the per capita consumption, but is also related to the affordability factor in disadvantaged areas.

RES = POP * PCU * AFD * (SER + COM) (l/d)

where POP = population residing in urban area,

PCU = per capita use (l/c/d),

AFD = affordability coefficient,

SER = service coefficient,

COM = commercial component.

PCU ranges from as low as 25l/c/d to as high as 350l/c/d. AFD is related to standard of living and can range from 0.3 to 7.5 in direct relation to PCU. SER is unity for fully serviced areas and ranges between 0 and 1 for partially serviced areas. COM is typically in the range 0.1 to 0.2

Industrial use (IND) is determined by the area covered by the various industries and the intensity of usage, expressed as a rate per unit area. The proposed equation for industrial use IND is thus very simple as shown below:

IND = IAR * IWU (kl/d)

where IAR = industrial area (ha),

IWU = industrial use coefficient (kl/ha/d).

IWU can have a very wide range, depending on the type of industry (e.g. wet and dry industries). IWU ranges from as low as 7 to as high as 540kl/ha/d.

Recreational use (REC) is of similar form to that for industrial use, i.e. it is based on an area multiplied by a usage intensity. The equation is as follows:

REC = RAR * RIQ (kl/d)

where: RAR = recreation area (ha),

RIQ = (irrigation) water applied (kl/ha/d).

RIQ is strongly related to climate, as is any irrigation-type requirement. For South Africa, RIQ ranges between 10kl/ha/d for the humid east coast to as high as 60 kl/ha/d for the arid north western interior.

4.4. Forecasting water supply system demand

The key problem affecting the development of water supply systems is forecasting of consumers' demands ahead in time. The long-term demand forecasts (10 to 30 years ahead) are usually estimated by means of population and per capita water demand growth rates. The short-term (i.e. hourly, daily, weekly or monthly) forecasts of water demand for the purposes of water supply optimisation (e.g. minimum cost pumping) are commonly based on the statistical parameters of historic data. Two components of demand are usually important to observe.

(a) deterministic demand component - indicating commonly the regular repetitive trends (e.g. long-term trends, seasonal trends or habitual trends).

(b) stochastic demand components are random events which are difficult to predict in their recurrence and magnitude (e.g. fire, pipe breakage's and outages).

The deterministic component of predicted water demand can be determined from the historic data by determining of relevant averages (i.e. factors).

$$\text{Demand} = (HF \times DF \times MF) \times AADD + RC \times AADD$$

where HF = hourly factor,

DF = daily factor,

MF = monthly factor,

RC = random component,

AADD = annual average daily demand.

Various models were developed internationally to adopt this concept onto microcomputers (e.g. ARIMA model, etc). Locally, Nel and Haarhof (1996) adopted this approach in sizing of municipal water storage tanks by Monte Carlo simulation on a 286 PC and 486 PC.

4.4.1. Short term demand forecasting (STDF)

The short term demands correspond to hourly defined consumption patterns over 24 hours or 7 days periods.

The short term demand forecasting for a water supply and distribution system has two major purposes:

(a) system design purposes providing for a range of short term demand patterns

(b) system operational control purposes, providing for prediction of expected demand patterns for each succeeding control period.

When adequate short term demand forecasts are determined, the design or control objectives will include the least cost operation for a given system (or sub-system). In the case of a pumping water supply system, the electricity consumption can be seriously reduced and considerable energy cost savings realised (over 10 per cent in most cases).

The prerequisite in short term demand forecasting is to determine the specific quantifiable features of typical demands for a specific geographical area (or system zone) for which demands are to be monitored and forecasted.

The total short term demand is made up of three major components as follows:

(a) metered or unmetered small user consumption (e.g. small communities, small holdings, etc.)

(b) metered supplies of large municipal, industrial, agricultural, recreational or other users consumption.

(c) metered or unmetered unaccounted consumption (i.e. wastage's on account of leakage's, breakage's, etc.)

It must be noted that the combined effect of large numbers of consumers formulates the overall demand profiles with certain well defined features as illustrated in Figs. 4.2 and 4.4.

The forecasting of short term demands for design and operational control of a water supply system typically involves the time-series methods with coefficients derived from analysis of historic data, superimposed over the study area supplied by existing or new supply system. This approach is known as exponential smoothing. In principle, the trends and error corrections are incorporated resulting in forecasted demand patterns reflecting the smoothed history of the previous period.

4.4.2. Small supply system demand forecasting

For planning purposes of a small water supply system, the interest is focused on the relationship between the population's demand for water and the socio-economic characteristics of domestic water use with regard to the attributes of the surrounding environment (i.e. temperatures, rainfall, etc.). Unfortunately domestic water demand cannot be directly linked to a single determining factor.

The method used in determining future water demand in small communities with a micro-level population is known as micro-simulation. This method involves the estimation of micro-level data using chain conditional probabilities.

Example 4.1: Modelling of supply system water demand

Urban water demand may be defined as the use of water by a developed or developing urban area supplied from schemes sustained by revenue from water sales. A suitable model consists of four main components as follows:

URB = TWL + DWL + RES + IND

URB is the total water demand for the urban area (i.e. town or city).

TWL is the loss incurred in the bulk water supply to the urban area. This includes losses at water purification works die to, for example, backwashing of sand filters. TWL represents a small portion of total use, ranging typically between 3% and 7% of URB.

DWL is the distribution loss incurred in supplying water to individual consumers. It includes real losses (i.e. leaking pipes and reservoirs) and unaccounted for water resulting from meter inaccuracies. DWL is a significant proportion of total use, ranging from 10% to 30% of URB in towns where lack of maintenance has become a problem.

RES is the residential usage, which includes public, commercial and recreational water use. This is nearly always the largest component of urban use, ranging typically from 50% to 70% of URB. RES is divided into two main components, RID and ROD. RID is the indoor component, which can vary from about 30% of RES in developed areas to as much as 90% of RES in developing urban areas. ROD is the outdoor component, ranging from 10% of RES in developing areas to 70% in high income developed areas.

IND is the water used by industries supplied by the local authority (e.g. municipality) and ranges typically from about 5% to 2% of URB.

5. WATER SUPPLY TRUNK MAIN OPTIMIZATION

5.1. Pipeline optimisation problem

The construction and maintenance costs of supply system main pipelines (commonly of large diameter up to 2400mm) represent a significant portion of the development capital for water supply.

The problem of pipeline main optimisation refers to the relationship between pipeline diameter, flow velocities, head losses and pipeline costs. Selecting the optimum pipeline diameter, reservoir storage volume and pumping rate can minimise the overall costs of water pumping supply. According to Stephenson (1995) a pumping main diameter should be selected by minimising the total capital plus operating costs brought to a common time basis. The relationship between the cost of pipe diameter and operating (i.e. power) cost is shown in Figure 5.1.

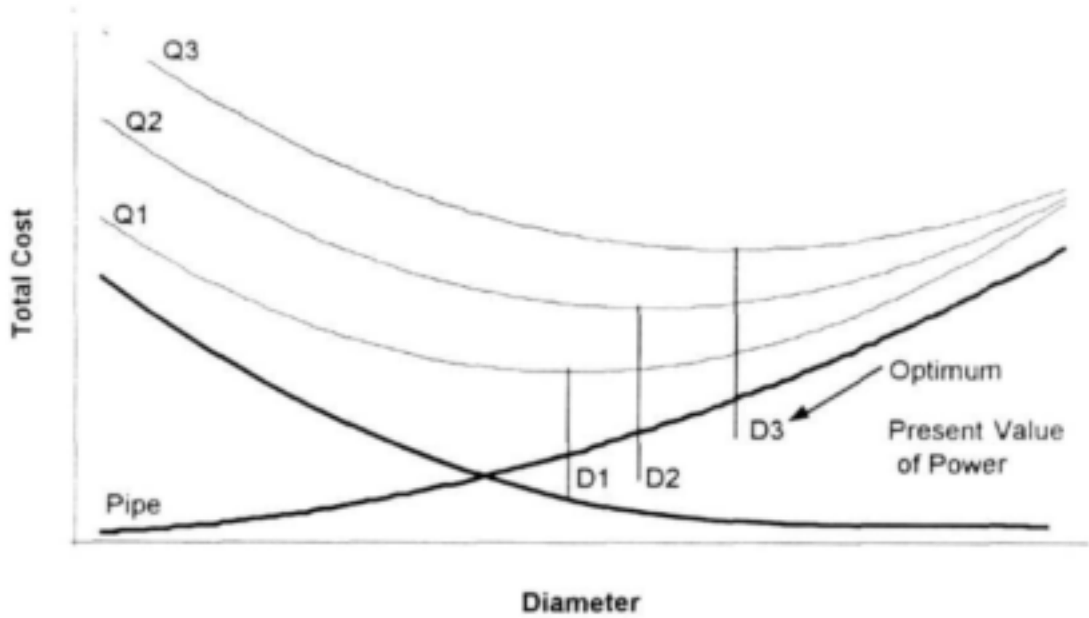


Figure 5.1: Cost optimisation of diameter vs. power

5.2. Alternative mains in water supply systems

5.2.1. Rising and gravity mains in water supply

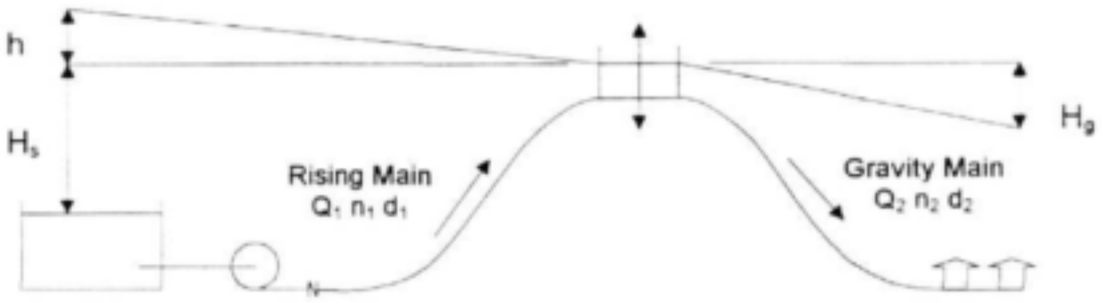


Figure 5.2: Rising and gravity mains

The flow velocity at optimum pumping main diameter varies from 0,6 to 2,0 metres per second, depending on flow and working pressure. The range is as follows:

i) about 1m/s for low pressure heads and a flow of 100 l/s;

ii) up to 2m/s for pressure heads to 400m and a flow of 1000 l/s

By doubling the diameter of the pipeline and other factors being constant (i.e. pumping heads) the pipeline capacity increases nearly six-fold (Lotz *et al*, 1993).

The water supply authorities are finding it economic to size the large pipelines for a service life of between 10 and 30 years. A longer life-span can be envisaged normally for small bore pipe diameters and low working pressures.

5.2.2. Siting of rising main storage

Alternatively the pumping main in conjunction with balancing storage can involve placing a storage to provide for appropriate residential pressures to satisfy the consumer demands on the supply system. Such a design approach is usually required in flat supply areas or generally in the featureless terrain. Fig 5.3 illustrates the concept of approach to design.

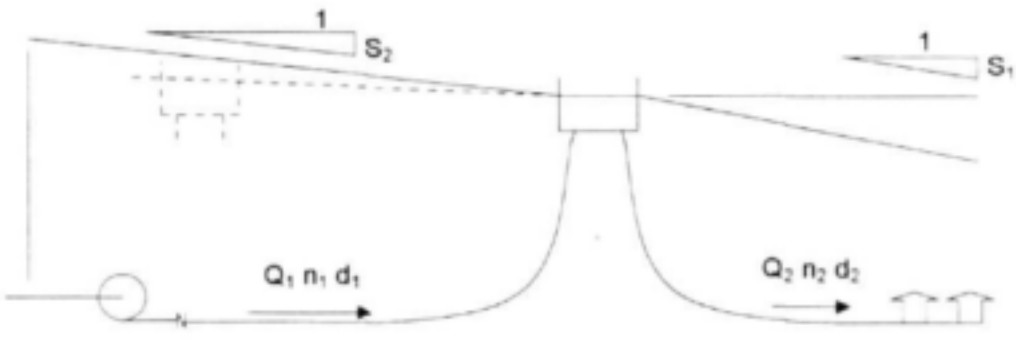


Figure 5.3: Siting problem of intermediate storage

The booster pump stations are usually installed at intermediate points with an objective either to increase or to reduce pressure along the trunk main route. If the objective is to increase flow pressures, the siting of the balancing storage will be very important in the trunk main size optimisation. The operating costs must be carefully optimised.

5.2.3. Gravity main with booster station.

This approach to the design of trunk main is required if the conditions of supply have to satisfy peak flows or quite often because residual pressures have become inadequate.

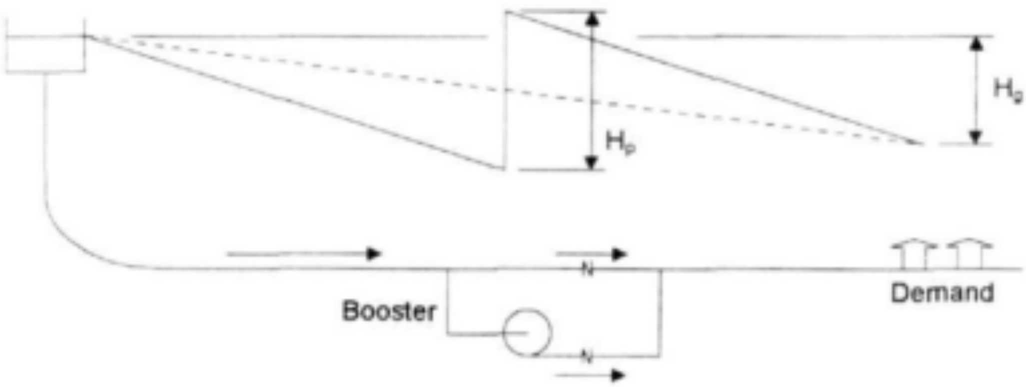


Figure 5.4: Gravity trunk main with booster station

It is assumed that boosting is used to provide the higher flow and gravity flow satisfies lower consumer demands, the concept of optimum choice of trunk main line size can be used to develop rules for its selection. However, the pipeline size may also be determined by a gravity flow rate.

5.2.4. Rising main with booster station

There might be also a need for a booster station to be installed at the intermediate location instead of pumping to a high pressure head at the input location and maintaining a high pressure along the entire route. Such an approach to the problem solution can show that the pressures to be reduced and the pipeline wall thickness may be minimised. The cost of a booster station has to be competitive to the gains from cost reduction on the trunk main pipeline. The consumer demands prediction may be essential to this problem solution as the booster station might not be required at the initial stage of the project development. It should be noted that the energy requirement costs increase steeply as the pipeline is reduced in diameter.

5.3. Trunk main pipe optimisation

The initial costs related to the pipe system are far greater in proportion to the operational and maintenance costs, so therefore the design stage has the greatest impact on the ultimate project cost.

The cost of replacing an existing pipe compared to pipe maintenance costs are greater, stressing the importance of correct design assumptions and specifications. Furthermore, the pipe size is a function of the operational costs of the system because it has a lot to do with influencing the pumping cost.

A range of pipe diameters will produce different friction heads for a particular pumping rate. The cost of pumping is directly related to the internal pipe friction, making the pipe diameter an important component in determining the pumping cost. The Darcy formula for friction is used in determining the optimal diameter. However the pipe manufacturer usually provides the Darcy headloss coefficients.

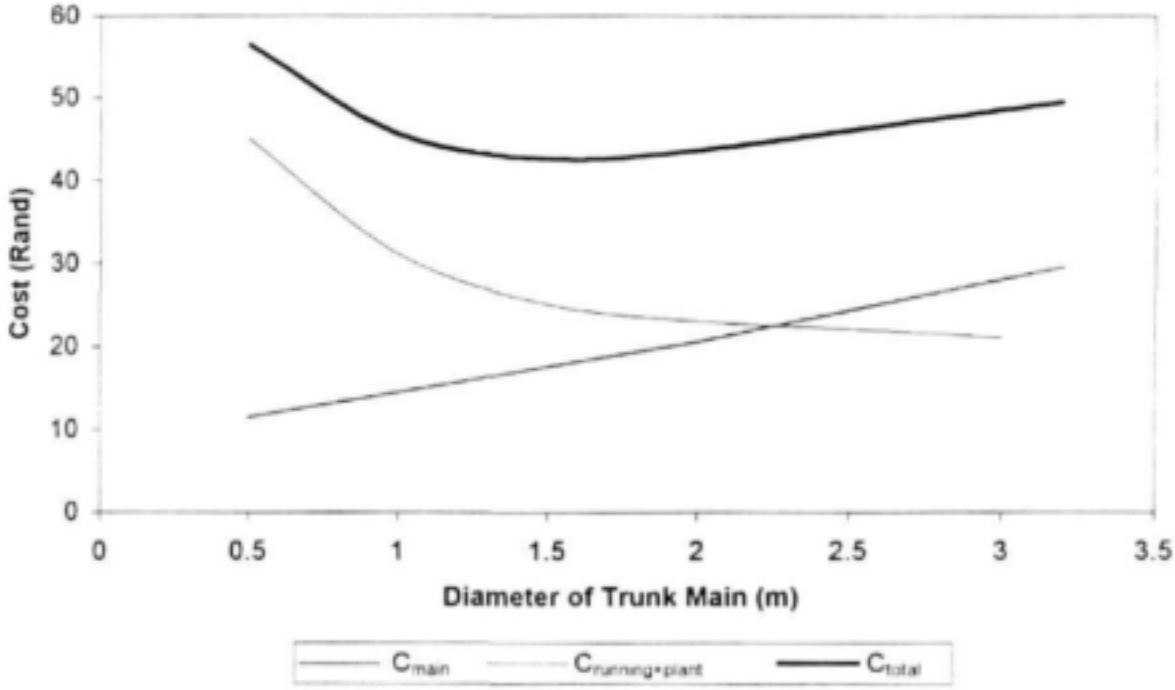


Figure 5.5: Trunk main pipe diameter optimisation

In Figure 5.5, the sum C_{total} is total annual cost of three annual costs presenting cost of a pumping trunk main (C_{main}) and pumping installation together with the power costs ($C_{running + plant}$).

For each pipe diameter there will be a corresponding cost of installation. The cost of construction works per meter are approximately the same for different diameters, the increase in costs relating to the cost per metre diameter of pipe.

5.4. Method of pipeline optimisation in water supply

The method of optimising a pipe diameter for a particular pumping rate and operating factor involves using a set of predetermined velocities which include the probable optimum or most efficient pipe velocity.

The characteristics for the pipeline such as the internal friction coefficient and the cost per metre are used in the calculation of the cost of pumping installation. The cost of pumping is calculated using the pump operating factor, the cost per unit of electricity and the energy lost in overcoming the pipe friction.

To obtain an optimum pipe diameter for the trunk main the diameter that costs the least to install and pump water through must be found. Once the most effective diameter has been found the pipe system has been optimised.

Example 5.1: **Minimum cost of pumping scheme**

The pump station is to elevate water under pumping rate of $Q(m^3/s)$ into the balancing reservoir at total pumping head of H(m). The length of delivery pipeline (i.e. pumping main) is L(m). Determine parameters as follows:

i) friction losses Δh (m) in the pipeline, the intake and outlet.

ii) Relationship between the total head and various pipeline diameters.

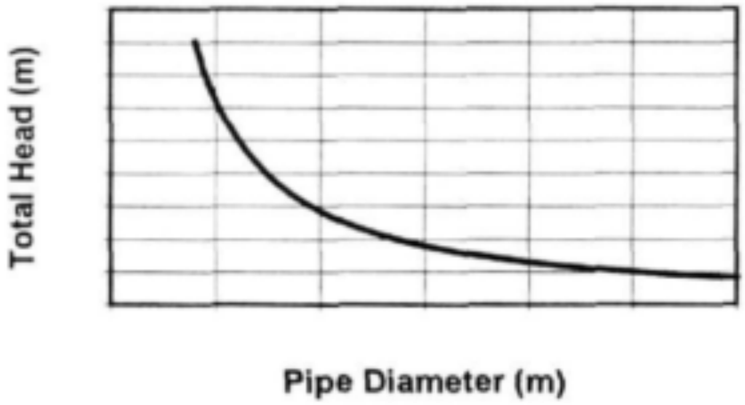


Figure 5.6: Relationship between diameter of pipe and total pumping head

iii) Relationship between the pipe diameter and pressure from known pipe diameter and applied pressures the thickness of pipe wall can be determined and estimates of the pipeline capital and annual cost can be determined.

$$AC(R) = \frac{TC * Rate * (1 + Rate)^{period}}{1 + Rate)^{period} - 1}$$

where: AC is the annual payment required to repay the loan used to construct the pipeline,

TC is the total capital of the loan obtained,

Rate is the difference between the interest rate and the inflation rate,

Period is the number of years over which the loan will be repaid.

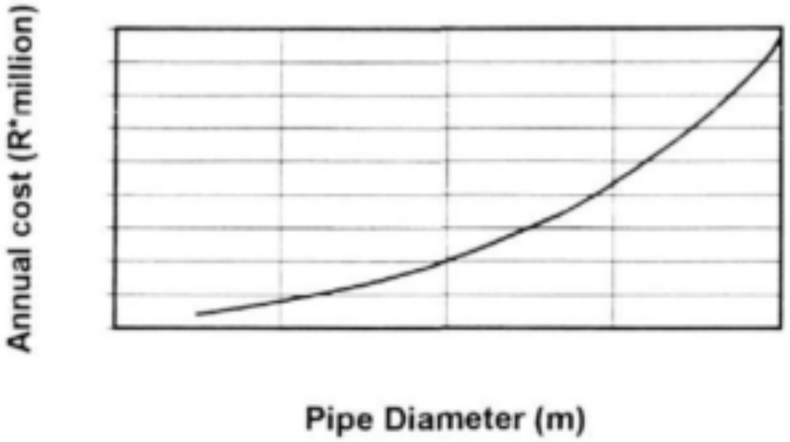


Figure 5.7: Relationship between diameter of pipe and annual cost of pipeline

iv) Cost of pumping and cost of energy.

(a) input power to motor (kW) =

$$\frac{9{,}8 \times \text{pumping rate} \times \text{total head}}{\text{combined efficiency of pump and motor}}$$

(b) energy consumption per day (kWh) =

input power to motor x 24 x load factor

Load factor is the proportion of time run by the pump (e.g. if 12 h per day load factor = 0,5).

(c) present value of energy over x-years =

energy consumption per day (kWh) x 365 x cost of electricity (R/kWh) x discount factor over x-years (i.e. present value of R1 per annum over x-years).

v) Finally determine the relationship between the pipe diameter and total cost of pumping with subsequent minimum cost of a pumping scheme.

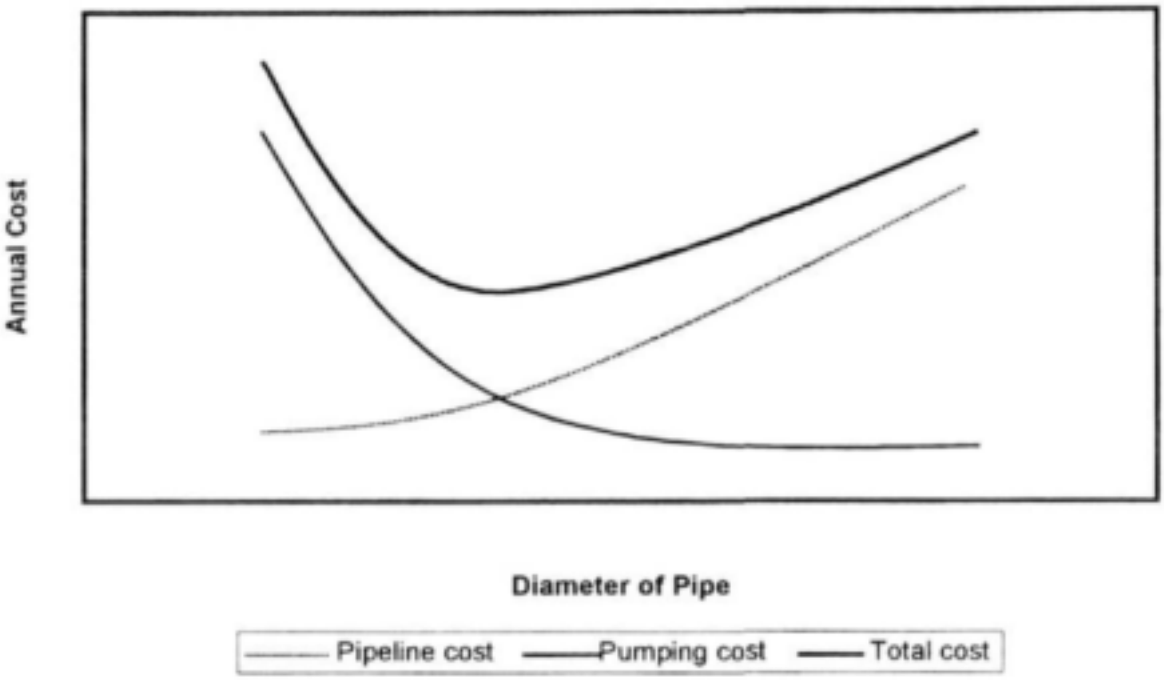


Figure 5.8: Minimum cost of a pumping scheme

It must be noted that this exercise should be repeated for all commercially available pipe sizes, for possible fluctuations in the reservoir levels, standby pumping capacity for all losses in the pipeline system, and other details that may have some influence on the change of the minimum total cost (i.e. optimum point). Principally the cost of water supply pipeline is governed by design constraints (e.g. capital available, etc.) and is dependent on the volume of water to be transferred and the distance and differences in elevation between the source and the destination.

The majority of the trunk main systems are designed to satisfy and regulate major demand fluctuations (i.e. average and peak demands) generated within a supply area. The initial supply regime for a trunk main is essentially determined with a provision for future likely developments within the designated supply area. Note that for a geographically large area, considerable energy is used in overcoming the natural topography.

A pumping trunk main conveys usually raw or potable water from a source (e.g. river or storage reservoir) to a distribution reservoir. The size of a pumping main pipe is determined with regard to the total capital loan charges and cost of power consumption against friction in the pipe and all fittings at a minimum cost. Stephenson (1989) illustrates the optimum pumping main diameter for a particular set of conditions in Figure 5.9.

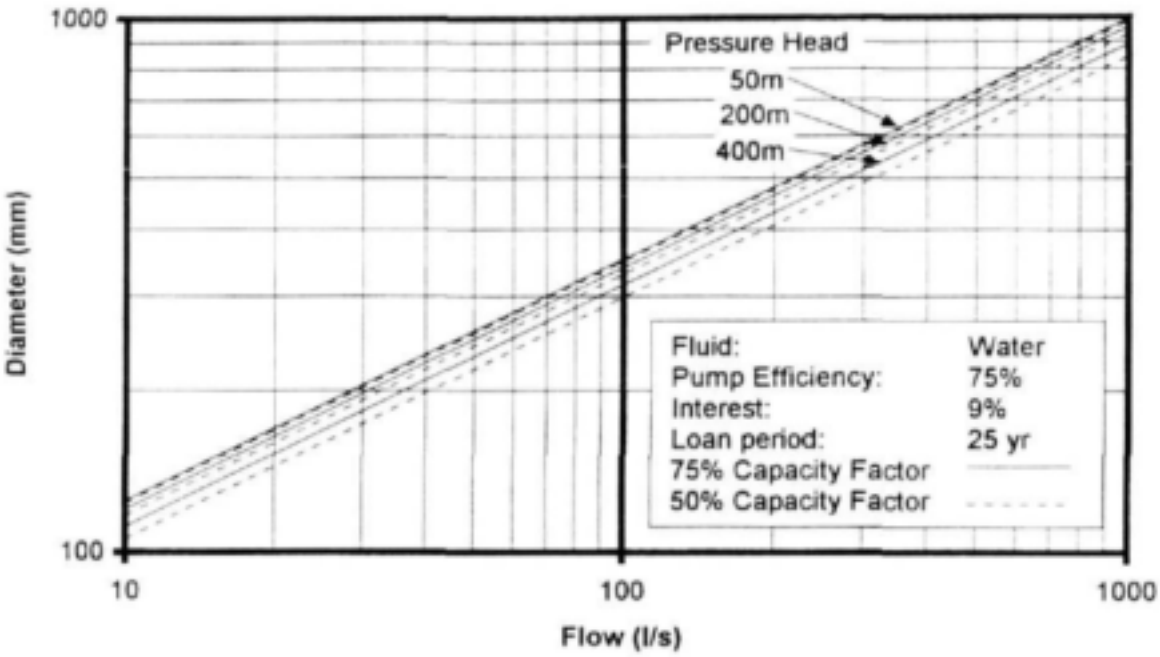


Figure 5.9: Optimum pumping main size for a particular set of conditions. (after Stephenson, 1989)

6. PUMPING OPERATION IN WATER SUPPLY

6.1. Purpose and type of pumping operation

The main purpose of the pumping operation is to transfer energy to the through-flow of water by effecting a step-increase in head or pressure. In water supply applications, pumps are typically running continuously and in most cases the required step-increase in pumping head is substantial.

In wastewater applications pumps are typically running in a start/stop mode and the required wastewater lift is usually not significant, the pumping operation being typically used to augment gravity flow systems.

The most common types of pumping operations are as follows:

i) Mine dewatering, usually without housing.

ii) Sewage pumping where deep pump stations are often required.

iii) Raw water schemes where the pumps draw water from dams. In this case pump stations can be in an intake tower in the dam lake, or incorporated in the dam wall or in a separate building below the dam wall.

iv) River pump stations where water is drawn from an intake in a river.

v) Purified water pumping stations where water has often to be pumped against a varying head, for example in pressure booster stations.

vi) Pumped storage schemes where the pumps often also act as turbines.

vii) Stormwater and some activated sludge pumping stations where large quantities have to be pumped against low heads.

viii) Oil and oil products pumping where the pumps are frequently in the open.

ix) Boiler feed and hot water circulating pumps in power stations where the pumps are subject to sever thermal shocks.

x) Slurry substances primarily for mining purposes.

6.2. Pumping Operation Selection

The most advanced approach to the selection of pumping installation should be based not only on the parameters of pumping head and quantity of liquid to be pumped, but on more broader assessments of intermediate or transient conditions of the whole pumping system.

An advanced method in pump installation selection is called the total integrated approach to pump selection. Such an approach requires access to sophisticated financial packages, production control software and computer aided design software. To date, most pumping installations are selected manually from catalogues based on very little information available.

A typical "user friendly" program for the designer of pumping installation is the AQUATEC program available in South Africa from the University of the Witwatersrand. The program is written for an IBM compatible PC with

DOS version 3.1 or higher, 640 KB RAM and a hard drive. The program will interface with a monochrome or colour monitor.

The input information on the technical parameters to the program are as follows:

- pumping system description: static head, suction sump, delivery piping (route and materials), valves and fittings, site altitude, flow rate and number of pumping units.
- fluid description, temperature, solid content, vapour pressure and viscosity.
- pump performance curves (H/Q, efficiency and NPSH).
- maximum allowable impeller diameter.
- maximum allowable speed.
- standard pipe and fitting sizes.
- friction factors for pipes and fittings in different materials.
- temperature/viscosity curves.
- pump and motor price lists.

The output information the user can expect to be selecting refers to the following:

- The pumping system losses and total head.
- The number of pumps in series or parallel.
- A choice of pump model, size and speed.
- The pump efficiency and power.
- $NPSH_A$ versus $NPSH_R$ (i.e. A indicates Net Positive Suction Head available and R indicates the NPSH required, usually lowered by a safety margin, e.g. 0.5 metres).

In addition the user has the facility to use standard pipe data, a manual selection of different pipe internal diameters or an optimised selection based on a pipe size the same as the pump discharge branch or based on a pipeline velocity. Up to five pipelines in parallel can be analysed and pumps in series or parallel can be selected.

The program compensates automatically for the fluid viscosity in establishing the pump H/Q characteristics.

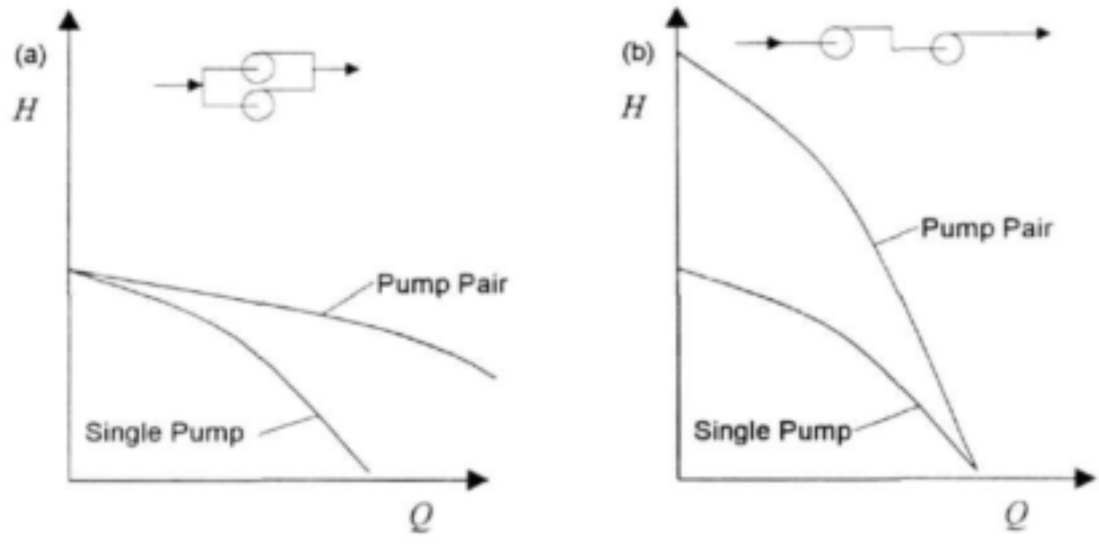


Figure 6.1: Characteristic H/Q curves for pump combinations: (a) in parallel; (b) in series.

The program can be used also quite independently to analyse and optimise pipeline systems for different types of pipes and their pressure ratings. The level of technology incorporated in the program means that a technically competent person is needed to benefit from it.

6.3. Pumping installation design

Before a pumping installation can be designed, it is necessary to decide on two matters:

(a) the length, diameter and type of internal lining of the suction and delivery mains, so that energy losses in the system can be calculated for the required rate of flow.

(b) the number, make and type of pumps to be installed.

6.3.1. Pump selection

In general, pump type selection is largely determined by the required duty, from which the specific speed can be calculated, thus indicating whether the pump should be of the centrifugal, mixed flow, or axial flow type. The number and size of pumps is normally selected to match the pattern of flow variation. Where the pumping demand is more or less constant, there is an obvious maintenance advantage in using a single size and type of pump. Except in very high head installations, pumps are used in parallel configuration.

The pumping efficiency will vary, depending on whether one, two or more pumps are working. Energy efficiency is obviously an important consideration in pump set selection, the normal design objective being the achievement of the highest possible operational efficiency, taking all modes of operation into account. While pump units of best efficiency in a particularly category may be chosen for clean water pumping, the designer has generally to settle for a lower efficiency in pumping wastewater, where the necessity to pass suspended solids calls for large flow passages and clog-free impeller geometry.

The provision of standby capacity is essential. The minimum standby capacity is that which will allow the station to operate at design load with any one of its pumps down for maintenance.

6.3.2. General layout of pumping installation

- The primary criteria for the design and costing are as follows:
- Vertical spindle installation;
- Horizontal spindle installation having either:
    - high pressure and low volume,
    - high volume and low pressure.

The underground structure of a pumping station consists of a wet well or sump, and a dry well in which the pump set is housed. The use of a vertical shaft pump allows a more compact dry well and has the environmental advantage of drive motor placement at ground level. The use of submersible pumps eliminates the necessity for a separate dry well.

6.3.3. Design of the wet well or sump

Pump sumps are preferably designed to provide a flooded pump suction (positive gauge pressure at the suction flange) at start-up and thus obviate priming problems. Also, the geometry of the pump and submergence of the inlet should be such as to avoid vortex formation and air intake.

Where the inflow rate to the sump is variable, as in sewage and stormwater pumping, it is necessary to provide a storage volume in the sump to avoid too frequent pump starting, which would lead to motor-starter burn-out. At the same time, it is also desirable to minimise solids deposition in pump sumps and the attendant septicity problems; hence, there is a need to avoid unduly large sumps. The sump volume required to satisfy the starting frequency criteria may be calculated on the following basis, where P is the pumping rate, Q is the inflow rate to the sump, and V is the effective sump volume:

$$\text{Time to empty sump} = \frac{V}{P - Q}$$

$$\text{Time to fill sump} = \frac{V}{Q}$$

If the pumping installation is to be situated lower than ground level, the designer must investigate a risk of flooding (relatively high risk). The installation must comply essentially with the following criteria:

i) drain sump equipped with preferably the submersible pump connected to the stand by generator having the switches above maximum potential flood level.
ii) one submersible pump of about 1kW motor.
iii) a location of the switches above maximum potential flood level for submersible pump and also the overhead crane (i.e. for an emergency purpose).
iv) sizing of the generator must comply with requirements of the submersible pump and the crane.

6.3.4. Control of pumping system

Pumping systems are normally automatically controlled, responding to signals generated by the particular duty regime. A variety of level-sensing devices is available, including electrode rods, floats, ultrasonic devices, pressure sensors, and so on. These sensors produce a signal which switches on a pump when the water level in the sump reaches the upper set point for that pump or switches off a pump when the water level drops to the cut-out level for that pump. Typically, each pump has its own cut-in and cut-out level. This means that only one pump is involved in any switching operation, thus minimising the resulting step-change in flow in the rising main. The minimum height difference between two adjacent water surface control levels should be such as to avoid simultaneous switching of these pumps due to water surface wave action.

Pump control may also be exercised in response to signals generated on the delivery side. This type of control is typical of water supply installations, where the requirement may, for example, be the maintenance of a fixed pressure at a specified location in the distribution pipe system. The use of variable-speed electric motor pump drives offers considerable control flexibility in this category of application, allowing a close matching of supply and demand and thus reducing energy costs.

Pump installations are typically fitted with a valve set, and with a gate or sluice valve on the suction side and with a gate valve plus a non-return valve on the delivery side. The gate valves allow the isolation of the pump and non-return valve for repair and maintenance purposes, while the non-return valve prevents the emptying of the rising main when the pump is not operating.

All pumping installations should be checked at the design stage for water hammer effects and provided with appropriate control devices, if required.

Table 6.1: Essential parameters for pumping operation design

Parameter	Suggested value
Efficiency - pump - motor	90% 97%
Overall efficiency	86%
Power factor (kW/kVA)	0,96
Motor size - up to 10 000 kW operating at 11 kV motor safety factor	10% in excess of pump power
Pump – breakdown allowance	10% p.a.

6.3.5. Costing of pumping design

In the costing of pumping operation design, there are usually numerous options to be added to the basic price. For most of cases specified by the application of pumping installation and sometimes by the scope of the water supply. A comprehensive quoting facility, customised for the pump user requirements would be essential. This is not always the case, even the pump and motor suppliers try to accommodate the designer in the best possible way. They should have the price databases available for the consumer's or designer's purposes.

Cost of pumping installation

In order to cost adequately and optimally a pumping installation, sound costing information on the past and current pumping installation components is needed.

Basically each pumping installation comprises the mechanical and electrical equipment which is usually housed within a civil structure. The designer must essential deal with the cost components as follows:

Pumping installation standard electro-mechanical equipment costs (i.e. pump, electrically driven motor, suction and delivery valves, pipework, metering and sundry equipment, manifold piping, manifold valves, switchgear, auxiliary transformers, battery chargers, crane and radiotelemetry).

Pumping installation non-standard equipment costs (i.e. standby generator, electrical elevator, telephone exchange, fire-fighting equipment, etc.).

Pumping installation civil structure costs (commonly the pump station area can be divided into: control room, pump basement, high voltage room, battery room, loading bay, staff room, passages and staircases, and occasionally roofed or unroofed store areas or workshop areas).

See Appendix A for more background information on costing of various components of a pumping installation. For a brief orientation, the prices of pumps for the ordinary pumping and booster stations are given in Table 6.2.

Table 6.2: Cost of pumps versus pumping head for ordinary and booster station

Pumping head (m)	Cost of pumps (R/kW) Ordinary pump station	Booster station
50	1450	625
150	1020	430
250	735	340
350	545	225
460	400	-
550	360	-

Source: Department of Water Affairs and Forestry (all costs escalated to 1996)

6.3.6. Cost of pumping

Most pumping installations are powered by electricity. The consumption of electrical energy subsequently dictates the cost of water delivered to the consumers. Primarily due to the considerable distances and differences in elevation between intake of water and distribution network, pumping represents for most water supply authorities the largest portion of the running costs. It is therefore most important to the water supply authorities that both sizing and operation of pumping installation are near to optimally managed.

Stallard (1980) based the costs of operating the electrically-driven pumps on the maximum daily pump-rate during the mouth and adopted to the cost of program as follows:

PUMPING COST = UNIT COST * PUMP RATE

The program embodies provision for penalty costs incurred on accounts, the costs of flood damage, water loss and the consequences of failing to meet consumer demand.

To find the cost of pumping, the power and energy expended must be calculated. Power input depends on the discharge and the pumping head. The discharge is determined by the demand for water that must be met. The head depends on the configuration of the system. The pumping system head includes the head requirements in the distribution network after allowing for friction losses. Usually the designer does not have available the full extent of future operational network for the distribution system. The system head curves have to be estimated by taking into account the variation of the friction loss from changes in demand. The changes in service (or distribution) reservoirs are usually neglected by assuming that reservoirs are supplied from the top and thus the lift requirements remain the same.

Monthly electricity use is based on metered use and applicable electricity tariffs. The electricity tariff includes charges based on electricity demand and actual use. Each pumping installation is usually metered separately for the electricity it uses.

In the following example, savings in energy achieved through speed control of one centrifugal pump (in the sense of adapting the pump flow to its real consumption of the energy) in relation to damping system will be shown.

Example 6.1: Annual energy savings in water supply pumping

Pump power in service point 1: (Q_1 = 550 m^3/hr, H = 60m)

P_1 = 122 kW

Pump power in service point 2: (Q_2 = 400 m^3/hr, H = 72m)

P_2 = 114 kW (system with damping)

Pump power in service point 3: (Q_3 = 400 m^3/hr, H = 32m)

P_3 = 47 kW (system with speed control)

Specific energy of pump (energy consumption per flow unit):

Service point 1: e_1 = 0,22 kWh/m^3

Service point 2: e_2 = 0,29 kWh/m^3 (system with damping)

Service point 3: e_3 = 0,12 kWh/m^3 (system with speed control)

It can be observed that the lowest specific energy in the system with damping is always obtained at the maximum flow. In the system with speed control, the specific energy does not only decrease by also reaches its minimum service point.

The amount of energy consumed in one day:

System with damping: 114 x 24 = 2736 kWh/day

System with speed control: 47 x 24 = 1128 kWh/day

Energy savings: 2736 – 1128 = 1608 kWh/day

Savings in energy per year: 1608 x 365 = 486,920 kWh

The opportunity for cost savings often exists if incentives are provided, in particular for electricity tariffs.

The main generating authority, ESKOM, produces about 95 per cent of all electricity consumed and also operates as a distribution authority in areas not served by an intermediate distribution authority (e.g. municipality, etc.). Most water supply authorities obtain electricity directly from ESKOM, enabling them to adopt most relevant tariffs. ESKOM has developed a long-term tariff plan which applies differential increases to the various available tariffs to become more cost effective. In particular, those tariffs for consumers taking supply at voltages of 6,6 kV to 132 kV. ESKOM stated that the price increase for 1997 has been 5 per cent on average over 1996 costs.

Table 6.3: Standard Rate tariff kVA demand versus the kW demand

Voltage supply	KVA Demand Increase (%)				KW Demand Increase (%)			
kWh	kVA	Equal	Cost LF	Average	kWh	kW	Equal cost LF	Average
<500V	5	7	0.76	6	5	8.4	0.83	6.7
>=500V to 66kV	5	6	0.72	5.5	5	7.4	0.79	6.2
>=66kV to 132kV	5	3	0.64	4	5	4.4	0.71	4.7
>132kV	5	3	0.64	4	5	4.4	0.71	4.7
Average	5	5	0.7	5	5	6.2	0.8	6

There is a cost implication for the consumers connected at lower voltages over those at higher voltages, with demand being the major cost portion. The benefits of converting to the kVA demand from kW demand are obvious from Table 6.3.

According to ESKOM, the consumers using the Standard Rate can be granted sliding scale diversity of demand without the pre-conditions being met. Such benefits can reduce overall demand charges by 5 per cent or more.

It should be borne in mind what effect a load factor has on proportional increase. The load factor is given by kWh/(730 * kW), where 730 is the average number of hours in a month.

The two part tariff which has an energy (kWh) charge and a demand (kW or kVA) charge, has a break even point when the two costs are equal:

$$\text{kWh} * \text{R/kWh} = \text{kW} * \text{R/kW}$$

Therefore for equal costs:

$$\text{Load factor (LF)} = (\text{R/kW})/(730 * \text{R/kWh})$$

This varies between 0,83 (below 500 V supply on kW demand) to 0,64 (above 132 kV supply on kVA demand). Subsequently the consumers with load factor lower than the "equal cost" load factor will therefore have a higher proportion of demand charges and vice versa.

6.4. Optimal pumping operation

As described in paragraph 1.4, this project is a two phase project with two separate but closely related subject. The optimal pumping operation is dealt with in the pumping rules phase report.

The aim of the pumping rules phase of this project was to develop a dynamic computer simulation model that could then be used to optimise the rules for pumping operation used by the operators of reservoir storage systems with respect to demand patterns, pumping costs, and storage volumes. This aim has been fulfilled with the completion of the computer program *Reservoir System Pumping Optimisation* (RSPO), which allows historical demand data to be imported and analysed, a stochastic model of that data to be fitted, and optimum operation policies to be calculated.

RSPO analyses the available historical data by fitting either a linear or exponential trend, whichever gives the best fit. This trend is then removed from the data, and annual, weekly and daily periodic trends are fitted. To model the serial autocorrelations of the data, a linear stochastic model is fitted.

The simulation option of RSO allows the user to calculate the total running costs of a given reservoir system using either historical, or synthetically generated, demand data. Both the cost functions and the operating policy used in the simulation can be specified by the user. This allows the user to test the effect of different cost and operation policies on the system, over any period the user chooses. The synthetic demand data generated using the simulation option has been shown to have a high statistical similarity to the historical data modelled.

RSPO also allows the user to calculate optimum operating policies for the system using either the Downhill Simplex method or the Dynamic Programming method of optimisation. The policies calculated by these methods have been shown to be both very close to optimum, and useful for the continued operation of the reservoir system. The effect of local optimum points in the cost space was investigated. It was found that although there are local optimum points that do tend to prevent the system from finding the global optimum solution these tend to be small and not reduce the accuracy of the solution by more than 2%. It was also found that the accuracy of the optimum solution found is not very sensitive to the length of the data set used in the optimisation process.

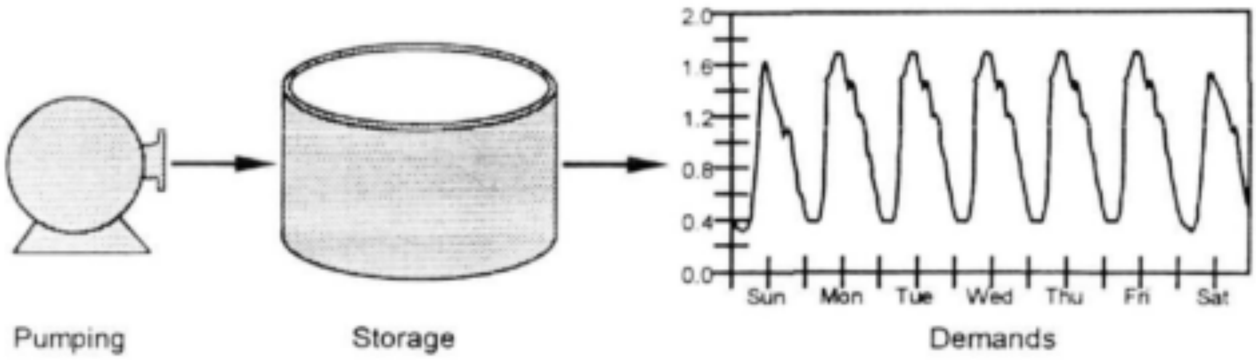


Figure 6.2: Reservoir System Pumping Optimisation (RSPO) model

7. RESERVOIR STORAGE OPTIMIZATION

7.1. Representation of storage reservoir in water supply system

The storage reservoir is a key component in both the gravity and pumping water supply systems. The prime functions of a storage reservoir are related to the following:

(a) balancing of variable water demands imposed on the supply system by various water users depending on such supply system. The water demands vary according to the category of water use (e.g. residential, commercial, industrial, power generation or agricultural, etc.), seasonal conditions and technical limitations of water supply infrastructure.

(b) providing for an emergency storage due to possible interruptions in water supply on account of failure.

(c) compensating for operational requirements, e.g. to shut the supply of the inlet, control provision of pumps and provision of a storage for sediment at the bottom of a reservoir.

(d) providing the fire-fighting storage - primarily in case of municipal water supply system.

General standards observed in the determination of quantity of storage indicated in (b), (c) and (d), are primarily based on the number of hours that are required to compensate for the duration of such events. It should be noted that increasingly storage is constructed to be sized to allow for optimum use of electricity tariffs, (i.e. in the case of a pumped water supply system).

7.1.1. Bulk storage reservoir (service operational storage)

The reservoir storage sizing in the pumped water supply system depends primarily on the system operation policy adopted by the supply authority. There are principally three approaches in determining such a policy.

(a) **Constant pumping approach** - if it is desirable to maintain the pumping operations as constant as possible, the effective (or active) storage of a balancing reservoir has to be increased, to store needed reserve water supply quantities to meet essentially varying demands. It must be noted that by adopting this policy, the energy costs may be increased because of the higher head due to the larger storage. Also the off-peak energy might be less utilised, although the price of off-peak energy is usually much cheaper.

(b) **Reduction in energy cost approach** - if it is desirable to reduce the energy costs, the reservoir water levels might have to be maintained as low as possible by meeting fluctuating water demands through adjusting the operating rate or number of operating pumps. The reduction in energy costs would assume an increase of the off-peak energy supply. Such an approach will depend upon the correlation between hourly water demand and energy price.

(c) **Supply system reliability approach** - if the reliability of supply is the prime concern in policy determination, the hourly variation in the water demand must be correlated with the accidental supply system breakdowns. The uncertainties in the variation and magnitude of water demand can be met with either marginal reservoir storage or by adjusting the rate of pump operation or by a combination of both.

By present measures the constant pumping approach in bulk reservoir storage policy determination as described in (i) appear to be far too obsolete for most pumped water supply systems. However the water supply utilities in South Africa do not necessarily define their bulk reservoir storage operational policies according to either (ii) or (iii) approaches as described above. This is primarily due to the unitary electrical energy tariffs offered to them by ESKOM (i.e. no differentiation is made between the day and night tariffs). At present the only stimulator in policy determination is considered to be a peak tariff imposed on users during "peak periods" (i.e. a period of excessive demand for power sold under higher tariff by a supplier).

The water supply utilities are trying not to exceed their historic peak demand by pumping the storage's just below the historic peak level. In the case of Rand Water, this is arranged by assuring that the storage in a reservoir will not fall below 70 per cent of the peak day demand which would be an equivalent to 91 per cent of the annual average daily demand (AADD).

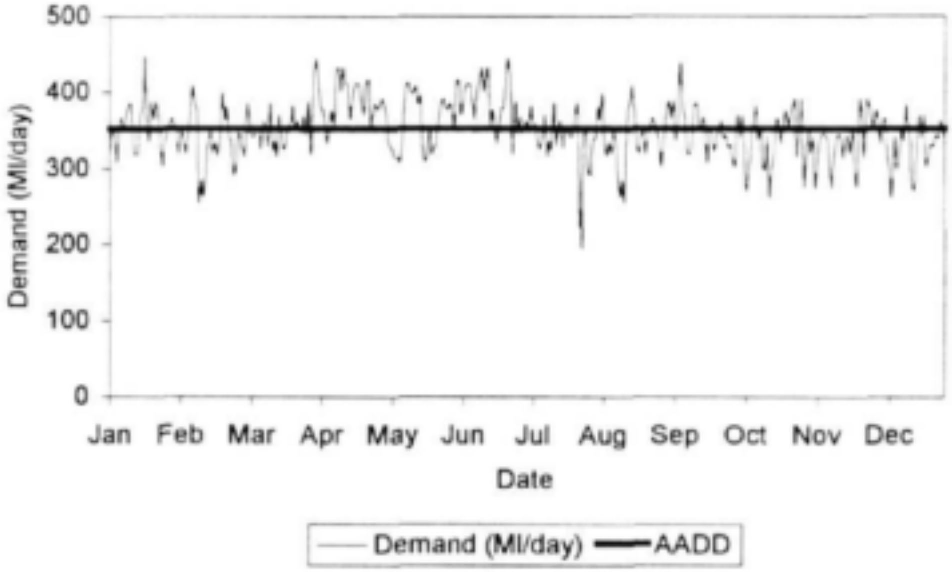


Figure 7.1: Determination of the Annual Average Daily Demand (AADD)

On the other hand to avoid over filling the reservoirs on account of a sudden drop in demand (i.e. to prevent a loss of water and possible flood damages) the levels in the bulk storage reservoirs are prevented from rising above 75 per cent of their capacity. The simple bulk storage reservoir policy is expressed as follows:

OPERATION STORAGE = MAXIMUM STORAGE (e.g. 75% of reservoir capacity) - MINIMUM STORAGE (e.g. 70% of the peak day demand)

Rand Water operates at any given time some 50 reservoirs ranging in size between 6 Ml and 600 Ml. With a total balancing storage of about 3900 Ml, this largest South African water supply authority has to cope with a wide range of operational requirements from its customers. In principle Rand Water supplies its customers (i.e. local authorities, mines and some end users) with their maximum day demand. However, the Board stipulates that the consumer's maximum demand must be drawn off the supply mains at a uniform rate over at least 20 hours of the day. The Board also stipulates that the customers must have reservoir storage capacity (i.e. primarily distribution storage reservoirs) equal to 36 hours of their average demand during their month of highest demand. The Board does not guarantee a specific residual pressure at any of its off-takes.

Table 7.1: Typical storage component sizes in the service reservoir (after Vorster et al., 1995, Reservoir capacity guidelines)

Land use or urban area character	Balancing volume (h x AADD)	Emergency volume (h x AADD)	Total volume (h x AADD)
Low density residential	12	12	24
Medium density residential	12	12	24
Industrial/ Commercial	9	15	24
High risk areas	12	24	36

AADD = Annual Average Daily Demand of a given supply area

Since 1995, Rand Water accepted in principle (Vorster et al., 1995) that additional storage capacity (i.e. above that required to compensate out peak demand) equals to 36 hours (or in some cases 48 hours) of average consumption during the month of highest consumption's.

7.1.2. Distribution storage reservoir (intermediate storage)

The distribution storage reservoir (or intermediate storage) is an integral component primarily of municipal water supply and distribution systems (i.e. third tier water supply authority). This storage is principally a buffer storage between the bulk storage reservoir and the end user, be it a residential household or mining undertaking. For such reason, the storage composition has to differ to some extent to that of storage's relevant to the bulk service utility reservoirs. The typical storage capacity of distribution reservoirs will range from 0,5 Ml to 50 Ml.

The intermediate storage has to balance directly all demands for water generated within the distribution networks, subsequently the assessment of peak demands for a distribution area is vital for determination of adequate reservoir size. The operational policy and design approach have to observe and accommodate requirements for the storage components.

Table 7.2: Typical storage component sizes in the distribution storage reservoir

Storage component:	Current design parameters:
Operational freeboard	1.5 hours AADD
Emergency storage (i.e. pipeline breaks, etc.)	Typically: 12 hours of the peak day
Demand storage (i.e. to balance the variable flow rates)	Typically: 12 hours of the peak day for industrial areas, 6 hours for all other areas
Fire-fighting storage	Varies according to fire-fighting regulations adopted
Bottom storage	1.0 hour AADD

Operational freeboard storage

It is recognised to be highly impractical to operate a storage reservoir at its 100 per cent capacity level. The operational or level sensing errors would then lead to water spillage through the emergency overflow. It is therefore advisable to allow for some freeboard storage between the emergency overflow level, and the reservoir maximum operational level. In the case of the pumped storage system, the reservoir has to act as a balancing sump to ensure an acceptable length of the pumping cycles. This operational volume provision for pumps should be added to the freeboard storage volume. The typical operational freeboard storage is determined as 1,5 hours of AADD.

Emergency storage

The emergency storage allocation in the water supply system reservoirs relates mainly to the amount of time to organise repairs of bursts or other difficulties related to interruptions in supply as follows:

(a) high supply pressures - causing deformation to the pipelines, joints, seals and possibly to the immediate environment above or near vicinity of the damaged pipe in the event of a burst.

(b) low supply pressures - causing collapse of the pipeline, leakage's into or from the pipeline with the possibility of water contamination.

(c) scheduled maintenance or unscheduled failure of electric motor or pumping installations - caused usually by damages to the brush gear or seals.

(d) extreme conditions caused by natural or political disturbances.

To determine an emergency storage component in a balancing reservoir related to unscheduled events is rather a precarious task considering that the process is influenced by the following factors:

(a) probability of interruption events;

(b) duration of such events;

(c) losses in water quantities due to pressures, etc.

(d) possible large cost incurred to the supplier and customers.

The simplistic approach to determination of emergency storage is as follows:

$$\text{Emergency volume } (V_e) = \frac{\text{Repair or outage time } (t_A)}{24 \text{ hours}} \cdot \text{max daily demand } (Q_d)$$

where: $V_e = Q_d$ assumed to be maximum

From the inputs on outages with regard to various pipeline diameters obtained from the City Engineer's Department of Cape Town, Goudveld Water and Rand Water the researchers compiled the graph of comparison of outage times for different diameter trunk mains. Such information allows, e.g. to determine the average outage time for the steel pipeline diameters between 200 and 450 mm, if the total annual outage time is known or estimated. For example, Johannesburg Metro (1996) lists on average 300 hours monthly on repairs of various pipe bursts. If the total annual average time of some 3600 hours is scaled against the duration of outages on Figure 7.2, some 5,2 hours of repair time is required on average for the steel pipe of 300 mm in diameter.

The probability (Prob) of pipeline failure can be determined using the Poisson probability distribution

$$\text{Prob} = 1 - e^{-\beta}, \text{ with } \beta = \gamma * L$$

where: β = expected number of failures per year on a pipeline,

γ = expected number of failures per year per unit length of pipe and

L = length of pipeline.

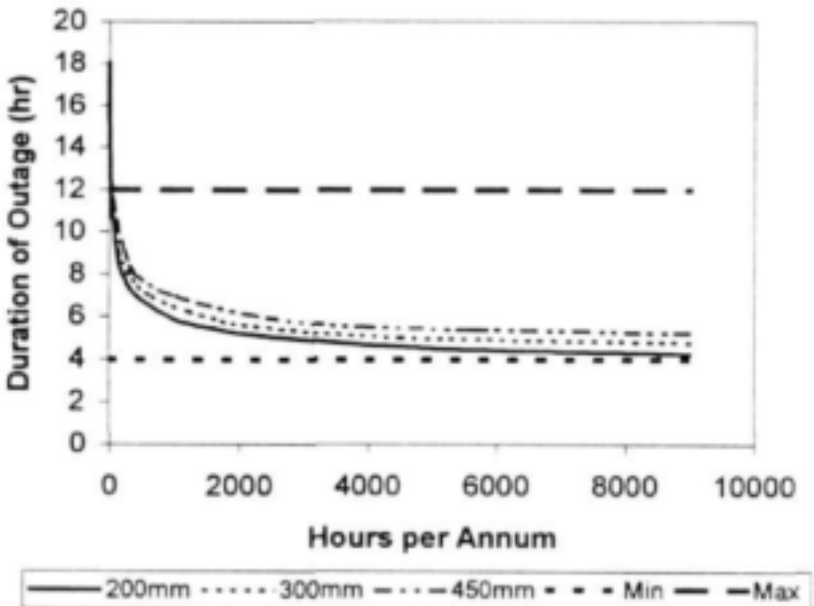


Figure 7.2: Outage times for different pipeline diameters based on research in South Africa

It must be noted that emergency volume (V_e) can be further reduced by assessment of overall system storage (i.e. storage in the supply and distribution network, terminal storage, etc.) and means of bypass pipeline and various pipeline interconnections. Figure 7.3 illustrates two examples of trunk mains arrangements commonly found in the Rand Water limits of supply area.

(a) three pipelines in parallel

(b) two pipelines in parallel with one interconnection

Figure 7.3: Multi-pipeline arrangements between two reservoirs

According to Wagner *et al.* (1988), the configuration of distribution network or trunk main supply system with regard to operational links and interconnections can be evaluated for its capacity to meet demands and subsequently the probabilities of failures to assess the supply system overall reliability.

At the Energy Laboratory of the Rand Afrikaans University, Edwards (1996) investigated various combinations of pipeline configurations subjected to a single or multi-failure situations using the binomial theory approach.

Example 7.1: Probability of system's pipeline capacity supply

Taking into consideration the Rand Water's Bloemendal/ Wildebeesfontein trunk main system with three pipelines in parallel, various failure scenarios of the trunk main due to pipe bursts or valve failure are observed as follows:

Table 7.3: Pipeline failure with regard to probability of flow

Pipeline failure (No.)	System's capacity (%)	Probability (P)
0	100,000	92,980
1	66,676	6,850
2	33,353	0,168
3	0,000	0,001

Based on the values listed in Table 7.3, the Probability of system's capacity supply = (0,92980 x 100) + (0,06850 * 66,676) + (0,00168 * 33,353) + (0,00001 x 0) = 92,980 + 4,567 + 0,056 = 97,603%.

This means that the system will function fully on average for about 98 per cent of the time. If various strategically placed interconnections between pipelines will be built into the system the probability of full capacity supply can be increased.

<u>Demand storage</u>

The demand storage in the balancing reservoir is the most critical component in determining total reservoir storage capacity. This storage component has to cater for varying consumption (i.e. average and peak demands) generated within the distribution network or another supply branch (i.e. adjacent supply zone) while supported by the constant or variable inflows. This means that the consumers situated in the supply area draw water from a storage reservoir at a variable rate, whereas the feeder trunk (or main) usually delivers water at a constant rate. The demand storage has to absorb differences between inflows and outflows. If the storage reservoir is an integral component of an existing pumping water supply system, a set of pumping rules will be required for the operators of reservoir storage with respect to consumer demand patterns. If the demand storage has to be determined for a new supply area (or township), the approach in obtaining and processing the consumer's demand patterns as discussed in Chapter 4 and paragraph 7.3.1 of this section should be observed.

The inflow into the storage reservoir to compensate for the deficits in the demand storage may be either of the following:

continuous (i.e. 24 hours per day), or

intermittent (i.e. n hours per day).

Both the continuous or intermittent inflows into the storage reservoir can be expressed, in the case of a pumped system, as the load factor. The average load factor is the proportion of time run by the pump (e.g. pumping for 12 hours per day, the load factor is 0,5; or 18 hours per day equal to the load factor of 0,75, etc.). The energy input calculations for reservoir optimisation are based on the size of pumping load factor.

The provision of demand storage is conventionally determined by the mass-curve method (Rippl, 1883) or improved stretch-tread method (Klemes, 1979). The mass-curve method (graphic or tabular) requires to determine accumulative water demand, expressed either volumetrically or as a percentage of the total demand over a peak day, or preferably peak day in a week. The demand storage is determined from the sum of the peak differences and the supply rate.

With regard to the current design guidelines, demand storage is typically installed at 12 hours for industrial areas and 6 hours for all other areas, based on the peak day within a supply area.

Fire-fighting storage

It is essential to have an adequate water supply available for fire-fighting. This is mostly supplied through the municipal water reticulation system, and is thus also drawn from the distribution storage reservoirs. In South Africa, it is a distinct responsibility of the local authority to provide storage for fire-fighting in their reservoirs. Most guidelines will specify an additional volume of water for which allowance must be made in the storage reservoirs.

The local (i.e. South African) fire-fighting water standards specify six fire risk categories, as follows:

high

medium, and

low, groups 1 to 4 based on the risk of a fire starting and spreading. The fire-fighting water requirements are then specified according to the fire risk category of a given area.

Haarhoff and van Zyl (1997) are proposing that a significant savings can be made in reducing the municipal fire-fighting storage by sharing the one off storage between building owners and local authority.

Table 7.4: Typical guidelines for the provision of fire-fighting storage in S.A. and elsewhere

Parameter	Fire risk category	Typical design guidelines			
		S.A. Nat. Housing (1994)	Holland (1977)	Germany (1978)	U.S.A. (1981)
Fire flow rate (L/min)	High Medium Low	12 000 6 000 900	6 000 1 500 1 500	3 200 1 600 800	17 700 11 800 3 800
Fire duration (hours)	High Medium Low	6 4 2	2 2 2	2 2 2	4 3 2
Volume of fire water (m^3)	High Medium Low	4 320 1 440 108	720 360 180	3 394 192 96	4 248 2 124 456

Bottom storage

Normally, it is not allowed that a storage tank is drawn down until it is completely empty. This would cause the sediment to be drawn into the supply system. It is therefore advisable to allow a minimum bottom storage volume, which will be a function of the treated water quality, frequency of reservoir cleaning and the hydraulic arrangement of the inlet and offtake pipework. The bottom storage volume has to be estimated for each case, and added to the total storage volume. The suggested value is 1 hour of AADD.

7.2. Typical guidelines for sizing of the total storage in reservoirs

According to the Guidelines for the Provision of Engineering Services and Amenities in Residential Township Development (1994), the capacity of the supply main to the reservoir and minimum reservoir storage capacity are suggested to be 24 hours and 48 hours AADD for gravity or pumped supply from one water source or two water sources respectively. The actual situation around the country is illustrated in Table 7.4 below.

Table 7.5: Typical current guidelines for sizing storage reservoirs and feeders in South Africa

Authority	Nature of supply	Feeder capacity	Reservoir capacity
Dept. of Water Affairs	gravity feed pumped main	1.5 x AADD 1.5 x AADD	24 h of AADD 48 h of AADD
Co-operation & Development	gravity feed pumped main	1.5 x AADD 1.5 x AADD	24 h of AADD 48 h of AADD
National Building Inst.	one source two sources	1.5 x AADD 1.5 x AADD	48 h of AADD 36 h of AADD
National Housing	gravity feed pumped main two sources	1.5 x AADD 1.5 x AADD 1.5 x AADD	20 h of AADD 30 h of AADD 66% of capacity with one source
Johannesburg City Council	gravity feed pumped main	1.5 x AADD 1.5 x AADD	36 h of AADD 54 h of AADD
Port Elizabeth City Council	pumped main	1.5 x AADD	48 h of AADD
Overberg Water	gravity feed pumped main	1.5 x AADD 1.5 x AADD	24 h of AADD 36 h of AADD
Goudveld Water	gravity feed	1.5 x AADD	24 h of AADD

N.B.: AADD = Annual Average Daily Demand of a supply area

Nel and Haarhoff (1996) listed in their technical paper the constraints related to the current guidelines for sizing of storage reservoirs and feeders (i.e. trunk mains) in South Africa. These are as follows:

(a) the size, character or unique features of the supply area are only partially reflected in the guidelines by adopting a single value of the AADD for a supply area.

(b) the pre-specified feeder pipe capacity into the storage reservoir does not allow for the optimum cost combination of supply pipe and storage size.

(c) the probability of water supply system failure due to the operational risks is not determined by the current guidelines (i.e. the inherent factors implied by the guidelines are not known).

In overall the current guidelines for sizing of storage reservoirs and their supply feed pipelines are insensitive to the performance criteria of water supply system reliability mainly with regard to the optimum design, management practices of water supply and distribution systems, and integration of design and proactive management based on the relevant risks analysis.

Due to increases in cost of water and other technical and institutional constraints, most supply systems in South Africa are in need of well researched and rational optimisation procedures, as well as adequate design guidelines leading to the optimum design of particularly new water supply systems with provision for continual extensions.

The major problem facing many existing urban water supply systems is continual process of supplying water to new demand areas by extending the existing infrastructure resulting eventually in inefficient water supply systems. It is estimated that about 80 percent of the very small and small water supply systems (O,5 to 30 Ml/day) can be classified as pumping systems requiring some form of energy input.

Table 7.6: Sizing of water supply systems according to delivery rate and energy input

System size	Delivery rate/ production of treated water (Ml/day)	Energy requirements	Location
Very small	0,5 to 10	none to low	Predominantly the community water supply systems (CWSS)
Small	10 to 30	moderate to high	Predominately municipal water supply systems (MWSS) and Government Water Schemes (GWS)
Medium	30 to 150	high	Most of GWS and water boards (WB) and some MWSS
Large	over 150	high	Large WBs and GWS

There is at present about 5000 Ml per day of installed urban water supply capacity administered by 17 water boards and numerous large municipal water supply systems in South Africa. There are also some of large urban government water supply schemes administered presently by the Department of Water Affairs and Forestry (DWAF). Some of these schemes are being or have a prospect to be privatised.

7.3. Optimal component sizing for a pumped reservoir storage

The design criteria for determining the total reservoir storage capacity are governed primarily by the following factors:

(i) varying demands for water, the most critical being:
  - systems pipeline and service pumps in relation to maximum seasonal and weekly demands; maximum storage replenishment rate, and maximum day demand plus fire-fighting storage (i.e. distribution reservoirs);
  - system's storage, service or booster pumps in relation to maximum day demand plus fire-fighting demand;
  - peak hour demand.

(ii) capital available for development or enhancement of a supply system

(iii) system's capacities of various components

(iv) system's operating policy

The optimal sizes of specific components for the total balancing storage of service or distribution reservoir can be obtained by optimisation procedures.

7.3.1. Optimal demand storage component sizing

The basic approach to demand storage component sizing is described earlier in this report. A more detailed approached has to follow essentially the following steps:

Step 1: determination of optimum supply main pipe diameter as a function of pumping rate, pressure head and pipeline costs, and

Step 2: optimisation of the relationship between reservoir demand storage component and supply main pipe capacity.

The Step 1 has been described in Chapter 5: Water Supply Trunk Main Optimisation and illustrated on relevant example determining the minimum cost of a pumping scheme.

The Step 2 evaluates the relationship between reservoir demand storage component and supply main pipe capacity and can be determined by various methods which were reviewed by Lauria (1983). The conventional Ripple mass-flow method is most commonly applied in reservoir storage design based on the simple fill-and-draw rule. The disadvantage of this method is that the single estimate of the capacity has no reliability explicitly attached to it.

Stephenson (1989) indicates that the most economic reservoir demand storage capacity varies from one day's supply based on the mean annual rate for short pipelines to two day's supply for long pipelines (i.e. over 60 km). The optimum storage capacity is determined by adding the cost of reservoirs and pipelines and capitalising running costs for various combinations and comparing them until the alternative with the least total cost is determined.

It must be noted that the demand storage component is the function of customers' consumption variability. The determination procedure of demand storage component for a service or distribution reservoir has to be based on the characteristics of the deterministic variables of the supplied area. These variables show usually regular, repetitive trends. Haarhoff and van Zyl (1996) stated that the relative magnitude of these variables can be estimated with reasonable certainty, as well as the time at which they might occur. The deterministic variables influencing the demand storage component sizing are as follows:

(i) **long-term demand trend** - represented in a designer's approach by the Annual Average Daily Demand (AADD) or more rightfully by the AADD estimated from a compound growth curve taking into the consideration the population and industrial developments increases. The long-term demand trend (or AADD) has to be determined according to characteristics of each individual supply area.

(ii) **seasonal trends** - represented by the climatological seasonal characteristics within a supply area (i.e. winter versus summer demand) or by seasonal activities of the population residing in a supply area (e.g. seasonal closure for some industries, school holidays, annual vacations, etc.). Such seasonal variation can be represented by the average monthly factors for every month in a year. The average monthly flow is then determined by multiplying the average annual daily demand by the monthly factors. The values of monthly factors for most areas in South Africa vary between 0,9 and 1,1.

(iii) **daily trends** - some days of the week have a higher or lower demand value than the average, such can be represented by weekly variation factors and the daily flow determined in a similar way as in (ii).

(iv) **hourly trends** - the variation of demand during the day is represented as the average hourly factors for each hour of the day. The characteristics for various supply areas can be determined from an analysis of historical demand profiles illustrated in section 4 of this report.

Note that if the historic demand trends are not known for a supply area, a synthetic demand records can be generated by means of the above described procedure. In this way the essential deterministic variables required for determination of the demand storage component can be obtained. This approach is commonly used in analysis of reservoir storage's (i.e. dam storage) in the hydrological systems. The steps taken in processing reservoir storage-yield analysis are similar to the steps which must be taken in processing of the pumped system demand storage component. The philosophy behind the reservoir storage simulation for a single estimate of storage capacity with inherent reliability is illustrated below:

Example 7.2: Reservoir storage simulation with inherent reliability

Step 1: Capacity $C_o = 0$; (i.e. reservoir is initially full)

Step 2: For period t = 1 to T calculate:

$C_t = \max(0, C_{t-1} + D_t - Q_t)$;

Step 3: If $C_T = C_o$ then go to Step 4; else if this is the first interpolation then set $C_o = C_T$ and go to Step 2; otherwise stop.

Step 4: C_a = maximum (C_t) over t = 1, ..., T,

where: C_t = storage deficit at the end of period t;

D_t = required delivery during period t;

Q_t = inflow during period t;

T = length of records (or design period);

C_a = active storage capacity.

The approach as described above has been adopted in designing of the demand storage component in the proposed capital cost optimisation program (CCOP). The technique of the logical statement method was built into the program. By using this method, three variables. i.e. demand pattern, storage and pumping rate, can be processed simultaneously by means of a micro-computer spreadsheet approach as one step in the optimisation procedure.

7.3.2. Optimal emergency storage component sizing

The general approach to determine an emergency reservoir storage component is described in paragraph 7.1.2.2. The South African typical trunk main and reservoir capacity guidelines including recommended sizes for emergency volumes for a specific urban land use are given in Table 7.5 and Table 7.1 respectively. The guidelines in Table 7.1 were compiled by Vorster et al. (1995) and based on the East Rand Regional Services Council administrative area which comprises numerous municipal water supply systems supplied almost exclusively from the Rand Water trunk water supply system. The available guidelines allow a designer to determine the first-hand sizes of a water supply system components before the final optimisation of a designed system or subsystem is performed.

The optimal emergency storage component is a function of the stochastic variables (i.e. non-supply events), contrary to the demand storage component determined primarily from the deterministic variables of the supplied area. The characteristics of stochastic variables for sizing of optimal emergency storage component can be determined from the analysis of all relevant non-supply events which might occur in a supply area with regard to the state of supply infrastructure, management procedures and preventative measures adopted within such a system. However, the non-supply events are difficult to predict with a certainty in terms of magnitude, duration, occurrence and financial losses on account of actual water loss, damage repairs and losses incurred by the consumers.

The concept of water losses incurred from a faulty water supply infrastructure components (i.e. mainly pipelines) due to non-supply events (i.e. bursts or leaks) of a water supply system is expressed as follows:

FAILURE/OUTAGE WATER LOSSES = FLOW RATE * AVERAGE DURATION * FREQUENCY

<u>Sizing of outage flow rate</u>

According to Lambert (1994) there are the continuous (or background) water losses present at the infrastructural components of all water supply and distribution systems. The UK trend in estimating of the background losses for the trunk mains assumes currently that there will be a water losses of 0,2 m^3/km of trunk main/year for each year of age. The service reservoir losses are estimated at 0,33 per cent of capacity per day. Lambert (1994) also estimated the proportional break-down between the background and failure/outage losses with 49 per cent and 51 per cent respectively. These estimates were based on a sample area of 70 m Average Night Zone Pressure (ANZP).

Table 7.7: Leak sizes and water losses encountered (after WRC, 1989)

Leak size	Litre per		m^3 per	
dia. (mm)	(minute)	(hour)	(day)	(month)
0,5	0,33	20	0,48	14,4
1,0	0,97	58	1,39	41,6
1,5	1,82	110	2,64	79
2,0	3,16	190	4,56	136
2,5	5,09	305	7,30	218
3,0	8,15	490	11,75	351
3,5	11,3	680	16,3	490
4,0	14,8	890	21,4	640
4,5	18,2	1 100	26,4	790
5,0	22,3	1 340	32,0	960
5,5	26,0	1 560	37,4	1 120
6,0	30,0	1 800	43,2	1 300
6,5	34,0	2 050	49,1	1 478
7,0	39,3	2 360	56,8	1 700

Note: Listed values calculated for a pressure of 5 bar or 500 kPa

In South Africa, the Water Research Commission Report No. 157/1/89 allows for the background water losses under certain pressure conditions to be estimated for various pipeline leak sizes as indicated in Table 7.7.

The supply pressures, particularly within the Rand Water limits of supply, can vary quite significantly. According to Coetze and Le Roux (1993) the actual pressures measured within the supply areas of Johannesburg Metro (e.g. Braamfontein, Observatory, Bellevue, etc.) experienced in some locations pressures of up to 750 kPa. In such areas, the pressure dependent leakage volumes can be estimated according to the UK's Pro-Aqua Systems (1994) equation as follows:

$$q_l = A_l * d * (2gh)^{1/2} + s * d * (2gh^3)^{1/2}$$

where: q_l = leakage flow (m^3/sec)

A_l = area of leakage path (m^2)

d = discharge coefficient (dimensionless, recommended value = 0,6)

g = acceleration due to gravity (m/sec^2)

h = average zone night pressure (m)

s = system expansion coefficient (%)

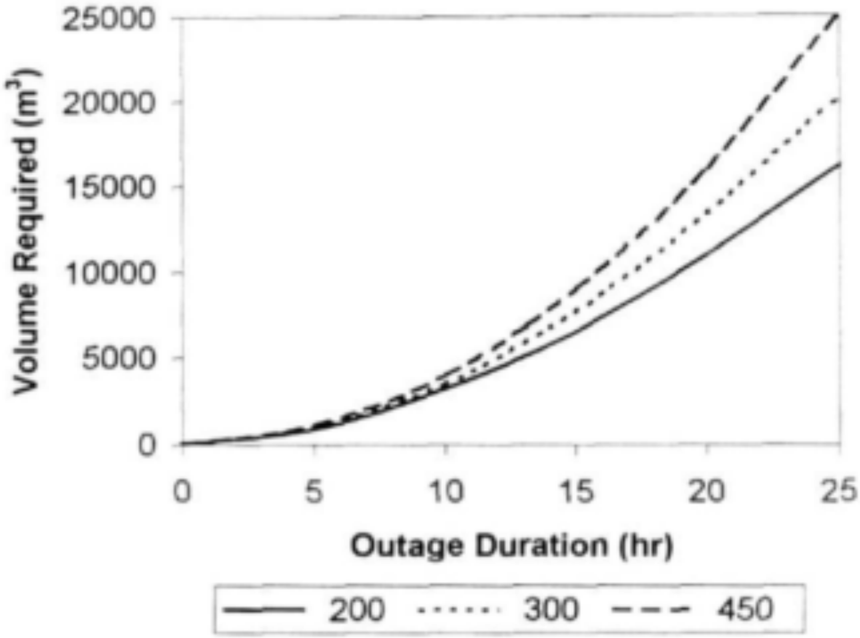


Figure 7.4: Relationship between outage and storage required for various pipeline diameters

From the information on stochastic characteristics (e.g. steel pipeline leaks and bursts) of non-supply events which occurred at various local urban water supply systems in 1996 (namely Goudveld Water, Cape Town Metro and Rand Water), the relationship between outage event duration and storage required, as a function of various pipeline diameters is illustrated in Figure 7.4.

Sizing of outage duration

Most water supply authorities, including the Rand Water, encounter commonly failures/outages on their supply mains as follows:

(i) holes (mainly due to corrosion in steel pipes)

(ii) joints (RW primarily on the pipelines with lead joints)

(iii) other breaks (e.g. split pipes, cracked weldings, etc.)

Generally, Rand Water can repair small holes within not more than an hour (usually by small sized wooden pinned wedges). The joints require on average 6 hours to repair. However, the abnormal conditions, so called "total breaks" (e.g. split pipes, usually in the control chambers) or other mechanical damages can take up to 12 hours to repair. Some very serious failures can take up to a few days to repair. A maximum known failure in recent history of Rand Water took 3 days to repair and reinstate supply to the subsystem.

The probability distribution of duration of failure/outage events is related to a Probability Density Function (PDF) which defines the probable duration of an emergency. The theory behind the PDF refers to the following:

$$Prob(a \le x \le b) = \int_{a}^{b} f(x)dx$$

where: $f(x)$ is a continuous function whose integral form $x = a$ to $x = b$ gives the probability that x will take on a value between a and b inclusive. PDF must satisfy the constraints as follows:

i) $f(x) \ge 0$ for all values of x (there is no negative probability that an event will occur)

ii) $\int_{\infty}^{\infty} f(x)dx = 1$ (there is the probability 100 per cent that one of the events will occur)

A simple PDF for a duration of an emergency can be adequately based on a triangular probability distribution through the minimum point (i.e. minimum possible emergency duration, as a function of the maintenance team's response time, usually min = 4 hours), the maximum point (i.e. maximum possible emergency duration, usually max = 24 hours) and most likely point (i.e. most likely duration of non-supply event, usually ml = 10 to 12 hours). This distribution is shown in Figure 7.5.

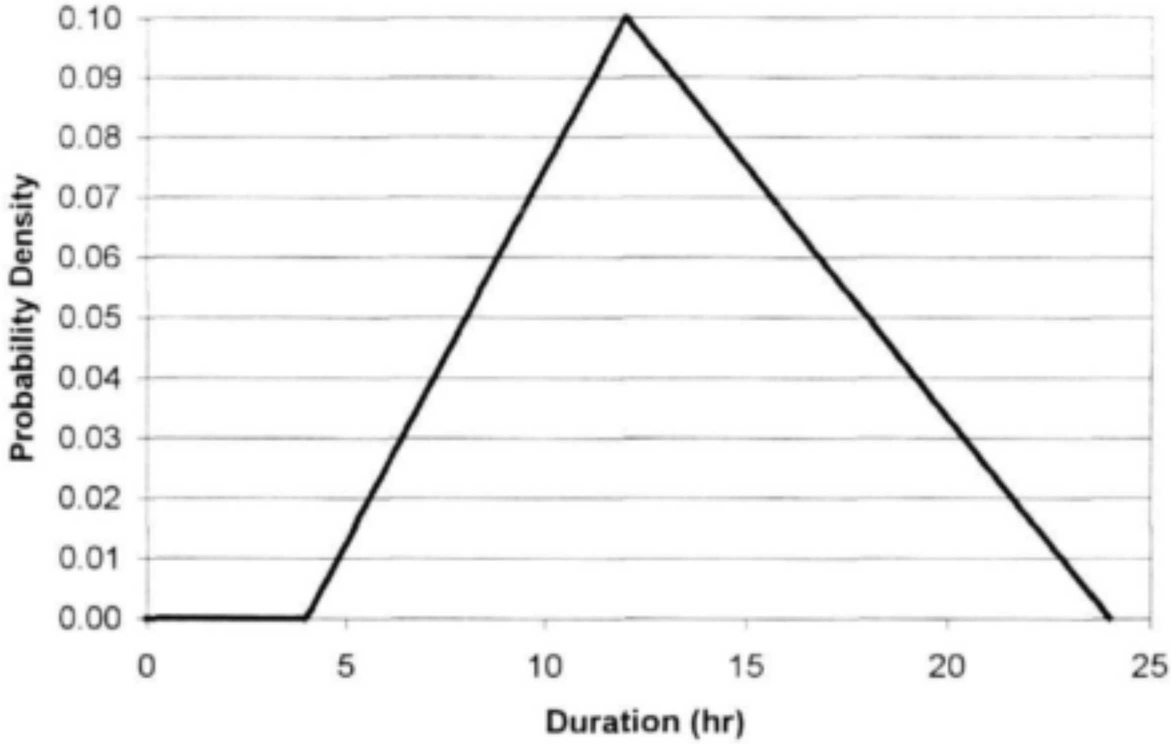


Figure 7.5: Simple triangular representation of a Probability Density Function (PDF)

From the PDF, the cumulative distribution function can be determined and as such is represented as follows:

$$F(a) = Prob(\alpha \leq a) = \int_{\infty}^{a} f(x)dx$$

For determination is emergency storage costs more appropriate representation of the cumulative distribution function is the Probability Exceedance Function (PEF) which is equal to 1 - F(a). This gives probability that an emergency with a duration exceeding *a* will occur.

$PEF(a) = 1 - F(a) = Prob(x \geq a) = \int_{a}^{\infty} f(x)dx$ The PEF representing most real-life non-supply event situations is illustrated in Figure 7.6 below:

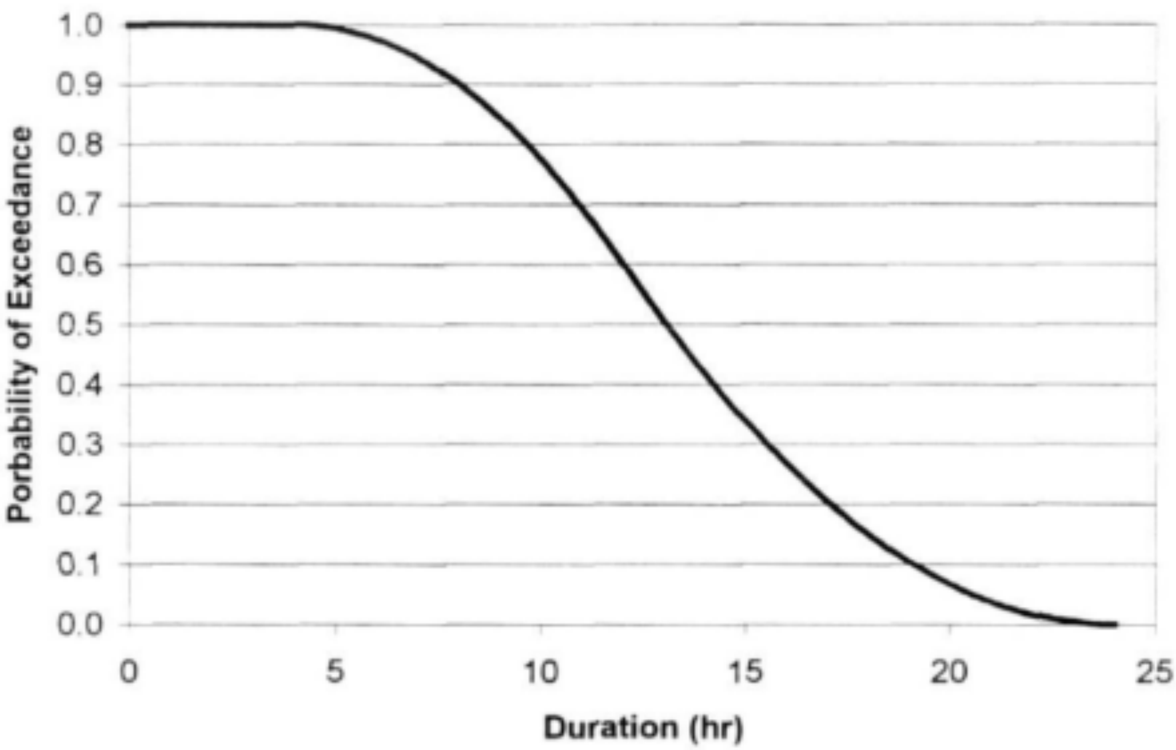


Figure 7.6: Probability of Exceedance Function on non-supply events for a supply and distribution systems

The expected value or mean (μ) of x can be determined as follows:

$$\mu = \int_{\infty}^{\infty} xf(x)dx$$

where μ is the expected duration of the emergency in hours.

Estimation of failure/outage frequency

The failure/outage frequency is dependent primarily on the conditions prevailing in a water supply and distribution system. Conditions such as ground subsidence situation, traffic loading, type of soils, supply pressures, pipe age and materials, climate, etc., matter in failure/outage frequency.

The information on the pipeline failures/outages compiled under this research project has its origin primarily from the local water supply authorities, (namely Goudveld Water, Cape Town City Engineers Department, Johannesburg Water and Gas Department and Rand Water Operation Division) as well as foreign literature.

Table 7.8: Pipeline failure/outage frequency for three combined supply systems in South Africa (1996)

Pipeline diameter range (mm)	Length of pipelines (km)	Number of failures per year (no)	Total outage time (hrs)	Average outage duration (hrs/failure)	Frequency of failure/ outage (per km/yr)
100-200	3898	645	2980	4,62	0,17
200-300	631	77	484	6,29	0,12
300-450	612	43	303	7,05	0,07
450-600	195	18	162	9,00	0,09
over 600	259	9	150	16,67	0,04
Total or average	5595	792	4079	average 5,15	average 0,14

The failure/outage frequency as determined in Table 7.8 relates primarily to the supply and distribution systems for the pipeline diameter ranges below 600 mm. The Rand Water operates mainly pipelines over 600 mm in diameter. Historic frequency of failure/outages with regard to recorded and repaired pipeline leaks within the Rand Water limits of supply are given in Table 7.9.

Table 7.9: Historic account of recorded and repaired leaks within Rand Water limits of supply (after Lamprecht, 1996)

Year (April to March)	Type of pipe leak			Total number of events	Total pipeline length (km)	Average frequency of non-supply events (/km/yr)
	Holes	Joints	Other			
1986/87	48	61	29	133	2450	0,05
1987/88	42	70	38	150	2520	0,06
1988/89	36	66	25	127	2430*	0,05
1989/90	42	47	27	116	2545*	0,05
1990/91	43	80	20	143	2560*	0,06
1991/92	36	55	29	120	2568*	0,05
1992/93	41	76	46	163	2572*	0,06
1993/94	13	55	14	82	2575*	0,03
1994/95	17	47	31	95	2580*	0,04
1995/96	16	59	17	92	2584	0,04

Note: * = Pipeline length estimated

The frequency of leaks within the Rand Water limits of supply is rather consistent with a significant reduction in recent years. The reasoning behind the reduction is pro-active management in cathodic protection of the steel pipelines implemented by the Rand Water in recent years.

Costs of pipeline failure/outage in supply

The costing details on pipeline failure or outage in water supply are not readily available, owing to mainly different approaches by the water supply authorities in gathering, compiling and processing of their costing (i.e. mainly budgetary) data. However, a limited costing information was eventually made available to the researchers by four major water supply authorities to enable basic orientation analysis in this sphere of optimisation research.

i) Total cost of non-supply event

According to the Rand Water's Operation Division personnel, the cost of pipeline failure or outage in water supply can be divided as follows:

C_1 = cost of repair time and material (commonly) somewhere between R5000 and R8000 in 1996).

C_2 = cost of water loss, varying according to the type of and size of break (a guideline unit cost is about R1000 per Ml in 1996).

C_3 = cost of reinstatement of environment in the immediate vicinity of the failure site (such cost can vary extensively and its magnitude can reach thousands of Rand).

C_4 = cost of customer's losses due to inconvenience of non-supply (not dealt with by the water supply authority).

C_5 = cost of increased maximum demand tariff resulting from having to refill reservoir after break.

TOTAL COST OF FAILURE/OUTAGE = $C_1 + C_2 + C_3 + C_4 + C_5$

Note that no adequate information has been found on aspects of C_4 cost component. Further research is needed to substantiate relevance of this cost component in overall optimisation procedure. The cost components C_2 and C_4 are relevant in determining an optimal size of emergency storage in a supply system.

Table 7.10: Average cost of non-supply event repair time and materials (1996)

Pipeline diameter range (mm)	Number of failures per year (no)	Total cost of repairs and materials (R*million)	Average cost of repair time and materials (R/failure)	Average outage duration (hr/failure)
100-200	645	2,243	3478	4,62
200-300	77	0,301	3909	6,29
300-450	43	0,312	7256	7,05
450-600	18	0,172	9556	9,00
Over 600	9	0,097	10778	16,67
Total or average	792	3,125	Average 3946	Average 5,15

ii) Annual costs of storage provision

The emergency storage in the service or distribution reservoir system is provided to ensure that the consumers receive an uninterrupted water supply in the event of a failure of supply to or from the reservoir. The types of possible failure are described in previous paragraphs.

The annual cost of emergency storage provision is related primarily to the cost components C_2 and C_4 of the total cost of non-supply event (i.e. failure). This cost is difficult to determine considering that both the duration and the number of non-supply events per year are of the stochastic nature. As illustrated in other paragraphs the number of failures per year will depend on the length of the reservoir or distribution supply pipe, its age, material and a number of other factors (e.g. extent of cathodic protection for steel pipelines, etc.).

The duration of emergency will depend primarily on the time it takes to detect the emergency, the response time of the maintenance crew and the severity of the emergency. The task is to find the optimum volume of emergency storage with regard to probable annual cost as a result of emergencies, as well as the cost of constructing a storage. The volume of storage that results in the minimum annual cost will be the optimum emergency volume the designer can recommend to be built into the supply system.

The annual reservoir storage cost is determined by means of mathematics of finance equation (*i.e.* capital recovery factor) as follows:

$$ACC = \frac{TCC * Rate * (1 + Rate)^{period}}{(1 + Rate)^{period} - 1}$$

where: ACC is the annual payment required to repay the loan used to construct the reservoir storage.

TCC is the total capital of the loan obtained from the regression analysis of costs for various sizes of reservoirs (see Chapter 8 for details).

Rate is the difference between the interest rate and the inflation rate.

Period is the number of years over which the loan will be repaid.

The annual expected cost of emergencies for a water supply system with no emergency storage is determined from the equation as follows:

$AEC = Q * \mu * n * C_L$;

where: AEC is expected cost of emergencies in a system with no emergency storage in Rands,

Q is the system supply capacity in m^3/hour,

μ is the expected duration of emergencies in hours,

n is the expected number of emergencies per annum,

C_L is the cost of water loss in R/m^3 (i.e. the cost to the supplier of the loss in water sales, and the cost to the consumers for the lack of available water).

Example 7.3: Sizing of the optimum emergency storage using an Emergency Storage spreadsheet model (EM_TRI.XLS)

The urban pumped water supply system is to be enhanced with a built-in service reservoir emergency storage based on a Peak Daily Demand of 766,6 m^3 per hour. An average of 9 non-supply emergencies per year (n) are assumed to take place most probably on the supply line. The estimated cost of water loss is R1 per m^3. The costing function is determined from Chapter 8.

If a reservoir system contains emergency storage, not all emergencies will result in financial loss. If the volume of emergency storage is greater than the duration of the emergency multiplied by the demand during the emergency, then the demands will be met from the storage, and no loss to consumers will occur. If the emergency goes on for too long, however, there will come a point when the consumer's demands cannot be met. If storage is provided, we can calculate a new Probability Density Function (PDF) for the probability that the available storage will be exceeded. The emergency duration PDF is represented as follows:

Minimum Duration (min)	4 hr
Most Likely Duration (ml)	12 hr
Maximum Duration (max)	24 hr

If we define n_e as the number of emergencies that exceeded the provided storage, then we can calculate the number of emergencies that can be expected to occur in a year within a given supply system from:

$$n_e = PEF(D) \times n$$

where D is the number of hours of storage provided at the peak daily demand

The expected cost of emergencies for a system that contains D emergency storage will thus be:

$$C_{ED} = Q \times \mu_e \times n_e \times C_L$$

where: C_{ED} is the expected cost of emergencies in a system with emergency storage D, in Rands,

Q is the system supply capacity in m^3/hr,

μ_e is the expected duration of the emergencies that exceed D in hours,

n_e is the expected number of emergencies that exceed D per annum, and

C_L is the cost of loss in R/m^3.

The total cost of providing emergency storage for a reservoir system is thus made up of the cost of providing the storage, and the expected cost of emergencies that exceeded the provided storage. The optimum storage will be the duration D that results in the minimum total annual cost.

The essential inputs into the program are as follows:

Emergency Duration of PDF

Minimum Duration (min)	4 hr
Most Likely Duration (ml)	12 hr

Maximum Duration (max) 24 hr

Emergency Duration Model

Emergencies per year (n) 9

Cost Calculations

Peak Daily Demand 766,56 m^3/hr

Reservoir Construction Costs ($a(QD)^b$) as per Chapter 8

a5052.293 R/m^3

b0.71109

Interest Rate 12%

Inflation Rate 6%

Loan Period 30 yr

Cost of Water Loss (C_L) R3,50/m^3

Time span of simulation 100 years

The cost of loss (C_L) is made up of two components (i.e. $C_2 + C_4$ as illustrated in Par. 7.3.2.4). The first is the cost of the loss of sales by the supply authority. For Rand Water it is about R1,00/m^3. The second component is the loss incurred by the consumers as a result of the outage. A value of R2,50/m^3 was assumed as this results in an optimum storage of 12 hours, which agrees with current practice and details on this cost are not available. Note that to simulate the operation of a reservoir system for 100 years time series will take about 40 minutes on a Pentium 166 to run. The process of simulation runs the expected emergency duration calculations resulting in production of the relationship between the Probability Density Function and Total Exceedance in hours per year is illustrated in Fig. 7.7.

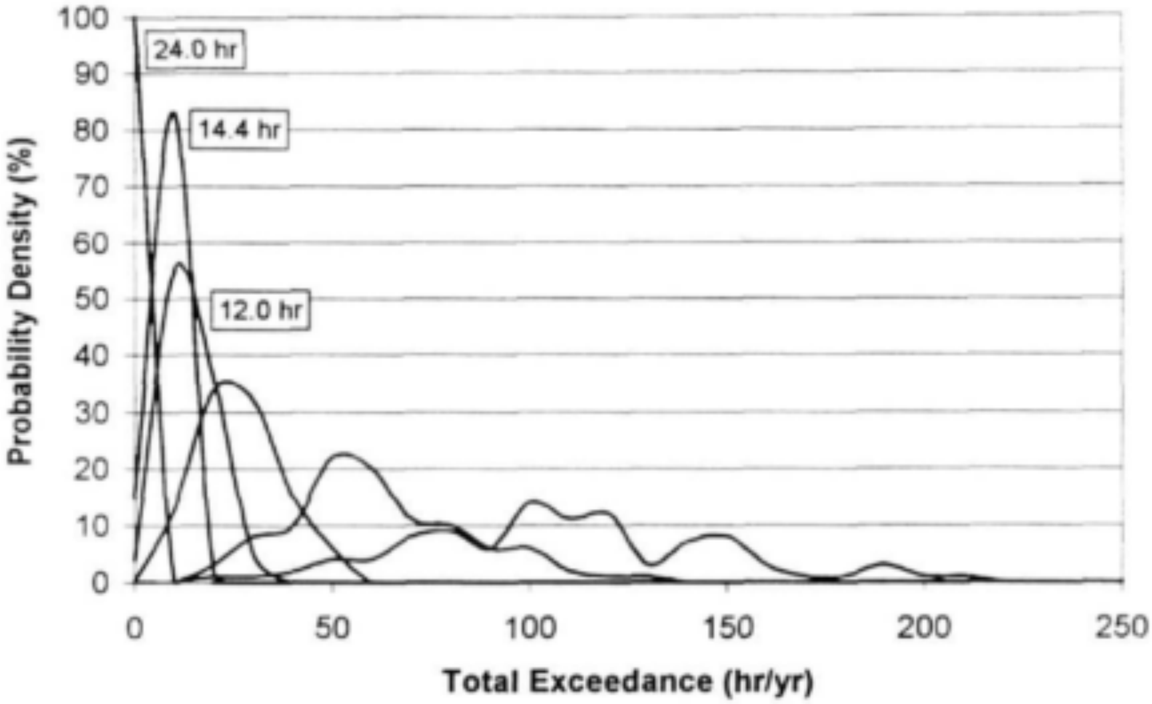


Figure 7.7: Probability density versus total exceedance of emergencies

The results of spreadsheet EM_TRI.XLS representing the total annual cost versus emergency storage provided are illustrated in Fig. 7.8.

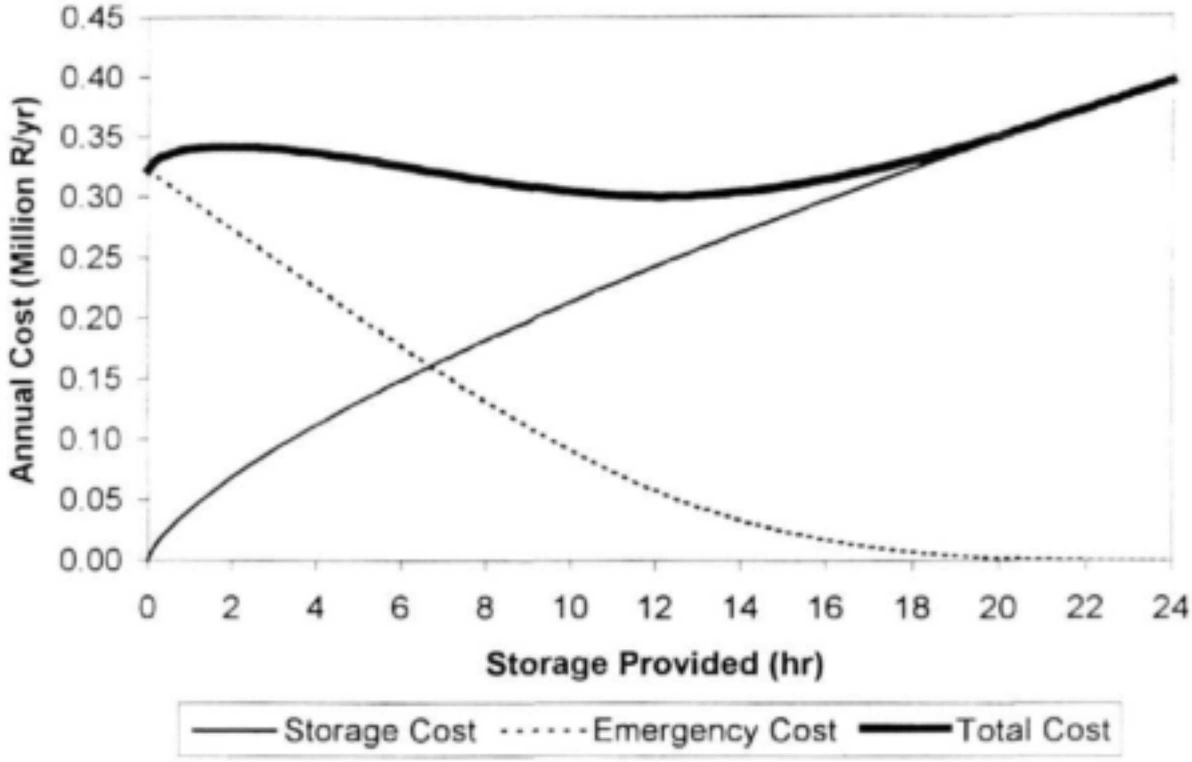


Figure 7.8: The relationship between the annual storage costs and storage provided

Fig. 7.8 illustrates that if the number of emergencies per annum, or the expected duration of the emergencies, or the cost of loss is high enough, it is possible to find an economically optimum volume of storage. In many cases, however, the expected cost of emergencies is fairly low, and the economical optimum volume of storage is zero. A system with no emergency storage will have a reliability of zero per cent. Such is usually not acceptable in urban water supply. However, this analysis leads to an aspect of the cost of reliability with regard to the annual cost of emergency storage in a system with balancing reservoir. This can be illustrated on the further step of this analysis in Fig. 7.9 which shows the economic optimum storage at 12 hours, and a reliability of 40 per cent at an annual cost of R300 000. If, for example, the water authority prefers a reliability at 90 per cent this would mean increasing the emergency storage to 19,2 hours at a cost of R340 000 per annum. It would then be up to the consumers, or a financing party for the balancing reservoir, to decide if the extra reliability is worth the extra costs.

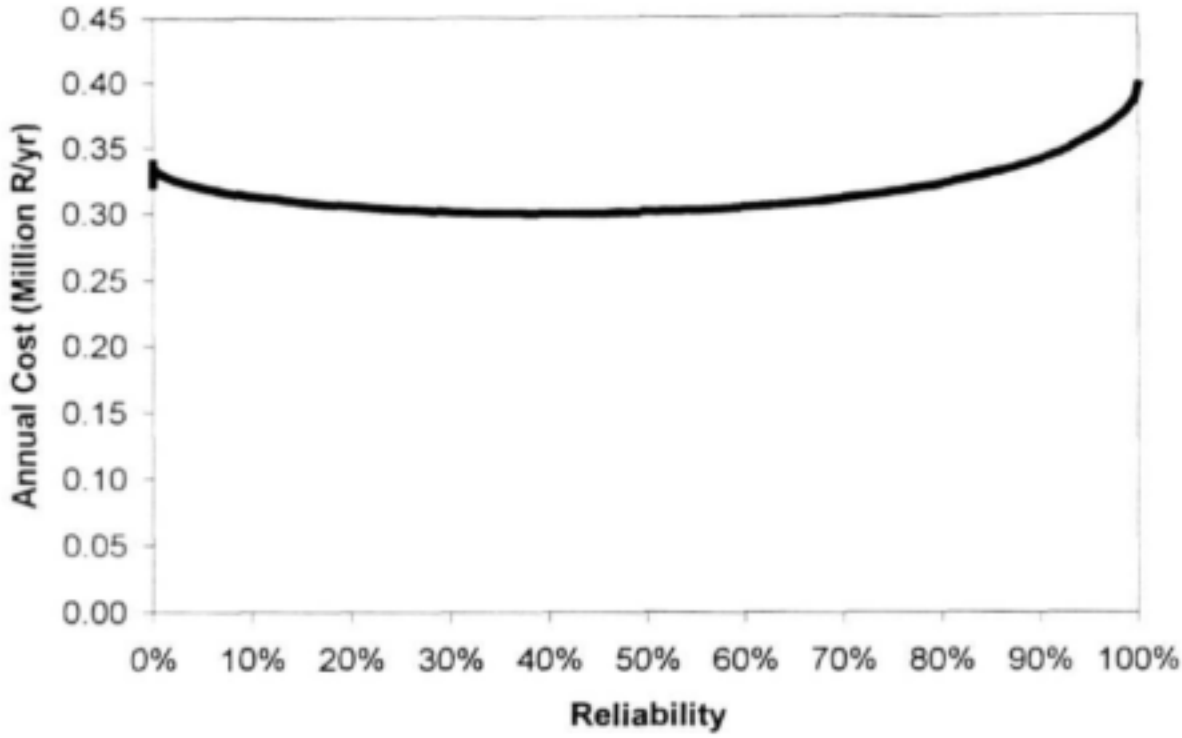


Figure 7.9: Relationship of reliability cost versus annual cost of emergency storage

7.3.3. Optimal fire-fighting storage component sizing

As stated previously, in South Africa, the distribution storage reservoirs (i.e. primarily municipal reservoirs) have to be provided with a fire-fighting storage component. The fire-fighting reservoir storage described in paragraph 7.1.2.4 of this report analysed in detail the current local (i.e. South African) water requirement standards for fire-fighting according to the risk categories. The current fire-fighting design criteria were scrutinised by van Zyl and Haarhoff (1997) on the background of historic real situation in "high risk" and "low risk" areas of Johannesburg metro (i.e. urban area within Rand Water limits of supply) and a small town, Kriel, respectively. Their objective was to determine the imbalance between the level of service and costs of providing an excess of water supply, primarily for the public water supply systems where the fire-fighting storage component is the only source of fire-fighting water with regard to the rate of flow, volume and pressures. The case study of the Johannesburg metro area is illustrated in Fig. 7.10 below.

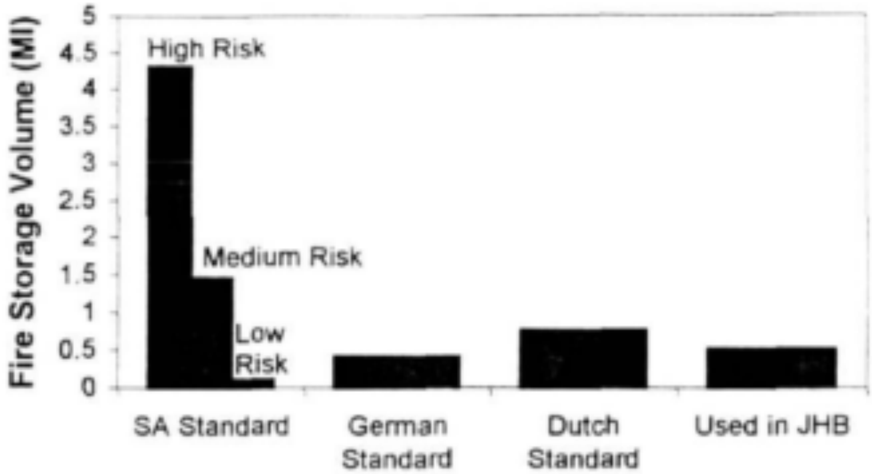


Figure 7.10: S.A. and international fire-fighting storage volumes compared to historic storage's used in Johannesburg area between 1980 and 1991 (after van Zyl and Haarhoff)

The comparison of historical average fire-fighting volumes used in the Johannesburg metro (i.e. high risk area) as illustrated in Figure 7.7 on the background of current S.A. and international standards, indicates that over more than one decade not even S.A. medium risk standards were exceeded. This leads to a strong impression that there is a large fire storage capacity under-utilised most of the time in most of the existing reservoirs. However, the data based compiled and analysed for only one (even most important) urban high risk area, will have to be extended into other major urban areas to verify further possible substantial relaxation of S.A. fire-fighting storage reservoir requirements. On the other hand, there is no doubt that the S.A. fire-fighting reservoir storage requirements if compared to the international requirements are far too high. There is a definite opportunity to lower the S.A. high risk fire storage requirement to say current medium level (e.g. to 2,0 Ml instead of 4,3 Ml in the case of the high risk category).

Further to their findings of quantitative assessment of fire-fighting storage requirements, van Zyl and Haarhoff (1997) analysed the cost sensitivity versus the fire storage volumes.

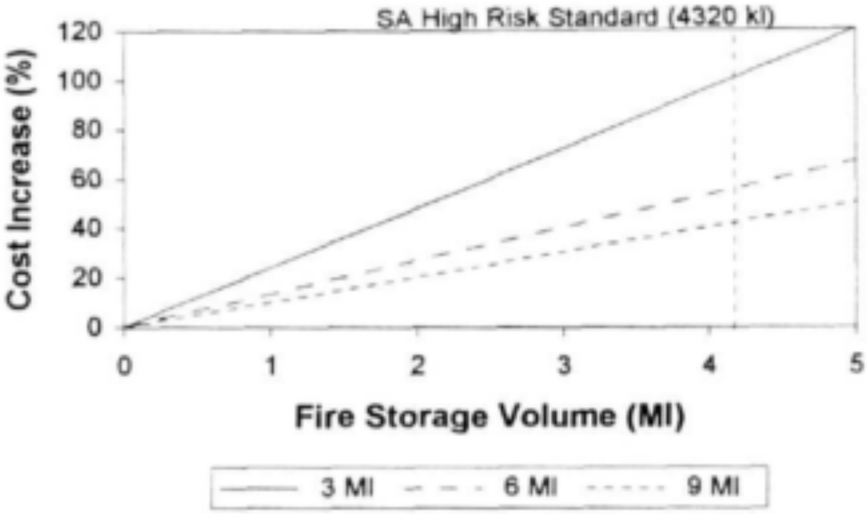


Figure 7.11: Cost increases for provision of fire storage in various sizes of distribution reservoirs (after van Zyl and Haarhoff)

The total balancing storage of a distribution reservoir (i.e. commonly municipal reservoir) is determined by adding the fire-fighting storage component over and above the sum of the demand emergency and operational storage components. The cost of fire-fighting storage component in comparison to the costs of other components of balancing reservoir situated in the fire high risk area is excessive, particularly for smaller size reservoirs.

Example 7.4: Cost implications from fire storage installation

Two distribution reservoirs are to be built in the fire high risk urban area. If the current design criteria as illustrated in Table 7.2 and reservoir costs (at 1996 prices) given in Figure 8.4 are adopted, the local water distribution authority will have to invest capital into the specific storage components as follows:

Table 7.11: Cost analysis for balancing reservoir including fire fighting storage component

Storage component	Reservoir size and costs (1996)			
	10 Ml		32 Ml	
	(R * million)	(%)	(R * million)	(%)
Operational freeboard	0,120	4	0,360	6
Emergency storage	0,920	34	3,060	51
Demand storage	0,460	17	1,560	26
Fire-fighting	1,120	41	0,780	13
Bottom storage	0,080	3	0,240	4
Total storage	2,700	100	6,000	100

The cost implications of an excessive fire high risk provision particularly in the case of smaller reservoirs (i.e. mainly below 10 Ml) are obvious. As the local water distribution authority (i.e. municipality or water board) is obliged to follow the current design criteria the overall decision has to attend to the advantages and disadvantages to build in a large fire-fighting reservoir storage against rather excessive capital investments which might not be necessarily approved by the consumers if they have a full opportunity to influence such decision. The advantages and disadvantages for a large fire storage are as follows:

Advantages:
- (i) better protection of life and property
- (ii) reduced fire losses
- (iii) reduced insurance premiums for customers.

Disadvantages:
- (i) uneconomical sizes of pipelines and storage capacity
- (ii) under-utilisation of capital investment
- (iii) possible stagnation of stored water in a system during low demand periods
- (iv) increased risk of water contamination and health hazard due to aspect as per (iii).

According to Hofer (1979) the actual size of fire-fighting component in already existing water supply systems is difficult to determine. If considered that adequate volumes of fire-fighting water supply must be available at all times, changing conditions within the supply area and ageing supply infrastructure are to influence availability of the fire-fighting water in existing systems. The conditions of a water supply system which can influence determinations of the fire-fighting component in an existing supply and distribution system can be summarised as follows:

- (i) densification extent of supply area
- (ii) changes in an urban area with regard to type of structures and materials
- (iii) enlargement of existing supply area
- (iv) construction of numerous high buildings
- (v) accumulation of deposits in pipelines and subsequent reduction in capacity of a system
- (vi) increases in pipeline leaks and bursts due to pressure increases and ageing of infrastructure
- (vii) negligence in maintenance and operational readiness of water supply system individual components (e.g. partially closed valves, etc.)
- (viii) changes or negligence of standards for residual fire pressures and/or fire hydrant spacing

The most critical scenario for the balancing reservoir is when the demand, emergency and fire-fighting requirements are combined together. Such a scenario can be simulated by the Monte Carlo simulation method, which gained in recent years popularity in replicating of real engineering systems. Nel and Haarhof (1996) adopted this method for an analysis of the Power Park (i.e. Rand Water limits of supply) storage reservoir, using the effect of the combination of the frequency of emergencies in the probability of evaluation of reservoir storage size. This approach is based on repetitive calculations of the water supply system performance, taking into consideration different combinations of input parameters at every step performed. The results are illustrated in Fig. 7.11 below.

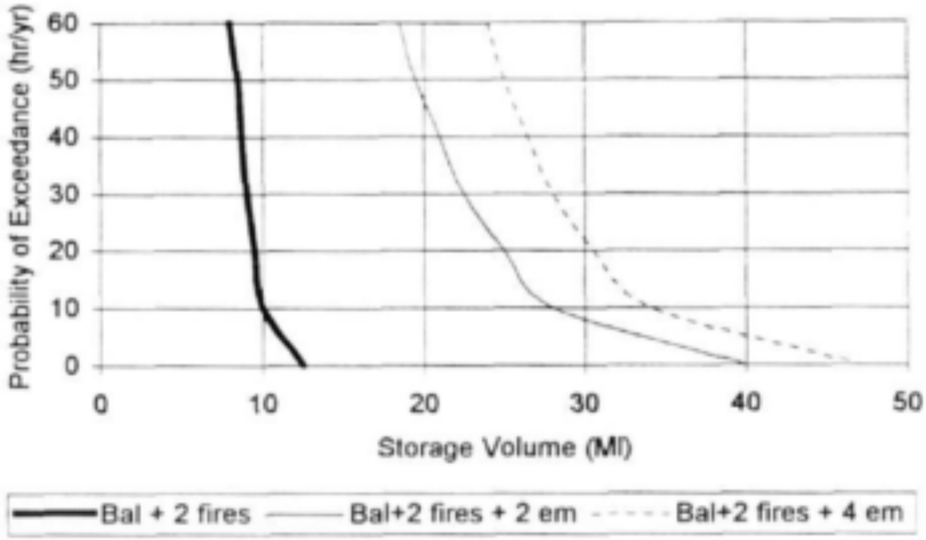


Figure 7.12: Monte Carlo analysis of the Power Park storage reservoir illustrating various emergencies (after Nel and Haarhoff, 1996)

This approach eliminates the limitation of single historic demand records by generating the likely historic sequences from specified or estimated parameters. This would be particularly important for the developing supply. The method does not evaluate the optimum storage size.

7.4. Risk and reliability in water supply with balancing storage

The classic criteria used in evaluating the cost and benefits of a water supply system are being gradually replaced in a system performance analysis. The criteria against which the satisfactory or unsatisfactory performance of a supply system is more frequently measured in recent years are summarised by Kaczmarek *et al.* (1996) as the risk in water supply, reliability, resilience, vulnerability or robustness of a water supply system.

All other concepts besides the reliability concept, as e.g. resilience, are used interchangeably with the reliability of a supply system. The concept of resilience has been adopted by Helling (1985) primarily in the environmental management sphere to describe systems that can survive intensive stress.

Resilience = Prob (S/F) = 1 - Prob (F/F)

where: S means "success" and

F means "failure" in the preceding time period

The reliability concept adopted in the optimal design of a pumped supply system with built-in balancing storage can ensure a reasonable trade off between the costs and efficiency. Morgan and Goulter (1985) established that if the design methods do not consider reliability constraints, the system's efficiency cannot be guaranteed under various critical demand loads (e.g. pipe burst or pump station failure). Billington and Allan (1984) defined. Reliability of a water supply system as the ability of such system to accomplish its required mission satisfactorily without a failure for a prior-defined unit of time, when it is subjected to an external stress.

The reliability of supply (i.e. water or energy system) is also defined in terms of required measures that reflect the magnitude, duration and/or frequency of service interruptions and supply shortfalls. An assessment of reliability for a water supply system involves stochastic events. The forefront research in this direction involves the cases of stochastic optimal control with regard to the constraint penalty functions which are to be minimised in order to ensure their feasibility.

Risk in water supply is a combination of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequence of the occurrence. The cost of risk is the product of the impact of uncertainty and its frequency. The increase in reliability while decreasing a risk of failure of water supply systems can be achieved by the size of capital expenditure. The principle is illustrated in Fig. 7.12.

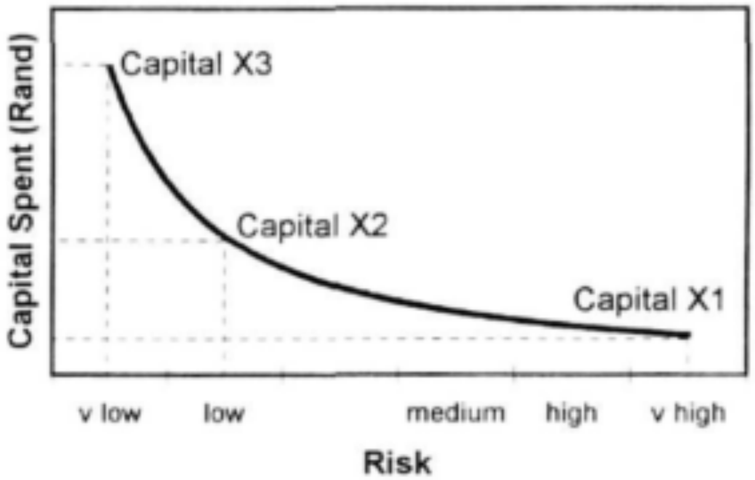


Figure 7.13: Capital spending vs. risk reduction

The principle outlined in Fig. 7.12 can be illustrated on an example of the risk reduction by preventively spent capital by Rand Water whose annual maintenance expenditure is about R1,5 million. This is particularly in the application of protective coatings and cathodic protection to the steel pipeline network. Fig. 7.13 indicates a significant decrease in the occurrence of pipeline leaks and holes and it can be assumed that the capital spent results in manageable number of emergencies.

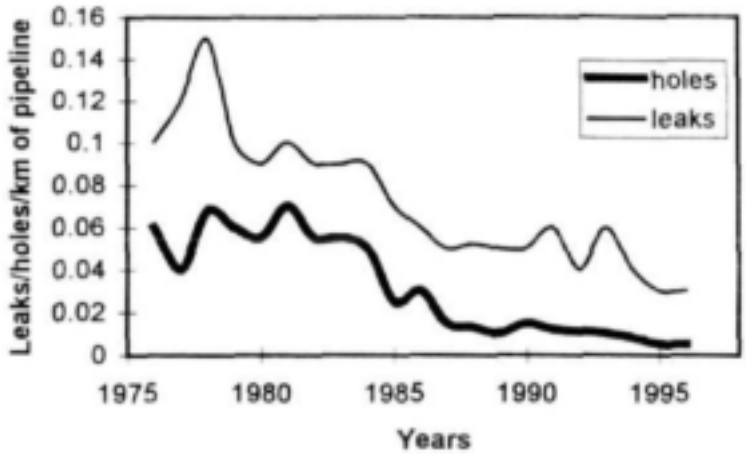


Figure 7.14: Rand Water historic trend reduction in pipeline failures

The risk in water supply system analysis is a measure of the probability of occurrence of various events and consequential loss of a specific hazard.

NET BENEFITS FROM RISK REDUCTION = BENEFITS FROM RISK REDUCTION -COSTS ASSOCIATED WITH INSTALLATION, OPERATION AND MAINTENANCE + COST OF DEALING WITH THE UNCONTROLLED RISK.

Risk control measures are essentially as follows:

(i) duplication and interconnections of pipelines (widely adopted for the strategic pipelines as described in Par. 7.1.2.2)

(ii) extension of life-span for various system components (e.g. cathodic protection and internal protection of pipeline.

(iii) proactive replacement of likely faulty components (e.g. pipe renewal every set period of years, etc.)

(iv) damage limitation (e.g. reliable detection programme, automatic closure, etc.)

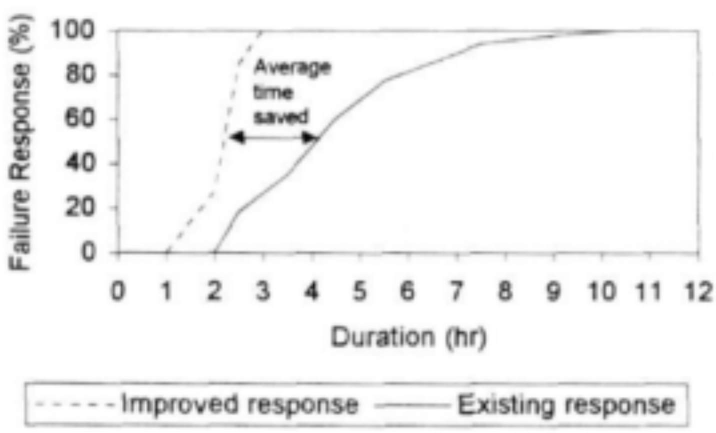


Figure 7.15: Damage limitation by early isolation of pipeline failure (after Kingham *et al.*, 1994)

Germanopulos *et al.* (1986) and Warren *et al.* (1989) recommended to analyse risk of a system failure based on an assumption that the probability of occurrence of failure events is represented by a Poisson distribution and the probability of the duration of the failure event is described by an exponential distribution. A combination of these distributions gives the probability of occurrence of failure events of duration greater at time t^* is represented as follows:

Prob($t > t^*$ for n failures in time T) = $\dfrac{(T/m)^n \exp[-T/m]}{n!}$ where: t = duration of a failure event

$m = \mu/\exp[-(t^* - r)/dr]$ is a mean period of time between failure events of duration greater than t^*

μ = expected period of time between successive failure occurrences

r = period of time between failure occurrence and beginning of repair

dr = expected duration of repair

Example 7.5: Reliability of supply measured as risk of failure.

The water supply utility identified and analysed water non-supply events for a period of recent 10 years to determine at overall reliability of their supply system. The most critical failure events formed the basis of probability analysis. It is assumed that it is possible to express level of reliability of supply in terms of acceptable probability of failure. The scenario of probability distribution of critical emergencies comprised the following:

(a) to isolate a pipe burst on the 600 mm diameter main near the one of two systems balancing reservoirs.

(b) at the same time the pumping station situated at the additional borehole water source is out of service for maintenance.

(c) the whole supply system is depending on one reservoir only with remaining storage of 6 hours of the Peak Daily Demand.

(d) it is estimated that to isolate the burst can take between 2 and 4 hours. However the full restoration of supply to consumers can take up to 36 hours.

(e) the utility management decided from the outset of burst alarm to persuade with restoration of pumping from boreholes.

(f) From the analysis of most critical failure events, it is estimated that the frequency of a burst of this type can occur twice in 10 years. The mean time to repair the 600 mm diameter pipeline is assumed to be 5 hours (i.e. dr = 5). The time to mobilise operation of pumping from boreholes may take 2 hours (i.e. r = 2). The probabilities of n failures ranging from 0 to 3 in a 10 year period for varying values of t^* are compiled in Fig. 7.15.

The acceptable probability of a given number of failures in the time period (T = 10 years) is referred to as the level of service (or reliability of supply system). Subsequently the utility considered to be acceptable the following:

(a) to have a 95 per cent probability that no emergency occurs in 10 years which exceeds the available storage, i.e. n = 0 and a t^* of about 21 hours is necessary.

(b) to have 95 per cent probability that no more than one event which can exceed the available storage represents t^* at 11 hours.

(c) or that supply failure of 5 hours would be acceptable under the same conditions if the required reliability of supply can be essentially reduced to 6 hours.

An illustration of the method for probability distribution of a water supply system failure is given in Fig. 7.15 below.

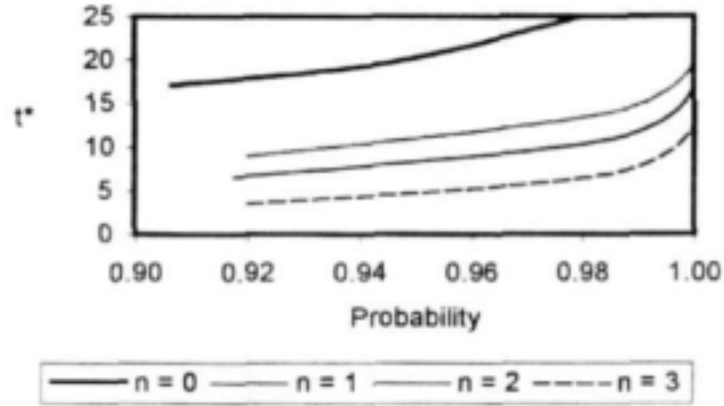


Figure 7.16: Illustration of t^* versus probability ($t>t^*$ for n failures in 10 years)

From the previous illustrations and example, it is obvious that the overall reliability of a water supply system can be measured in terms of risk of failure of vital components of such a system.

System Reliability (R_s) = 1 - F_s,

where F_s = probability of failure of the system

The water supply system failure is an inability of such systems to supply the desired flow rates at adequate pressures. The probability assessment can be conducted from the surveys of available incidents and failure reports.

Two types of likely failures are usually considered:

(a) hydraulic failure (i.e. an inability of a system due to the failure of e.g. a pump, pipe or valve)

(b) demand failure (i.e. a failure to provide adequate pressures due to excessive consumer demand).

Ormsbee and Kessler (1990) characterised the overall reliability in terms of three different levels:

(a) component reliability (i.e. the probability of a particular system's component remaining in operation over a specified time period)

(b) topologic reliability (i.e. the probability that the sub-system depending on a system remains physically connected over a specific period of time)

(c) hydraulic reliability (i.e. the probability of a system being able to supply a fixed set of flow rates at adequate pressures over a specified period of time).

Four major stages constitute the methodology for the optimal design of reliable water supply systems:

Stage 1: To formulate an optimal design problem of a water supply system under a number of loadings (i.e. demand patterns) in which the objective function is minimisation of system cost. The constraints are on the continuity of flow and energy, pressure heads and length of each pipeline.

The decision variables are the vector flows in all pipes for each loading condition, pumping heads for each pumping station and loading condition, the pipe segment lengths and the maximum power of each pumping station.

Stage 2: To identify the backup subsystems which will maintain a prescribed level of service under each of the loadings when a failure occurs. A backup is a subset of links of the full system. Two backups can be defined such that if any one link in the full system fails, then one of these two backs "survives", i.e. remains intact. In order to consider a lower probability of occurrence, two backups are usually required.

Stage 3: The hydraulic laws and consumer demands are formulated separately for each of the backups and loading conditions, to define for each the level of service required.

Stage 4: The models for the backups are added to the model of the complete system for the optimisation model to find the solution.

The probabilities of failure for various components of a complete water supply system are listed in Table 7.12 below.

Table 7.12: Probabilities of failure for various components of a complex water supply system (after Gray and Powell, 1988)

System component	Type of failure	Failure probability
PIPE/SIPHON	Pipe burst: Cast iron Steel	1/2.2 yr 1/100 to 1/300 yr.
VALVE	Burst valve Leaking valve Jammed gate	1/40 yr 1/5 yr 1/10 yr
PIPE CROSSING	Severe leak Pipe/joint burst Deformation of structure	less than 1/200 yr 1/200 to 1/300 yr 1/500 to 1/1000 yr
CONDUIT	Full structural failure Partial structural failure Damage to roof	1/50 to 1/1000 yr 1/50 to 1/500 yr less than 1/50 yr
TUNNEL	Full collapse Partial collapse Severe cracking	1/200 to 1/1000 yr 1/50 to 1/500 yr less than 1/50 yr

Table 7.12 above as well as Tables nos. 7.2, 7.4, 7.5, 7.7, 7.8 and 7.9 will enable a designer to estimate the first hand design parameters and possible failure probabilities. The actual inputs into the Reservoir System Design Optimisation model have to be obtained from surveys and analysing a specific water supply system.

8. COSTING FUNCTIONS FOR WATER SUPPLY OPTIMIZATION

8.1. Costing of water supply system design

The objectives in the designing process of a new or enhanced water supply system are related primarily to determination of the lowest initial cost, lowest running cost and the longest working life of the whole system and its components.

It is common that the costing during the conceptual design of a water supply system often excludes the costs for operation and maintenance. The initial development cost is usually taken as a function of the design capacity only. The shortage of information on costing functions for individual components of the supply system is usually the major reasoning for such simplification. This cannot be accepted in optimising of the supply system as it may lead to an erroneous decision being taken.

The development cost of a sizeable trunk pumping water supply scheme excluding the costs of the water treatment and distribution network might be as much as 40 to 50 per cent of the overall capital cost of the entire scheme. This would mean that it is essential during the costing of a supply system design that the costing functions for optimisation purposes are based on as wide an information base as possible. Such is not necessarily a common practice in real-life design situations.

Traditionally, the construction unit costs are retained in confidentiality by the construction specialists primarily for tendering purposes in a highly competitive market. The designer will usually adopt most current or past escalated market costs, providing that these will be made available to him by the relevant specialist, into the optimisation procedure commonly without determining the cost sensitivity characteristics. Generally there is little costing data available through public domain channels for optimisation purposes of water supply systems. For example, the shortage is particularly acute for components such as pumping stations and booster stations.

8.2. Whole Life Costing (WLC) of water supply system

In principle, the Whole Life Costing (WLC) of a supply system represents not only the initial costs and energy consumption during the operation, but also the down-time due to premature failure and planned maintenance. In adopting the WLC approach, the consideration is given to the aspects as follows:

(i) selection procedure of energy consuming components:

this refers to an extra margin of 10 per cent on account of electrical motor or the pump impeller diameter, both margins are taking the pump further away from the best pump efficiency duty.

(ii) effect of "wear and tear": which refers to the deterioration process due to unavoidable wear of some components (i.e. pumps, motors, etc.).

(iii) marginal differentiation within a system: commonly the designer of the system will tend to keep rounding up to the next standard pipe size or adopt margins in pump selection etc.; the total effect is likely to be wider margins between the anticipated performance and actual performance on account of higher energy consumption.

(iv) on-site performance: this means that in some cases the site performance of a component deviates from the test performance with an effect of higher energy consumption.

(v) operating procedure: marginal energy losses are also caused by frequent shut-down and re-starting sequences.

From the host of constraints listed above, the capital cost optimisation program can attend practically only the aspects of "wear and tear" and marginal differentiation within a water supply system design.

From the literature available on the WLC, it may be suggested that if all aspects of the WLC approach are implemented, the long-term energy savings can reach about 10 per cent for any given pumping water supply system.

8.3. Determination of costing data for this project

The cost functions for various key components of a water supply system are usually determined from the sets of cost data represented by the best curve fitting. Such costing information can be obtained from analysis of developed or projects presently being constructed.

The functions can have the linear, quadratic or exponential form and can be expressed as a function of unit design capacity, diameter, pumping rate, etc. The least square method is usually used to estimate the equation constraints. The strongest correlation of costing data sets, more refined costing functions are available for optimisation procedure.

The prime source of costing information for this project were the Department of Water Affairs and Forestry, Rand Water and several consulting firms situated in Gauteng. The SA Federation of Civil Contractors made available sizeable extracts from their costing data base on civil water-related projects constructed between 1993 and 1996. The information related mainly to pipeline and storage reservoir projects and unfortunately in most cases the total cost only was provided without other essential qualifying information (e.g. diameter for pipelines or storage size for the reservoirs). Some of the data available came in use in the determination of the reservoir costing function coefficients.

8.4. Costing functions for optimisation program

The total annual cost of the trunk water supply system can be obtained by summing the cost functions for fixed and variable costs. The annual costs for various system components can be determined as the product of the initial investment and the capital recovery factor which is a function of the rate of interest and the expected life of the installation. Operational costs are mainly associated with the pumping system and are functions of power consumption and associated labour. The annual maintenance costs apply to all system components and are usually taken as a percentage of the total cost over the economic life of the installation.

In the UK and USA, the work and research in determining the costing functions is undertaken by specialised units either within public domain research centres or consulting engineers are funding specifically to determine empirical functions relevant to the local conditions where they can be adapted. In the absence of such facilities in South Africa, the designer must determine cost functions from data available for his specific purposes. The following functions and cost coefficients were adopted in the design of the Reservoir System Design Optimisation (RSDO) program.

8.4.1. Pipe cost functions: New pipeline

$$\text{Cost(Rand)} = a * [\text{Length(m)}]^b * [\text{Diameter(m)}]^c * [\text{Thickness(m)}]^d$$

The cost related variables a, b, c and d are determined from the costing data for the relevant pipe parameters. Suggested average values for a = 153216,86; b = 1,0; c = 0,785431 and d = 0,808436 (1996 prices).

Figs. 8.1 and 8.2 illustrate graphically costing information on steel and asbestos standard size pipes available in South Africa.

The overall breakdown of pipeline installation cost is as follows: P&G - 20%; Supply - 30%; Construction - 20%; Contingency - 16%; VAT - 14%

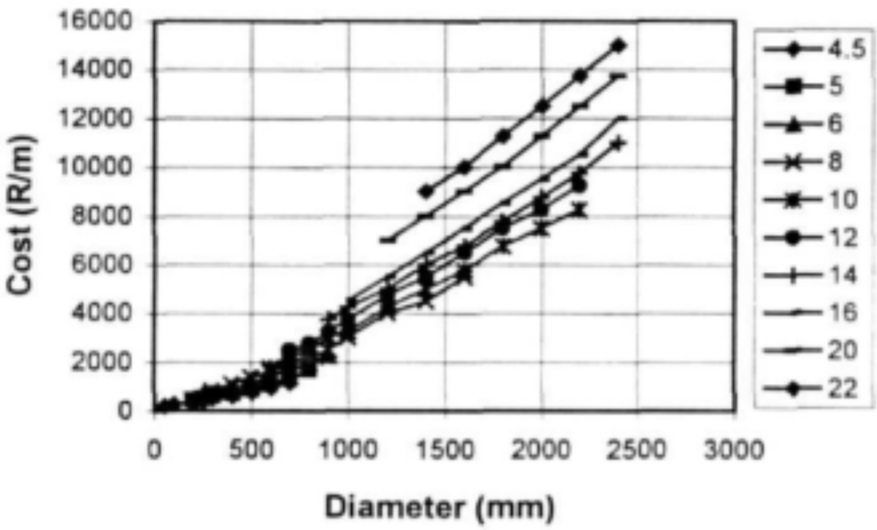


Figure 8.1: Steel pipe unit costs as a function of diameter and wall thickness

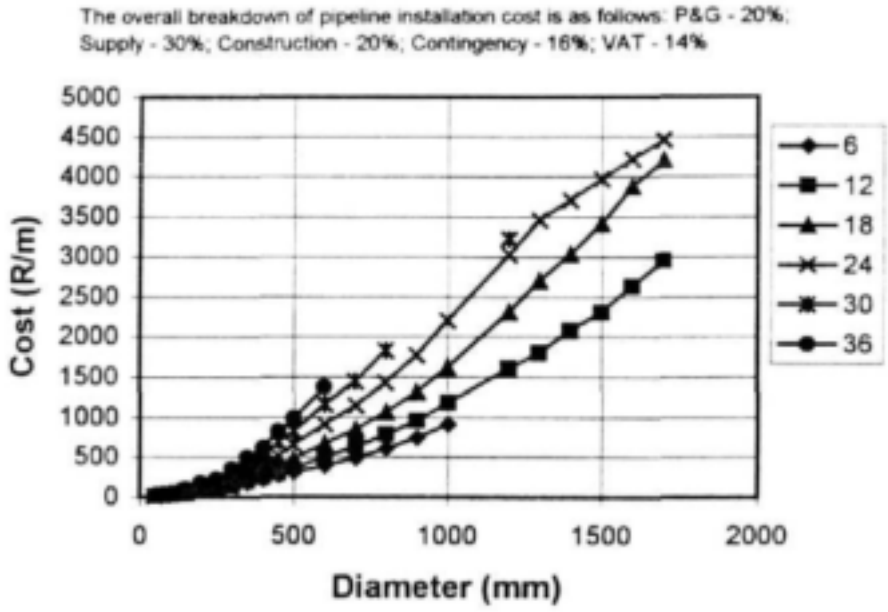


Figure 8.2: Unit cost of asbestos pipes as a function diameter and class

8.4.2. Pipe cost function: Refurbished pipeline

The enhancement of an existing water supply might require the designer to consider the alternative of cleaning and lining existing mains as compared to replacement or parallel installation of a new main.

Due to the decay of the water supply pipelines which can show itself in the increase of pipe breaks or malfunctioning of valves, and a general decrease of the system's capacity, the designer might consider to optimise an option with the costing functions related to the refurbishment of the existing pipeline instead of installing the new pipeline. However, the alternatives to replacement or refurbishment would be:

(i) to increase the system pressure by additional pump power or booster pumping;

(ii) if increased pumping will not work, a new pipeline parallel to the existing will most likely be installed.

The Rand Water has experimented in recent years with various methods in refurbishing existing pipelines to increase their capacity and extending their life. According to Trebicki (1996), the large pipeline diameters (up to 2400mm) can be successfully refurbished for less cost than direct replacement.

The steel pipeline is usually refurbished with external ring stiffeners and ± 10mm thick in-situ applied cement mortar lining or an epoxy lining. Another option is to apply a steel cylinder prestressed concrete pipe, which appears to be more expensive than the other two options.

The unit cost rates for the refurbishment of smaller diameter steel pipes are at 1996 prices as follows:

(a) dia. 350 to 600mm R200 per m^2 of pipe

(b) dia. over 600mm R150 per m^2 of pipe

Internationally, Walski (1985) and Walski et al. (1987) provide valuable research in refurbishment and rehabilitation of pipelines.

8.5. Pumping installation cost functions:

Each pumping installation comprises mechanical and electrical equipment which is housed within a civil structure.

8.5.1. Pump station capital cost function:

Cost (Rand) = aQH^b = a * [Power Capacity)kW)]^b

where: H = total manometric head in metres, (i.e. static head, friction loss and secondary losses),

Q = installed capacity in m^3/s (including standby capacity)

The costing variables a and b vary according to the pumping head. Suggested values (1994 prices) for quick reference follow:

(a) pumping head up to 30m: a = 650 000; b = 0,33;

(b) pumping head over 30m: a = 200 000; b = 0,65.

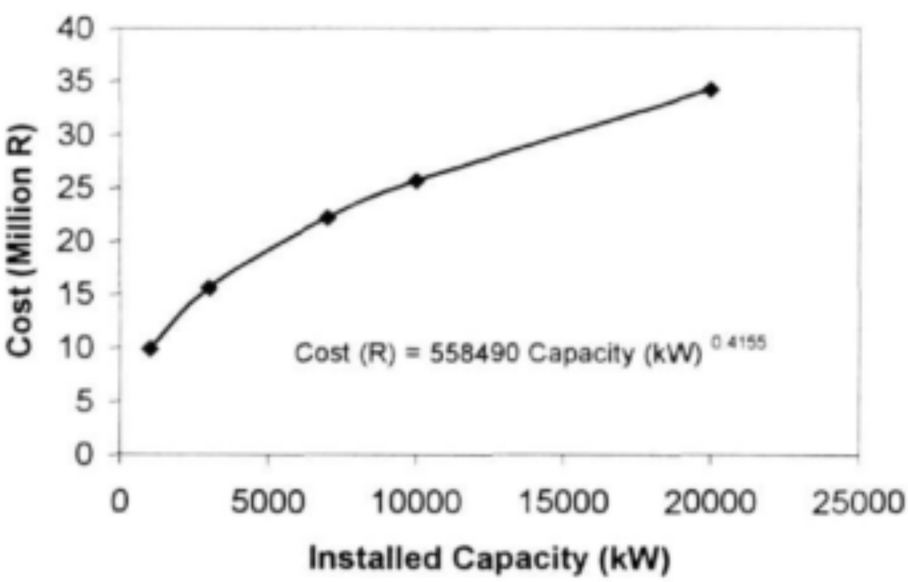


Figure 8.3: Cost of total pumping installed capacity (1996)

The specific input to the Reservoir System Design Optimisation Program (RSDO) can be determined from Fig. 8.3 according to the designer's choice.

The costing information compiled in Fig. 8.3 is based on analysis of fourteen pumping stations of various sizes. The total cost of the installed capacity includes the cost components for horizontal spindle pumps, electric motors, valves, connecting pipework, manifold piping, switchgear, transformers, sundry equipment, crane and civil structure.

To enable better orientation, an example is given to illustrate determination of the total installed capacity for costing data input into the program.

Example 8.1: Total installed pumping capacity

A pumping installation is required to pump water storage into a large reservoir with 200 000 cubic metres per day of ultimate demand. It is estimated that three duty pump units will satisfy the design capacity of 2,40 cubic metres per second, pumping against 180 metres of manometric head.

Hydraulic power per pump unit = $9{,}8 \times \frac{2.40}{3} \times 180 = 1411$ kW

Assuming 80 per cent overall efficiency for the pump and electric drive,

Pumping power per pump unit = 1411/0,80 = 1764 kW.

Add 10 per cent for electric motor rating (breakdown allowance) = 1764 x 1,10 = 1940 kW.

Standby capacity for 3 duty pumps is estimated at 33,3 per cent (1 stand by unit).

Total Installed Capacity = 1940 x 3 x 1,333 = 7760 kW (see Fig. 8.3 for cost information).

8.5.2. Pumping operation cost function

$$\text{Cost (Rand)} = \frac{\rho gQHTFY}{1000E_P}$$

where: T = number of hours of operation per annum,

F = energy cost per kWh,

Y = lifespan of the pump in year,

E_p = overall pump efficiency.

The suggested values of various parameters needed in determination of operating costs are listed below:

<u>Parameters for energy calculations</u>

(a) Efficiency of pump:	90%
(b) Efficiency of motor:	97%
(c) Maximum allowable lost in efficiency between major overhauls	3%
(d) Average loss in efficiency to be used for energy calculations	1,5%
(e) Overall efficiency:(0,90 × 0,97 × 1 × (1 - 0,015 x 100%) =	86,0%
(f) Power factor (MW/MVA)	0,96
(g) Unit Energy charges (see ESKOM charges)	c/kWh
(h) Peak Demand charges (see ESKOM charges)	R/kW

Note that a major overhaul is usually required every fifteen years or when efficiency has dropped by 3% for clean or settled water, and shorter periods of time for water containing silt, sand or other abrasive media.

Number of pump sets and standby capacity:

(a) Breakdown allowance: 10% per annum (resulting from power failures, pump/ motor/switchgear breakdown)

(b) Limitation on pump size: minimum 2 duty pumps plus 1 standby

Sizing of electrical motor:

(a) Up to 10 000 kW, operating at 11 kV;

(b) Safety factor: motor to be sized 10% in excess of calculated power absorbed by pump;

(c) Energy charge: between R0,10 and R0,15 per kWh + demand charge of R1,30/kWA/month.

Possible additions/cost components:

(a) power transmission lines from national grid;

(b) access roads;

(c) specific site conditions.

Note that these cost components are commonly considered to be site specific and for that reason not included in the cost functions for design.

8.6. Balancing reservoir storage cost functions

Cost (Rand) = a * [Volume(m^3)]^b

where V = volume of storage in m^3. The values of costing variables a and b are dependent primarily on the type of material used for the construction of manufacture of reservoirs.

The costing analysis of reinforced concrete round reservoirs based on the data from several sources (e.g. Rand Water, DWAF, SAFCC and various consultants) yielded the costing information given in Figure 8.4 and Table 8.1.

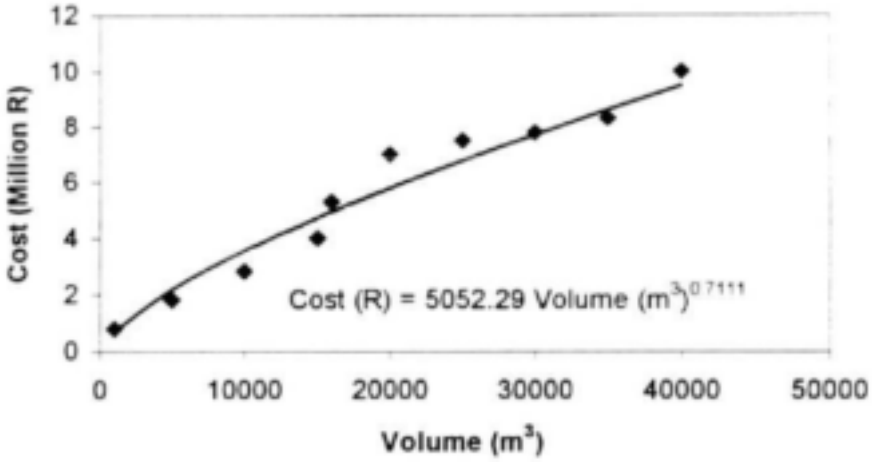


Figure 8.4: Reservoir storage costs

It must be noted that the costing data available from most information sources with the exception of Rand Water fall into the range of 5 to 45 Ml. The cost information on balancing storage obtained from the Rand Water refers to the reservoir storage sizes over 45 Ml and up to 50 Ml. For that reason, this data has been processed separately. The costs of storage over 45 Ml are indicated in Table 8.1.

Table 8.1: Cost data for large reservoirs

Storage size (Ml)	Cost (Rand Million)	Year completed
50	9,2	1997
120	26,7	1998
200	40,5	1997
400	66,3	1999
650	67,0	1996

8.7. Labour and maintenance costs

These costs can be expressed as follows:

(i) Labour costs: $L = l_1Q + l_2$;

where Q = design flow in Ml/day. The cost variables l_1 and l_2 are not readily available in South Africa and the labour cost component is assumed to be lump sum at present of ± R60 000 per annum per staff person.

(ii) maintenance costs: $M = m_1q + m_2$;

where q = actual daily flow in Ml/day ($q \leq Q_{design}$). The cost variables m_1 and m_2 are related to the type of materials replaced. These are not readily available in South Africa and annual maintenance costs are taken as a proportion of the total annual cost instead.

Table 8.2: Annual maintenance costs

Cost component	Economic life	Cost proportion (%)
Civil	45 years	0,25
Pipeline	45 years	0,50
Pump	30 years	4.0
Electrical/mechanical equipment	30 years	4.0

8.8. Formulation of the capital cost optimisation program

8.8.1. General description

The program calculates minimum costs related to transport and storage of water through near-to-optimal pipe size based on predetermined load factor.

8.8.2. Program user objectives

(a) meet demand within the limits of supply;

(b) minimisation of total or partial costs of investments;

(c) optimisation of operation design;

(d) improving the project viability and reliability of the supply system.

8.8.3. Program user limitations

(a) shortage of relevant input data;

(b) minimum and maximum velocities in the pipelines;

(c) commercial sizes and operation ranges of various components (e.g. pipes, pumps, etc.).

8.8.4. Applications for Capital Cost Optimisation Program

Parameters to be optimised:

(a) pipe diameters;

(b) balancing volumes of the reservoirs;

(c) pumping rates/operating factors of pumping stations;

(d) project life.

Introduction to Capital Cost Optimisation Program

The program accounts for the physical parameters of the supply system such as lift, pipe length and the economic factors such as cost of electricity, inflation, interest rates, etc. The optimal or most efficient cost for the operating system is calculated in comparison to the pump operating factor. The total cost of the system is the sum of the costs related to infrastructure, installation and energy required to pump water in that system.

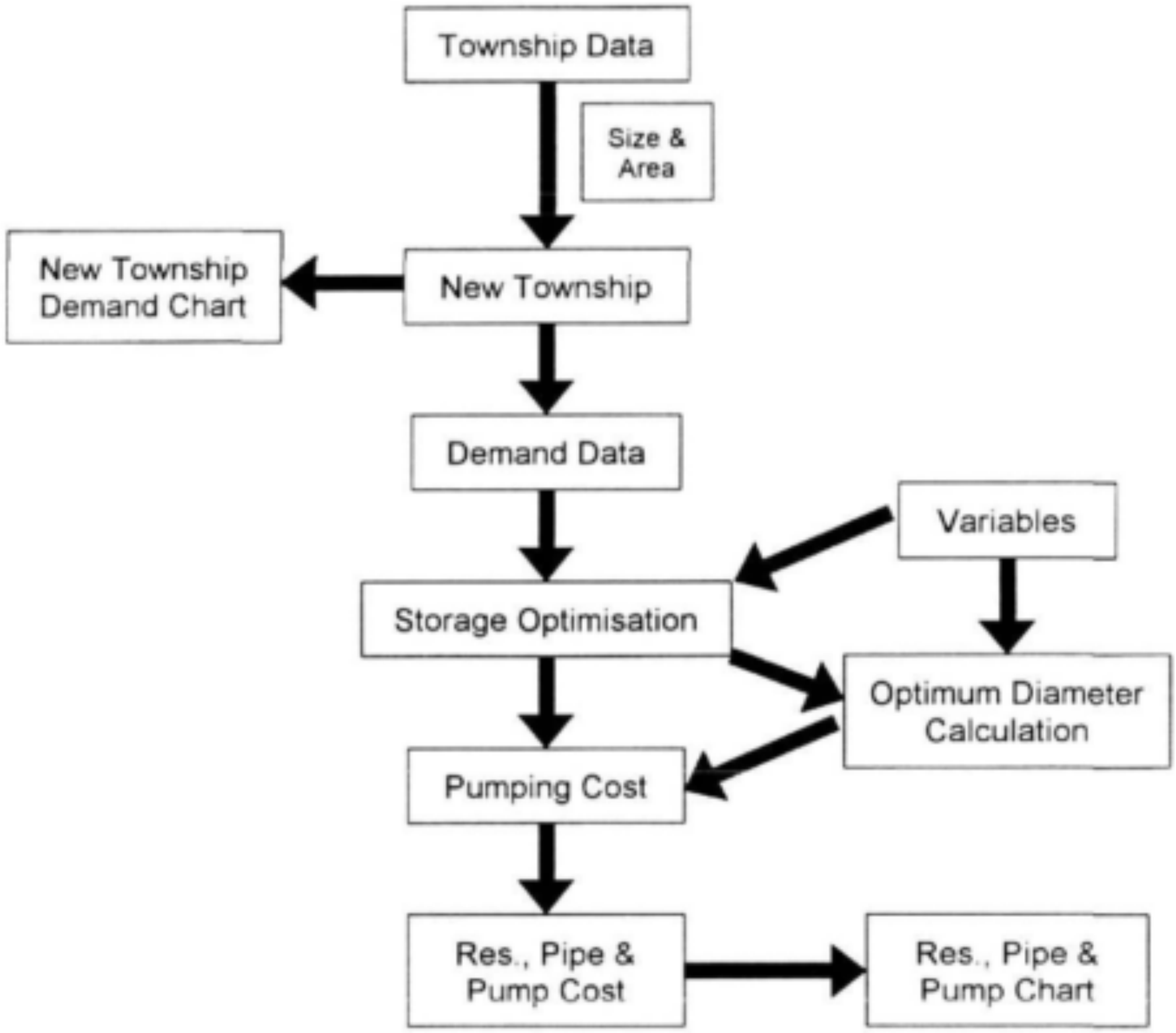


Figure 8.5: Flow chart of capital cost optimisation program

Optimisation Inputs vs. Outputs

Inputs	Outputs
System parameters	Final costs
Pumping rates	Operating factors
Pipe velocities	Optimal diameter
Demand data	Pipeline velocities
Furnish cost parameters	Reservoir volumes
Pumping cost)	Cost of * storage
Electrical cost)	* pipeline
Inflation rate)	* pump station
	* energy
Total design cost	

Cost escalation, interest rates and inflation

All historical costs used in costing analysis were escalated to the 1996 design time base. The Construction Year Deflector Index (YDI) has been adopted in the escalation procedure. Historically the interest rates and inflation in South Africa were recorded to be rather high in the light of an international comparison. However, South African economy is slowly moving into a position that would allow for an easing in monetary policies. See Appendix A for details on the YDI. Interest and inflation rates past and present are listed in Appendix A for the user's convenience.

8.8.5. Demand profiles for a supply area

Weekly demand profiles for the design of new or underdeveloped supply areas are used in calculating the demand (or balancing) storage component cost of water supply. The supply areas analysed have been categorised into residential, industrial or commercial, and can be differentiated by their characteristic demand patterns. By classifying the areas the user is afforded flexibility when integrating different supply zones and analysing a complex area.

8.8.6. Pump operating factor

The program calculates a total cost related to storage, pumping and the transport of water by pipeline for a particular pump operating factor. The cost for each cost component is optimised to find the minimum in relation to the other two, i.e. most efficient pipe diameter for cost of pumping at a certain discharge rate and pipe installation costs.

8.8.7. Program calculated outputs

Maximum and average demands are obtained from the demand profile, which gives the user a fairly accurate idea of the range of pumping rates required by the system. To run an analysis the user must determine a range of pumping rates which fall into the range of pumping rates calculated from the demand profile. From the user-defined range of pumping rates, the program will calculate:

(a) the operating factor for each pumping rate,

(b) balancing storage required for each pumping rate and the cost of that storage,

(c) the cost of pumping at each operating factor, and

(d) the cost of each pipeline using the most efficient pipe diameter.

The costs are then plotted against the operating factors and a relationship between the total cost and operating factors for that system can be determined.

The program allows the user to manipulate the demand patterns being analysed. A demand pattern for a new or underdeveloped area can be calculated by incorporating consumption data of another area that is perceived to have similar demand characteristics. The program uses proportions of size to calculate the increase or decrease in demand of the new area.

8.8.8. Reservoir System Design Optimisation (RSDO) software

A new supply area or the future demand for a developing supply area can be simulated by modelling the new area on an existing area. A new supply area uses demand data taken from an existing supply area for which the demand profiles are thought to be the same. The program allows for complex supply areas where there may be a combination of residential, commercial, industrial or other users. Each separate component of the supply area, i.e. residential or industrial may be modelled on existing areas. Increase in demand due to expansion or growth can be calculated by increasing the size of the supply area.

9. CONCLUSION AND RECOMMENDATIONS

The design development phase of a water supply system or its vital components is the phase in which the sizes and characteristics of each system's components are determined for a given system layout and capital cost.

The optimal design problem is to find the component characteristics (e.g. pipe diameter, pump heads and reservoir volumes) that minimise the total system cost. The capital cost optimisation program - Reservoir System Design Optimisation (RSDO Version 1.01) compiled at the Water Systems Research Group (WSRG), University of the Witwatersrand is a useful tool available to the water supply industry in South Africa. By means of this computer tool a rapid assessment of various cost optimisation scenarios involving the optimal sizing of the pumps, pipe diameter and balancing storage can be performed on a standard computer facility.

The Water Services Act of 1997, which replaced the previous Water Act, encourages a greater commercialisation of the South African water supply industry. Although at present the biggest risk for most water supply authorities operating in South African water supply industry is the inability of the third-tier water consumers (e.g. local authorities) to meet their liabilities in the event of non-payment by end water users, the new more advanced computer programs (as RSPO and RSDO) enable the water supply authorities in coping more efficiently with regard to pumping, pipeline and balancing storage design problems.

The capital cost optimisation program provides the individual designer with the costing model and the design parameters calculated from it, allowing thus for him to improve the current design methods and also to scrutinise the current design standards. The spheres of concern about application of the current design standards were identified in this research project as follows:

(a) the size, character or unique features of the supply area are only partially reflected in the guidelines by adopting a single value of the AADD for a supply area.

(b) the pre-specified trunk main pipe capacity into the storage reservoir does not allow for the optimum cost combination of supply pipe and storage size.

(c) the probability of water supply system failure due to the operational risks is not determined by the current guidelines (i.e. the inherent factors implied by the guidelines are not known).

(d) the reliability of supply concept inherent to the optimal design of a water supply by means of capital cost optimisation can ensure a reasonable trade off between the costs and efficiency, contrary to the applications of standard guidelines.

(e) the current design standards do not allow for the consumers or a financing party to decide if they are prepared to absorb the financial consequences with regard to various levels of supply reliability to be based on the economical optimum storage volumes for emergency, and particularly fire-fighting, purposes.

With respect to determination of optimal size of emergency storage (i.e. a specific component of total balancing reservoir storage) installed in most service and distribution reservoirs, the research concluded that the revision of current standards can be based on an assumption that it is possible to express level of reliability of water supply in terms of acceptable probability of water supply failure. The probability of occurrence of system failure events is determined by a Poisson probability distribution and the probability of the duration of the failure events is determined by an exponential distribution. However, the most critical failure events (or their combined scenarios) must form the basis of probability analysis for a particular supply system.

In view of the above identified constraints, various recommendations are proposed to be considered in design procedure and as a background for further research in water supply optimisation.

(a) As wide as possible publicity for application of available optimisation computer software, methods and techniques in water supply industry, taking into consideration the effects and requirements brought up by the new Water Services Act of 1997. This approach will also increase opportunities in saving pumping energy, water quantities and financial resources.

(b) To introduce more intensive and rationalised water supply data gathering and analysing by all water supply utilities to allow primarily for variations to standard design criteria to be tested and new available methods and techniques to be adopted.

(c) The water supply authorities must be made aware of high value of specific information as for example the pipeline failure, outages, refurbishment of pipelines, protective measures (i.e. cathodic protection) and all sorts of financial data for the optimisation analysis.

(d) The application of methods for forecasting of future water use in developing areas must take into consideration both deterministic demand component but most importantly the stochastic demand random events which influence the reliability and risks in water supply.

(e) More serious attention should be given by the designers to the Whole Life Costing (WLC) of supply systems. This means that not only the initial capital costs and energy consumption are considered in the original design, but also the down-time due to premature failure and planned maintenance. If all aspects of the WLC approach to design are implemented, the long-term energy savings can reach about 10 per cent for any given pumping water supply system.

REFERENCES

Basson S.M. and Grosman, D.D. (1985). "Stochastic modelling and risk analysis applied to urban reservoir design", SAICE.

Bligh, W.R. and Quazor, D. (1990). "Optimum design of a pumping station with daily variable inflow". Hydraulic Engineering Software Applications, Computational Mechanics Publications.

Cabrera, E. and Vela, A.F. (1995). "Improving Efficiency and Reliability in Water Distribution Systems", Water Science and Technology Library, Kluwer Academic Publishers.

Carstensen, J., Kjolby, M., Harremoës, P. and Madsen, H. (1996). "Interpretation of a pumping station as an on-line flow measurement", 7th International Conference on Urban Storm Drainage, Vol. III, Hannover, Germany, 1725-1730.

Casey, T.J. (1992). "Water and Wastewater Engineering Hydraulics", Oxford University Press, U.K., 187.

Caves, J.L. and Earl, T.C. (1980). "Computer Applications: A Tool for Water Distribution Engineering", An AWWA Management Resource Boo, 14-18.

Coetzee, P. and le Roux, P.J. (1993). "Case Study : water loss management in the Johannesburg CBD", Water Sewage and Effluent, Vol. 13, No. 3, 25-28.

Coulbeck, B. and Orr, C. (1988). "Computer Applications in Water Supply", Vol. 1, Systems Analysis and Simulation, Research Studies Press Ltd., U.K.

Coulbeck, B. and Orr, C. (1988). "Computer Applications in Water Supply", Vol. 2, Systems Optimisation and Control, Research Studies Press Ltd., U.K.

Cross, M. and Moscardini, A.O. (1985). "Learning the art of mathematical modelling". Ellis Horwood Series Mathematics and its applications.

Deb, A.K. (1978). "Optimisation in design of pumping systems". ASCE, Environmental Engineering Division, Vol. 104.

Edwards, G.T. (1996). "Betroubaarheid van Waternetwerke". RAU, S.A.

Eiselt, H.A., Pederzoli, G. and Sandblom, C.L. (1987). "Continuous Optimisation Models", Operations Research, Walter de Gruyter & Co., Berlin.

Featherstone, R.E. and El-Jumaily, K.K. (1983). "Optimal diameter selection for pipe networks", Journal of Hydraulic Engineering, Vol. 109, No. 2.

Germanopoulos, G., Jawitt, P.W. and Lumbers, J.P. (1986). "Assessing the reliability of supply and level of service for water distribution systems". Proc. Inst. Civ. Engrs., 413-428.

Greenberg, M.R. and Thordon, R.M. (1976). "Water Supply Planning, A Case Study and System Analysis", the State University of New Jersey.

Haarhoff, J. and van Zyl, K. (1997). "South African fire water guidelines and their impact on water supply system cost". SAICE Journal, First Quarter, S.A.

Harris, R.U. (1984). "Computer modelling in water system planning and design", Journal AWWA, Management and Operations, Jul, 78-81.

ICEC (1985). "New Technology in Water Services". Proceedings of a symposium organized by the Institution of Civil Engineers, Feb, London.

Johnsen, A. (1978). "Economies in water works operation". International Water Supply Association, 12th Congress in Kyoto, Japan, Oct.

Jowitt, P.W. and Lu, C. (1993). "Predicting pipe failure effects in water distribution networks", Journal of Water Resources Planning and Management, Vol. 119, No. 1, Jan/Feb, 18-31.

Kaczmarek, Z., Strzepek, K.M., Somlyódy, L. and Prizhinskaya, V. (1996). "Water Resources Management in the Face of Climatic/Hydrologic Uncertainties", Inter. Inst. for Applied Systems Analysis, Kluwer Publishers, 174-179.

Kingham, T.J. and Von Lany, P.H. (1994). "An approach to risk analysis and management of large water transfer pipelines", IWSA Yearbook 1994, U.K., 337-352.

Kinnersley, D.J. (1986). "Ensuring the operation and maintenance of water supply systems", Aqua, No. 3, 147-149.

Klemes, V. (1979). "Storage mass-curve analysis in a systems-analytic perspective". Water Resources, 15(2), 359-370.

Lambert, A. (1994). "Accounting for losses : The Bursts and Background Concept". Water and Environmental Management, IWEM. 1994 Annual Symposium, U.K.

Lamprecht, J.M. (1995). "Corrosion Control Practice at Rand Water", Rand Water Internal Report, Johannesburg, S.A.

Lauria, D.T. (1983). "Research Needs for Capacity Planning", Journal of American Water Works Association, 75 (1), 14-19.

Lotz, J.J.S., Stephenson, D. and Rodrigues, J.P.V. (1992). "Economic study of alternatives for boosting the capacity of a water pipeline". SAICE Journal, Vol. 35, No. 1.

Loubser, F.B. and Gessler, J. (1991). "Computer aided optimisation of water distribution networks". IMEISA, Vol

Loucks, D.P., Stedinger, J.R. and Haith, D.A. (1981). "Water Resource Systems Planning and Analysis", Cornell University.

Manson, N.J. (1996). "Optimising Water Supply and Distribution". Proceedings IAHR Southern African Division Biennial Congress, Sun City, SA.

Martin, Q.W. (1990). "Linear water supply pipeline capacity expansion model". Journal of Hydraulic Engineering, Vol. 116, No. 5, May, 675-689.

McCloud, R.J. and Lowndes, R.D. (1981). "Planning water supply systems", National Development, Apr, 36-38.

Meredith, D.D., Wong K.W., Woodhead, R.W. and Worman, R.H. (1985). "Design and Planning of Engineering Systems". Prentice-Hall.

Nel, D.T. (1996). "Sizing of Reservoirs", Report No. WR0016-01-00-0596, Johannesburg Administration, Water and Gas Department, Apr.

Nel, D.T. and Haarhoff, J. (1996). "Sizing municipal water storage tanks with Monte Carlo simulation", Aqua, Vol. 45, No. 4, 203-212.

Nitivattananon, V., Sadowski, E.C. and Quimpo R.G. (1996). "Optimisation of water supply system operation", Journal of Water Resources Planning and Management, Vol. 122, No. 5.

Ormsbee, L. and Kessler, A. (1990). "Optimal upgrading of hydraulic-Network reliability". Journal of Water Resources, Planning and Management, Vol. 16, No. 6, 784-785.

Ormsker, L.E. and Lansey, K.E. (1994). "Optimal control of water supply pumping systems", Journal of Water Resources, Planning and Management, Vol. 120, No. 2, Mar/Apr, 237-252.

Ostfeld, A. and Shamir, U. (1996). "Design of optimal reliable multi quality water supply systems". ASCE, Journal of Water Resources Planning and Management, Vol. 122, No. 5.

Pro-Aqua Systems (1994). "Pressure Dependent Leakage", World Water and Environmental Engineering, U.K., 10.

Rippl, W. (1883). "The capacity of storage reservoirs for water supply", minutes; Proc. Inst. of Civ. Eng., 270-278.

Saleet, M.H. and Helweg, O.J. (1990). "Peaking storage tanks for water supply systems", Water Resources Bulletin.

Sharp, B.B. (1985). "Economics of pumping and the utilisation factor". Journal of Hydraulic Engineering, Vol. 111, No. 11, Nov, 1386-1395.

Smook, A.W. (1985). "Determination of the optimum supply and storage capacities for municipal service reservoirs", SAICE, Aug, 429-438.

Stallard, K. (1980). "Economic Optimisation of the RWB System by Simulation Modelling". Rand Water Board Study, Johannesburg.

Stephenson, D. (1989). "Pipeline Design for Water Engineers". Developments in Water Science 40, 3rd revised and updated edition, Elsevier, Amsterdam.

Stephenson, D. (1995). "Factors affecting the cost of water supply to Gauteng". Water SA, Vol. 21, No. 4.

Teasdale, L.M. (1994). "Whole life costing of a pumped system", World Pumps, Special Feature, Oct, Elsevier Science Ltd.

Topping, R.E. and Puccia, R.J. (1984). "A computer assisted water system analysis and design for Charleston S.C.", Journal AWWA, Jul, 83-89.

Trebiki, R. (1997). Personal communication, Rand Water, Johannesburg.

Turner, K.N. and Manson, N.J. (1995). "Water Demand Patterns in Gauteng". Water Systems Research Group, WRC Report No. 488/1/95.

Twort, A.C., Hoathen, R.C. and Law, F.M. (1974). "Water Supply", 2nd edition.

Vorster, J., Geustyn, L., Loubser, E., Tanner, A. and Wall, K. (1995). "A strategy and master plan for water supply, storage and distribution in the East Rand region". SAICE, Vol. 37.

Wagner, J.M., Shamir, U. and Marks, D.H. (1988). "Water distribution reliability simulation methods", Journal of Water Resources, Planning and Management, Vol. 114, No. 3, 279-285.

Walsh, G.R. (1975). "Methods of optimisation", John Wiley & Sons.

Walski, T.M. (1985). "Cleaning and lining versus parallel mains", Journal of Water Resources, Planning and Management, Vol. 111, No. 1, 43-52.

Walski, T.M. (1987). "Water Supply Systems Rehabilitation", ASCE, New York.

Warren, R.P., Cook, S.C., Lumbers, J.P. and Fenner, P. (1989). "Optimization of water supply systems : risk analysis and reliability of supply", IWEM : 1988 Annual Symposium, Paper No. 7, U.K.

Water Practice Manual (1984). "Water Supply Systems, 4: Water Distribution Systems".

APPENDIX A: SUPPORTING INFORMATION

Construction Year Deflator Index (YDI)

Year	Construction Deflator Index	Construction Inflation	Year on Year
1946	5.97	3.00	
1947	6.52	3.27	9.26
1948	6.88	3.46	5.57
1949	7.20	3.61	4.60
1950	7.73	3.88	7.44
1951	8.89	4.46	14.90
1952	9.87	4.95	11.06
1953	9.72	4.88	-1.51
1954	9.41	4.72	-3.16
1955	9.66	4.85	2.65
1956	10.00	5.02	3.53
1957	10.04	5.04	0.33
1958	10.14	5.09	1.01
1959	10.31	5.18	1.74
1960	10.96	5.50	6.25
1961	10.98	5.51	0.18
1962	11.29	5.66	2.80
1963	11.62	5.83	2.96
1964	11.93	5.99	2.69
1965	12.58	6.32	5.45
1966	13.05	6.55	3.70
1967	13.75	6.90	5.37
1968	14.06	7.06	2.27
1969	14.18	7.12	0.82
1970	14.89	7.47	5.01
1971	15.58	7.82	4.66
1972	16.99	8.53	9.05
1973	19.14	9.61	12.64
1974	22.32	11.20	16.64
1975	26.77	13.44	19.93
1976	31.08	15.60	16.10
1977	34.46	17.30	10.88
1978	38.35	19.25	11.29
1979	44.76	22.47	16.70
1980	52.39	26.30	17.06
1981	60.30	30.27	15.10
1982	69.75	35.01	15.66
1983	79.88	40.10	14.53
1984	87.73	44.04	9.83
1985	100.01	50.20	13.99
1986	115.07	57.76	15.06
1987	130.52	65.52	13.43
1988	152.18	76.39	16.59
1989	175.58	88.14	15.38
1990	200.50	100.64	14.19
1991	226.50	113.70	12.97
1992	246.50	123.74	8.83
1993	267.51	134.28	8.52
1994	290.70	145.92	8.67
1995	313.95	157.60	8.00
1996	342.21	171.78	9.00

Current Cost = Base Cost × (Current Year Index/Base Year Index)

Consumer Price Inflation and Interest Rates Graphs

CONSUMER PRICE INFLATION

INTEREST RATES

Specimen of Questionnaire 1

QUESTIONNAIRE ON WATER SUPPLY SYSTEM OPTIMIZATION

(Please fill and/or tick where appropriate)

1. **Name** of water supply system or responsible authority ....
2. **Contact** person and address :..
Tel:............................ *Fax:*......................... *Email*..
3. **Sources of water:**......................... *own* ....*(%)*; *external*.........*(%)*; *other*.............*(%)*
4. **Type of supply system:** *single-source single reservoir:* .... ; *multi-source single reservoir:* ....; *multi-source multi-reservoir:* ....; *other:* ....
5. **Purpose of system's supply:** *domestic/commercial:* ....*(%)*; *industrial:* ....*(%)*; *mining:* ....*(%)*; *agricultural/stockwatering:* ....*(%)*; *other:* ....*(%)*.
6. **Total supplies of raw or treated water** *(Ml):* ....*(1995)*; ....*(1996)*.
7. **Total capacity of supply system:**......................... *(Ml/d)*; *treatment:* .... *(Ml/d)*; *pumping:* .... *(Ml/d)*; *losses:* ....*(%)*; *number of subsystems:* ....
8. **Total energy requirements:** .... *(MWh/a)*, *treatment:* .... *(kWh/a)*; *pumping:*................. *(kWh/a)*, *other:* .... *(kWh/a)*; *cost per unit:* ....*(R/kWh)*
9. Please describe **flow monitoring policy**: ....
....
10. **System configuration:** (*If possible, attach a schematic sketch or recent annual report*)
Number of pumpstations: .... ; *number of booster stations:* ....
type/number of pumps: fixed speed .... / .... ; *variable speed* .... / .... ;
variable throttle: .... / .... ; *other type:* .... / ....
Please describe **pumping policy**: ....
....
Total **reservoir capacity**: ....*(Ml)*; *number of reservoirs:* ....
Please describe **reservoir operation policy**: ....
....
Total **length of pipelines**: ....*(km)*; *min dia* : .... ; *max dia* : ....
11. Please describe briefly the **distribution subsystem(s)** (*e.g. population supplied, type of township i.e. low, medium or high income, peak demands experienced, losses, age of infrastructure, etc.*): ....
....
....
12. Please describe what **methods and techniques** are used in process of **demand assessment** for your supply system capacity expansion: ....
....
....
13. Please describe system's **infrastructural and/or operational constraints** approbating preoptimization or full scale optimization: ....
....
....
14. Please discuss **costing methods or functions** adopted by your authority in optimization processes: ....
....
....
15. Any **other comments**: ....
....

Specimen of Questionnaire 2

Fax: **Date:**

To: **Att:**

Subject: QUESTIONNAIRE ON WATER SUPPLY SYSTEM OPTIMIZATION

Dear Sirs

In addition to our questionnaire on Water Supply Optimization dated March 18, 1997 with anticipated return by May 15, 1997 we would like to ask you for further information as follows:

a) annual number of breakage's on pipeline of x size and pipe type (e.g. 35; 300mm; steel);
b) duration of outage (i.e. non-operational time) in hours per various pipe diameter and pipe type;
c) occurrence of breakage's (or outages) within various supply zones with regard to balancing reservoir;
d) cost of repair and maintenance to water supply authority;
e) cost of water based on quantities lost;
f) loss to consumer due to the outage (please indicate how important this aspect is to the supply authority and how the cost of loss can be estimated).

Our thanks to those who responded to our questionnaire; the data contribution will be used to serve nation-wide.

B. Barta **Water Systems Research Group, WITS**
Tel: 011-716 2516 Fax: 011-403 2062
Email: amanzi@civen.civil.wits.ac.za
Pages: 1 of 1

APPENDIX B: SOFTWARE USERS MANUAL

Reservoir System Design Optimisation

User Manual

Version 1.02
February 1998

Neil Manson
Water Systems Research Group
Private Bag 3
University of the Witwatersrand
2050
n-manson@civen.civil.wits.ac.za

1. INTRODUCTION

Reservoir System Design Optimisation, or *RSDO* for short, is a tool developed for designers of service and distribution reservoir systems. It allows the designer to calculate optimum capacities for the pumping, storage and pipeline components of distribution and service reservoir systems.

This software was developed as part of a project entitled "Development of a Model to be used for the Economic Optimisation of the Pumping and Design Policies of Reservoir Systems", which was funded by the Water Research Commission and conducted by B. Barta and N. Manson of the Water Systems Research Group, University of the Witwatersrand. This manual describes how to use the software, but details of the calculations used are given in Water Research Commission report No 757/2/98.

1.1. Reservoir Systems

Reservoir systems consist of a number of components, which each require capital costs to install or running cost to operate, or both. Optimising a reservoir system means finding the combination of capacities for each of the components that results in the lowest lifetime cost for the system. Figure 1 shows a schematic of the model used by *RSDO*.

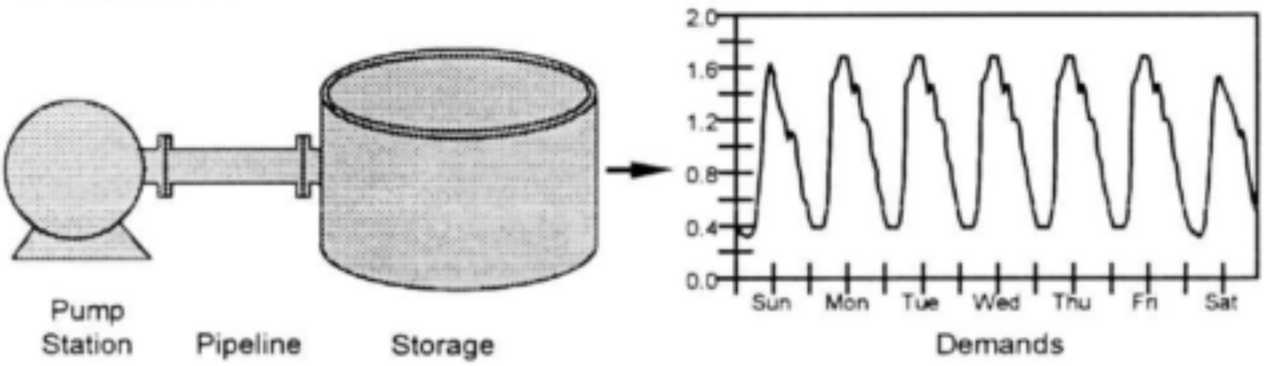


Figure 1 *RSDO* System Model

The cost of constructing the pump station and the pipeline are a function of the maximum possible pumping rate. The size of the reservoir, and therefore its installation cost is a function of the relationship between the maximum pumping rate and the maximum demand rate. If the pumping rate can meet any demand rate, then no storage would be required. If the maximum pumping rate is only equal to the average demand then a maximum amount of storage would be required. The size of the pipeline affects both the capital and the running costs. A large diameter pipeline would be expensive to install, but would have low frictional resistance. This means that the running costs for pumping would be lower.

The optimum capacities for each of the components depends on the relative costs involved and on the nature of the demand that must be met. *Reservoir System Design Optimisation* enables a designer to calculate these optimum capacities simply and quickly.

2. GETTING STARTED

The first part of this manual is written as a tutorial for you to follow. It is recommended that you work through the tutorial sections, and use the example files supplied, as this is the quickest way to see what the program can do, and how to use it.

2.1. System Requirements

RSDO was written for the Microsoft® Windows™ operating system, version 3.1. It will also run under Microsoft® Windows™ 3.11 and Microsoft® Windows95™. Any computer that is capable of running any of these operating systems will be able to run *RSDO*. At least a 486 processor is recommended as *RSDO* performs some intensive mathematical calculations that can take some time to complete. The faster the processor speed, the quicker these calculations will be completed.

2.2. Installing the Software

To install *Reservoir System Design Optimisation* you need to insert the program diskette into your 3½-inch 'Stiffy' drive. This will usually be the 'A:' drive, but it may be the 'B:' drive on your computer.

Follow the steps for the operating system you are using as given below:

<u>Microsoft® Windows™ 3.1x</u>

1. From *Program Manager's* menu, select **File** and then **Run**.
2. Type in 'A:\SETUP' and press ENTER.
3. The *Setup* program will then run and the Welcome Screen will be displayed.
4. Click on the NEXT button or press ENTER.
5. Select the drive and sub-directory in which you want *RSDO* to be installed. You can also type in a new sub-directory. Then click on the NEXT button or press ENTER. To accept the default directory of 'C:\RSDO', just click on the NEXT button or press ENTER.
6. You will then see a list of optional program components. If you do not want to install any of these components, click on them or press the SPACEBAR to deselect them. When only the options you want are selected, click on the NEXT button or press ENTER.
7. A confirmation screen will be displayed. Check that the options are what you want. If the are not, click on the BACK button to change the options. If the options are correct, click on the INSTALL button or press ENTER.
8. *Setup* will now copy some files to your hard drive and unpack the files. It will also install a new program group in *Program Manager*. When *Setup* is complete, click on the DONE button or press ENTER to exit from *Setup*. To start *RSDO*, double click on the *Res. Sys. Design Optimisation* program group in *Program Manager*, and then double click on the *Reservoir System Design Optimisation* item, which appears as .

Microsoft® Windows95™

1. Click on , and select **Run**.
2. Type in a:\setup.exe and press ENTER.
3. The *Setup* program will then run and the Welcome Screen will be displayed.
4. Click on the NEXT button or press ENTER.
5. Select the drive and sub-directory in which you want *RSDO* to be installed. You can also type in a new sub-directory. Then click on the NEXT button or press ENTER.
6. You will then see a list of optional program components. If you do not want to install any of these components, click on them or press the SPACEBAR to deselect them. When only the options you want are selected, click on the NEXT button or press ENTER.
7. A confirmation screen will be displayed. Check that the options are what you want. If the are not, click on the BACK button to change the options. If the options are correct, click on the INSTALL button or press ENTER.
8. *Setup* will now copy some files to your hard drive and unpack the files. It will also install an *RSDO* item in your start menu.
9. To run *RSDO* click on , select **Res. Sys. Design Optimisation** and then **Reservoir System Design Optimisation**.

2.3. How to use this Manual

If this is the first time you have used the program *Reservoir System Design Optimisation*, it is recommended that you read through the first three chapters of this manual, **SETTING UP A SYSTEM, ENTERING DESIGN INFORMATION** and **CALCULATING AN OPTIMUM DESIGN**. These chapters explain in a step-by-step manner how to use the program. You can follow along using the example data supplied with the program. The remaining two chapters, **MENU OPTIONS** and **RSDO FILES**, give reference information on all the available commands and the files used by *RSDO*.

2.4. Manual Conventions

This User Manual uses a number of conventions to assist in explaining the use of the Program *RSDO*.

Italics	Italic text will be used for program and data file names.
Bold	Bold text will be used to indicate menu options that the user needs to select.
SMALL CAPS	Small capital letters will be used to name any of the keys on the keyboard that the user needs to press, or for the names of buttons on the screen that the user should click on.
Double Underline	Double underlined text will be used to indicate cross references to other sections and chapters in this manual.

Now that you have installed the software, and know how to use this manual, let us proceed with optimising a design.

2.5. Disclaimer

This software and the associated data and files are supplied "as is". The Author, the Water Research Commission, the Water Systems Research Group and the University of the Witwatersrand cannot and do not guarantee that any functions contained in the Software will meet your requirements, or that its operations will be error free. The entire risk as to the Software performance or quality, or both, is solely with the user and not the Authors. You assume responsibility for the installation, use, and results obtained from the Software.

The Author makes no warranty, either implied or expressed, including with-out limitation any warranty with respect to this Software documented here, its quality, performance, or fitness for a particular purpose. In no event shall the Authors be liable to you for damages, whether direct or indirect, incidental, special, or consequential arising out the use of or any defect in the Software, even if the Authors have been advised of the possibility of such damages, or for any claim by any other party.

All other warranties of any kind, either express or implied, including but not limited to the implied warranties of merchantability and fitness for a particular purpose, are expressly excluded.

3. ENTERING DESIGN INFORMATION

When you start *RSDO*, you will see the following screen. You can perform actions using either the menus or the toolbar, and by clicking on each of the page tabs, you can display a different page of information.

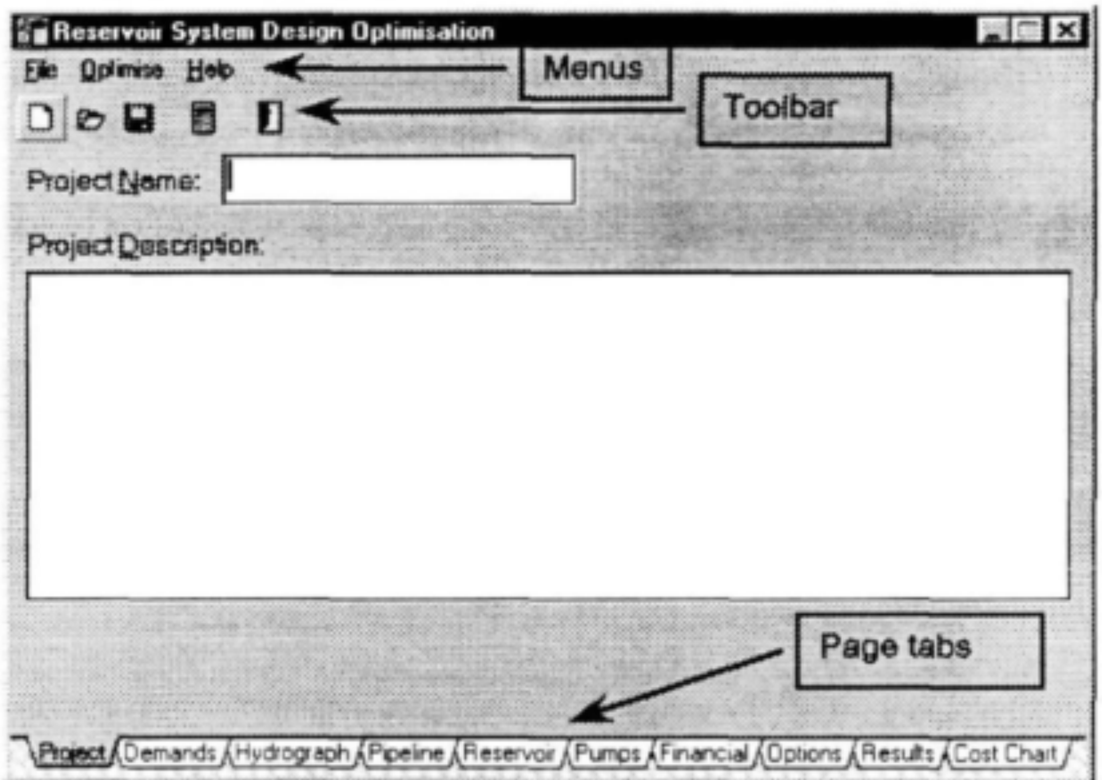


Figure 2 *RSDO* Screen Layout

3.1. Project Options

The page that is currently displayed is the Project Options page. On this page you can enter a project name and a project description. You can enter anything you want to in these fields. They are not used in the optimisation, but are simply for your reference. Figure 3 shows the Project Options page with some example data filled in.

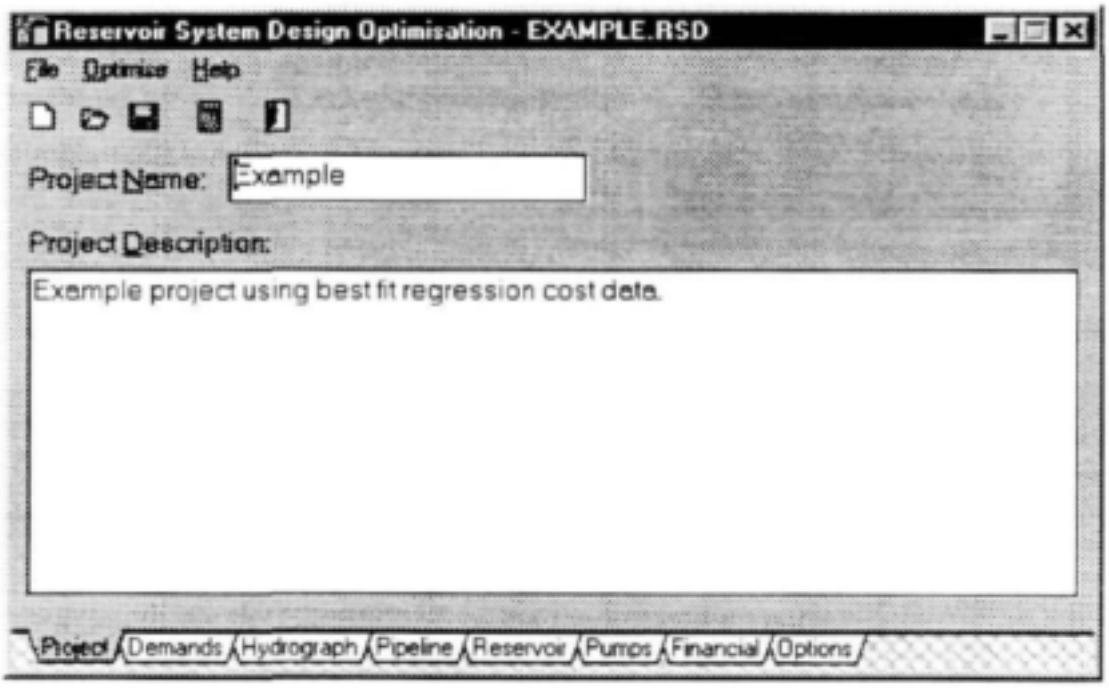


Figure 3 Project Options Page

The data shown on this screen, as with all the other screens, is contained in the *Example.RSD* file that was copied onto your hard drive when the program was installed.

It is a good idea to save your work regularly. To do this, choose **File|Save** from the menus. You can also click on the  toolbar button, or press CTRL+S on the keyboard. The first time you save a file, the Save As dialog will be displayed, as shown below:

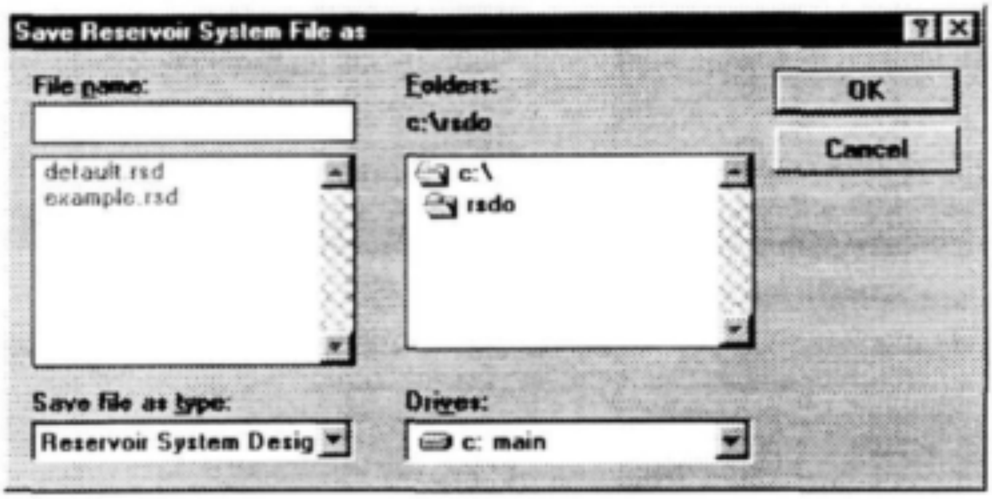


Figure 4 Save As Dialog

Using this dialog, you can select a drive and directory in which to save your file, and you can type in a file name. When you have entered a filename, click on the OK button. Your file will now be saved to the hard drive.

You can recall a previously saved file by choosing **File|Open** from the menus, clicking on the toolbar button, or pressing CTRL+O. An Open dialog, which is similar to the Save As dialog above, will be displayed. You can then select the file you want to open, or type in the filename.

If you would like to start a new project, you can choose **File|New** from the menus, click on the toolbar button, or press CTRL+N. This will clear the current data, and present you with a blank project in which you can enter new information.

You can also save the current data in a new file by choosing **File|Save As** from the menus.

When you have finished using *RSDO* you can exit by choosing **File|Exit**, clicking on the toolbar button, or by pressing ALT+F4.

3.2. Consumer Demand Data

Once you have entered the project options information you can click on the Page Tab labelled Demands, or press CTRL+PAGEDOWN to display the Consumer Demand Data page, as shown below.

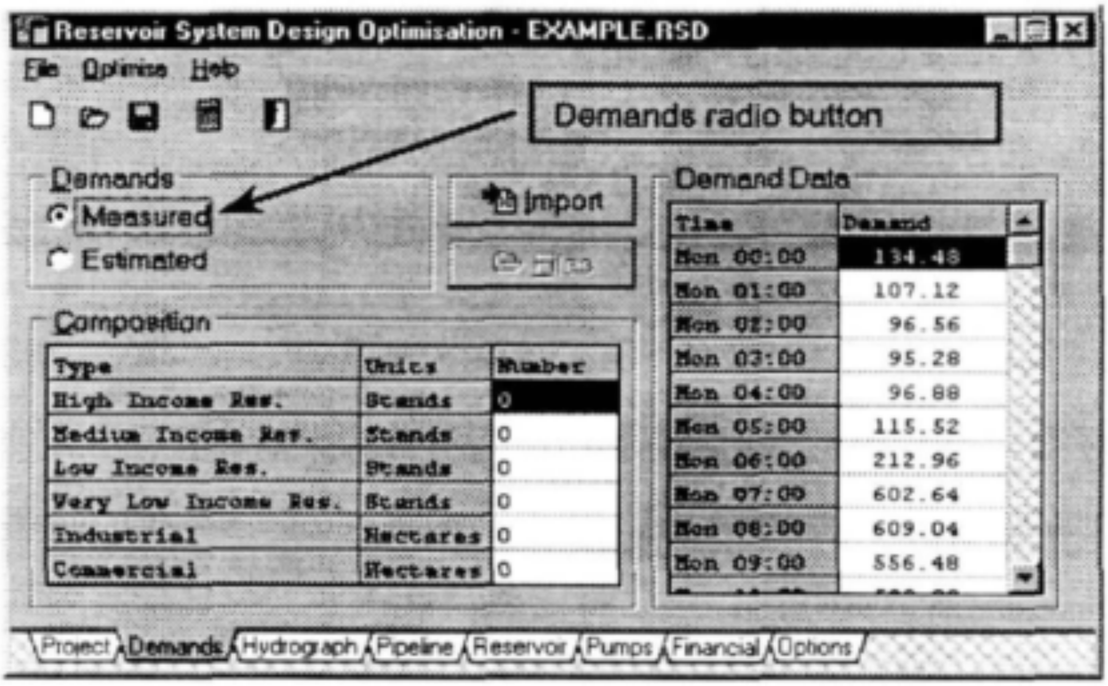


Figure 5 Consumer Demand Data Page

On this page you need to enter one week of hourly demand data. This information will be used to find the average and peak daily demand, and should be as representative of the consumers your system is intended to supply as possible. There are three ways to input this data, manually, importing it from a text file, and by estimation based on the composition of the area supplied.

To enter data manually, ensure that the Demands radio button is set to Measured, as shown above, and simply type in the values that you want into the Demand Data table.

To import data from a *Text Data File*, select the IMPORT button. A standard File Open dialog will be displayed, and you can select the *Text Data File* you would like to import. Details of the format that this *Text Data File* must be in are given in the section entitled Input Files later in this manual. This data would ideally come from flow monitoring done in the are that the reservoir system is intended to supply.

In many cases there is no measured demand data for a new reservoir system, so it is necessary to estimate the demand. *RSDO* allows you to do this based on the composition of the supplied area. Firstly, set the Demands radio button to Estimated, and then type in the number of stands or hectares of each type of consumer in the Composition table. *RSDO* calculates the consumer demands by summing the demands for each category of consumer, and displays the results in the Demand Data table.

You can also change the data that is used to estimate the demand data for each category by clicking on the FILES button. This will cause the Edit Data files dialog to be displayed, as shown below.

Figure 6 Edit Data Files Dialog

For each of the categories of consumer, you can click on the button to display a standard File Open dialog and select the *Category Data File* you would like to use. The required format of the *Category Data File* is given in the Input Files section of this manual. If you click on the OK button, these files will only be used for the current project. If you choose the SAVE AS DEFAULT button, they will be saved for use with each new project started.

3.3. The Demand Hydrograph

When you have entered the consumer demand data that you want to use, click on the Hydrograph page tab to see a graph of these demands. A typical graph is shown below. It is useful to see a chart of the demand data to check that the data is correct and is as you expect it to be.

You can export both the demand data and the chart for use in other programs as follows. To export the demand data to a text file, choose **File|Export|Demand Data** from the menus. A standard Save As dialog will be displayed which allows you to enter the name of the file in which you want to save the demand data. When you choose the OK button, the demand data will be exported to a tab delimited *Demand Text File*, which you could then import into a spreadsheet program for further processing.

You can also export the chart to a *Demand Chart File* by choosing **File|Export|Demand Chart** form the menus. This will save the chart in a *Windows Metafile*, which is a standard graphics file format that can be used by many different word processing and graphics programs.

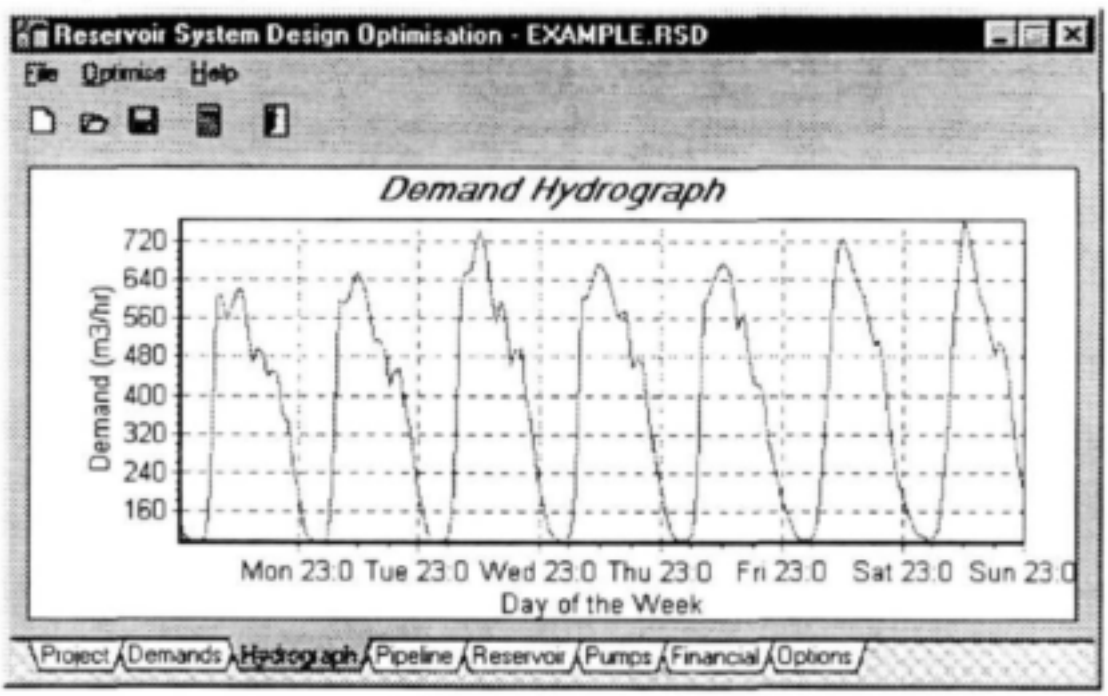


Figure 7 Typical Demand Hydrograph

3.4. Supply Pipeline Details

The next page is the Supply Pipeline Details page, where information about the pipeline between the pump station and the reservoir is entered. Typical information is shown in the figure below.

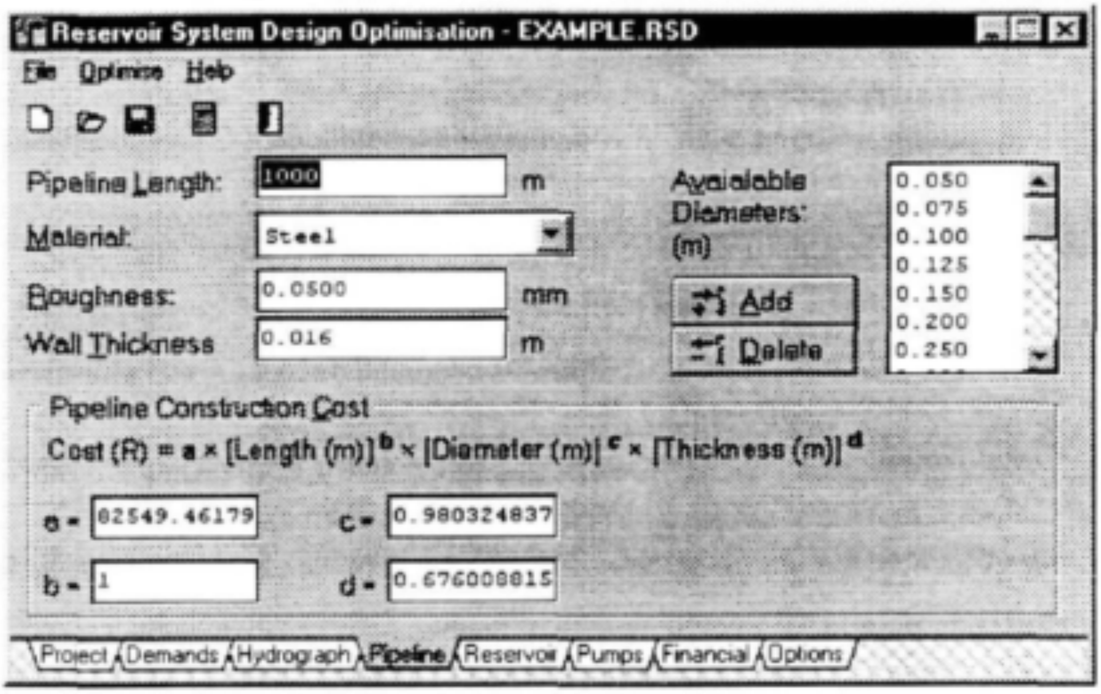


Figure 8 Pipeline Details Page

Type in the length of the pipeline in metres. Select a material by clicking on the ▾. If the material you want is not in the list, choose Custom. If you select a custom material you need to enter a roughness in millimetres. The wall thickness you enter will depend on the pressure in the pipe, the pipe material, and the thickness' available. It may be necessary to change this value a few times and recalculate the optimum in order to find the minimum thickness that will meet your pressure requirements.

Enter all the pipe diameters that are available in the material you have chosen in the Available Diameters list. Note that these must be the internal diameters and not the nominal diameters. Click on the ADD button to add a new diameter, or select a diameter and click on the DELETE button to remove it.

The last four fields contain the coefficients of the equation used by *RSDO* to calculate the cost of constructing the pipeline. These should be found by performing a regression analysis on historical costs of constructing pipelines. The costs used should be the all-inclusive costs of construction. It is very important that these costs are as accurate as possible as the optimum solution is very dependant on the balance of the costs of the various components.

3.5. Reservoir Details

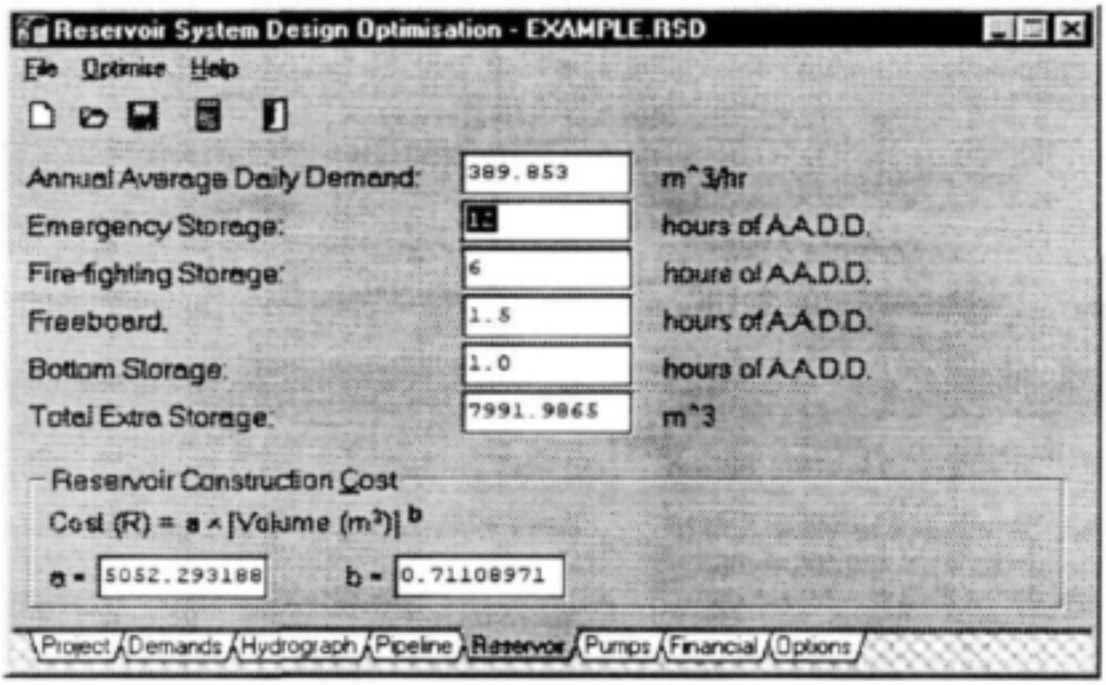


Figure 9 Reservoir Details Page

This page contains the information relating to the reservoir in the system. The in Water Research Commission report No 757/2/98, contains information on how to calculate the volume of storage required for emergencies and fire-fighting, or you can use your local regulations.

The operational freeboard and bottom storage are typically of the order of 1,0 to 1,5 hours of average annual daily demand, but need to be evaluated for each specific reservoir.

The coefficients in the cost equation will again need to found be regression analysis of historical reservoir construction costs.

3.6. Pump Station Details

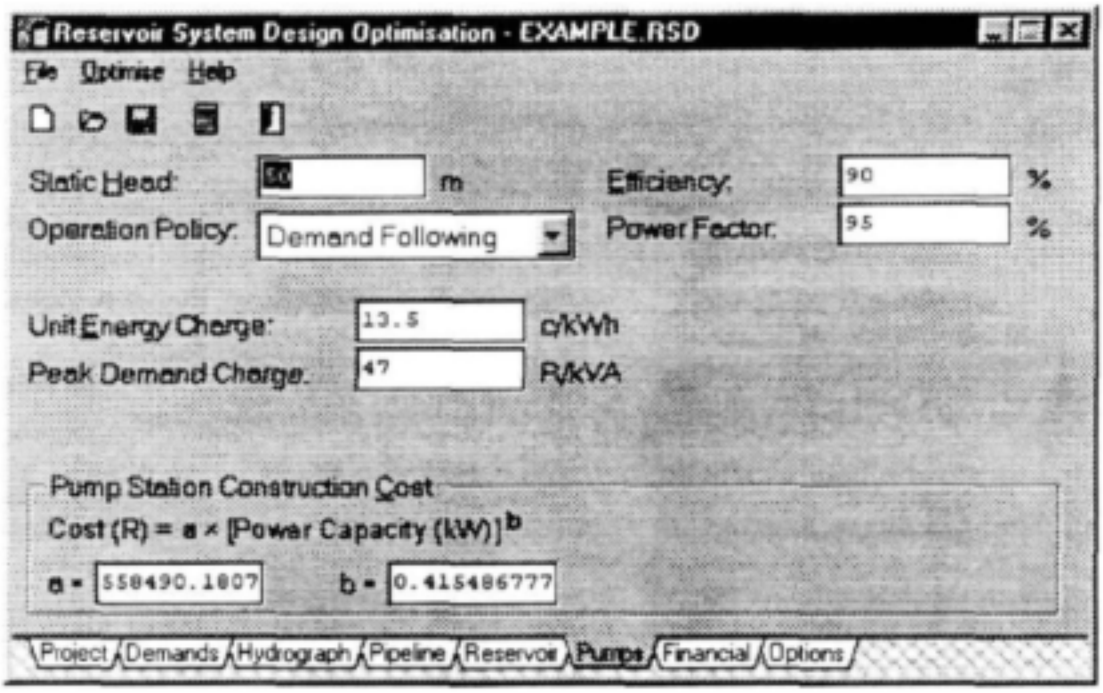


Figure 10 Pumping Details Page

On this page you need to enter the data used to calculate the cost of pumping and the cost of building a pump station. The information required is

- The static head, or difference in elevation between the pump station and the reservoir
- The combined pump and motor efficiency
- The pump Operation Policy. Select one of the available policies by clicking on . The Basic On/Off policy assumes that the pumps run at full capacity whenever the reservoir is less than 100% full, and do not run at all when the reservoir is full. The Demand Following policy assumes that the pumping rate is equal to the demand when the reservoir is full, and is a maximum when the reservoir is less than 100% full.
- The power factor is a function of the type of motors used to drive the pumps. If the driving force is not electrical motors then this value should be set to unity.
- The unit cost of power. This will usually be the cost of electricity used to drive the pumps, but if the pumps are going to be driven by an energy source other than electricity, the cost of that energy must be entered here.
- The peak demand tariff. Most electricity suppliers charge a peak tariff, but it may be set to zero if the energy source is not electricity.
- The coefficients of the regression equation for the all-inclusive cost of constructing the pump station.

3.7. Financial Information

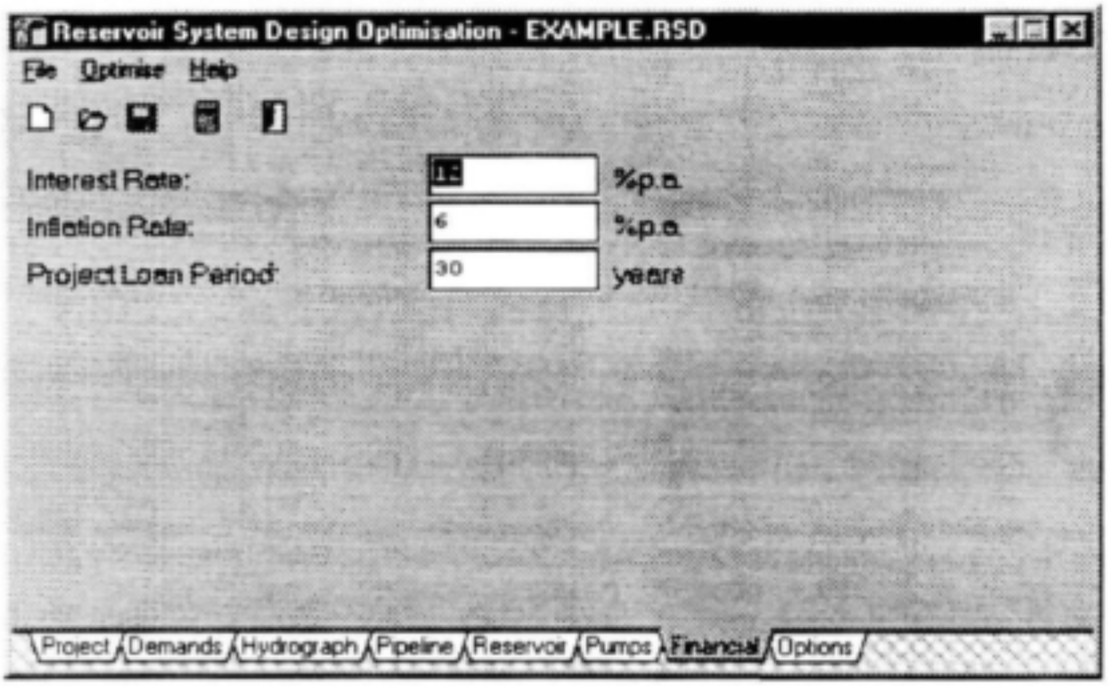


Figure 11 Financial Information Page

The information on this page assumes that a loan will be raised to fund the capital cost of constructing the reservoir system. The Interest Rate is the rate at which interest will have to be paid on the loan, and the Project Loan Period is the time, in years, over which the loan will have to be repaid. The inflation rate is an estimate of the inflation over the period of the loan.

3.8. *Optimisation Options*

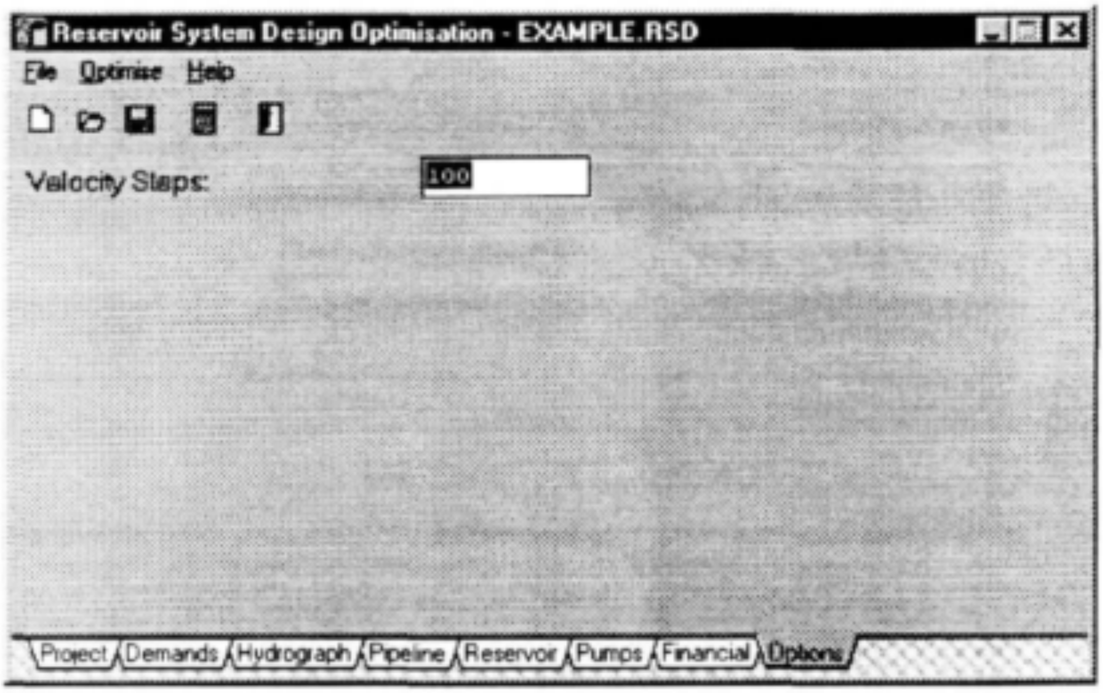


Figure 12 Optimisation Options Page

These options do not affect the optimum solution calculated, but only how the solution is displayed. The Velocity Steps field contains the number of different velocities for which results are calculated. The larger this number the finer the resolution of the solution, but the longer it will take to calculate.

Once all the required data has been entered, it is a good idea to save the file. Then you can calculate the optimum solution as discussed in the next section.

4. CALCULATING AN OPTIMUM DESIGN

To calculate the optimum reservoir, pipeline and pump station capacity, choose **Optimise|Calculate Optimum** from the menus, press CTRL+C, or click on . A status dialog will be displayed that shows the progress of the calculation. When it is complete, press ENTER, or click on the OK button. Two new sheets will then be displayed, containing the results of the optimisation.

4.1. Results Table

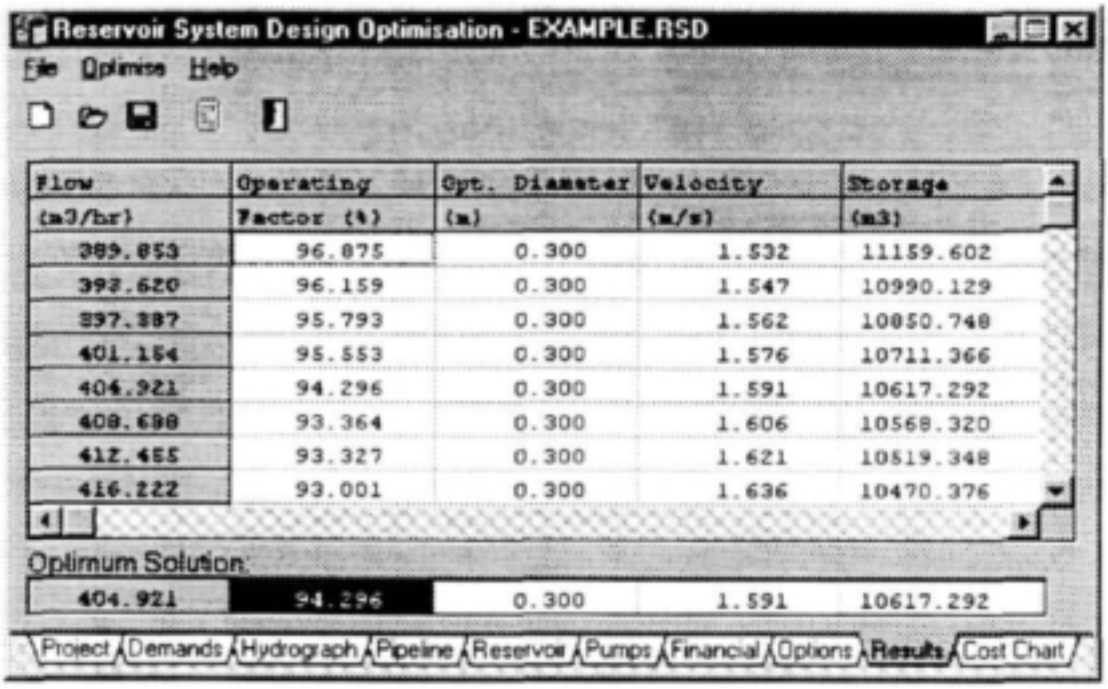


Figure 13 Results Table Page

The Results Table lists the operating factor, optimum diameter, velocity, storage required, and all the component costs for each of the flows analysed. The optimum flow grid at the bottom shows the results for the flow that results in the minimum total cost.

These results can be exported for printing or further analysis by choosing **File|Export|Results Data** from the menus. A File Save As dialog will be displayed that you can use to enter the name of the *Results Text File* in which you want to save the results. When you click on the OK button or press ENTER, these results will be exported to the specified file.

4.2. Results Chart

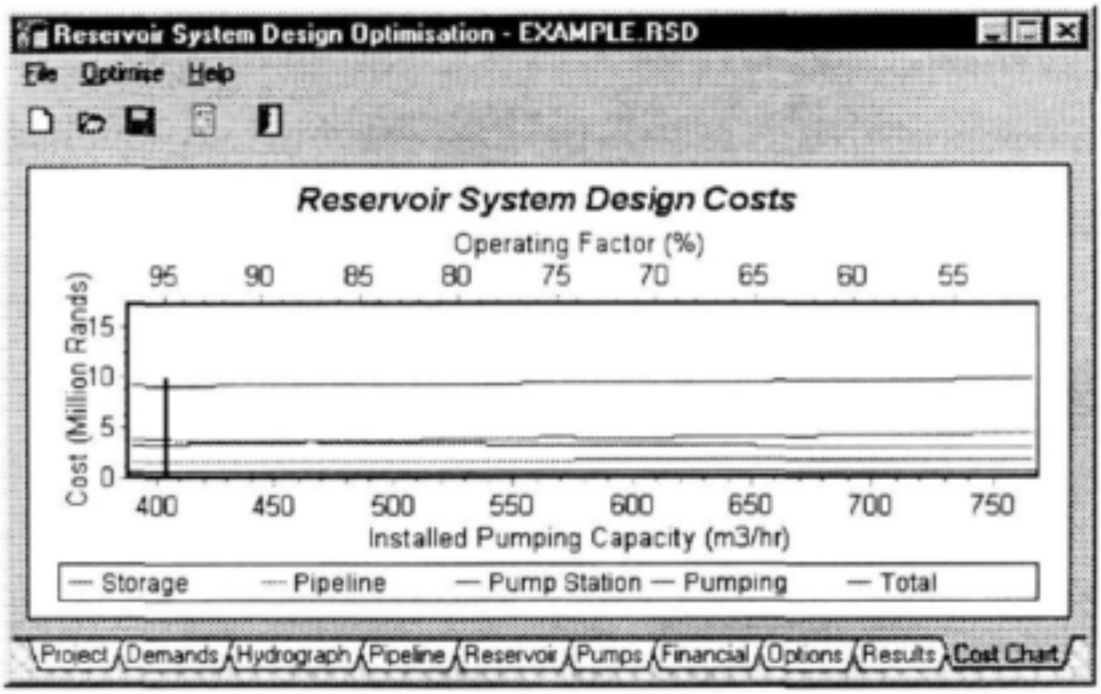


Figure 14 Results Chart Page

The results chart gives a graphical representation of the results so that you can easily see the relationships between the various cost components. The vertical line on the chart indicates the position of the optimum solution. It can also be exported by choosing **File|Export|Results Chart** from the menus. The chart will be saved in a *Windows Metafile* format that can be imported into many other programs.

5. MENU OPTIONS

5.1. File|New

Clears any existing data and resets the program to its starting state. If the current data has not been saved then it prompts the user to save the current file before clearing.

Shortcut: None

Toolbar button 

5.2. File|Open

Displays a standard Open File dialog box that allows you to select an existing *Reservoir System Design File* (*.RSD). The data in the *Reservoir System Design File* is loaded, and the name of the file is shown in the title bar of the program window.

Dialog Box Options

File Name

Select or type the name of the document you want to open. This box lists documents with the filename extension selected in the List Files Of Type box. To see a list of files with a particular extension, type an asterisk (*), a period, and the three-character extension, and then press ENTER.

List Files Of Type

Select the type of file you want to see in the File Name list. The types available will depend on the particular function that is being executed. For example, if the **File|Open** menu was selected, then the only file type available will be *Reservoir System Design Files* (*.RSD).

Drives

Select the drive that contains the file you want to open.

Directories

Select the directory that contains the file you want to open.

Shortcut: CTRL+O

Toolbar button 

5.3. File|Save

Saves the current data to the current *Reservoir System Design File*. If there is no current *Reservoir System Design File*, then the Save As dialog is displayed, as described below.

Shortcut: CTRL+S

Toolbar button 

5.4. File|Save As

Displays a standard Save As dialog box that allows you to enter the filename of a new *Reservoir System Design File* (*.RSD). The current data is saved in the *Reservoir System Design File.*

Dialog Box Options

File Name

Type in the name of the document you want to save to. This box lists existing documents with the filename extension selected in the List Files Of Type box in a grey font to indicate that they already exist. If you select an existing file it will be overwritten, although you will be prompted to confirm this.

List Files Of Type

Select the type of file you want to see in the File Name list. The types available will depend on the particular function that is being executed. For example, if the **FileSave As** menu was selected, then the first file type available will be *Reservoir System Design Files* (*.RSD).

Drives

Select the drive on which you want to save the file.

Directories

Select the directory in which you want to save the file.

Shortcut:	None
Toolbar button	None

5.5. File|Export|Demand Data

Exports the current demand data to a tab delimited ASCII text file. Displays the standard Save As dialog in order for the user to enter or select a filename for the *Demand Text File.*

Dialog Box Options

File Name

Type in the name of the *Demand Text File* you want to export the demand data to. This box lists existing documents with the filename extension selected in the List Files Of Type box in a grey font to indicate that they already exist. If you select an existing file it will be overwritten, although you will be prompted to confirm this.

List Files Of Type

Select the type of file you want to see in the File Name list. The types available will depend on the particular function that is being executed. For example, if the **File|Export|Demand Data** menu was selected, then the first file type available will be *Demand Text Files* (*.DTX).

Drives

Select the drive on which you want to save the file.

Directories

Select the directory in which you want to save the file.

Shortcut:	None
Toolbar button	None

5.6. File|Export|Demand Chart

Exports the current demand chart to a Windows™ Metafile. Displays the standard Save As dialog in order for the user to enter or select a filename for the *Demand Chart File*.

Dialog Box Options

File Name

Type in the name of the *Demand Chart File* you want to export the demand data to. This box lists existing documents with the filename extension selected in the List Files Of Type box in a grey font to indicate that they already exist. If you select an existing file it will be overwritten, although you will be prompted to confirm this.

List Files Of Type

Select the type of file you want to see in the File Name list. The types available will depend on the particular function that is being executed. For example, if the **File|Export|Demand Chart** menu was selected, then the first file type available will be *Demand Chart Files* (*.WMF).

Drives

Select the drive on which you want to save the file.

Directories

Select the directory in which you want to save the file.

Shortcut:	None
Toolbar button	None

5.7. File|Export|Results Data

Exports the current results data to a tab delimited ASCII text file. Displays the standard Save As dialog in order for the user to enter or select a filename for the *Results Text File*.

Dialog Box Options

File Name

Type in the name of the *Results Text File* you want to export the demand data to. This box lists existing documents with the filename extension selected in the List Files Of Type box in a grey font to indicate that they already exist. If you select an existing file it will be overwritten, although you will be prompted to confirm this.

List Files Of Type

Select the type of file you want to see in the File Name list. The types available will depend on the particular function that is being executed. For example, if the **File|Export|Results Data** menu was selected, then the first file type available will be *Results Text Files* (*.RTX).

Drives

Select the drive on which you want to save the file.

Directories

Select the directory in which you want to save the file.

Shortcut:	None
Toolbar button	None

5.8. File|Export|Results Chart

Exports the current results chart to a Windows™ Metafile. Displays the standard Save As dialog in order for the user to enter or select a filename for the *Results Chart File*.

Dialog Box Options

File Name

Type in the name of the *Demand Text File* you want to export the demand data to. This box lists existing documents with the filename extension selected in the List Files Of Type box in a grey font to indicate that they already exist. If you select an existing file it will be overwritten, although you will be prompted to confirm this.

List Files Of Type

Select the type of file you want to see in the File Name list. The types available will depend on the particular function that is being executed. For example, if the **File|Export|Results Chart** menu was selected, then the first file type available will be *Results Chart Files* (*.WMF).

Drives

Select the drive on which you want to save the file.

Directories

Select the directory in which you want to save the file.

Shortcut:	None
Toolbar button	None

5.9. File|Exit

Closes the *Reservoir System Design Optimisation* program. You can also close *RSDO* by pressing ALT-F4. If the current data has not been saved, you will be prompted to save it.

Shortcut:	ALT+F4
Toolbar button	

5.10. Optimise|Calculate Optimum

Calculates the optimum capacities for each of the reservoir system components. Displays a status dialog box during calculation to indicate the progress.

Shortcut: CTRL+C

Toolbar button 

5.11. Help|About

Displays information about your copy of *Reservoir System Design Optimisation*, including the version number; the copyright statement, and the author's e-mail address. Unfortunately, a full on-line help system was not implemented for this release of the software, so no other help functions are available.

Shortcut: None

Toolbar button None

6. RSDO FILES

The following sections describe all the files used or generated by *RSDO*. Each file type description starts with the name of the file type, and then has the file extension in brackets. This is then followed by a description of what the file contains, where it is used, and any formatting details.

6.1. Project Files

Reservoir System Design Files: (*.RSD)

These files contain all the data used in the calculation of the optimum reservoir, pipeline and pumping capacities. They are read whenever a **File|Open** is selected, and written when either **File|Save** or **File|Save As** are chosen.

6.2. Import Files

Text Data Files: (*.TXT)

These files contain the demand data that can be imported by pressing the IMPORT button on the Demands page. These files can contain any number of header rows, as long as the rows do not contain any tab characters. These rows are ignored during the importation. The first row containing a tab character is assumed to contain the column headings, and is also ignored. There after the file must contain 168 rows, each with a date and time string and a demand value, in cubic metres per hour, separated by a tab character. Any information in the row after the demand value is also ignored.

Category Data Files: (*.TXT)

Category files contain demand data for each of the different consumer categories, and are used to estimate the demand based on the number of consumers in each category. They are imported by pressing the FILES button on the Demands page. These files must have the same format as *Text Data Files*, and they must contain the following demand values, depending on the category of consumer:

Category	Unit	Number
High Income Residential	Stands	100
Medium Income Residential	Stands	100
Low Income Residential	Stands	100
Very Low Income Residential	Stands	100
Commercial	Hectares	100
Industrial	Hectares	100

6.3. Export Files

Demand Text Files: (*.DTX)

These files are created when the user chooses **File|Export|Demand Data**. They have the same format as the *Text Data Files*, with the following header lines. Line 1 contains the name of the reservoir system, line 2 contains the name of the *Reservoir System Design File* from which the data was exported, line 3 is blank, and line 4 contains the headings for each column of data. The next 168 lines contain a date and time string, and a demand value separated by a tab.

Results Text Files: (*.RTX)

These files are created when the user selects **File|Export|Results Data** from the menus. Lines 1 and 2 contain the name of the reservoir system and the name of the *Reservoir System Design* filename from which the data used to calculate the results were obtained. The remaining lines in the file contain the same data as in the Results Table, in a tab delimited format.

Demand Chart Files: (*.WMF)

These files are created when the user selects **File|Export|Demand Chart** from the menus. The file is a standard Windows Metafile, which is a format defined by Microsoft® Windows™ and can therefor be used by many Windows™ application programs such as word processors and graphics packages.

Results Chart Files: (*.WMF)

These files are created when the user selects **File|Export|Results Chart** from the menus. The file is a standard Windows Metafile, which is a format defined by Microsoft® Windows™ and can therefor be used by many Windows™ application programs such as word processors and graphics packages.