Chapter 1

Water Reclamation and Reuse

1.1 Introduction

The availability of fresh water supplies throughout the world has been getting scarcer over the past several decades. The information on availability of water resources generated world-wide continually supports the notion of existing or impending water shortages in many regions. It has been reported that up to 40% of the world's population in over 80 countries and regions are experiencing water stress, where water reserves are explored faster than they are being replenished (Wallace and Austin 2004). In this context, water reuse has emerged as a genuine and reliable alternative that can be used to supplement, and in some cases substitute traditional water sources.

The population growth combined with growth of urban centres and a tremendous increase in the number and intricacies of sources of water pollution, uneven temporal and spatial distribution of water resources, climate change and shifts in weather patterns towards extremes are just some of the factors which are making the option to reuse water more attractive throughout the world, and necessary for some regions. Other influencing factors vary between the industrialised countries, where the main drivers are the lack of dependable water supplies and wastewater disposal sinks, and developing countries, which need to provide economically feasible new water supplies and protect existing water sources from pollution (Asano 1999). The common element is the need to offset future expansion of water supply infrastructure and the protection of existing water resources. Environmental benefits of water recycling and reuse are also becoming more attractive, including the decrease in diversion of freshwater from sensitive ecosystems, decrease in discharge to sensitive water bodies, creation or enhancing of wetlands, and reduction and prevention of pollution (North Carolina Division of Water Resources 1996; USEPA 1998). Along with all the benefits mentioned above, there are also some obstacles to implementing reuse projects, and three that have been identified as most important are cost effectiveness, information dissemination and education, and water quality and health issues (Ahlstrom 1991).

Water from several sources can be reused: rainwater, greywater (wastewater from household baths, sinks, washing machines, and dishwashers), and wastewater. While all three types of water reuse are practiced in different parts of the world, the issues associated with each type can be quite different, due to different pollutant types and their concentrations, and scale and location at which the water reuse is practiced (Booker 2000). The focus of this thesis is on municipal wastewater reclamation and reuse systems in urbanized settings, which typically include one or more treatment facilities receiving conventional wastewater treatment plant (WWTP) effluent, and a distribution system for reclaimed water.

It should also be mentioned that the basic condition that needs to be met for a municipal wastewater reclamation program to be considered is that the area must be sewered. This presents opportunities for developing countries, where only 10% to 50% of urban areas are serviced by sewers, and where it is suggested that the water reclamation should be included in the sewerage planning projects (Okun 1991). In the developed world, inclusion of water reclamation into sustainable water management strategies for new developments is already progressing, particularly in areas that have experienced water shortages in the recent past.

In this Chapter, possible uses of reclaimed municipal wastewater are first presented, along with a brief history and current status of wastewater reuse. The projections for water reuse in the near to medium term are presented next, followed by a review of issues relating to the decision support systems (DSS) for wastewater reuse. The last section presents the objectives of this research and provides the thesis outline. A substantial part of the research presented in this thesis was conducted as part of author's contribution to the AQUAREC (2006) project on "Integrated Concepts for Reuse of Upgraded Wastewater" under the Fifth European Community Framework Programme.

1.1.1 Uses of Reclaimed Water

Prior to considering the uses of reclaimed water, a standardisation of terminology used in water reuse needs to be made. Although there is no consensus on the terminology, the widely accepted definitions cover the following terms (Metcalf and Eddy 2003):

- Wastewater reclamation involves treatment of municipal wastewater to a predetermined water quality level, which facilitates reuse,
- Reclaimed water is treated effluent of a quality suitable for a specific reuse application,

- Water reuse is the use of treated wastewater for beneficial purposes,
- Water recycling or recirculation typically refers to industrial systems, in which the effluent is recovered, usually treated and returned back into the industrial process.

Reclaimed water can be used to supplement or replace the use of natural water resources in numerous ways. The intended reuse application is also the key influencing factor in determining the degree of wastewater treatment required and the reliability of wastewater treatment, processing and operation. Several different classifications of end uses of reclaimed water have been proposed in the past, and a brief overview of general classifications based on the nature of use and reuse application is presented here (Dinesh 2002).

The first classification depends on whether the reclaimed water is blended with water from other sources, and the distinction is made between the direct and indirect reuse. Examples of direct reuse include irrigation, industrial use and use through dual distribution systems. Indirect reuse involves mixing, dilution and dispersion of reclaimed wastewater, and an example would be a groundwater recharge application. The second classification simply distinguishes between planned and unplanned wastewater reuse. The former is the focus of this review, and refers to systems which are designed with the specific purpose in mind for use of reclaimed water, while the latter refers to a scenario where the treated wastewater is being discharged into a water body which also serves as a source of water supply. A typical example of unplanned reuse is use of river water as potable water supply downstream of a discharge from a wastewater treatment plant, a scenario very common throughout the world (although not necessarily always acknowledged as water reuse).

Classification of end uses of reclaimed water based on the reuse application is typically related to a particular guideline or criteria for end use. A general classification into seven categories is shown in Table 1.1, in a decreasing order of volumes currently used in the world. The use of reclaimed water for agricultural irrigation is applicable and indeed applied widely, in both the industrialized and developing countries. Landscape irrigation is practiced mostly in industrialized countries, and in many cases involves dual distribution systems. The most common industrial use of reclaimed water is for cooling water, and different uses of reclaimed water in the industry impose specific and often very high water quality requirements. Ground water recharge with reclaimed water is practiced for several purposes shown in Table 1.1, and can be done either using recharge basins or by direct injection to aquifers. Recreational and environmental uses of reclaimed water are numerous, and in many cases involve impounding of reclaimed water in man-made or natural water bodies that bland into the surrounding landscape. Non-potable urban uses are again practiced in industrialized countries, and are usually combined with landscape irrigation for economic reasons. Finally, the potable use is the least common of all end uses, largely due to public acceptance reasons.

Table 1.1 Categories of Municipal Water Reuse (adapted from (Metcalf and Eddy 2003) and (Dinesh 2002))

No.	Wastewater Reuse Category	Applications
1	Agricultural Irrigation	Food crops, pastures, nurseries
2	Landscape Irrigation	Parks, school yards, highway medians, golf courses, cemeteries, greenbelts, residential
3	Industrial	Cooling water, boiler feed, process water, heavy construction
4	Groundwater Recharge	Groundwater replenishment, saltwater intrusion control, subsidence control
5	Recreational/Environmental	Lakes and ponds, marsh and wetland enhancement, stream-flow augmentation, fisheries, snowmaking
6	Non-potable Urban	Fire protection, air conditioning, toilet flushing
7	Potable	Blending in water supply reservoirs, pipe-to-pipe water supply

1.1.2 Evolution and Extent of Water Reuse

Although the concept of wastewater reuse tends to be considered as more recent and related to advancements in wastewater treatment technologies, it has been in practice for much longer. Asano and Levine (1996) summarized the evolution of wastewater reuse extending to the earliest examples of agricultural irrigation with wastewater 5,000 years ago. In the early nineteenth century wastewater reuse was unplanned, and involved inadvertent wastewater collection systems discharges to potable water supplies, which caused catastrophic epidemics of waterborne diseases. In the later part of nineteenth century, wastewater reuse started shifting to planned use, with a number of sewage farms built in Europe and the US that served primarily for waste disposal, but also as a source of water for crop production (Metcalf and Eddy 2003).

In the first half of the twentieth century, most of the developments in wastewater reuse were concentrated in the United States, as shown in Table 1.2. The earliest systems were developed in California and Arizona in the 1920s to provide water for irrigation, after the California State Board of Public Health adopted the first regulation for use of sewage for irrigation in 1918. Since then, the number of reuse systems in the US has been growing rapidly, particularly in the last three decades, including a number of potable reuse projects outlined by Hamann and McEwen (1991). Today, more than 1,500 utilities have a water reclamation program with large scale groundwater recharge practiced in California, New Jersey and Georgia. The technological advances in processing of water and wastewater contributed to the growth in the number of reuse systems throughout the world in the last three decades. A brief overview of the development of some of the more prominent water reuse schemes is presented below.

One of the largest projects recently completed, Orange County's Groundwater Replenishment System in California, uses reclaimed water for seawater intrusion barrier at a cost of nearly US$500 million (ENR 2004). In Florida, reuse is an integral part of wastewater management, water resource management, and ecosystem management, with daily volumes of reclaimed water produced exceeding two million cubic meters. During the last 20 years, the primary driving force behind the implementation projects in Florida has been effluent disposal (York and Crook 1991). The City of St. Petersburg in Florida has been operating a dual distribution system since 1977, making the reclaimed water available in a separate piping system for landscape irrigation throughout its boundaries. Reclaimed water has also been used to supplement potable supplies by the Upper Occaquan Sewage Authority in Alexandria, Virginia, since 1978 (IETC 2000).

Water reuse is also practiced in Canada, a country that enjoys abundant water resources, albeit on a relatively small scale. A review of the history of water reuse in Canada, presented by Exall (2004), identified isolated agricultural and landscape irrigation projects in several provinces, and recognised further potential for other non-potable urban reuse applications as well, such as use of effluent heat energy and snow-making. Regulatory guidance for water reuse currently exists only in British Columbia and Alberta, however, the need for the national guidelines has been clearly identified in a workshop organized by Canadian Council of Ministers of the Environment (Marsalek et al. 2002).

Table 1.2 Examples of Historic Development of Water Reuse (adapted from (Metcalf and Eddy 2003) unless otherwise shown)

Year	Location	Application
1890	Mexico City, Mexico	Irrigation of agricultural area
1912-1985	Golden Gate Park, San Francisco, CA, USA	Watering lawns and supplying ornamental lakes
1926	Grand Canyon National Park, AZ, USA	Toilet flushing, lawn sprinkling, cooling water and boiler feed water
1929	City of Pomona, CA, USA	Irrigation of lawns and gardens
1942	City of Baltimore, MD	Metals cooling and steel processing
1960	City of Colorado Springs, CO, USA	Landscape irrigation
1961	Irvine Ranch Water District, CA, USA	Irrigation and toilet flushing
1962	County Sanitation District of Los Angeles County, CA, USA	Groundwater recharge
1962	La Soukra, Tunisia	Irrigation of citrus plants and groundwater recharge
1968	City of Windhoek, Namibia	Augmentation of potable water supplies
1969	City of Wagga Wagga, Australia	Landscape irrigation
1976	Orange County Water District, CA, USA	Groundwater recharge (Water Factory 21)
1977	City of St. Petersburg, FL, USA	Irrigation and cooling tower makeup water
1977	Dan Region Project, Tel-Aviv, Israel	Groundwater recharge, crop irrigation
1984	Tokyo Metropolitan Government, Japan	Toilet flushing
1987	Monterey Regional Water Pollution Central Agency, Monterey, CA, USA	Irrigation of food crops eaten raw
1989	Consorci de la Costa Brava, Girona, Spain	Golf course irrigation
2000	Flag Fen Sewage Treatment Works Peterborough, UK[1]	Power station water supply
2003	NEWater, Singapore[2]	Industrial process water, augmentation of potable supplies

[1] (Murrer and Macbeth 2004)
[2] (PUB 2004)

Even though Europe has apparent abundance of water resources, several factors have supported the development of wastewater reuse schemes. In Southern Europe, the water scarcity has been the primary factor for wastewater reuse in irrigation, while the environmental protection in response to increasingly stringent environmental regulations has been the main driver in Northern European countries (Angelakis et al. 2003). In France, the largest scheme is Clermont-Ferrand, where the reclaimed wastewater is used for irrigation of over 700 ha of maize. Smaller wastewater reuse schemes exist throughout the Mediterranean European countries, with the primary use of reclaimed water being agricultural irrigation. In the UK, two schemes have recently been implemented: in Essex, over 30,000 m^3/day of tertiary treated effluent is being used for augmentation of flow in the River Chelmer and the volume stored in the reservoir for water supply (Lunn 2004), and in Peterborough, where up to 1,000 m^3/day of highly treated tertiary effluent is used as boiler feed (Murrer and Macbeth 2004). Further opportunities for large scale municipal wastewater reuse in the UK have been recognised in a recent report from the House of Lords Science and Technology Committee (2006), which also recommended that the Government and relevant agencies explore, encourage and support schemes for the planned indirect reuse. The lack of uniform guidelines has been identified as an impediment to wider reuse both in the UK by this report, and throughout Europe by the AQUAREC (2006) project, which also served as a vehicle to provide much information used in this research.

In Africa, the use of reclaimed water in Tunisia has evolved from the 1960s, when some of the wastewater from Tunis started being used for irrigation and to reduce the impact of salt water intrusion due to excessive pumping of groundwater, to the point where the reclaimed water is now a part of Tunisia's overall water resources balance (Bahri 2002). The fraction of reclaimed wastewater reached 7 percent of sewage collected in South Africa in 1988, with primary use being agricultural irrigation (IETC 2000). An example of direct potable use of reclaimed water often cited in literature is the Windhoek Water Reclamation Plant in Namibia, which was constructed during a devastating drought in 1968. The percentage of reclaimed water in the blend with raw water sometimes was at times as much as 50% since construction for short periods of time (IETC 2000), and the scheme is still operating and is being expanded (Leeuwen 1996).

In Israel, wastewater reuse in irrigation began in the seventies, for cotton production. One of the first, and currently the largest and most important wastewater reuse system in Israel, is Dan Region, whose development started in 1977. The system treats in excess of 120 million m^3 of reclaimed water per year, and conveys it from a densely populated urban area to a desert area, thus converting the desert into agricultural land

(Icekson-Tal et al. 2003). Other municipalities, such as Arad, Eilat, Beer-Sheva, Netania, Afula and others are directing most of their treated effluent to agricultural irrigation while some places use the treated effluent for augmentation of streams used for recreation (Shalef 2001). The development of numerous other reuse installations over the years led to a point where today, more than 70 % of Israel sewage is reused in agriculture irrigation, making the reclaimed water an integral part of its water resources.

A number of water reclamation and reuse projects also exist throughout the Far East. In Japan, the practice of reusing treated wastewater for recreational impoundments started in the 1970s, and is today widely used in a range of scales. On a smaller scale, the reclaimed water is used for toilet flushing using on-site installations in buildings or groups of buildings. Large installations involve conventional wastewater treatment facilities reclaiming water for stream augmentation needs (Suzuki et al. 2002). One of the most recent large-scale reclamation projects in the Far East has recently been implemented in Singapore. The reclaimed water (marketed as NEWater) is used in the electronic manufacturing industry and also contributes about 1% of the potable water supply, with plans for a higher percentage in the future (PUB 2004).

Large scale wastewater reuse in Australia started in the late 1960s. The primary driver of earlier installations has been the scarcity of water supplies for agricultural irrigation, with environmental protection of beach areas gaining importance in the recent years. The largest Australian wastewater reuse schemes is the Virginia pipeline project, which is used to deliver recycled water from the Bolivar Waste Water Treatment Plant to irrigators in the Northern Adelaide Plains, and also uses poor quality aquifers to provide temporary storage of reclaimed water (Kracman et al. 2001). Today, reclaimed water is recognised in Australia as a valuable economic commodity, with the volume of sewage effluent reused in 1999/2000 exceeding 7% of wastewater treatment plants effluents, amounting to 112 million cubic meters annually (Dillon 2000).

This section provides a general overview of drivers for water reuse and key installations operating in the world today. A wider review of wastewater facilities around the world, including the use of treatment technologies and management practices, can be found in (Bixio et al. 2004) and (IETC 2000).

1.1.3 The Future of Water Reuse

As indicated in previous sections, several factors have contributed to an increased use of water reclamation in recent years. This trend is expected to continue in the future, with some projections indicating that the global water reuse capacity will rise more than 180% in the next decade, from the present capacity of 19.4 million m^3/day to 54.5 million m^3/day in year 2015 (Global Water Intelligence 2005). The largest expected growth markets, identified by the same authors, are shown in Table 1.3.

Table 1.3 Largest Water Reclamation Markets (Global Water Intelligence 2005)

Market	Expected Additional Capacity by 2015 ($x10^3$ m^3/day)	Percent Annual Increase
China	10,790	29
MENA	5,589	12
USA	4,473	12
Western Europe	3,895	10
South Asia	3,750	14

An important factor that needs to be considered in the context of planning of future water reclamation undertakings is the scale of the projects. The water reclamation projects are driven by demand and lack of alternative sources, which lead to projects being considered, or even made feasible, by securing one or several large customers. As the demand for reclaimed water rises in the future, the number of potential end-users will also increase, leading to larger and more complex projects requiring adequate tools for their development. In the City of Chicago, for example, which currently reuses only 2% of effluents discharged form its seven wastewater treatment plants, a study of future water reuse opportunities identified over 800 potential users (Meng 2005). To author's knowledge, a detailed evaluation of water reclamation and reuse alternatives has not been conducted to date for projects approaching this magnitude.

1.1.4 The Need for Decision Support

Reclaimed water projects typically include construction of new or upgrades to a municipality's treatment systems to treat wastewater to the required quality level, and construction of distribution systems for reclaimed water. Their realisation requires deciding on many relevant issues that will be discussed later, but the three broad questions that need be answered for every potential project are: 1) how much of the available wastewater should be treated, 2) what level of treatment needs to be provided and 3) how is the reclaimed water going to be distributed. A Decision Support System (DSS), defined as "a computer-based system that aids the process of decision making" (Finlay 1994), can be of aid in making these decisions. Since the decision support tools can form an integral part of planning process of wastewater reclamation and reuse projects, it is important to first consider how a DSS could fit into the overall process.

The planning of wastewater reuse schemes can be distinguished into the following three stages: conceptual planning level, preliminary feasibility investigation and facilities planning (Asano and Mills 1990). At the conceptual planning stage, preliminary cost estimates are prepared for alternatives that include few, if any, site-specific considerations. Feasibility investigations involve evaluation or analysis of the potential impacts of a proposed project or program (Urkiaga 2004), and for wastewater reuse projects include assessment of consumer demand, evaluation of techno-economic feasibility, environmental and institutional feasibility (Dinesh 2002). The final stage in the planning process involves the development of a wastewater reclamation and reuse facilities plan. The plan needs to consider numerous aspects shown in Table 1.4, as all of them have been found to affect the evaluation of water reclamation and reuse projects (Metcalf and Eddy 2003).

Table 1.4 Considerations for Wastewater Reuse Facilities Plan (Asano and Mills 1990; Metcalf and Eddy 2003)

1. Study area characteristics - geography, geology, climate, groundwater basins, surface waters, land use, population growth
2. Water supply characteristics and facilities - agency jurisdictions, sources and qualities of supply, details of major facilities, water use trends, future facilities needs, groundwater management and problems, present and future freshwater costs, subsidies and prices
3. Wastewater characteristics and facilities - agency jurisdictions, quantity and quality of treated effluent, flow and quality variations, future facilities needs, needs for source controls, details of existing reuse, if any
4. Treatment requirements - discharge and reuse standards, user-specific requirements
5. Potential customers - reuse market analysis, user surveys
6. Project alternative analysis - capital and operation and maintenance costs, engineering feasibility, economic, financial and energy analysis, water quality impacts, public and market acceptance, water rights impacts, environmental and social impacts, comparison of alternatives and selection
7. Recommended plan - description of proposed facilities, preliminary design criteria, projected costs, potential users and commitments, variation of reclaimed water demand in relation to supply (quantity and quality), reliability of supply and need for supplemental or backup supply, implementation and operational plans
8. Construction financing and revenue - sources and timing of funds, reclaimed water pricing, cost allocation, projections on future conditions, analysis of sensitivity to changed conditions

A DSS could be used to encapsulate most (if not all) of the information presented in Table 1.4 and aid in the decision making process. The main value of a DSS for wastewater reclamation and reuse, however, would be in conducting the project alternative analysis and the development of a recommended plan. These activities include the following evaluations (Asano and Mills 1990):

- Treatment alternatives,
- Alternative markets, based on different levels of treatment and service areas,
- Pipeline route alternatives,
- Alternative reclaimed water storage locations and options,
- Freshwater alternatives, and
- Water quality management alternatives.

Steps similar to those outlined above have also been incorporated into regulations dealing with sewer plans. For example, some legislation in the US requires that reclaimed water feasibility analysis be included in wastewater plan submissions, consisting of the following steps (Washington State Department of Health 2000):

- Identify existing and future potential uses for reclaimed water (not limited to service boundaries),
- For each of the uses identified, estimate the annual or seasonal volume of reclaimed water required,
- Determine the level of treatment required,
- Evaluate the ability of current or proposed facilities to meet both the treatment and the reliability requirements for the intended uses,
- Screen for water rights (determine if there is a reliance on the wastewater discharge),
- Discuss the general layout of a reclaimed water distribution system for the likely uses identified (including a map showing potential routes for trunk lines),
- Identify plans to meet future water demands if reclaimed water is not used,
- Evaluate technical feasibility (compare the various treatment, water supply, distribution and use alternatives and provide a basis for selection or rejection of alternatives), and

- Evaluate economic feasibility (using life cycle costs, compare unit costs of potable water and wastewater treatment to reclaimed water for the same market under present and future conditions).

It is very likely that a water reuse system could have many possible design options: different sets of served end-users, with various water quality requirements; type and degree of treatment; number and location of treatment plants; number and location of pumps/pumping stations; number, size and location of storage tanks; layout and size of distribution pipe network. These elements are all linked, to give multiple interactions and a very large number of design combinations, even for apparently small systems.

A simple trade-off example that highlights the interaction between system components can be made considering a hypothetical scenario with only two potential users. The first user is an industry located relatively close to the WWTP requiring a constant supply of high-quality reclaimed water for process use, and the second user, located further away from the reclamation plant, has the seasonal need for reclaimed water of lower quality for restricted irrigation. Providing reclaimed water to the first user would likely require a more expensive treatment option, but less expensive distribution system and no need for seasonal storage, whereas a more extensive distribution system and seasonal storage would be required to satisfy the second user's demands, albeit with less treatment. Even if an assumption is made that the annual reclaimed water requirements for the two users are the same, there is an obvious need to perform a comprehensive analysis of design options prior to making the final decision.

The trade-offs involved in determining the optimal water reclamation system configuration illustrated above on a very simple case is amplified if different combinations of available treatment processes are contemplated and/or a greater number of potential end-users is considered. The complexity associated with planning of water reuse schemes is therefore very high due to a very large number of design combinations possible, and establishes the need for use of DSS to aid in the planning process.

1.2 Research Objectives and Thesis Outline

From the discussion contained in previous sections, it is evident that the practice of water reclamation and reuse is a proven way of supplementing conventional water supply options. Due to the projected tremendous growth of water reclamation practice, driven by a variety of factors throughout the world, the water reclamation projects are likely to become more complex. Although structured approaches have been developed and applied in planning of water reuse schemes, there is a definite need for tools that would enable the planners and decision-makers in this area to explore large numbers of alternatives in a systematic way, which will be required in elaborate schemes of large scale.

The main objective of this research is to develop a methodology that can be used to perform two tasks: efficiently screen alternatives for integrated water reuse systems, and assist the planner in identifying the most promising alternatives through optimisation. As indicated earlier, the focus is on centralised municipal wastewater reclamation and reuse systems, delivering reclaimed water for any of the number of potential types of end-users.

In order to address the efficiency aspect of the main objective, the methodology is encapsulated in the simulation component of the DSS, which includes information needed for planning of various water reclamation system components, means for specifying components comprising the integrated system, and methods to calculate the performance of the user-specified system. Specific goals related to this task set out in the development of the simulation model are:

- Present a completely open modelling environment that will allow users flexibility in terms of information on water reclamation system components (e.g. treatment unit processes and their characteristics, pollutants to be considered, use types and quality requirements, rules for combining unit processes in a treatment train, etc.),
- Provide suggestions for complete treatment trains based on the influent quality (or current level of treatment provided in the case of existing wastewater treatment facilities) and quality requirements for “standard” end uses of reclaimed water,
- Include the distribution system in the reuse scheme evaluation, by allowing users to specify the locations of pumping, transmission and storage facilities and providing a least-cost preliminary sizing of the distribution system that meets operational requirements.

Due to the potentially large number of design alternatives that can be expected, the identification of most promising alternatives is addressed in the DSS by developing and incorporating mathematical optimisation techniques for integrated water reuse systems. The optimisation of complex integrated water reuse schemes has not been attempted to date, and it presents essentially a large scale, highly constrained combinatorial problem. The methods developed for optimisation of this problem are applied to find potentially useful design principles, which is the secondary objective of this thesis.

The development of the DSS was preceded by a comprehensive review of issues relevant to the integrated water reuse planning problem, and available tools developed to deal with these issues. The results of the review and details of issues considered in the development of the current DSS that resulted from the review are presented in Chapter 2.

Chapter 3 deals with the development of methodologies used for evaluation of integrated water reuse schemes that were implemented in the DSS. Beginning with the introductory section, this chapter has three sections dealing with the principal components of the DSS. The first of these outlines the development of the knowledge base of information needed for evaluation of integrated water reuse systems. The two sections that follow deal with approaches used to conduct the preliminary sizing and performance evaluation of two principal components involved in water reclamation and reuse (treatment train and distribution system).

Chapter 4 presents methodologies developed for optimisation of integrated water reuse systems that were implemented in the DSS. The size of the optimisation problem is quantified in this Chapter, and the rationale for different optimisation techniques used in the DSS is presented. Details of the three optimisation methodologies, enumeration, simple genetic algorithm (GA) and comprehensive GA, are presented in subsequent sections, followed by the Chapter summary and conclusions.

Chapter 5 presents the results of the software testing and sensitivity analyses, which were carried out on a test case studies to examine the modelling and optimisation methodologies developed, and the results of these activities are presented in this Chapter. The chapter ends with a summary section, which includes the conclusions reached.

DSS application on a larger case study in Waterloo, Canada is presented in Chapter 6. The chapter includes sections that describe the general description of the study area, outline the model assumptions, and present the details of design alternatives identified by the optimisation as most promising for detailed investigations. Design principles arrived from application of the DSS are also described in this Chapter.

In Chapter 7, the thesis summary is provided. Conclusions reached from the development, testing and application of the DSS are also presented, and the chapter concludes with a summary of recommendations for further research.

Four Appendices are also included in this thesis. Appendix A contains detailed information sheets on unit processes contained in the knowledge base of the DSS.

Appendix B covers the implementation of DSS in a user-friendly hydroinformatics tool, used to manage and process the information entered by the user and generated during the evaluation process. The developed tool is named Water Treatment for Reuse with Network Distribution (WTRNet). Appendix C contains results of the sensitivity analyses carried out on the methodology used for optimal allocation of reclaimed water amongst potential end-users and sizing of storage elements. Detailed WTRNet input data for the case study considered in this research are provided in Appendix D.

Chapter 2

Decision Support for Planning of Water Reuse Projects

2.1 Introduction

The components of a DSS for evaluation of integrated wastewater reuse schemes can broadly be divided into treatment and distribution components. Therefore, the literature reviewed as part of this study is summarised separately for these two areas. The final component of the literature review examined the methodologies addressing the integral aspects of wastewater reuse schemes, by examining approaches taken in the past dealing simultaneously with wastewater treatment and distribution/allocation. The three sections included in this chapter provide an overview of literature reviewed, and introduce the concepts adopted in this research for the development of the DSS.

2.2 Wastewater Treatment

The number of treatment processes used to treat the wastewater has been steadily growing over the years. This is particularly true for advanced treatment technologies capable of treating wastewater to the degree of quality appropriate for reuse, making the selection of the most suitable sequence of processes (treatment train) for any potential reuse situation more complex. The challenges experienced by planners and designers of water reuse systems include deciding on suitable treatment trains from a large number of unit process combinations (Chen and Beck 1997) as well as handling the multiple objectives that treatment systems need to satisfy (Balkema et al. 2001). The first part of the literature review focused on examining the key issues involved in the methodologies used for generation, evaluation and optimization of treatment alternatives, and their integration into a structured decision process using a DSS.

The planning and selection of appropriate treatment alternative for a water reclamation scheme can be considered in a three-stage process (Rossman 1989): selection of treatment alternatives, pilot plant studies, and selection of preferred alternative for detailed design. A planning process that can then be used in the development of water reclamation project was presented by Langoria and Lewis (1991), which includes four steps: project definition and formation, creation of baseline data, treatment process evaluation and selection, and implementation. The focus of this review was on the planning stage involving evaluation of performance and cost of a number of treatment alternatives to select the most appropriate ones for more detailed evaluation. The tasks involved can further be divided into: generation or synthesis of treatment trains, evaluation of synthesized treatment trains, and selection of optimum (or near optimal) treatment train(s) for detailed investigation (Dinesh 2002). The same author provides an extensive review of techniques used in the past to perform these three functions. The discussion provided here summarizes key approaches that were deemed relevant to this research.

2.2.1 Synthesis of Treatment Trains

The issue of generation of treatment trains for municipal wastewater treatment was first explored by Rossman (1979) in the development of the EXEC/OP model, aimed at generating a set of attractive design alternatives for wastewater treatment. The author defined the synthesis as the "specification of both a system structure – the choice and arrangement of unit processes and operations – and the design of the individual units within that structure so that a set of design objectives is fulfilled". The model includes eight conventional wastewater treatment processes, ranging from influent pumping to chlorination, and thirteen sludge treatment processes. The rules in which the various treatment processes could be combined were indicated in a relatively simple flow diagram.

The same author later developed a hybrid approach to generate alternatives (Rossman 1989), which included a structured knowledge base containing the following information:

- List of unit processes and information for estimating their performance,
- Rules for excluding a unit process based on acceptable configurations and area limitations,
- Unit process pre-treatment requirements, and
- Procedures for estimating real and pseudo-costs.

A branch-and-bound algorithm was then used to determine treatment trains that met the treatment goals by sequentially adding unit process, and using a bounding condition to stop the addition of processes if the pseudo-costs exceeded a user-specified value.

Chen and Beck (1997) took a more computationally expensive and more exhaustive approach of Monte Carlo simulation to synthesise treatment trains, and identify more sustainable urban wastewater treatment options. The authors considered 22 liquid and 11 solids treatment technologies, and specified rules through process stages and specific rules that ensured that the treatment trains would include conventional treatment as well as the emerging novel techniques. They then generated randomly 50,000 feasible treatment trains (that met all the rules), and analysed the results to identify the unit process technologies most likely to be used.

Krovvidy et al. (1991) took the approach of developing treatment trains according to rules based on unit process performance data and external rules specifying the interactions between these technologies. The authors used decision trees and data contained in the treatability database developed by the Hazardous Waste Treatment Research Division of the United States Environmental Protection Agency (USEPA) to generate about 230 rules that describe the removal properties of a particular technology on a given compound at a given concentration (Krovvidy 1998). Rules are represented with if-then constructs, such as *IF compound = X and influent concentration is between A and B AND technology = Y THEN effluent concentration is between C and D.* External rules were obtained by knowledge engineering. A typical external rule they used is *Never (Activated Sludge, Aerobic Lagoons)*, which means that the treatment train can contain only one of the two technologies. In the actual synthesis phase, the method uses an A*-based heuristic search technique to generate alternative treatment trains to reduce the pollutant concentration to required levels using fuzzy possibility curves, in increasing order of cost. Yang and Kao (1996) used a similar approach, that incorporated fuzzy preference rules specified by the user in the selection process. The treatability database used (and indeed the problem the researchers were dealing with) is focused on industrial wastewater treatment, where the number of potential contaminants of concern is greater compared to municipal wastewater treatment. Also, specific processes are often used for removal of specific pollutants, which makes the selection of treatment processes for industrial wastewater treatment quite different from the municipal wastewater treatment selection process.

Chin-Tien and Jehng-Jung (1996) developed an expert system to find an appropriate treatment process design for a given waste stream, in which they use a knowledge base that contained a treatability database. The system used a fuzzy logic based approach to capture the user's preference of treatment techniques, defined on the basis of the treatment efficiency and the cost of a technique.

The approach taken by Elimam and Kohler (1997) in the sequencing of wastewater treatment processes considered 22 unit processes, ranging from preliminary to tertiary treatment. The cost and pollutant removal efficiencies were provided for each process. Their sequencing rules were specified using an acyclic directed network reproduced in Figure 2.1, where each unit process is represented by a single labelled arc. The unlabeled ‘dummy’ arcs are used to encapsulate the precedence relationships between the processes.

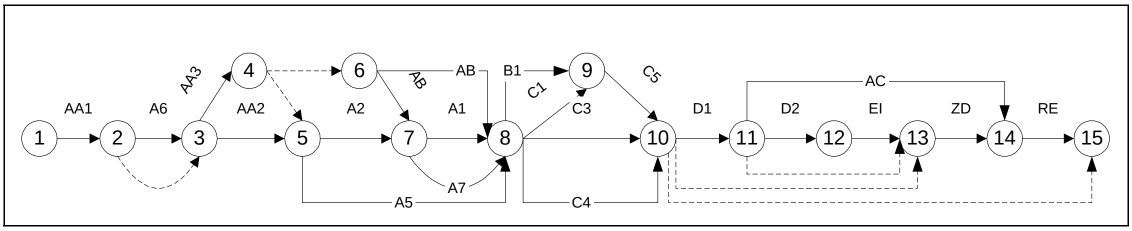


Figure 2.1 Wastewater Treatment Network Representation (Elimam and Kohler 1997)

Another approach taken by some researchers was to consider pre-determined combinations of unit processes forming treatment trains. For example, Tang et al. (1997) included 46 and 94 combinations for wastewater and sludge treatment, respectively, and used the Analytical Hierarchy Process (AHP) to select the best treatment alternatives. The focus of the expert system developed by Economopoulou and Economopoulos (2003) was on finding the lest-cost natural treatment alternatives for small and medium communities. The methodology deals simultaneously with the treatment, reuse and disposal problems and is used to develop a rational management scheme that satisfies all relevant treatment requirements and receiver water quality standards. Addou et al. (2004) as well as Bick and Oron (2004) used AHP for the selection of most appropriate wastewater treatment option, and each author used five different treatment trains in their evaluation. In general, the focus of these researchers was on a large number of technical, environmental and socio-economic factors considered in the selection process rather than the large number of possible treatment alternatives.

Dinesh (2002) included a knowledge base containing design and costing information on 39 wastewater treatment and six sludge treatment and disposal processes in MOSTWATER, a software package for optimum selection of technologies for wastewater reclamation and reuse. The knowledge base also contains a series of rules used in the generation of treatment trains, covering acceptable process configurations, site-specific considerations, and maximum allowable process influent quality. The generation of treatment trains is performed by the genetic algorithm (GA), which is discussed in more detail in the following sections.

The WAWTTAR decision support tool developed by Finney and Gerheart (2004) includes the most comprehensive information of all tools reviewed, covering 200 different water and wastewater treatment processes. The information contained in the knowledge base on each unit process is classified under five categories (general, construction, O&M (operation and maintenance), siting, impacts, and on-site miscellaneous) covering a wide range of information ranging from costs to specifics of resources required for each unit process. The generation of treatment trains, however, is not guided and the tool requires users experience and familiarity with treatment processes in generation of treatment trains.

2.2.2 Evaluation of Treatment Trains

Once a methodology is selected and applied to synthesise treatment trains, the criteria against which these generated treatment trains are evaluated have to be specified prior to any screening, ranking or optimisation. In this section, an overview of criteria used by researchers in the past is provided first. The criteria that were considered in this work are summarised next, with a brief discussion on each of the 'short listed' criteria.

2.2.2.1 Review of Evaluation Criteria

The choice of evaluation criteria varies quite drastically in the approaches taken in the past, but mainly depends on whether or not considerations are made for factors pertinent to developing countries. For example, Ellis and Tang (1991) identified 20 parameters including technical, economic, environmental and socio-cultural factors in their wastewater treatment optimization model that can be applied to communities in developing countries. Similarly, the SANEX™ decision support tool uses 50 technical, socio-cultural and institutional criteria to compare the treatment alternatives with regard to their implementation feasibility and sustainability (Loetscher and Keller 2002). Abu-Taleb (2000) investigated treatment options for a World Heritage Site in Jordan, where cultural and environmental concerns played a significant role in deciding not only what treatment technologies would be used, but also how the wastewater is collected and where it is treated. The author considered sludge handling, power required, odour potential, extent of resistance to preliminary treatment failure, mechanical complexity, construction difficulty, groundwater contamination potential, local spare parts availability, and local process experience in deciding on the optimal wastewater collection and treatment process. The WAWTTAR DSS also considers factors applicable to developing countries, and assesses the applicability of treatment processes based on extensive community information grouped in the following five categories: demographics, resources, hydrometeorology, finances, and site characteristics (Finney and Gerheart 2004; McGahey 1998).

For the developed countries, Metcalf and Eddy (2003) indicate 23 important factors that should be considered when evaluating and selecting unit processes in conventional wastewater treatment, ranging from cost and performance to climatic constraints and compatibility with existing treatment processes. Rossman (1979) used eight design criteria (initial construction cost, annual operation and maintenance costs, total equivalent annual cost, energy consumption, energy production, net energy consumption, total land utilisation and a subjective “undesirability” rating) in the selection of appropriate treatment trains. The approach taken in the EXEC/OP model was embodied in the implicit enumeration optimisation technique, where the evaluation criteria were combined into a weighted objective function that was minimised.

In MOSTWATER, evaluation of treatment trains is conducted on fourteen criteria, ranked into technical, environmental and economic types (Dinesh 2002). The technical criteria considered are performance, reliability, adaptability to upgrade, varying flow rate and change in water quality, and ease of O&M and construction. The environmental criteria consider power and chemical requirements, odour generation, impact on groundwater, land area requirements and sludge production, while the project costs (total, annual or lifecycle) are the economic criteria used. Of these, four are calculated (performance, cost, land area requirement and sludge production), while the others are considered as qualitative.

The selection criteria used by Chen and Beck (1997) centred on sustainable development and consists of land area requirement, capital and operating costs, odour emissions, desirability (impact of process outputs on environment), robustness, solids produced (volume and water content), and the amount of carbon, nitrogen and phosphorus recovered. More recently, Balkema et al. (2002) presented a comprehensive review of indicators used in the literature to compare wastewater treatment systems. The same author used sustainability indicators grouped in four areas (van der Vleuten-

Balkema 2003): functional (adaptability, maintenance, reliability, robustness and waste), economic (investment and O&M costs), environmental (emissions and utilisation of resources: energy, space, nutrients and water) and socio-cultural (related to acceptance, expertise, institutional, participation and sustainable behaviour).

2.2.2.2 Selected Evaluation Criteria

It is evident from the examination of evaluation criteria used by different researches that the criteria used most often for screening of treatment trains are their performance and cost. In some approaches (Aramaki et al. 2001; Chang and Liaw 1990), these were the only criteria used in determining the most appropriate treatment alternatives. Also, these criteria are typically quantified, along with land requirements and the utilisation of resources (in some cases). Functional criteria, such as reliability and adaptability, and socio-cultural criteria, on the other hand, are typically considered in a qualitative way. The evaluation criteria considered in this thesis are similarly grouped into quantitative and qualitative criteria, the latter of which are further divided into technical and environmental, as shown in Table 2.1. A brief discussion on each of these criteria is provided in this section, and the detailed information that was incorporated in the knowledge base is provided in Section 3.2.

Table 2.1 Short List of Treatment Train Evaluation Criteria

Type of Criteria	Name of Criteria
Quantitative	Effluent quality Cost Land requirements Labour requirements Sludge production Concentrates production Energy consumption
Qualitative Technical	Reliability Adaptability to upgrade Adaptability to varying flow rate Adaptability to varying water quality Ease of O&M Ease of construction Ease of demonstration
Qualitative Environmental	Chemical requirements Odour generation Impact on groundwater

Performance / Effluent Quality

The performance is obviously a decisive factor in the selection from candidate unit processes, as the generated treatment trains have to meet specific criteria with regards to treated water quality. The criteria themselves vary both in terms of the contaminants that have to be considered for various end-uses of reclaimed water, as well as in their required maximum concentrations. For example, the 5-day Biochemical Oxygen Demand (BOD_5) limits of reclaimed wastewater to be used for agricultural irrigation (WHO category C – products consumed cooked) in Israel are 35 mg/L, while the USEPA guidelines specify concentrations <30 mg/L and in Kuwait they are 10 mg/L (Salgot and Hueartas 2004). The literature review paid attention to the inclusion of contaminants in planning and evaluation of water reclamation schemes and particularly the information on their allowed concentrations included in a decision support tools.

The number of contaminants considered in wastewater reclamation and reuse worldwide is indeed growing, with continually emerging new contaminants of concern in various water sources. Although bulk of the materials reviewed dealt with conventional wastewater treatment, some explicitly considered reuse. The MOSTWATER considers 11 different pollutants relevant to wastewater reclamation and reuse, and effluent quality limits for 34 types of end-uses for nine different guidelines. Finney and Gerheart (2004) provide a list of 64 water quality parameters to the user, and concentration limits for 17 different end-uses/standards.

A typical approach used by other researchers in specifying the capacity of a unit process to remove pollutants from wastewater has been to express it in terms of percent removal. While it is well known that the process performance will vary depending on the influent conditions (quantity and quality), process design and operating practices, this simplification was deemed appropriate for screening of treatment trains. In this thesis, this simplification was relaxed to some extent by using additional expressions to quantify the effluent quality from a unit process, as detailed in Section 3.2.

Cost

The cost of wastewater treatment has been the criterion used most often for evaluation of alternatives and the selection of preferred treatment, and a number of methodologies have been developed to estimate wastewater and water treatment costs (Gillot et al. 1999; Hydromantis 2003; Moch et al. 2002; Richard 1996; Sipala et al. 2003; USEPA 1981; USEPA 2000; USEPA 2001; Wilbert et al. 1999). In addition to these methodologies, several tools that can be used by planners have been developed or upgraded recently. The Computer Assisted Procedure for the Design and Evaluation of Wastewater Treatment Systems (CAPDET) (USEPA 1981) is one of the early models that was used to evaluate wastewater treatment alternatives (McGhee et al. 1983), which has been updated and is currently available as CapdetWorks (Hydromantis 2003). The Plan-it STOAT^{TM} software (Landon et al. 2003) for calculation of the capacity, effluent quality and capital and operating costs for wastewater treatment unit processes was developed by Water Research Centre in collaboration with Camp, Dresser & McKee.

The capital costs involved in constructing wastewater treatment facilities cover the costs of building and installing unit process equipment, land costs, and also ancillary costs associated with the provision of standby units, emergency power, instrumentation and alarms, process piping, site development, administration buildings, etc. Although capital costs have been used frequently in the past as the single criterion, the O&M costs associated with wastewater treatment cannot be ignored. These costs include the staff salaries, and expenses related to power consumption, chemicals consumption and repair and maintenance of equipment. The lifecycle cost, computed by combining amortized capital costs with annual O&M costs (Richard 1996), is used as an estimate of the overall costs of treatment alternatives.

Costs of various treatment processes vary according to local conditions, and in the case of some proprietary technologies the information may not even be available. The methodologies for cost estimation of wastewater treatment processes are numerous, but not easily comparable or globally appropriate due also to the assumptions used in their development, which are also not always specified. In this research, the cost information for unit processes was obtained from partners involved in the AQUAREC project, who compiled the information from their experience in constructing and operating WWTPs, and also from a number of published sources.

Land Requirements

Since unit processes for treatment of wastewater utilise different mechanisms for removal of different pollutants, the footprint they occupy to process the same quantity of wastewater differs tremendously. Processes designed for removal coarse pollutants, such as screens, occupy just several square meters, while comprehensive processes such as lagoons require long residence/treatment times and occupy areas measured in hectares. Therefore, differently composed treatment trains can also occupy vastly different spaces, and the land they occupy can be a key determining factor in their selection, particularly in densely populated urban areas. Several different methods for specifying the unit process land requirements are included in the methodology developed here, with details provided in Section 3.2.

Labour Requirements

The labour required to operate different wastewater treatment processes varies both as a result of process design, and with different levels of automation used. Although this research is not specifically targeting developing nations, where the availability of skilled labour is one of the key criteria, the labour required to operate treatment trains needs to be considered in the selection process. While it is often desirable to minimise the labour requirements, it has been suggested that a country with high unemployment should consider a labour intensive treatment option more preferable from the sustainability point of view (Balkema 1998).

A difficulty in estimating labour requirements for individual unit processes arises from the fact that operating staff hours are typically distributed to activities dealing with several processes, as well as administration, which makes the development of expressions for individual unit processes more difficult. Nevertheless, this work includes several expressions and default values for specifying the labour requirements of individual unit processes which are used to estimate total treatment train labour requirements. The expressions and default values were developed using literature values and from experience of AQUAREC project partners gained in operating wastewater treatment facilities.

Sludge and Concentrate Production

Both the quantity and quality of sludge produced by unit processes varies, and they have been considered in the past in determining most appropriate wastewater treatment trains. The sludge, comprised of solids and biosolids and typically containing 0.25 to 12 percent solids depending on the operations and processes used (Metcalf and Eddy 2003), can be treated, disposed of and reused in a number of ways. Concentrates, a by-product of wastewater treatment by membrane processes, can also be disposed of in a number of ways including blending with other wastewater flows in smaller facilities. For larger facilities, the ocean discharge is most commonly used and least costly option, which is not available for inland facilities that require more expensive and environmentally sensitive options such as transmission through long pipelines, evaporation, deep well disposal or spray irrigation. The choice of ultimate disposal of sludge and concentrates is driven to a large extent by local conditions and regulations, and could potentially have significant implications in treatment train selection.

In this work, the quantities of sludge and concentrates produced are estimated but the quality is not considered explicitly. Since the local experience and regulations heavily influence the selection of disposal alternatives, the choice is left open to the user and is viewed as having a financial impact in the optimisation of treatment alternatives. Provisions are made in the methodology to specify the unit costs of treatment and disposal options with default unit costs developed based on operating experience with real WWTPs of AQUAREC project partners in Belgium.

Energy Consumption

Most of the wastewater treatment processes utilise some amount of electrical energy in their operations. The usage varies depending on the electrical equipment used, from activated sludge processes requiring large amounts of energy for aeration to lagoon systems that require little or no energy for operation. The energy consumption is used in the evaluation of treatment trains, since it represents a large portion of the overall operation costs involved in treating wastewater. While it has been reported that this portion may be 30 percent or higher for conventional wastewater treatment (Metcalf and Eddy 2003), in the case of water reuse this portion is even larger, due to the need for advanced treatment. Finally, the availability of energy infrastructure in some countries may preclude the use of some energy-intensive processes altogether. The methodology developed here includes energy consumption estimates for unit processes, which can be used to provide an approximation of energy consumed by treatment trains.

Reliability

Wastewater treatment plant reliability, defined by Metcalf and Eddy (2003) as the percent of time that the effluent concentration meets specified permit requirements, is of particular importance in water reuse where inadequate treatment can have immediate and serious health consequences. While statistical and graphical methods can be used to assess the reliability of different unit processes based on performance data, at a planning level this information is not available and qualitative indicators are quite commonly used (Dinesh 2002). The approach of qualitative indicators was taken in this work, and each unit process was given a qualitative mark for reliability as described in Section 3.2.

Adaptability to Upgrade

Water infrastructure is typically developed in stages due to the growth in demand, and this is also the case for wastewater reclamation projects. In potable water systems, the quality requirements of the water delivered to users is constant, but in reclaimed water systems this may change over time as new customers are added. Treatment trains used in water reclamation may have to be modified according to the demand, both in terms of quantity and quality, by adding additional treatment steps or by combining with other technologies. For some packaged unit processes, this may be relatively easy, but processes that need large tanks and accordingly sized equipment to operate may require extensive investment to meet future demand. Therefore, adaptability to upgrade is used in this work to reflect the ease with which treatment trains could be upgraded or combined with other processes.

Adaptability to varying flow rate and changes in water quality

The adaptability to varying flow rate and quality of the inflow refers to the resilience of the treatment system to the changes in operating conditions. Although all unit processes are designed for certain influent conditions, some are more adaptable to changing conditions in terms of flow and quality of inflow. This is the case even with preliminary treatment processes where, for example, a grit chamber might not cope with changes in operating conditions as well as a bar screen, due to its limited storage capacity and detention time. These factors are accounted for by using separate qualitative marks assigned to each unit process in the evaluation of treatment trains.

Ease of O&M

The difference in efforts required to operate and maintain different treatment processes can be quite large. Natural treatment processes such as lagoons, for example, require only periodic maintenance while biological treatment and membrane processes require extensive monitoring and control. The ease with which each unit process can be operated is reflected in qualitative marks assigned to them.

Ease of construction

Construction of treatment processes can require specialised knowledge and skills as well as certain type of site conditions. On the other hand, packaged treatment processes are typically constructed off-site and put in place with no extensive construction activities. The criterion is used in this work to reflect this with qualitative marks given to each unit process.

Ease of demonstration

In order to determine the operational policy that needs to be applied to achieve the required performance, some treatment processes need extensive pilot studies. This is particularly the case with proprietary technologies that use chemical to enhance the pollutant removal. On the other hand, well-established and not overly complex treatment processes are sized using design standards that ensure certain level of performance without the need for extensive testing. A qualitative mark assigned to each unit process is used here to indicate the effort required to demonstrate the performance of the process.

Chemical requirements

The addition of coagulants is necessary for certain treatment processes, designed to achieve high levels of contaminant removal in conjunction with high throughput. Other processes that use filtration as the primary pollutant removal mechanism require periodic cleaning with chemicals, while chemicals are used as the primary pollutant removal in processes such as chlorination. On the other and of the spectrum, mechanical and natural treatment processes require very little chemicals in their operation. The level of use of chemicals for treatment by different processes is indicated with qualitative marks.

Odour generation

Different processes that may achieve the same level of removal of pollutants can emit very different levels of air pollution. Although odour control equipment can be used to virtually eliminate this concern, it requires additional costs and operational complexity, which is not required if certain processes are used. To reflect these differences between processes' odour generating characteristics, a qualitative mark is used.

Impact on groundwater

The final qualitative criterion used to evaluate treatment trains' environmental impact relates to their potential for groundwater pollution. Although the potential for groundwater pollution is very low for the majority of municipal wastewater treatment processes, some processes such as Soil Aquifer Treatment (SAT) have the potential to seriously degrade groundwater. To reflect this, a qualitative mark is assigned to each unit process.

2.2.3 Optimisation of Treatment Trains

The earliest reported research in the area of waste treatment system optimisation models (Rossman 1979) was the development of a linear programming (LP) model for wastewater treatment plant design presented by Lynn et al. (1962), and various LP derived approaches have been used since. The wastewater treatment design problem was formulated as a multiple-constrained shortest path models by Elimam and Kohler (1997), and optimised using Integer Linear Programming (ILP) to find the least-cost design alternative. Wu (2002) defined the optimal treatment train as that meeting a given treatment requirement at a minimal cost, and used Stochastic Integer Programming to determine the optimum combination and efficiencies of various unit processes in multistage treatment plant.

A series of approaches were developed beginning with the late 1960s, which were based on dynamic programming (DP), geometric programming and nonlinear programming. The approach presented in (Evenson et al. 1969) minimizes the cost of treatment, subject to a constraint on the amount of pollutant removal (BOD in this case). Using DP, the state variable is the pollutant concentration, while the stages correspond to treatment processes. The decision variable at each stage is the amount of pollutant removal by the treatment process that is bounded within some limits. The unit process costs are represented with nonlinear functions, but an assumption is used that the pollutant concentration changes linearly with the process level. Schwartz and Mays (1983) used DP for optimal (least-cost) selection of treatment alternatives over time, and Ellis et al. (1985) used a stochastic DP to determine least-cost treatment sequences that produce the effluent of required quality for a probabilistically-generated influent, by representing the inflow contaminant concentrations as log-normally distributed random variables.

The works of Ecker and McNamara (1971) and much later Smeers and Tyteca (1984) were primarily concerned with developing the optimal cost curves for various levels of performance that could be used in regional water quality models. Both groups of researchers used general (but different) representations of wastewater treatment plants, and optimised the design variables of each unit process using geometric programming. Berthouex and Polkowski (1970) optimized a wastewater treatment plant by combining different kinetic models for different processes of a “typical” treatment train. They used nonlinear programming to determine optimal values of decision variables (cross-sectional areas of the primary and secondary clarifiers, concentration of secondary clarifier underflow solids and effluent BOD), and also incorporated their uncertainty which, interestingly, led to only slight increases (3-4%) in total cost above the minimum cost without uncertainty.

Uncertainty in design loads, as well as stoichiometric and kinetic parameters used in the design of treatment facilities was also considered in (Doby 2004), who developed a design procedure using stochastic programming taking both cost and reliability into account. The methodology allows the designer to choose for a level of uncertainty in stoichiometric and kinetic parameter the design values with an optimum cost-reliability trade-off.

Various heuristic methods have also been used over the years for optimal planning of treatment alternatives. Chang and Liaw (1990) dealt with the large number of treatment combinations that exist when a number of unit processes are considered in each stage of treatment and the number of stages in a system increase using total enumeration and implicit enumeration to eliminate infeasible treatment combinations and identify the least-cost treatment system. The authors proposed a bounded implicit enumeration approach to make the implicit enumeration more efficient, and concluded that the bounded implicit enumeration is consistently more efficient than the total and implicit enumeration approaches in identifying least-cost and feasible treatment alternatives. The approach by Krovvidy (1998) uses an A*-based heuristic search to generate least-cost treatment trains that meet the effluent quality requirements, as well as a Case Based Reasoning (CBR) approach that utilises knowledge from a case base of previously optimised treatment trains in the selection process. The CBR was reported to significantly reduce the search effort.

Loughlin et al. (2001) presented a comparison of three optimisation techniques applied to wastewater treatment plant design: random search, conjugate-gradient procedure and genetic algorithm (GA). To evaluate potential designs and identify the least-cost design, the authors used a WWTP design model based on one developed by the Water Research Council of South Africa for nutrient removal plants, and optimised twelve design parameters. They concluded from a case study conducted that the GA-based approach was able to produce solutions that were almost always more cost-effective than designs produced by other two techniques.

The MOSTWATER software, developed by Dinesh (2002), used GA for the selection of appropriate treatment trains using a standard single objective GA in which the objective function includes the cost of treatment, performance, land requirement and sludge production as well as a range of qualitative criteria. The features of the GA optimisation incorporated in MOSTWATER are discussed further in Chapter 4.

Various screening and ranking techniques have also been applied to the selection of appropriate treatment alternatives. Ellis and Tang (1991) and Tang and Ellis (1994) addressed the multicriteria issue that exists in the selection of wastewater treatment technology in developing countries using AHP, a technique that was also used in (Bick and Oron 2004) and (Addou et al. 2004) to generate appropriate wastewater treatment plant for communities in Israel and Morocco, respectively. Finney and Gerheart (2004) incorporated the Delphi selection process in the WAWTTAR DSS for the selection of appropriate treatment, while Loetscher and Keller (2002) developed and implemented a two-step selection process based on Conjunctive Elimination and Multiattribute Utility Technique in the SANEX™ DSS for the selection of sanitation systems in developing countries.

It should also be mentioned that a number of different optimisation approaches have been applied in the field of chemical engineering for optimisation of industrial water recycling schemes. These problems involved not only treatment process selection, but also pipe networks supplying water of various qualities between water-using operations and treatment processes. Jodicke et al. (2001) developed a mixed integer linear programming (MILP) model for least-cost design of industrial water reuse network, which requires data such as process location, current water demand, and binary information on the reuse possibilities of wastewater streams as input. Gunaratnam and Smith (2002) used Mixed Integer Non-Linear Programming (MINLP) for designing an integrated water network that considers simultaneously water-using operations and effluent treatment systems at least cost. A toolbox called GAPinch was developed by Prakotpol and Srinophakun (2004) which uses GA for design of an optimum water usage network leading to minimum freshwater consumption.

2.3 Distribution of Reclaimed Water

The second aspect of the literature review dealt with distribution of reclaimed water. The first and perhaps most obvious component of any water distribution system is the network of pipes that convey the water over distances. Therefore, the literature review covered methodologies used in the planning and optimal design of pipe networks. A review of the DSS related to storage facilities in reclaimed water distribution systems was considered as being of key importance, given that most water reclamation schemes include significant reservoirs. The sizing and operation of reservoirs in reclaimed water systems, however, differs from the design considerations typically considered in potable water systems, as it is focused on the annual water balances in addition to the daily operation. Therefore, the literature review also extended into the area of optimal design and operation of regional reservoir systems.

2.3.1 System Layout

The layout of the distribution system for any water reuse scheme will be governed by a number of factors, including the locations of existing or planned water reclamation facilities in relation to potential customers, topography, soil and geological conditions, land use, right-of-ways, existing and planned roadways, etc. Identification of potential reclaimed water users can be carried out using aerial photographs, maps and databases on existing water customers that may be interested in substituting part of potable supply with reclaimed water, and identifying possible new uses. To perform these tasks, various approaches based on Geographical Information Systems (GIS) have been developed, which are presented below. The use of GIS to perform the task of identification of potential reclaimed water users more efficiently was described in (Francis and Dotson 2000). The authors used a GIS-based database to conduct a comprehensive search of 1,500 potential new customers in the City of Tuscan in Arizona that included land parcels with uses generally associated with turf irrigation, such as golf courses, parks, schools, agriculture, hotels/resorts, cemeteries, and uses with large common areas, such as commercial and multi-family residential. In his assessment of opportunities for municipal wastewater reuse, Meng (2005) also used GIS to integrate data on WWTPs and publicly available information on potential reclaimed water users such as parks, golf courses and major industrial facilities in the Chicago area.

Once the potential reclaimed water users are identified, the distribution system layout needs to be developed, including the size and location of key facilities (pump stations, storage reservoirs) (Holliman 1996). For determining the system layout, Luettinger and Clark (2005) used GIS-based route selection process to provide a rational basis selecting a single alignment corridor for a pipeline from hundreds of potential alternatives. Kim et al. (1998) developed an information system based on GIS and Multicriteria Decision Analysis (MCA) technique, which was used not only for suggesting some alternatives for pipeline layout but also finding the best alternative among them, by taking into account several hydraulic, economic, and socio-technical factors. Methodologies for determining optimal layout of waster distribution networks include iterative procedures (Bhave and Lam 1983), evolutionary algorithms (EA) (Davidson 1999; Stanic et al. 1998; Walters and Lohbeck 1993; Walters and Smith 1995), and harmony search (Geem et al. 2000).

A variety of methodologies have been proposed for simultaneously deriving optimal distribution system designs by simultaneously addressing the layout and sizing aspects. Rowell and Barnes (1982) presented a two-level hierarchically integrated system of models for the layout of single and multiple source water distribution systems, where a non-linear programming model is used to select an economical tree layout for major pipe links and an integer programming model adds the loop-forming links to satisfy a specified level of reliability. Morgan and Goulter (1982) used two linked linear programs, where one solved the layout and the other determined the leas-cost components sizes, which they later improved by incorporating a Hardy-Cross network solver (Morgan and Goulter 1985). Further improvements to these approaches were made by Lansey and Mays (1989) and Duan et al. (1990), who included sizing and location of pumps, storage tanks, and valves as well as pipes in the optimisation. A two-step approach was also used by Cembrowicz (1992), who used GA to determine the optimal layout and LP for establishing the least-cost pipe diameters. Taher and Labadie (1996) presented a GIS-based DSS called WADSOP, in which an NLP-based network solver and an LP-based optimal design model are used interactively in a convergent scheme to determine least-cost design, including the layout. Tanyumboh and Sheahan (2002) developed a maximum entropy based approach in considering jointly layout, reliability and pipe sizing optimisation problem. Finally, a GA approach for optimum layout and optimum hydraulic design of a branched pipe network was developed by Hassanli and Dandy (2005). The authors reported that the application took in the order of several minutes to find optimal solutions when applied on a case study consisting of 11 demand nodes and a source node.

In practice, the reclaimed water pipelines are typically placed beneath city streets and other public rights-of-way, such as alleys or easements. The selection of layouts is therefore heavily influenced by the locations of these corridors. In general, the reclaimed water distribution networks are not as extensive as those used for potable water distribution. A survey of reclaimed water providers in the United States, for example, showed that less than ten percent of respondents actually provided service to single-family residences (Griffith 2003). Further reviews of existing and planned water reclamation schemes that involve extensive distribution confirmed that, at a planning level, it is sufficient to concentrate on branched or tree-like layouts. Due to these factors, the methodology developed in this research requires that preliminary pipeline layouts, as well as locations of pumping and storage facilities, be determined by the user and used as input to the DSS.

2.3.2 Sizing of Pipes and Pumps

In the context of optimal water distribution system design, simulation models are typically used in conjunction with mathematical optimisation, with the design problem often viewed as a least-cost optimisation problem with pipe diameters and pump capacities being main decision variables, assuming a pre-determined pipe layout. A comprehensive review of the models available for piped distribution networks is available in (SKAT 2002). Labadie and Herzog (1999) summarised the difficulties that arise in designing municipal water distribution system, which are:

- large-scale and spatially extensive,
- composed of multiple pipe loops to maintain satisfactory levels of redundancy for system reliability,
- governed by nonlinear hydraulic equations,
- designed with inclusion of complex hydraulic devices such as valves and pumps,
- impacted by pumping and energy requirements,
- complicated by numerous layout, pipe sizing, and pumping alternatives, and
- influenced by analysis of tradeoffs between capital investment and operations and maintenance costs during the design process.

A number of methodologies have been developed in the past that consider not only the pipe-sizing problem, but also include other important aspects of water distribution system design such as cost data implications, reliability and redundancy of designs, uncertainty, and sizing decisions influencing future development and demands. An extensive overview of issues addresses in optimal planning and design of water distribution system design can be found in (Walski et al. 2003), while Ostfield (2005) classified the optimisation methods applied to the pipe-sizing problem as follows:

- methods that decompose the problem, sizing the pipes for fixed set of flows using LP and altering the flows using a gradient or a sub-gradient technique,
- methods that link network simulation programs with general nonlinear optimisation code,
- methods that use a straightforward NLP formulation, and

- methods that link network simulation programs with evolutionary techniques such as GA, ant colony and shuffled frog leaping algorithm.

The focus of the literature review was on techniques considered relevant to current research, which is concerned with wider issue of planning of integrated water reuse systems. Also, the methodology developed here considers branched systems, where the calculation of flows in each pipe is straightforward for a given demand, whereas in looped systems, which are looped to provide network reliability, resilience and flexibility, an infinite number of flow distributions exists that meets the demands of connected users. Therefore, computationally simpler methodologies can be used for optimal sizing of branched systems compared to those used for traditional municipal water distribution systems.

The methodology commonly used for optimisation of both branched and looped distribution networks has been LP. The LP approach for optimal sizing of branched water distribution networks offers significant computational efficiency over other mathematical optimisation techniques. Other advantages of LP, highlighted in (Labadie 2004), are:

- ability to efficiently solve large-scale problems,
- convergence to global optimal solutions,
- initial solutions generally not required from the user,
- well-developed duality theory for sensitivity analysis, and
- ease of problem setup and solution using readily available and low-cost LP solvers.

Labye (1966) first presented a LP methodology for optimal sizing of pipes and determination of reservoir levels in a branched pressurised water network. Subsequently, approaches presented in (Gupta 1969) and (Gupta and Hassan 1972) used LP-based design procedures for water distribution systems with single and multiple supply points, respectively, in which the decision variables were the lengths of pipes chosen for each link from a set of candidate diameters (e.g. commercially available pipes). These approaches, capable of producing realistic split pipe solutions, require unit costs of different diameter pipes to compute the least-cost design.

Alperovits and Shamir (1977) adopted the same split pipe approach with regards to pipe sizing, and further expanded the methodology by considering multiple loading conditions and by including the sizing of pumps, location of valves and sizing of operational reservoirs in the optimisation. Since the candidate diameters in this approach have to be specified in advance, the solution generated by LP may not be globally optimal if a link is found to have its entire length composed of a single pipe whose diameter is at the limit of adopted pipe sizes. To address this issue and limit the search space, Bhave (1979) developed a method based on the critical path concept for selection of the optimal sets of pipe sizes to be included in optimisation by LP.

The method by Lai and Schaake (1969) and later by Quindry et al. (1981) considered pipe sizes as continuous design variables, which was also adopted by for optimisation of looped distribution networks. A critical review of this approach by Templeman (1982) highlighted that the substitution of the non-linear cost function in the least-cost optimisation with a linear one may lead to solutions that represent a global minimum, a local minimum, or not even a minimum at all.

More recently, Samani and Mottaghi (2006) demonstrated the use of ILP in an iterative procedure for the selection of pipe diameters for each link that achieve minimum total water distribution system cost. Its limitations are the lack of ability to determine the pump discharges and heads, as well as the reported fact that the success and speed of the optimisation process both depend on the selection of initial values for decision variables.

2.3.3 Storage Sizing

In using traditional ground and surface water resources, the source of water usually provides the storage function. This is not the case in reclaimed water sources, which provide a relatively constant flow throughout the year, but typically no storage function, therefore presenting different challenges with regards to the management and allocation of water. The sizing and operation of reservoirs in reclaimed water systems also differs from the design considerations in potable water systems, as it is often focused on the annual water balances in addition to the daily operation. The operational storage, required to balance the diurnal fluctuations in water demand, should be sized to hold up to two times the average summer day demand volume (Holliman 1996), so it is not substantial compared to volumes required to satisfy seasonal variations. Seasonal storage requirements can have substantial impact, particularly on the capital cost of integrated water reuse schemes, depending on the volume and pattern of estimated reclaimed water demands (USEPA 2004). Therefore, it was considered imperative to include seasonal storage elements in the development of the DSS, and the literature review extended into the area of optimal planning and design of reservoir systems.

The focus of a large number of optimisation methodologies developed in the past has been the optimal operation of reservoir systems, and a comprehensive review of these methodologies can be found in (Labadie 2004). Majority of the methodologies reviewed deal with operational and management aspects of multireservoir systems. As such, they are typically not capable of formulating the problem so that the optimal size of individual storage elements can be determined concurrently with the operating strategy, which is the aspect that needed to be addressed in the development of this DSS.

A methodology which was found to be capable of simultaneously optimising the operational policy and storage sizing in multireservoir systems belongs to a class of water supply models that use network flow representation, a concept first presented in (Evanson and Mosley 1970), which can be formulated and optimised in terms of linear programming. The approach essentially represents elements of interconnected reservoir systems as capacitated flow networks, and is also referred to as Network Flow Programming (NFP). The elements included are both real and artificial nodes and arcs (links), with real elements representing the physical components of the system and artificial elements introduced to ensure the mass balance is satisfied, as illustrated in Figure 2.2. Different approaches used variations of the out-of-kilter method (Ford and Fulkerson 1962) and the primal simplex algorithm (Kennington and Helgason 1980) to solve the formulated problems, and later research identified the dual ascent RELAX algorithm (Bertsekas and Tseng 1994) as the most efficient network solver (Kuczera 1993).

A number of researchers subsequently adopted the network flow optimisation approach, including the work of (Kuczera 1989), who introduced several new elements to the basic formulation. These new elements covered shortfalls in supply, seasonal reservoir targets, instream flow requirements, variable pumping costs and evaporation. Khaliquzzaman and Chander (1997) took these improvements to the basic NFP representation, and extended it in an attempt to determine optimum capacities of each reservoir in a system of interconnected multiple reservoirs serving multiple demand areas. This methodology, which was adopted in the development of the DSS for integrated water reuse, is described in more detail in Section 3.4.

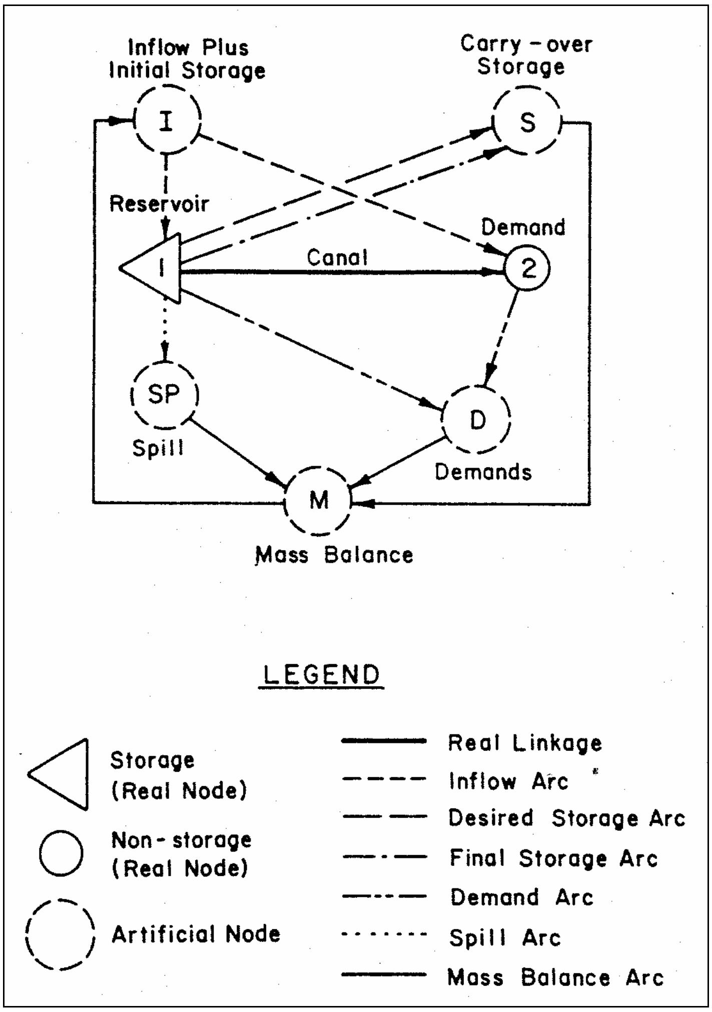


Figure 2.2 Network Flow Representation of a Reservoir System (Labadie et al. 1986)

2.3.4 Changes in Reclaimed Water Quality

It is well acknowledged that the removal of contaminants found in wastewater to the point where its quality is comparable to that of potable water is possible with advanced treatment technologies. However, that level of treatment is typically not provided, so the reclaimed water that is transmitted in the distribution system to customers is typically of lower quality. A wide variety of literature dealing with characterisation, planning, and management of water quality in potable water systems exists and has been reviewed. The summary presented here focuses primarily on investigations that dealt with reclaimed water.

The key concern in reclaimed water distribution systems deals with changes in microbial quality of water from its source to the destination. In addition to increasing the risk to public health and safety., the presence of certain microbes in reclaimed water increases the problems related to biofilm formation and growth, which leads to increases in pipe corrosion, adverse affects on pipe hydraulics, generation of bad tastes and odours, and proliferation of macroinvertebrates (USEPA 2002b). Factors that influence microbial proliferation in water distribution systems have been summarised by Levi (2004) as transit times, system condition, construction materials, water temperature, disinfectant residual, hydraulic conditions and initial physical, chemical and microbial characteristics of the treated water. The complexity of determining and managing microbial growth in distribution systems was illustrated by LeChevallier et al. (1996), who presented the results of an 18-month survey of 31 water systems in North America focused on determining the factors that contribute to the occurrence of coliform bacteria in drinking water. They concluded that its occurrence in a distribution system is dependent upon a complex interaction of chemical, physical, operational, and engineering parameters, all of which must be considered in devising a solution to the regrowth problem.

In contrast to the large number of studies dealing with water quality in potable distribution systems, very few investigations have focused entirely on reclaimed water. Funamizu et al. (2000) demonstrated a method for measurement of bacterial growth potential in reclaimed water, and concluded that the reduction of initial bacteria count delays their growth, but does not control the maximum level. Therefore, the authors concluded that the retention time in the distribution system (including storage) should be accounted for in designing and operating the disinfection process, but that the bacterial growth could be controlled by regulating organic carbon concentrations. With respect to the selection of treatment processes, the authors concluded that the ozonation increased the growth potential while carbon adsorption reduced it. Laurent et al. (2005) also reported that high-dosage ozonation increased the potential for the regrowth of fixed and suspended bacteria substantially in raw water samples, and that the application of full treatment (coagulation-ozonation-filtration) or high-dosage chlorination/de-chlorination supported very little bacterial regrowth.

The criteria for the quality of reclaimed water are typically specified in various guidelines as concentrations of various contaminants at the treatment facility. The same guidelines typically include recommendations on operating practices that will ensure that the quality of the reclaimed water that reaches the end-user is not compromised in the process of transmission. The merit of this assumption can be argued both ways. On the one hand, some operators of water reclamation facilities have contended that the design of reclaimed water systems is typically approached from a hydraulic perspective only, which makes the management of water quality difficult once the systems are built and operated (Jew 2005). In addition, a recent survey of reclaimed water providers in California found no clear link between maintenance practices and water quality in the distribution system, with results that also indicated that very few agencies had a regular written maintenance program despite the fact that quite a few experienced customer complaints (Daniel 2005).

Manios et al. (2006), on the other hand, recently conducted extensive sampling of water quality in a reclaimed water distribution system in Crete used to convey tertiary treated wastewater that seems to support the approach taken in many guidelines. The analyses of samples collected at the WWTP and throughout the distribution system showed that three (COD, conductivity and pH) out of the four measured parameters were not easily affected by either the distance or the time gap between a remote location and the exit of the WWTP. Similarly, a study dealing with risk of using reclaimed water in public park irrigation conducted by Navarotto and Jiménez (2005) found no important changes in BOD_5, Turbidity, Conductivity, Total Suspended Solids and Total Coliform in the distribution system under normal operating conditions, although high microbial activity was detected in periods where the system was stagnant for prolonged periods of time.

The changes in reclaimed water quality also occur in storage reservoirs, particularly if they are of the seasonal variety with large volumes. The ability of reservoirs to remove pollutants is utilised in many water reclamation schemes that include significant storage volumes not only for the purpose of balancing the seasonal demand, but also as active treatment elements. Juanicó and Dor (1999) presented a comprehensive overview of the issues related to the design and operation of these facilities, including their expected performance, and also developed models for predicting their efficiency in removal of pollutants. Barbagallo et al. (2003) described the development of non-steady-state first-order kinetic model used to predict the microbial water quality of the effluent from wastewater storage systems used to provide tertiary treatment in Sicily, Italy. These and a number of other authors have concluded that it is necessary to use non-steady-state models to represent the complex physical, chemical and biological interactions that occur in these facilities.

Various management options can be implemented to improve the reclaimed water quality in distribution systems, including implementation of a flushing/ superchlorination program, strategic placement of re-chlorination stations within the system, system looping and decreasing pipe diameter size (Jew 2005). Detailed investigations of the effects of implementation of these options, and particularly their combinations, seem to be an area that needs to be developed. Icekson-Tal et al. (2003) reported that biofilm growth in the long pipelines conveying tertiary treated effluent in Israel is controlled effectively by intermittently applying chlorine based compounds at a 10 mg/L dosage for a few hours. For the same system, earlier investigations showed the severity of the impact of inadequate maintenance regime on the hydraulic performance of the system, where the Hazen Williams Coefficient changed from 142 (clean pipe) to 90 (fouled pipe) in ten years of operation resulting in a tremendous increase in pumping costs (Sack et al. 2002).

The main purpose of this review was to evaluate how the changes in water quality in the reclaimed water distribution system could potentially be included in the DSS. From the material presented above, it is evident that this issue is of concern from the public health and safety perspective, and also with regards to the management and cost issues. It is also apparent that this issue, which is complicated by numerous factors, is highly dependant on the configuration of the distribution system, its operation, and management and maintenance activities employed. While some aspects of system configuration and its impacts on the reclaimed water quality changes could potentially be integrated at planning level, the operational issues remain the most important aspect that can only be assumed to be appropriate at a planning stage. Therefore, the DSS developed here does not explicitly account for changes in reclaimed water quality in the distribution system.

2.4 Integrated Approaches

Various factors that need to be considered in the development of wastewater reclamation and reuse facilities plan and the complexity of the process have been highlighted in Section 1.1.4, which emphasised the need for integrated decision support tools. The purpose of this section is to present a summary of the literature reviewed, which either integrated some of the elements in a decision support framework, or was deemed relevant to this research.

With regards to integrated approaches developed for water reuse management modelling, several approaches have been developed dating back more than three decades. The work presented by Bishop and Hendricks (1971) was concerned with determining the optimal allocation of water between different sources and users. The authors formulated the problem as a trans-shipment problem with simplified linear treatment and transmission costs, and solved it using LP. Pingry and Shaftel (1979) later considered an expanded problem that minimised the total cost of delivery and disposal of water using an NLP formulation, but did not include different levels that may be offered by wastewater treatment. The costs included are those incurred for piping, treatment, source and disposal. A more comprehensive water reuse planning model in a multiperiod planning scenario was presented by Ocanas and Mays (1981), which considered different levels of treatment by water and wastewater treatment plants. The least-cost optimisation was carried out using two different techniques (generalised reduced gradient and successive LP with rejection), where the objective function included costs of piping, pumping and both construction of new and expansion of existing treatment facilities. Schwartz and Mays (1983) introduced a model for least-cost water reuse allocation and wastewater treatment alternatives, consisting of two DP models. The first model is used to allocate water to secondary users, and included wastewater treatment and water transportation costs (pipelines and pumping). Its results are used by the treatment DP model that determines the minimal total present cost treatment alternatives over time.

Oron (1996) developed a management model for integrative wastewater treatment and reuse systems, in which he considered the following components in the objective function: selection of treatment method, treatment costs, effluent quality, transportation and effluent storage costs, costs/benefits for environmental and health control, operation and maintenance expenses and design and contingency expenses. The model was applied on a case study that involved supplying six consumption sites from a centralised WWTP with reclaimed effluent meeting agricultural irrigation criteria, and optimal sizing of treatment and distribution facilities for two different layouts was conducted. The least-cost optimisation of pipe diameters was carried out using LP, while a trial-and-error procedure was employed for optimal location of reservoirs.

A study presented by Abu-Taleb (2000) addressed environmental policy tradeoffs with three objectives: determining the best wastewater collection system, deciding on the best site for the WWTP, and determining the wastewater treatment processes to be used. The study area of this work was a World Heritage Site in Jordan, which necessitated that a large number of evaluation criteria be used, and the selection process was carried out using multicriteria decision making (MCDM). The actual number of alternatives involved selecting from two alternative collection systems, nine potential sites, and three treatment options.

The treatment and distribution aspects of water reuse were also addressed by applying the concepts of industrial ecology to water management in an industrial park. Keckler and Allen (1998) demonstrated that a number of economical water reuse opportunities may exist for the network of facilities analysed. While this work considered water reuse opportunities between industries by allowing blending of various industrial effluents, Nobel (1998) developed a methodology which uses LP within a GIS framework to determine both feasible and cost-optimal reuse scenarios in a water reuse planning problem. The methodology identifies water reuse opportunities by simple quality matching and accounts for water purchase, treatment, and transportation costs, which are calculated in basic terms from straight-line distances between the potential sources and demand sites.

Zhang (2004) also used GIS for basic data processing and information gathering in the development of a comprehensive urban water reuse planning and management model, which used a network flow optimization model for modelling an urban water network. This approach included simplified modelling of multi-level wastewater treatment processes for water reuse and employed stochastic optimization methods to quantitatively model uncertainty issues in urban water reuse planning and management.

While the methods presented above include a number of optimisation algorithms that were applied for integrated treatment and distribution aspects of water reuse, there is a notable absence of random search techniques in this considerable body of research. Without going into further details, a note is made here that methods based on GAs have been applied in areas that bear relevance to the integrated municipal water reuse problem, such as the optimal design of water reuse networks in the chemical industry (Prakotpol and Srinophakun 2004), optimal design and operation of multiquality water networks (Ostfield 2005), optimisation of multi-stage logistic chain networks (Admia et al. 2002), allocation of waste-load in water bodies and watershed planning (Burn and Yulianti 2001; Cho et al. 2004; Yeh and Labadie 1997).

2.5 Summary and Conclusions

The literature review summarized in Section 2.2 of this Chapter revealed that there are several existing decision support systems for evaluating water reuse projects that focus entirely on the treatment aspects. Significant variety is apparent in existing decision support systems in terms of the number, types and characteristics (performance, cost, etc.) of unit processes included in their knowledge bases. In that respect, the most comprehensive knowledge base is contained in the WAWTTAR DSS, although its development was focused primarily on developing countries. Another feature worth noting is that the performance of unit processes was typically represented in terns of the percent of pollutant concentration that they were capable of removing.

With regards to synthesis of treatment processes, several of the approaches presented incorporated some form of rules that guide either the user or the optimisation algorithm in the development of treatment trains from a list of unit processes. The rules are kept quite rigid in some approaches, with the only methodology that claimed to include the flexibility of modifying or adding new rules to the user being the MOSTWATER tool. The sludge treatment and disposal options were also included in knowledge bases of some tools and included in the selection of optimal treatment trains.

An overview of the treatment train evaluation criteria used by different researchers is presented, and the choice of criteria was largely influenced by their intended purpose. The cost of wastewater treatment, however, was frequently recognised and included as perhaps the most important, in addition to the effluent quality achieved. A short list of criteria is developed that are used in this work and discussed in more detail. The selected criteria are both quantitative, such as the cost and performance of a treatment train which are expressed in numerical terms, as well as qualitative (technical and environmental), represented on an ordinal scale.

The review of literature also showed that different optimisation methods have been used in the past for optimal selection of treatment trains. Many early approaches used variations of LP in optimising the process selection. Later approaches recognised the economies of scale that exist in construction of wastewater treatment alternatives, and used methods capable of accounting for these non-linearities such as different enumeration techniques, NLP, DP, geometric programming, as well as screening methodologies with many objectives. The use of GA for optimal selection of treatment trains was demonstrated in MOSTWATER.

It has been stated that “no single factor is likely to influence the cost of water reclamation more than the conveyance and distribution of the reclaimed water from its source to its point of use” (USEPA 2004). The review of literature therefore extended to the area of optimal planning and design of distribution systems, where a particular attention was paid to methodologies relevant to distribution of reclaimed water. Several approaches are presented that deal with the selection of customers and choice of the system layout, as well as the methodologies that dealt with the drinking water system layout and sizing concurrently. Due to the reduced complexity of reclaimed water distribution systems, when compared with potable systems, it was decided that preliminary pipeline layouts and locations of pumping and storage facilities be determined by the user and used as input to the DSS.

The difficulties associated with design of municipal water distribution system are highlighted, and a classification of optimisation methods applied to the pipe-sizing problem is presented. The review of literature related to sizing of pipes and pumps in a water distribution networks then focused on LP-based approaches, which offer numerous advantages in application to sizing of branched pipe networks.

The importance of seasonal storage elements in reclaimed water distribution is discussed, and a review of network flow approaches capable of simultaneously optimising the operational policy and storage sizing in multireservoir systems is presented. Again, the focus of the review was on computationally inexpensive methods relevant to the overall aim of the DSS developed here.

Recognising that the quality of reclaimed water can potentially change in distribution systems, the final section of the literature review related to distribution systems focused on causes, concerns and management of these changes. Microbial changes in reclaimed water quality, which can lead to health and safety concerns as well as increased instances of biofouling, are highlighted as being of utmost importance. However, it is concluded the operational issues are the most important aspect impacting on the reclaimed water quality changes. Therefore, the DSS developed here does not explicitly account for changes in reclaimed water quality in the distribution system.

Lastly, a review of integrated approaches that account for both the treatment and distribution aspects of water reuse is presented. The review shows that the integrated models that include the distribution aspects of reuse schemes either do not incorporate provisions for the assembly of treatment trains from individual unit processes, or consider the treatment train components in a simplified manner by considering only general levels of treatment. In addition, both the layout of the distribution system and the optimal design of its components (pipes, pumping stations and storage reservoirs) are in most cases greatly simplified. While a number of optimisation algorithms have been applied in the reviewed approaches, no application of random search methods to the problem was found, although a number of GAs-based applications in related fields are referenced.

Chapter 3

DSS Evaluation Methodologies

3.1 Introduction

The approach for the development of DSS in this research was influenced by the results of the literature review presented in the previous chapter, and the intention to create a practical tool that would enable efficient planning level evaluation of integrated water reuse schemes. In this chapter, the key components of the methodology developed for evaluation of integrated water reuse schemes are presented in the context of approaches introduced in the previous chapter. Author's involvement in the AQUAREC project provided access to resources that facilitated the collection of information and also provided feedback throughout the development of the DSS methodology, both of which were very valuable in satisfying the practicality aspect of the developed tool.

A logical progression was followed in the development of the DSS that involved gathering of relevant information, development of methods to evaluate the integrated water reuse systems presented in this chapter, and the development and integration of optimisation routines. The aspects of water reuse systems included in the decision support tool were first identified, and the information requirements for each component were drafted. Much effort was then directed towards gathering this information, development of the knowledge base architecture that would accommodate all the requirements and populating the knowledge base with collected data. The results of these efforts are presented in the next section. Part of this information dealt with performance evaluation of treatment trains that are assembled during the generation of alternatives, and Section 3.3 describes the implementation of this information in the DSS. The sizing of the distribution system components is addressed in Section 3.4, which presents the methods used to size the pumping, storage and conveyance components. The development of the optimisation methodology that used the treatment and distribution system evaluations described here to conduct the optimisation of integrated water reuse systems is presented in Chapter 4.

3.2 Knowledge Base

As indicated earlier, the knowledge base approach was adopted in this research. Several examples of integration of a knowledge base into a DSS were presented in the previous chapter, which demonstrated that the choice of information that was captured in the model knowledge base was dictated by its intended use. In the following three sub-sections, the following elements of the developed knowledge base are described: unit process design and performance data, rules governing their combination and information relevant for preliminary evaluation of reclaimed water distribution systems

3.2.1 Unit Processes

The unit processes used in wastewater reclamation are generally classified according to the level of treatment they provide and their position in the treatment train. The classification adopted here is shown in Table 3.1, which also summarises the unit processes included in the knowledge base. The processes integrated in the knowledge base include conventional wastewater treatment processes used extensively in Europe and throughout the world. The determination of which processes to include in the knowledge base was also influenced by discussions with AQUAREC project partners (Joksimović 2003).

Preliminary processes, which remove coarse solids and floatables, are used to protect the downstream treatment processes. Although they are sometimes classified as primary, a distinction is made here primarily to reflect that the three processes included in the knowledge base under this category do not offer significant removal of majority of pollutants found in wastewater.

Six processes are included in the knowledge base under the primary treatment category. Of the processes included, most are conventional processes that partially remove suspended solids and organic matter from wastewater through the process of sedimentation. In the simplest of these processes, pollutants are removed by gravity requiring long residence times. In addition, anaerobic ponds reduce the organic matter content of wastewater by enhancing the activity of gas-producing microbes. Processes such as Dissolved Air Floatation (DAF) and Actiflo® include the addition of chemicals to accelerate the settling of suspended solids (and pollutants attached to them), while the direct membrane filtration is included as a novel technology that is still in experimental stage (Ravazzini et al. 2005).

Table 3.1 Unit Processes Included in the Knowledge Base

Category	ID	Unit Processes Name	Abbreviation
Preliminary	001	Bar Screen	BScr
	002	Grit Chamber	GrCh
	003	Coarse Screen	CScr
Primary	101	Fine Screen	FScr
	102	Sedimentation w/o Coagulant	Sed
	103	Sedimentation w/ Coagulant	SedC
	104	DAF w/ Coagulant	DAF
	105	Membrane Filtration	DMF
	106	Actiflo®	Acti
	107	Stabilization Pond : Anaerobic	SPAnbc
Secondary	201	High Loaded Activated Sludge + Sec. Sedim.	HLAs
	202	Low Loaded Activated Sludge w/o de-N + Sec. Sedim.	LLAs
	203	Low Loaded Activated Sludge w/ de-N + Sec. Sedim.	LLAsN
	204	Trickling Filter + Secondary Sedimentation	TF
	205	Rotating Biological Contactor	RBC
	206	Submerged Aerated Filter	SAF
	207	Stabilization Pond : Aerobic	SPAbc
	208	Stabilization Pond : Aerated	SPAer
	209	Stabilization Pond : Facultative	SPFac
	210	Constructed wetland: Free-Water-Surface Flow	WetFWS
	211	Constructed wetland: Subsurface Water Flow	WetSUB
	212	Membrane bioreactor	MBR
	213	Excess Biological Phosphorus Removal	EBPR
	214	Phosphorus Precipitation	Ppre
Tertiary	301	Filtration over fine porous media	MedF
	302	Surface filtration	SurF
	303	Micro filtration	MF
	304	Ultra filtration	UF
	305	Nano filtration	NF
	306	Reverse osmosis	RO
	307	Granular Activated Carbon	GAC
	308	Powdered Activated Carbon	PAC
	309	Ion exchange	IE
	310	Advanced oxidation – UV/O_3	AOO3
	311	Advanced oxidation – UV/H_2O_2	AOH2O2
	312	Soil Aquifer Treatment	SAT
	313	Maturation pond	MP
	314	Constructed wetland - polishing	WetPOL
	315	Flocculation	Floc
Disinfection	401	Ozone	O3
	402	Paracetic acid	PA
	403	Chlorine dioxide	ClO2
	404	Chlorine gas	ClG
	405	Ultraviolet radiation	UV

In secondary treatment, a combination of physical, chemical and biological processes are used to remove suspended or dissolved contaminants from primary effluents. Fourteen processes are included in the knowledge base under this category. The most commonly used process for secondary treatment is the activated sludge process, which has been combined in the knowledge base with secondary clarifiers. Rotating Biological Contactors and Submerged Aerated Filter are included in the knowledge base as efficient fixed film wastewater treatment technologies, used extensively in municipal and industrial wastewater treatment. Several natural treatment technologies, often regarded as environmentally friendly but land intensive, are also included in the knowledge base: stabilization ponds or lagoons (aerobic, aerated and facultative), and constructed wetlands (free-water-surface and subsurface). The Membrane Bioreactors couple the activated sludge process with a low-pressure membrane and their application in water reclamation has been growing in recent years. As the names suggest, the last two processes, Excess Biological Phosphorus Removal and Phosphorus Precipitation, are used primarily for removal of phosphorus from wastewater.

The list of unit processes included in the knowledge base as tertiary starts with processes used for filtering, or polishing, of secondary effluents through granular media. This is followed by a series of membrane filtration technologies, designed for removal of micro-pollutants, whose application in water reuse has increased tremendously in recent years due in part to significant reduction in their cost. The next two technologies (*GAC* and *PAC*) are used to remove negative ions from the water (e.g. ozone, chlorine, fluorides and dissolved organic solutes) by absorption onto the activated carbon, while the ion exchange is used for reduction of hardness or removal of nitrogen, heavy metals and total dissolved solids. Two types of advanced oxidation technologies are also included, which are used to oxidise complex organic constituents into simpler end products. Several natural treatment processes are also included in the knowledge base. The *SAT* is a sub-surface process which uses the soil matrix for removal of wide range of organic and inorganic constituents. Maturation ponds are used primarily for the removal of pathogens and nutrients through algal activity and photo-oxidation, and constructed wetlands used as a polishing step are also included as a technology that offers removal of nutrients at low cost and maintenance. Finally, flocculation is included in the knowledge base as a separate tertiary clarification processes intended to remove phosphorus and for the precipitation of silica, metals, and other inorganic compounds (Reardon 2005).

The final category of processes included in the knowledge base is disinfection. Ozone is a disinfecting agent that has been used for perhaps the longest time to address the microbial pollutants in water, in addition to Paracetic acid and chlorine. All three methods are included in the knowledge base, in addition to Ultraviolet disinfection, which has advantages over the other methods since it does not involve addition of any chemicals.

For each unit processes shown in Table 3.1 , the following information needed for analyses of treatment trains was assembled: process efficiencies for a series of pollutants, cost estimates, land and labour requirements, sludge and concentrates production, energy consumption and preference scores on qualitative evaluation criteria introduced in Table 2.1. The information on each unit processes that was assembled and included in the knowledge base is discussed in the remainder of this section, while the actual values used for each unit process are provided in the fact sheets included in Appendix A.

3.2.1.1 Efficiencies

The ability of a unit process to remove pollutants is a key function it performs in the process of reclaiming wastewater, and is perhaps its defining characteristic. However, due to the dynamic nature of the process, complexity of physical, chemical and biological processes that occur simultaneously, and variations in sampling and measurement of wastewater, the description of unit process performance is not an easy task. In addition, monitoring of water quality is typically performed only for a relatively small number of pollutants, and the performance is usually monitored for the ability of treatment facility to meet treatment targets and not to assess individual unit processes. The list of parameters considered in this research is shown in Table 3.2, which is followed by a discussion on methods that were use in estimating performance of unit processes covered by the knowledge base.

Table 3.2 Wastewater Parameters Included in the Knowledge Base

Parameter	Units	Short Identification
Turbidity	NTU	Turb
Total Suspended Solids	mg/L	TSS
Biochemical Oxygen Demand (5-day)	mg/L	BOD
Chemical Oxygen Demand	mg/L	COD
Total Nitrogen	mg/L	TN
Total Phosphorus	mg/L	TP
Faecal Coliforms	No/100mL	FC
Intestinal Nematode Eggs	No/100mL	INEggs
Escherichia Coli	No/100mL	Ecoli

A typical method used to describe process performance is by specifying the minimum, average and maximum percent of pollutant it is capable of removing. This approach was taken by many researchers in the past in the development of DSS for wastewater treatment and reuse. It is acknowledged that the true performance of a unit process depends on a variety of factors, including the plant location, influent characteristics, environmental conditions, loading and condition of the plant (Dinesh 2002). Although most of these factors cannot be accounted for at a planning level, a greater flexibility in terms of expressions used to calculate the average performance of unit processes was used in this research.

A total of fourteen expressions that can be used to calculate the removal efficiencies of processes are included in the knowledge base, all of which were developed based on literature review and inputs from AQUAREC project partners (Joksimović 2003; Joksimović 2004). A review of expressions is shown in Table 3.3, where the first is a percent removal equation, used frequently in other DSS to describe the pollutant removal performance of unit processes. The second expression is used to simply specify the minimum, average and maximum concentration of pollutant in the unit process effluent. The removal of microbial contaminants is typically expressed as log-removal, and the third expression is used for this purpose, while the fourth expression specifies the effluent quality as a function of the BOD removed. Expressions 5 and 6 are used for lagoon processes, where the removal of FC has been found to be a function of the retention time and temperature, and expressions 7 to 9 can be used to identify the relation between the influent and effluent water qualities as a function of the inflow quality and the surface overflow rate. Expressions 10 and 11 also incorporate the temperature and pH as factors influencing the unit process performance, and expression 12 is used to specify the effluent quality simply as a function of the surface overflow rate. Expressions 13 and 14 can be used to select the effluent quality as a function of influent and raw sewage concentrations, respectively. The exact expressions used in the knowledge base for each unit process/pollutant combination, as well as the values of coefficients, are provided in Appendix A.

Table 3.3 Expressions Used for Calculation of Pollutant Removal Efficiency

Type	Expression for Process Pollutant Removal Efficiency
1	$C_{eff} = C_{inf} \cdot (1 - R_i), R \in \{R_{\min}, R_{\text{avg}}, R_{\max}\}$
2	$C_{eff} \in \{C_{\min}, C_{\text{avg}}, C_{\max}\}$
3	$C_{eff} = 10^{(\log_{10}(C_{inf}) - LR_i)}, LR \in \{LR_{\min}, LR_{\text{avg}}, LR_{\max}\}$
4	$C_{eff} = C_{inf} \cdot (1 - BOD_{rem} \cdot C_i), C_i \in \{C_{\min}, C_{\text{avg}}, C_{\max}\}$
5	$C_{eff} = C_{inf} \cdot \left(1 - C_1 \cdot e^{(C_2 \cdot HRT)}\right)$
6	$C_{eff} = \dfrac{C_{inf}}{1 + (k_t \cdot HRT)}, k_t = 2.6 \cdot 1.19^{(T\text{-}20)}$
7	$C_{eff} = C_1 \cdot C_{inf} + (C_2 \cdot SOR)$
8	$C_{eff} = C_{inf} \cdot e^{\frac{C_1}{SOR}} + C_2$
9	$C_{eff} = C_1 \cdot SOR^{C_2} \cdot C_{inf}^{C_3}$
10	$C_{eff} = C_{inf} \cdot e^{\left\{-\left[C_1 \cdot C_2^{(T-20)}\right] \cdot \left[HRT + C_3 \cdot (pH - 6.6)\right]\right\}}$
11	$C_{eff} = C_{inf} \cdot \left[1 - (C_1 \cdot T + C_2)\right]$
12	$C_{eff} = C_i \, {}^{mg}\!/_{m^2 \cdot day}, C_i \in \{C_1, C_2, C_3\}$
13	$C_{eff} = C_1 \cdot C_{inf}^{\;C_2} + C_3$
14	$C_{eff} = C_i \cdot C_{Raw}, C_i \in \{C_1, C_2, C_3\}$

C_{inf} = Influent concentration
C_{eff} = Effluent concentration
R_i = Removal efficiency(%)
LR_i = Log removal efficiency
BOD_{rem} = BOD removed
C_i = Removal coefficients
HRT =Hydraulic retention time
T = Temperature
SOR = Surface overflow rate
C_{Raw} = Raw wastewater pollutant concentration

3.2.1.2 Costs

It was pointed out earlier that cost is the most frequently used criteria used for evaluation of wastewater treatment options regardless of considerations for reuse. Since lifecycle costs of treatment trains are computed in this research, methodologies for estimating both the construction and O&M costs of treatment options were developed and built into the knowledge base.

As Dinesh (2002) summarised, wastewater treatment costs are dependant on many factors including the plant capacity, reuse option selected, design criteria used, process configuration, site conditions, land cost, climate, size of competition and local and national economic conditions. Therefore, despite a considerable body of literature reporting costs of unit processes and their combinations, planning level estimates of treatment options are not easily transferable from existing projects, and are generally produced with a wide margin of accuracy of -30% to +50% (Landon et al. 2003).

Most of decision support tools presented in the previous chapter incorporated expressions, or cost curves, that are then used to predict the cost of treatment options, and a similar approach was adopted in this research. The full list of expressions included in the knowledge base for calculating construction costs of unit processes is shown in Table 3.4. The expressions included cover polynomial equations such as expression 1, 2 and 6, all of which are a function of design flow rate. A review of the literature and inputs received from AQUAREC project partners showed that construction costing of some processes is better expressed as a function of BOD loading (expression 3), serviced population (expressed as population equivalents and captured in expression 4), occupied land area (expression 5) and annual treated volume (expression 7). In addition to calculating construction costs of individual unit processes in the knowledge base, a provision is made to specify the percentage of the construction cost that is attributed to the cost of Electro-mechanical (EM) equipment. This provision is made to so that the lifecycle cost of a treatment process can be calculated accounting for different life expectancies for civil works and EM components. Expression used in O&M costing, shown in Table 3.5, can similarly be expressed as a function of the serviced population (expression 1), annual processed volume (expression 2), design flow rate (expression 3), capital cost (expression 4), BOD loading (expression 5) and the occupied land area (expression 6).

Table 3.4 Expressions Used for Calculation of Capital Costs

Type	Expression for Process Construction Cost
1	$CC = C_1 \cdot Q_i^{C_2}, Q_i \in \{Q_{avg}, Q_{pday}, Q_{dwf}\}$
2	$CC = C_1 \cdot Q_i^2 + C_2 \cdot Q_i + C_3, Q_i \in \{Q_{avg}, Q_{pday}, Q_{dwf}\}$
3	$CC = C_1 \cdot BOD_{load} + C_2$
4	$CC = C_1 \cdot PE^{C_2} + C_3 \cdot PE + C_4$
5	$CC = C_1 \cdot A$
6	$CC = C_1 \cdot Q_i^{C_2} \cdot e^{(C_3 \cdot Q_i)}, Q_i \in \{Q_{avg}, Q_{pday}, Q_{dwf}\}$
7	$CC = C_1 \cdot V_{ann}$

CC = Process capital cost
C_i = Capital cost coefficients
Q_{avg} = Average flow
Q_{pday} = Peak daily flow
Q_{dwf} = Dry weather flow
BOD_{load} = BOD loading
PE = Serviced area population equivalents
A = Process area
V_{ann} = Annualy processed volume

Table 3.5 Expressions Used for Calculation of O&M Costs

Type	Expression for Process O&M Cost
1	$OMC = C_1 \cdot PE^{C_2} \cdot C_3 \cdot PE + C_4$
2	$OMC = C_1 \cdot V_{ann}$
3	$OMC = C_1 \cdot Q_i^2 + C_2 \cdot Q_i + C_3, Q_i \in \{Q_{avg}, Q_{pday}, Q_{dwf}\}$
4	$OMC = C_1 \cdot CC$
5	$OMC = C_1 \cdot BOD_{load} + C_2$
6	$OMC = C_1 \cdot A$

OMC = Process annual O&M cost
C_i = O&M cost coefficients
Q_{avg} = Average flow
Q_{pday} = Peak daily flow
Q_{dwf} = Dry weather flow
CC = Process capital cost
BOD_{load} = BOD loading
PE = Serviced area population equivalents
A = Process area
V_{ann} = Annualy processed volume

3.2.1.3 Land Requirements

Five different expressions are included in the knowledge base for calculation of land requirements for unit processes, as shown in Table 3.6. The expressions cover calculations based on process loading in terms of flow rate (expressions 1, 2 and 4), serviced population (expression 3) and BOD removed (expression 5). Land costs are not included in the estimates of unit process costs presented in the previous section. Instead, they are calculated separately for each unit process included, and subsequently added to the total cost of treatment train along with a separate fixed cost of land acquisition specified by the user.

Table 3.6 Expressions Used for Calculation of Land Requirements

Type	Expression for Process Land Requirements
1	$LaR = C_1 \cdot Q_i^2 + C_2 \cdot Q_i + C_3, Q_i \in \{Q_{avg}, Q_{pday}, Q_{dwf}\}$
2	$LaR = \dfrac{Q_i}{SOR}, Q_i \in \{Q_{avg}, Q_{pday}, Q_{dwf}\}$
3	$LaR = C_1 \cdot PE^{C_2}$
4	$LaR = C_1 \cdot Q_i^{C_2}, Q_i \in \{Q_{avg}, Q_{pday}, Q_{dwf}\}$
5	$LaR = C_1 \cdot BOD_{rem}$
$LaR =$ Process land requirement $Q_{avg} =$ Average flow $Q_{pday} =$ Peak daily flow $Q_{dwf} =$ Dry weather flow	$C_i =$ Land requirement coefficients $SOR =$ Surface overflow rate $BOD_{rem} =$ BOD removed $PE =$ Serviced area population equivalents

3.2.1.4 Labour Requirements

As indicated in Chapter 2, a provision is made in this methodology for labour requirements to be specified for each individual unit process. Three separate expressions are provided in the knowledge base, as shown in Table 3.7, which allow the unit process labour requirements to be expressed as a function of design flow rate (expression 1) or serviced population (expressions 2 and 3).

Table 3.7 Expressions Used for Calculation of Labour Requirements

Type	Expression for Process Labour Requirements
1	$LbR = C_1 \cdot Q_i^{C_2} + C_3, Q_i \in \left\{Q_{avg}, Q_{pday}, Q_{dwf}\right\}$
2	$LbR = C_1 \cdot PE^{C_2}$
3	$LbR = C_1 \cdot PE^{C_2} \cdot C_3 \cdot PE + C_4$
$LbR =$ Process labour requirement Q_{avg} = Average flow Q_{pday} = Peak daily flow Q_{dwf} = Dry weather flow	C_i = Land requirement coefficients $SOR =$ Surface overflow rate BOD_{rem} = BOD removed $PE =$ Serviced area population equivalents

3.2.1.5 *Production of Sludge and Concentrates*

The production of sludge is calculated in terms of the weight of dry solids produced by each unit process, using expressions summarised in Table 3.8. The first expression contained in the knowledge base is used for processes for which the annual sludge production is best estimated from the serviced population numbers. The three expressions that follow are used to calculate the sludge production from the BOD, solids and phosphorus removed by the process, and the last expression simply specifies the sludge as a product of the annually processed volume of wastewater. The annual volume of concentrate (brine) produced by each membrane process is specified by the recovery of the feed water.

Table 3.8 Expressions Used for Calculation of Sludge Produced

Type	Expression for Process Sludge Production
1	$SlP = C_1 \cdot PE$
2	$SlP = C_1 \cdot BOD_{rem}$
3	$SlP = C_1 \cdot SS_{rem}$
4	$SlP = C_1 \cdot P_{rem}$
6	$EnC = C_1 \cdot V_{ann}$
$SlC =$ Process sludge production C_1 = Sludge production coefficient $PE =$ Serviced area population equivalents V_{ann} = Annualy processed volume	 BOD_{rem} = BOD removed $SS_{rem} = SS$ removed P_{rem} = P removed

As indicated in Section 2.2.2, several options for the treatment and disposal of sludge are considered, for which the user needs to specify the unit costs. The sludge treatment and disposal options are summarised in Figure 3.1. The sludge thickening is included as a compulsory step, following which the options for dewatering and digestions are available in addition to direct land disposal of thickened sludge. Three other disposal options are included (incineration, composting and landfilling), giving a total of ten possible options to the user. Default unit costs for all sludge treatment and disposal options are shown in Table 3.9.

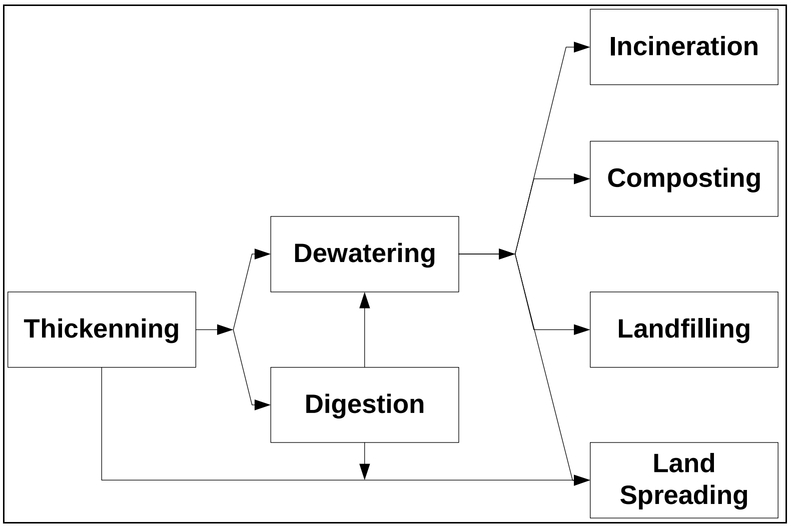


Figure 3.1 Sludge Treatment and Disposal Options

Table 3.9 Default Costs of Sludge Treatment and Disposal

Option	Unit Cost (€/ton of dry solids)
Thicken, dewater, incinerate	550
Thicken, digest, dewater, incinerate	600
Thicken, dewater, compost	500
Thicken, digest, dewater, compost	550
Thicken, dewater, landfill	600
Thicken, digest, dewater, landfill	450
Thicken, land spread	200
Thicken, digest, land spread	300
Thicken, dewater, land spread	400
Thicken, digest, dewater, land spread	450

3.2.1.6 Energy Consumption

The energy consumption is the final quantitative criteria used in the DSS for evaluation of treatment trains, and three expressions are included in the knowledge base to estimate the energy consumed by unit processes. As the expressions shown in Table 3.10 indicate, the energy consumption can be specified as a function of either the serviced population or the volume of treated wastewater.

Table 3.10 Expressions Used for Calculation of Energy Consumption

Type	Expression for Process Energy Consumption
1	$EnC = C_1 \cdot PE$
2	$EnC = C_1 \cdot V_{ann}$
EnC = Process energy consumption C_1 = Energy consumption coefficient	PE = Serviced area population equivalents V = Volume of wastewater processed annualy

3.2.1.7 Qualitative Criteria Scores

As described in the previous Chapter, several qualitative criteria are considered in the DSS for evaluation of treatment trains. The treatment train score for each criterion is computed using the weighted average technique described in Section 3.3.2, which necessitates that each unit process is given individual scores on all criteria considered. In the DSS developed here, a methodology similar to that used by (Dinesh 2002) is employed, in which the scores used for each unit process and for each qualitative evaluation criteria are chosen as NIL, LOW, MEDIUM or HIGH.

The classification of the qualitative criteria used here, distinguishing between technical and environmental, coincides with the criteria being positive or negative. For positive (technical) criteria a score of HIGH can be used to indicate that the unit process is, for example, highly reliable based on operating experience or adaptable to varying conditions. Conversely, a score of HIGH would be assigned to negative (environmental) criteria to indicate that a unit process, for example, consumes large quantity of chemicals, generates a lot of odours, or has a high potential for groundwater pollution. The scores assigned to each unit process on all qualitative criteria are provided on process “fact sheets”, included in Appendix A.

3.2.2 Treatment Train Synthesis Rules

The objectives of this research related to assembling of treatment trains necessitated the development of a structured process that could be used to guide their synthesis. In the use of the DSS, the rules are used to provide aid to users not having detailed knowledge of each process in the knowledge base in assembling treatment sequences, but also need to be flexible to allow the user to easily make modifications according to their own experience and local conditions. At the same time, the process has to be comprehensive so that it can be used for the automated creation of treatment trains used in the optimisation process. As shown in the previous section, the classification of unit processes in the knowledge base was kept relatively simple since each process was only classified as belonging to one of the five categories indicated in Table 3.1, and assigned an identification number according to its category.

Three types of rules for assembling treatment trains were taken into account in developing of the knowledge base:

- rules that dictate possible starting points depending on the influent water quality,
- general rules that restrict the formation of treatment trains in sequences of the generally accepted engineering practice, and
- specific rules that capture the specific preferences of the user for combinations of treatment processes.

An example of a rule of the first type is: if raw wastewater is used as the source, it has to receive preliminary treatment prior to application of any additional treatment, unless lagoon systems are used. The second type of rule could be that an MBR can be used only for effluents from one of the primary treatment processes, excluding the Actiflo® process and anaerobic ponds. Any number of rules of the third type can be cited since they depend highly on the DSS user, but one example could be the use of different types of wetlands in a single treatment trains, which may or not be acceptable to different designers (Dinesh 2002).

Other considerations in the development of the treatment train assembly synthesis process were its overall simplicity and tractability, and they were handled by other researchers typically by introducing a series of rules. For example, Dinesh (2002) used 25 rules classified into category specific and process specific types in MOSTWATER. The category specific rules guide the selection of unit processes classified into technology categories (e.g. attached growth reactors, ponds/lagoons, etc.), and specify processes from the same or different categories that are allowed or forced to coexist in a treatment train. The process specific rules are similar to the category specific ones, except that they are specified at the individual unit process level. The general structure of the rules can be summarised with the following expression: *IF (unit process A / unit* *process(es) from category X) IS (present / absent) THEN (unit process(es) B / unit process(es) from category Y) (can / must / cannot) be present*. Although individual rules are comprehensible and easy to follow, their representation using logical conditions and operator values becomes convoluted even with a relatively small number of rules.

The final consideration in the development of a process for structured synthesis of acceptable treatment train was the ability to incorporate it into the GA-based optimisation methodology. In this respect, the methodology used in MOSTWATER was deemed to have potential for improvement since the checking of the rules in the optimisation required substantial computational burden. A process that could be easily incorporated in optimisation was sought that would reduce this computational burden and improve both the speed and the accuracy of the algorithm.

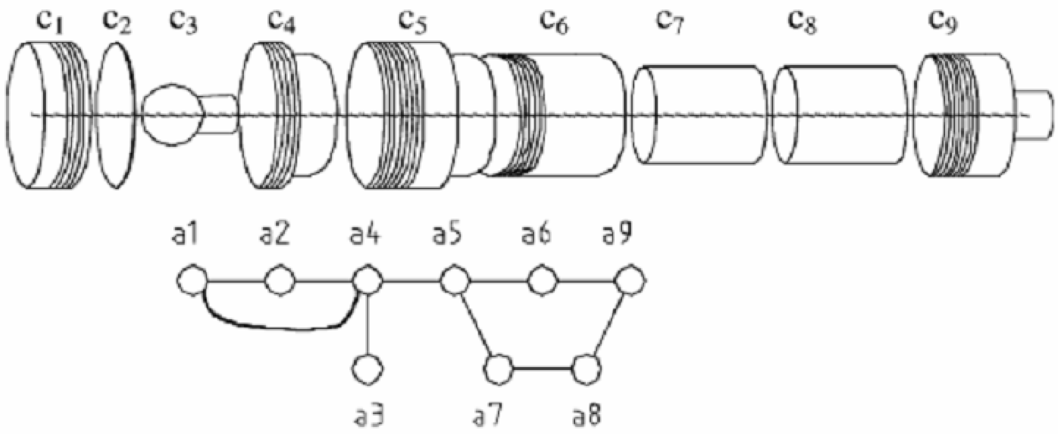
The first set of treatment train rules, dealing with possible starting unit processes, are addressed simply by specifying in the knowledge base the starting processes that can be used for different influent qualities. The information included in the knowledge base is shown in Table 3.11, which was compiled from inputs received from AQUAREC project partners, literature dealing with design of WWTPs (Metcalf and Eddy 2003), and publications in that which provide layouts of existing water reclamation facilities (Asano 1998; Bixio et al. 2004; Juanicó and Dor 1999; USEPA 2004).

In reviewing the literature relevant to the development of the process for specifying rules in which unit processes can be assembled in a treatment train, several publications were reviewed that dealt with a similar problem in the area of manufacturing – Assembly Sequence Planning Problem (ASPP). The ASPP is, similarly to the problem at hand, a highly constrained, combinatorial problem concerned with determining the optimal sequence for assembling a manufactured product composed of potentially large number of parts, under a given set of rules that govern how the parts can be joined. The assembly process rules can be described using a simple, connected, undirected graph of liaisons, or a table of liaisons, which is essentially the adjacency matrix of the graph of liaisons.

Table 3.11 Starting Unit Processes

Influent	Allowed Starting Processes
Raw sewage	Bar Screen Grit Chamber Coarse Screen Stabilization Pond : Anaerobic High Loaded Activated Sludge + Sec. Sedimentation Low Loaded Activated Sludge w/o de-N + Sec. Sedimentation Low Loaded Activated Sludge w/ de-N + Sec. Sedimentation Stabilization Pond : Facultative
Primary effluent	High Loaded Activated Sludge + Sec. Sedimentation Low Loaded Activated Sludge w/o de-N + Sec. Sedimentation Low Loaded Activated Sludge w/ de-N + Sec. Sedimentation Trickling Filter + Secondary Sedimentation Rotating Biological Contactor (RBC) Submerged Aerated Filter (SAF) Stabilization Pond : Aerobic Stabilization Pond : Aerated Stabilization Pond : Facultative Constructed wetland: Free-Water-Surface Flow Constructed wetland: Subsurface Water Flow Membrane bioreactor Excess Biological Phosphorus Removal (EBPR) Phosphorus Precipitation
Secondary effluent	Filtration over fine porous media Surface filtration Micro filtration Granular Activated Carbon (GAC) Powdered Activated Carbon (PAC) Soil Aquifer Treatment (SAT) Maturation pond Constructed wetland - polishing Flocculation Ozone Paracetic acid Chlorine dioxide Chlorine gas Ultraviolet radiation (UV)

These representations of the ASSP were illustrated by Marian et al. (2003) on a simple product, an electric torch shown in Figure 3.2, where the product parts are shown on top. Immediately below, the graph of liaisons is used to illustrate in an intuitive way the order in which parts of the electric torch can be joined in an assembly process. Since the graph cannot be easily processed for computing and also may become very complicated with large number of parts involved, it is translated into the table of liaisons shown at the bottom of Figure 3.2. The table of liaisons, which is symmetrical, is used to capture the information in matrix form simply by indicating a value of 1 if there is a link between nodes (parts) and 0 if the two parts cannot be joined. This basic idea of tables of liaisons is used as a foundation in the development of the process that can be used for specifying the rules for assembling wastewater treatment processes, and is described next.



	a1	a2	a3	a4	a5	a6	a7	a8	a9
a1	0	1	0	1	0	0	0	0	0
a2	1	0	0	1	0	0	0	0	0
a3	0	0	0	1	0	0	0	0	0
a4	1	1	1	0	1	0	0	0	0
a5	0	0	0	1	0	1	1	0	0
a6	0	0	0	0	1	0	0	0	1
a7	0	0	0	0	1	0	0	1	0
a8	0	0	0	0	0	0	1	0	1
a9	0	0	0	0	0	1	0	1	0

Figure 3.2 Assembly Rules for an Electric Torch (Marian et al. 2003)

The process for capturing the sequences in which unit processes can be combined to form treatment trains uses the following operators to capture the relation of each unit process (UP) to all other processes included in the knowledge base:

- null operator (“ ”) is used to indicate that UP_i and UP_j cannot coexist in a treatment train,
- precursor operator (“<”) is used that UP_i can precede UP_j in a treatment train,
- immediate precursor operator (“<<”) is used that UP_i can immediately precede UP_j in a treatment train,

- postcursor operator (“>”) is used that UP_i can follow UP_j c in a treatment train, and
- immediate postcursor operator (“>>”) is used that UP_i can immediately follow UP_j in a treatment train.

The precursor (“<”) and postcursor (“>”) operators have essentially the same purpose, which is to specify that two processes can coexist in a treatment train, in addition to specifying the general positioning of a unit process in a treatment train. For example, they can be used to indicate that an MBR or a stabilization pond should not be considered in a treatment train that already has an activated sludge process. These operators are the opposite entries in the matrix of treatment train assembly rules, which is shown in Figure3.3.

The “<<” and “>>” operators are also complementary, but their primary function is to identify processes that must be used in the treatment train as pre-cursors. For example, the influent to a fine screen has to be free of any larger objects that could damage or blind the screen, so it has to be preceded by a coarse screen that provides adequate water quality. Similarly, the MF and UF processes require relatively clean feed water that can be provided by several primary processes (if no secondary treatment processes are used), and the choice of primary treatment processes is even more limited if NF is to be included in the treatment train.

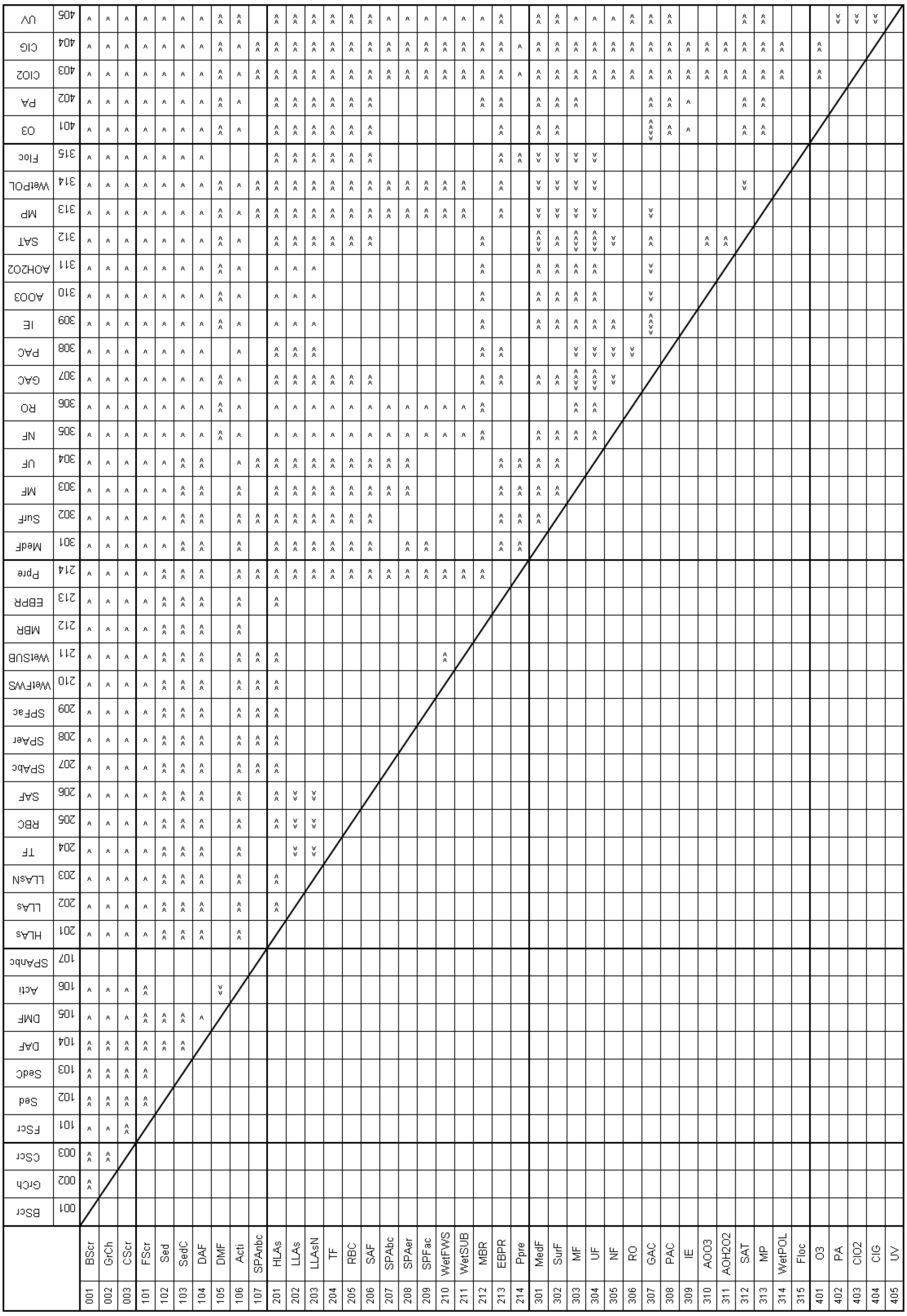


Figure 3.3 Matrix of Treatment Train Assembly Rules

3.2.3 Distribution System Components

The information contained in the knowledge base on the distribution system components is primarily concerned with the capital and operating costs of components. The components of the distribution system covered are pumping facilities, storage elements and of course the pipes that convey the flows. The primary source for expressions used in the knowledge base to calculate costs of pumping and storage facilities were the equations reported in (Heaney et al. 1999), except where otherwise noted.

Equation 3.1 is used in the knowledge base for calculation if capital cost of pumping stations. A default value of 5% of the capital cost is used for annual maintenance costs, while the annual energy costs are calculated using Equation 3.2 (Oron 1996). Default values of 0.05 €/kWh and 65% are used in the knowledge base for the unit cost of energy and pump efficiency, respectively.

$$CC = 21{,}715 \cdot H \cdot Q^{0.52} \tag{3.1}$$

Where:

CC = pumping station capital cost (€)
H = required pumping head (m)
Q = design flow rate (L/s)

$$CE = \theta_{hp} \cdot C_e \cdot \frac{\left(V_{ann} \cdot H\right)}{2.7 \cdot \eta} \tag{3.2}$$

Where:

CE = annual cost of energy required for pumping (€)
θ_{hp} = conversion factor to kWh ($\theta_{hp} = 0.746$)
C_e = unit cost of energy(€/kWh)
V_{ann} = volume of water pumped annually (m^3)
H = pressure head required at the pump(m)
η = pump efficiency(%)

Three types of storage facilities are included in the knowledge base: reservoir, concrete tank, covered concrete tank and earthen basin. Equation 3.3 is used to calculate the capital costs using the cost coefficients presented in Table 3.12. A default value of 0.5% of the capital cost is used for the annual O&M costs for all types of storage facilities.

$$UCS = C_1 \cdot V^{C_2} \tag{3.3}$$

Where:

UCS = Unit cost of storage facility (€/m³)
C_i = Cost coefficients
V = Storage volume (m³)

Table 3.12 Storage Facilities Cost Factors

Storage Type	C_1	C_2
Reservoir	15,093	-0.60
Concrete tank	1,238	-0.19
Covered concrete tank	5,575	-0.39
Earthen basin	128	-0.24

The cost curves for pipes were derived from the data on the costs of installed pipes provided by UK water companies (OFWAT 2000), reported in (USEPA 2002a). The expression used for calculating pipe unit costs is indicated by Equation 3.4, where different cost coefficients are used depending on the land use where the pipe is to be installed as shown in Table 3.13. The knowledge base also includes a multiplier for the unit cost of pipes (a default value of one), which can be used to indicate increased cost for pipe installations in difficult terrains or conditions. The annual O&M cost for pipes is set at 3% of the annualised capital cost.

$$CP = C_1 \cdot e^{C_2 \cdot D} \tag{3.4}$$

Where:

CP = pipe unit cost (€/m)
D = pipe diameter(m)
C_i = cost coefficients

Table 3.13 Pipe Unit Cost Factors

Land use	C_1	C_2
Grassland	47.47	3.51
Rural/suburban	96.19	3.07
Urban	129.42	2.72

The final information contained in the knowledge base deals with the potential end-users of reclaimed water. Each end-user can be specified as belonging to one of the six default types stored in the knowledge base and shown in Table 3.14. Additional information in the knowledge base specifies the maximum contaminant concentrations for each end-user type for pollutants indicated in Table 3.2.

The end-user demand for reclaimed water can be specified as either being constant, or seasonal, in which case daily demands need to be specified for each month of the year. Two other parameters are required to fully describe the hydraulic component of end-user requirements: pressure and peak demand factor, both of which are used in sizing of the distribution system components. The information related to connection and usage charges, used in calculations of potential revenues from reclaimed water, is summarised in Table 3.15. The last term, unit cost of alternate supply, can be used for prioritisation the allocation of reclaimed water between the potential customers. No default values are provided in the knowledge base for any of these parameters, as they are likely to vary considerably between applications of the DSS by different users.

Table 3.14 Classification of Reclaimed Water End-users

No.	Wastewater Reuse Category	Applications
1	Industry	Industrial cooling, except for the food industry
2	Potable	Indirect potable use
3	Urban	Private uses (in house), heating and cooling systems, car washing Urban uses and services. Green areas with public access (public parks, sport fields, etc.), street washing, fire fighting, ornamental fountains
4	Groundwater recharge	Indirect (through soil or vadose zone) Direct (injection wells)
5	Environmental and recreational	Golf courses irrigation Streams, urban lakes, with public access allowed Irrigation of woodland and green areas with restricted access Streams, urban lakes, with public access restricted
6	Agriculture	Greenhouses irrigation Fodder, crops to be processed, ornamental flowers, cereal. Aquiculture Unrestricted irrigation (vegetables). Fruit trees sprinkler irrigated

Table 3.15 Information on Potential Reclaimed Water End-users

Symbol	Unit	Description
Con	€	Connection charge
Mon	€/month	Monthly service charge
Trf1	€/m^3	Tariff for usage up to Trf1L
Trf1L	m^3/day	Limit for application of Trf1
Trf2	€/m^3	Tariff for usage exceeding Trf1L
AltS	€/m^3	Unit cost of alternate supply

3.3 Treatment Train Performance

The assessment of treatment train performance is carried out by conducting evaluations of individual unit processes of which it is composed, and post-processing the results of these evaluations along with additional information. This process is illustrated in Figure 3.4, where individual procedures carried out during the course of treatment train evaluations are presented in the sequence in which they are carried out in the DSS. Each procedure is discussed in further detail in sub-sections that follow.

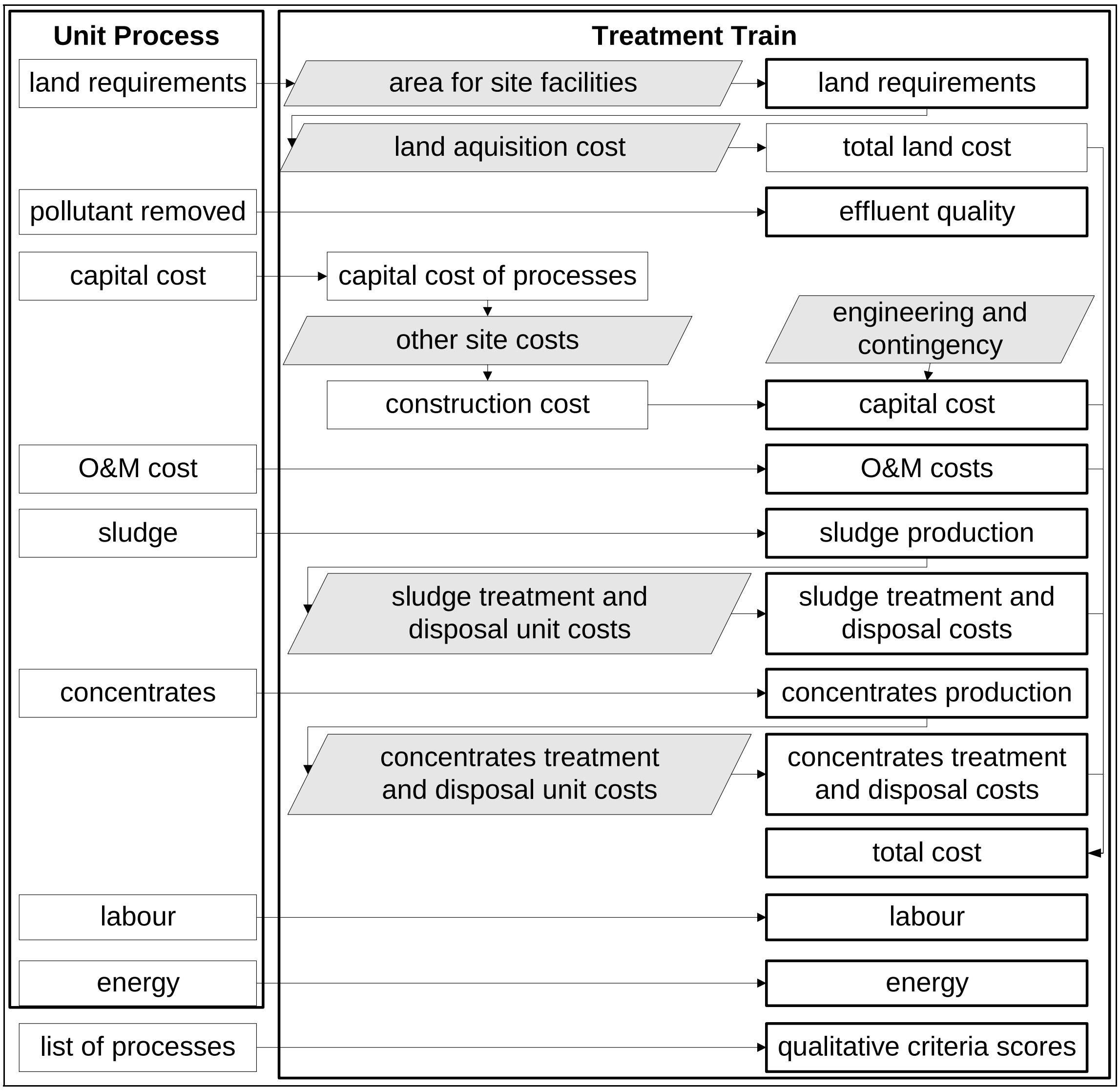


Figure 3.4 Treatment Train Evaluation

3.3.1 Quantitative Criteria

The first procedure conducted on the unit process level calculates the land required for the unit process. The reason for this is that some expressions used for calculating the pollutant removal efficiencies require the process area from which the surface overflow rate and the amount of pollutant removed are calculated. Once the land areas for all unit processes are calculated, the total required area for the treatment train is calculated using Equation 3.5. A default value of 15% of the total area required for unit processes is added to account for the land occupied by site facilities such as access roads, fencing and administrative buildings (Dinesh 2002). The unit cost of land and the land acquisition costs are the additional two values provided by the user to calculate the total land cost for the treatment train.

$$LaR_{TT} = 1.15 \cdot \sum_{i=1}^{N} LaR_i \qquad (3.5)$$

Where:

LaR_{TT} = Treatment train land requirement
N = No. of unit processes in the treatment train
LaR_i = Land requirement for unit process i

The calculations dealing with the removal of pollutants are carried out sequentially, according to the order of unit processes that form the treatment train. For each unit process, the preceding process effluent quality is used as input, and the amount of pollutant removed is calculated. The treatment train effluent quality is simply the quality of effluent from the last process in the treatment train.

The sum of construction costs of individual unit processes is used as a basis in calculating the total capital cost of the treatment train. The treatment train construction cost is defined as the sum of the construction costs of unit processes cost, to which common facility costs whose default values are shown in Table 3.16 are added. The engineering and contingency costs (12% and 15% of construction cost, respectively) are then added as percentages of the treatment train construction cost to generate the total capital cost of the treatment facility. O&M costs of individual unit processes are added to form the annual treatment train O&M cost. Similarly, the total production of sludge and concentrates for the treatment train is determined by adding the results of individual unit processes. The cost of sludge and concentrates treatment and disposal is calculated from these values and respective unit costs. The treatment train labour requirements and energy consumption are calculated by adding the respective numbers determined for each unit process in the treatment train.

Table 3.16 Common Treatment Facility Costs

Cost Description	Amount
Piping	8%
Controls and instrumentation	8%
Site electrical	9%
Site development	8%
Site works	6%

Using the cost of land, capital cost and annual costs of sludge and concentrates disposal and O&M, additional cost functions are used to calculate the lifecycle, annualised and unit costs of treatment. These require the Capital Recovery Factor (*CRF*) to be determined first, which is based on the user supplied discount rate (*r*) and the planning period (*n*), as shown by the following equation:

$$CRF = \frac{r}{1-(1+r)^{n}} \tag{3.6}$$

The lifecycle cost of treatment is calculated by adding and properly discounting all payments associated with initial capital investment, future replacements of civil and EM works, sludge and concentrates disposal costs, and O&M costs over the life of the project using Equations 3.7 and 3.8. The annualised treatment cost (AC_{TT}) is calculating simply by multiplying the lifecycle cost of treatment with *CRF* (Equation 3.9), while the unit cost of treatment (UC_{TT}) is computed using Equation 3.10, by dividing this amount with volume of reclaimed water produced annually (V_{ann}).

$$LC_{TT} = CC_{TT} + \sum_{i=1}^{N} PW\left(RC_i^n\right) + \frac{OMC_{TT} + SlC_{TT} + ConC_{TT}}{CRF} \tag{3.7}$$

Where:

LC_{TT} = Treatment train lifecycle cost
CC_{TT} = Treatment train capital cost
RC_i^n = Cost of replacing a component of unit process i in year *n*
$PW(x)$ = Present (discounted) cost of *x*
OMC_{TT} = Treatment train O&M cost
SlC_{TT} = Annual cost of sludge treatment and disposal
$ConC_{TT}$ = Annual cost of concentrates treatment and disposal

$$PW(RC_i^n) = \frac{RC_i}{(1+r)^n} \tag{3.8}$$

$$AC_{TT} = CRF \cdot LC_{TT} \tag{3.9}$$

$$UC_{TT} = \frac{AC_{TT}}{V_{ann}} \tag{3.10}$$

3.3.2 Qualitative Criteria

The treatment train qualitative criteria scores, as well as the overall treatment train score, are calculated from the list of processes of which the treatment train is composed, and the unit process scores contained in the knowledge base and described in Section

3.2. In the calculations, the unit process scores of NIL to HIGH are represented with numerical values of 0 to 3, which are then normalised. In addition, the importance of each of the criteria is indicated by the user in the form of weights that can range from 0, indicating no importance, to 10, representing the highest importance to the user. The process of determining the treatment train qualitative criteria scores proceeds in this order: calculate average criteria scores using Equation 3.11, normalise the scores according to the criteria type (positive/technical using Equation 3.12, negative/environmental using Equation 3.13), and calculate the overall treatment train score using Equation 3.14.

$$AEC_i^{TT} = \frac{\sum_{j=1}^{N} EC_{ij}^{UP}}{N} \tag{3.11}$$

Where:

AEC_i^{TT} = Treatment train average score for criteria i
EC_{ij}^{UP} = Unit process j score for criteria i
N = Number of unit processes in the treatment train

$$NEC_i^{TT} = \frac{1}{3} \cdot AEC_i^{TT} \tag{3.12}$$

Where:

NEC_i^{TT} = Normalised treatment train score for criteria i

$$NEC_i^{TT} = 1 - \left(\frac{1}{3} \cdot AEC_i^{TT} \right) \tag{3.13}$$

$$QS_{TT} = \frac{\sum_{i=1}^{M} W_i \cdot NEC_i^{TT}}{\sum W_i} \tag{3.14}$$

Where:

QS_{TT} = Overall treatment train qualitative criteria score
W_i = Weight of criteria i (user assigned)
M = Number of qualitative evaluation criteria

3.4 Distribution System Sizing

The sizing of the distribution system is conducted automatically in the DSS, based on user entered locations and capacities of production (treatment) facilities, locations and specifications of demands (potential reclaimed water users) and outlined distribution system components (storage and pumping facilities and transmission mains). The methodology employed is a two-stage process, in which the operational policy is conducted simultaneously with sizing of seasonal storage elements as the first step, followed by sizing of pumping stations and pipes. These two steps are described the next two sections, followed by the description of the implementation of this sequential approach in Section 3.4.3.

3.4.1 Operational Policy and Storage Sizing

In the developed DSS, the operational policy is determined simultaneously with finding the optimal sizes of seasonal storage elements, using a network representation of the distribution system monthly flows. This approach is based on the optimisation of multireservoir multiperiod linear programming models by Kuczera (1989), formulated as network linear programs (NLP), and that presented by Khaliquzzaman and Chander (1997), who expanded the approach to include deficit supplies in the optimisation using network flow programming (NFP). The two terms, NLP and NFP, are used interchangeably in the literature.

The network representing the multiperiod flows in a hypothetical reclaimed water distribution network is shown in Figure 3.5. The representation of the physical components of the water reuse systems is contained in each of the three panels representing different time periods (t). The system includes three supply nodes (e.g. wastewater reclamation plants), labelled S_1 to S_3, from which the reclaimed water is conveyed to their respective reservoirs (R_1 to R_3). The water can also be transferred from the middle reservoir (R_2) to the adjoining reservoirs, prior to being conveyed to the five users indicated by demand nodes (D_1 to D_5). The pipes conveying flows between the supply, storage and demand nodes are represented by solid lines (DP), whereas the conceptual links, indicated by various dashed lines, represent the volumes of water transferred in different time periods and to/from the conceptual master source/sink node (A).

The inflow arcs (I) and spill (SP) arcs represent the available and excess monthly volumes of reclaimed water, respectively, that is produced by the wastewater reclamation facilities. The spills here refer to treated wastewater in excess of demand that would be discharged (rather than reused), and not spills used in the classical reservoir operation, which are overflows from storage facilities. The storage carryover arcs (ST) represent the volume of reclaimed water stored at the end of each time period (month), where the arcs from the last period are connected to the first period to ensure that quantities remaining in storage at the end of the simulation correspond to those available at the beginning. The outflow arcs (*O*) represent the monthly volumes required by the users, while the shortfall arcs (*SH*) are used to represent the portions of demanded volumes that are not met. The balancing arc, which is not labelled, provides a connection for the source/sink node to ensure the circulatory nature of flows in the network. The two nodes labelled A in Figure 3.5 are actually a single node but are shown separately for visual clarity.

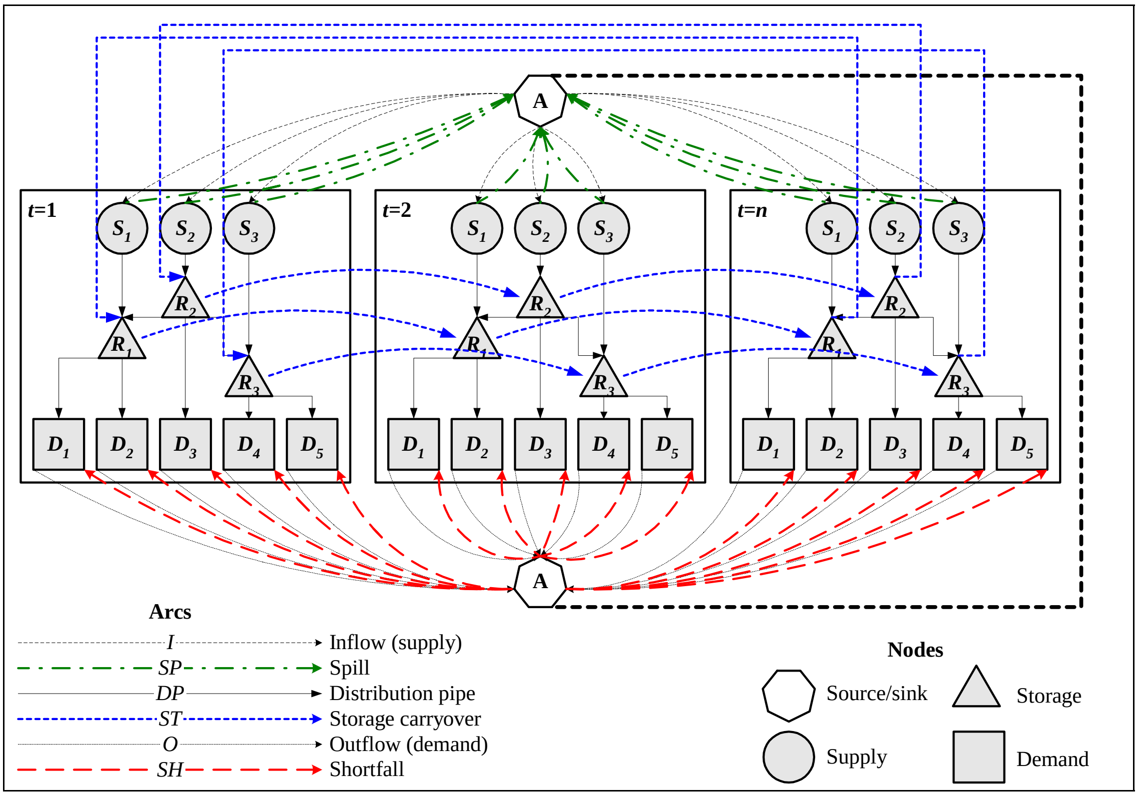


Figure 3.5 Network Representation of Multiperiod Multireservoir Sizing

The need for application of the algorithm depends on the monthly and total supply and demand volumes specified by the user, and is checked in the DSS prior to formulating or solving the NLP problem representing the system. The procedure that is performed, illustrated in Figure 3.6, modifies the NLP procedure to eliminate the spill arcs if there is an overall shortage of reclaimed water to satisfy the demand. This is done under the assumption that all reclaimed water, regardless of when in the year it is produced, will be utilised by the users by employing seasonal storage to balance the monthly supplies and demands using seasonal storage. If, on the other hand, there is an excess of reclaimed water produced yearly and all monthly supply flows are in excess of demand for reclaimed water, the NLP model will not be formulated and seasonal storage elements will not be sized. Finally, if there is annual surplus of reclaimed water, but volumes produced in individual months are less than those demanded, seasonal storage might be employed along with discharging some of the reclaimed effluent.

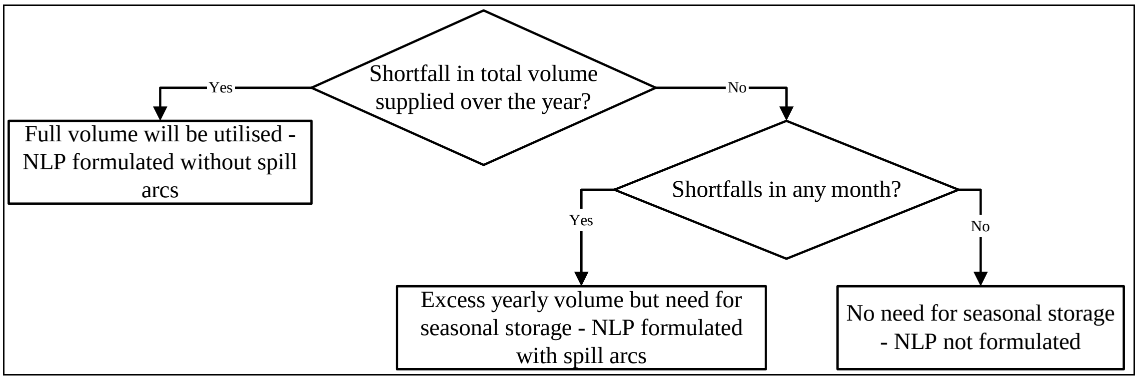


Figure 3.6 Specification of NLP Procedure Used in the DSS

If the NLP model is deemed to be required by the procedure presented above, the objective function needs to be formulated first. The simplest objective might be to minimise the amount of storage by minimising the values of carryover arcs. However, this strategy would not take into account the differences that might exists between characteristics of various storage elements (e.g. type, maximum capacity and unit cost), preferences that might exist for satisfying end-user requirements based on alternatives to providing reclaimed water, and end-user location/elevation relative to reclamation and storage facilities. The least-cost objective function is thus formulated to take into account these factors, as shown in Equation 3.15. The unit cost of storage carryover arcs and distribution pipe arcs is calculated using the lifecycle costing information from the knowledge base presented earlier. In order to avoid the spills from occurring prior to meeting all demands, a penalty value of 100,000 €/m³ is used for the unit cost of all spill arcs, which was determined through testing. The unit costs of shortfalls are determined from unit costs of alternative supply, specified separately for each end-user. These values are multiplied by 1,000 €/m³ to ensure that supplies are met to the maximum extent possible prior to creating shortfalls.

$$Min\ Z=\sum_{t=1}^{12}\left(\sum_{i=1}^{NR} CST_i \cdot ST_{it} + \sum_{j=1}^{ND} CSH_j \cdot SH_{jt} + \sum_{k=1}^{NS} CSP_k \cdot SP_{kt} + \sum_{l=1}^{ND} CDP_{lt} \cdot DP_{lt}\right) \tag{3.15}$$

Where:

CST_{it} = Unit cost of flow through storage carryover arc *i*

CSH_{jt} = Unit cost of flow through shortfall arc *j*

CSP_{kt} = Unit cost of flow through spill arc *k*

CDP_{lt} = Unit cost of flow through distribution pipe arc *l*

NR = Number of storage nodes *R*

ND = Number of demand nodes *D*

NS = Number of supply nodes *S*

ND = Number of distribution pipes

The minimisation of the objective function is subject to a number of constraints that ensure the continuity at supply nodes (Equation 3.16), storage nodes (Equations 3.17 and 3.18) and demand nodes (Equation 3.19). To ensure the overall continuity at the source/sink node, the calculated yearly shortfall or excess volume (V_A) is calculated using Equation 3.20, and used as demand of the master source/sink node in the constraint indicated by Equation 3.21. The final set of constraints deals with specifying the arcs capacities. The capacities of inflow arcs are made equal to monthly supply node inflows (negative demands), as indicated by Equation 3.22. Similarly, the capacities of outflow arcs are equated to monthly demands at their originating nodes (Equation 3.23). The capacities of distribution pipe arcs are determined from the LP model, which is explained in the next two sections, while the carryover storage arc capacities are set to maximum values specified by the DSS user.

$$I_{i,t} = \sum_{j=1}^{OUT_i} DP_{j,t} + SP_{i,t}, t = 1..12 \tag{3.16}$$

$$ST_{i,t+1} = ST_{i,t} + \sum_{j=1}^{IN_i} DP_{j,t} - \sum_{j=1}^{OUT_i} DP_{j,t}, t = 1..11 \tag{3.17}$$

$$ST_{i,1} = ST_{i,12} + \sum_{j=1}^{IN_i} DP_{j,12} - \sum_{j=1}^{OUT_i} DP_{j,12} \tag{3.18}$$

$$O_{i,t} = \sum_{j=1}^{IN_i} DP_{j,t} + SH_{i,t}, t = 1..12 \tag{3.19}$$

$$V_A = \sum_{t=1}^{12}\sum_{i=1}^{NS} S_{i,t} - \sum_{t=1}^{12}\sum_{j=1}^{ND} D_{j,t} \tag{3.20}$$

$$\sum_{i=1}^{NS}\sum_{t=1}^{12}\left(I_{i,t}-SP_{i,t}\right)-\sum_{j=1}^{ND}\sum_{t=1}^{12}\left(O_{j,t}+SH_{j,t}\right)=V_A \tag{3.21}$$

$$I_{i,t}^{cap}=S_{i,t}, i=1..NS, t=1..12 \tag{3.22}$$

$$O_{j,t}^{cap}=D_{j,t}, j=1..ND, t=1..12 \tag{3.23}$$

In the above equations, the following terms (not previously defined) are used:

$OUT_i =$ Set of arcs leaving node i

$IN_j =$ Set of arcs entering node j

$I_i^{cap} =$ Capacity of inflow arc i

$O_j^{cap} =$ Capacity of outflow arc j

Once formulated, the NLP model of the distribution system is solved using the RELAX algorithm (Bertsekas and Tseng 1994). Since the formulation of the model requires the non-linear storage unit costs, which are specified in the knowledge base as a function of the volume, an iterative procedure is used to update the unit costs with previously calculated results and re-solve the model. The testing of this approach showed that a single iteration was always required to determine the optimal (least-cost) solution. The optimisation output includes flows for each of the model arcs, which are interpreted to determine the monthly volumes of reclaimed water conveyed by each pipe and delivered to the end-users, monthly shortfalls and sizes of storage facilities.

3.4.2 Sizing of Pipes and Pumps

As indicated earlier, the sizing of pipes and pumps is based on a pre-determined branched pipe layout and preferences of the user for locating the pumping facilities. The least-cost sizing of the distribution system facilities is ensured by incorporating a LP algorithm. The model is limited to branched distribution networks, typical in water reuse schemes and appropriate at the planning level of analysis, and uses standard representation of the network in the form of links and nodes. Each link is further conceptualized as consisting of a series of segments of user-selected standard pipe sizes, where the user selects the sizes they wish to be considered for each of the network links.

Prior to formulating and solving the LP problem, a series of operations are perform to verify the data consistency and calculate the values required for LP formulation. Data consistency checks include confirming the existence of supply and demand nodes, and absence of loops in the user defined network. If the network passes all the data consistency checks, the following three procedures are performed: 1) identification of routes from supply nodes to demand nodes, 2) calculation of peak flows in all links, and

3) calculation of unit head losses using the Hazen-Williams or Darcy-Weisbach equation for standard (available) pipe sizes. The LP problem is then formulated using objective function indicated by Equation 3.24, and constraints provided in Equations 3.25, 3.26 and 3.27.

The formulated LP problem is solved using the MILP solver lp_solve (Berkelaar et al. 2004). The output of the distribution system performance module includes optimal values for pipe sizes for all node connections and capacities of pumping facilities (head and flow rate). These values are used to calculate the capital, O&M, and annualized costs for each pipe and pumping station of the distribution system, and for the distribution system as a whole.

$$Min\ Z = \sum_{ij\ \in A} \sum_{m=1}^{M_{ij}} c_{ijm} l_{ijm} + \sum_{k\ \in B} c_k p_k \tag{3.24}$$

$$\sum_{k \in C} p_k - \sum_{ij \in D} \sum_{m=1}^{M_{ij}} h_{ijm} l_{ijm} \geq H_r,\ \forall_r \tag{3.25}$$

$$\sum_{m=1}^{M_{ij}} l_{ijm} = L_{ij}, \forall_{ij} \in A \tag{3.26}$$

$$l_{ijm} \geq 0, \forall_{ijm} \tag{3.27}$$

Where:

A - set of all network links, described as ij node pairs
B - set of all network pumps
M_{ij} - number of diameter segments considered for link ij
c_{ijm} - unit cost of diameter segment m for link ij (€/m)
l_{ijm} - length of pipe of diameter segment m for link ij (m)
c_k - unit cost of pumping station k (€/m)
p_k - head provided by pumping station k (m)
C - set of all network pumps on route r
D - set of all network links on route r
h_{ijm} - unit head loss of diameter segment m for link ij (m/m)
H_r - required head for supply node of route r (m)
L_{ij} - length of link ij (m)

3.4.3 Sequential Approach

The execution of methodologies presented in the previous two sections for sizing of distribution system elements in the DSS required that a sequential procedure be developed and implemented, since the two models require each others output. In the case of the NLP model, used for determining the operating strategy and sizing of seasonal storage, head losses in the distribution system are needed to compute the unit costs of transmitting the reclaimed water in the system. In order to calculate the head losses, the sizes of pipes and pumps are needed, which are determined by the LP model. Conversely, the monthly volumes of water conveyed by the distribution system components, calculated by optimising the NLP model, are used by the LP model to determine the least-cost design of system pipes and pumping stations.

The sequential approach implemented in the DSS is illustrated in Figure 3.7. An initial assumption that demands of all users are fully met is made, and optimal sizing of the pipes and pumps is carried out accordingly using the LP model. The solution of the LP model is then used to calculate head losses in the system and unit costs of conveying the water through pipes. The NLP model is updated with this information and solved to provide optimal sizing of storage nodes and the operating strategy, from which the actual monthly pumped and conveyed volumes are extracted. This information on actual monthly flows in the system is used to determine the necessary input parameters for the LP model, and the lifecycle distribution system cost is determined. The procedures of updating and optimising the NLP and LP models are then iterated until no changes in the system cost are observed. The testing of the approach on a hypothetical system showed that one to two iterations are typically sufficient to provide the least-cost sizing of the overall distribution system, although this result cannot be generalised without further testing on other systems.

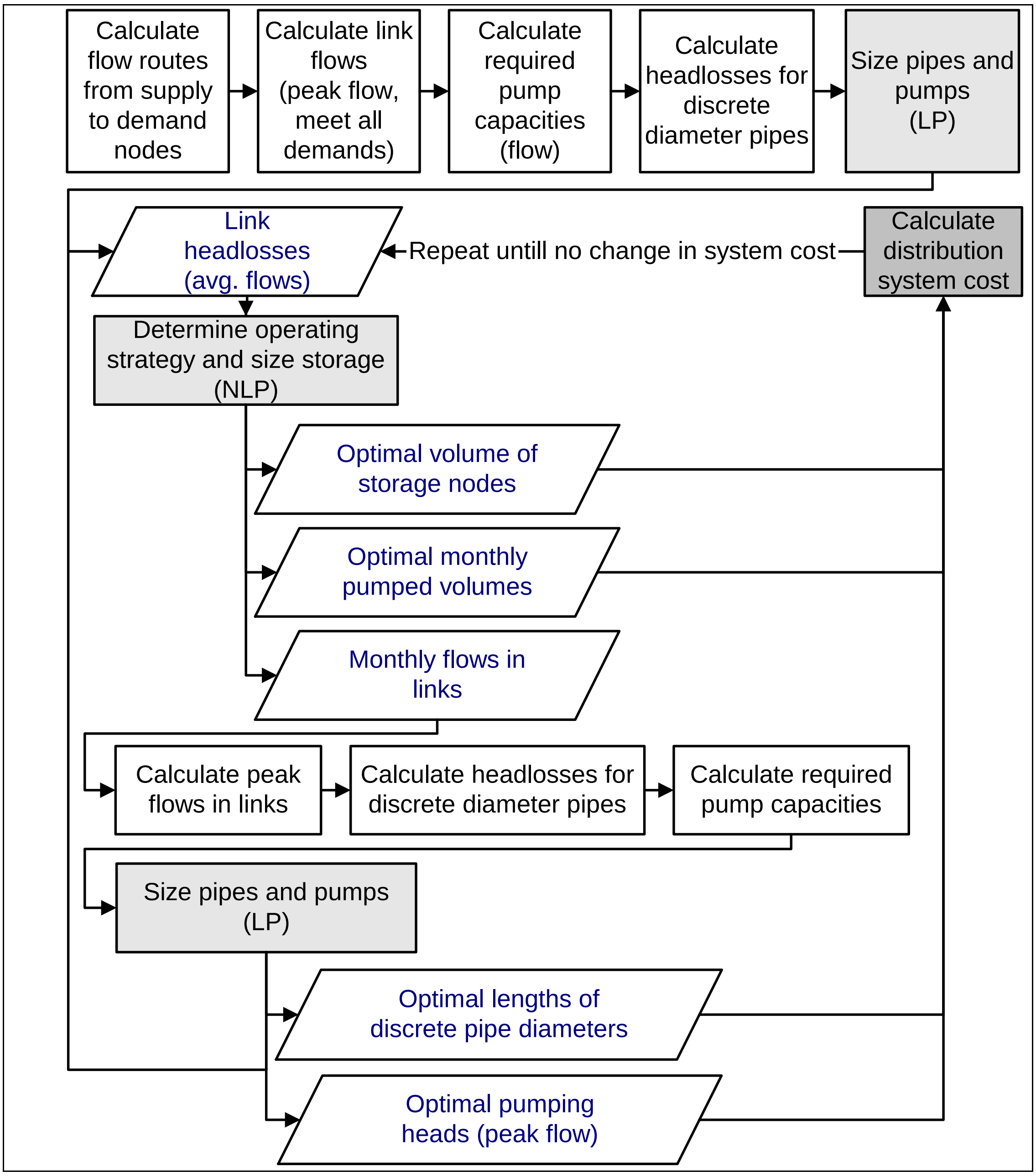


Figure 3.7 Sequential Approach for Sizing of Distribution System

3.5 Summary and Conclusions

Following on the review of literature related to development of a decision support tool for integrated water reuse, this Chapter presents the details of the developed DSS. The first part of the Chapter deals with the DSS knowledge base, large part of which deals with includes the information needed for assessing the wastewater treatment for reuse.

Forty-four unit processes included in the knowledge base are presented, grouped into categories. Several expressions that describe the efficiency of each process in terms of pollutant removal are presented, which are considered to offer greater flexibility than the existing approaches that allow the process efficiency to be expressed only as percent removal. Similarly, a number of different expressions are included for capital and O&M costs, which are used to compute the lifecycle cost of treatment. The resources required for building and operating the treatment facility include land, labour and energy, and several expressions are used to capture the requirements for individual unit processes. The production of treatment by-products, sludge and concentrates (brine), is also quantified through a number of expressions for the former and calculated from the unit process recovery for the latter. Finally, each unit process is assigned a score for each of the qualitative criteria included in the knowledge base.

In the process of assembling treatment trains, it was particularly important to develop a process that is simple enough to be comprehensible, but at the same time comprehensive enough to permit the automated generation of treatment trains in GA optimisation. Based on these two principles, a methodology for capturing policies for assembling treatment trains was developed consisting of three types of rules. The first of these are used to identify possible starting processes based on influent quality, while the remaining two restrict the formation of treatment trains in sequences of the generally accepted engineering practice and according to user preferences. The approach developed for storing the last two types, inspired by approaches developed for Assembly Sequence Planning Problem, includes a matrix in which five types of operators are used to fully encapsulate the rules for treatment train assembly.

The knowledge base also includes information required for sizing and costing of the distribution system components, as well as information used for calculating potential revenues from the provision of reclaimed water. Costing expressions included cover the capital and O&M costs of reclaimed water distribution system pipes, pumps, and four types of storage facilities. The information on potential end-users of reclaimed water is also included here, and covers their quality requirements (set as a function of one of the six end-use types), quantity requirements (peak and average flows and pressures) and different rates that are anticipated to be charged for provision of reclaimed water (connection and monthly fees, two volume-dependant tariffs).

Following the detailed description of the knowledge base, the issues of evaluating the treatment train performance and sizing of the distribution system are discussed. The former is explained using a flowchart that depicts the order in which different procedures are carried out to arrive at the overall treatment train performance figures. These include figures on the quantitative criteria and an overall treatment train score on the qualitative criteria used. The sizing of the distribution system is carried out in the

DSS using a sequential approach, in which the operational policy and storage sizing are carried out concurrently by an NLP model and used iteratively with a LP-based procedure for sizing of system pumps and pipes. As a result, the approach produces the least-cost design in an efficient manner that can then be used in the optimisation of integrated water system.

Chapter 4

Methodologies for Optimisation of Integrated Water Reuse Systems

4.1 Introduction

The previous Chapter describes methodologies for evaluation of integrated water reuse systems that were developed and integrated into the DSS, which formed the basis for the optimisation presented in this section. The approach taken here for determination of optimal water reuse schemes is hierarchical, in that it detaches the optimal sizing of the distribution system from the optimisation of the overall water reuse scheme. This approach is not uncommon, and has been taken by several researchers in recent years to solve difficult non-linear optimisation problems by decomposing them into this type of framework, several of which are mentioned here.

In the work of (Silva et al. 2000) the optimal operation of a pipeline network of a petroleum field problem was decomposed. A GA was used for decisions about which pumps will be operating during each interval of time and LP was applied to determine the flow rate of each selected pump. Two non-linear optimisation problems were addressed by (Cai et al. 2001): a reservoir operation model and a long-term dynamic river basin planning model. The authors identified so called complicating variables for each problem, which, when fixed, render the problem linear in the remaining variables, and developed the GA-LP approaches to solve them. The problem of optimal design of water distribution systems was solved using a GA-LP framework by (Ostfield and Karpibka 2005), where the LP was used for optimal pipe sizing based on fixed flows while the flows were altered in a GA optimisation procedure.

In this work, the “inner” problem of optimal distribution sizing is considered linear and solved using the sequential approach. The solution of the “outer”, non-linear problem of selecting treatment options and end-users is addressed using different methodologies discussed in this section. The methodologies were developed to address a wide range of alternatives, from those involving a small number of potential end-users and available secondary effluent, to schemes involving no existing treatment and a large number and assortment of potential end-users. They are introduced in the first section of this Chapter based on classification of problem size, and discussed in detail in the sections that follow.

4.2 Number of Design Alternatives

In order to develop an appreciation for the scale of the optimisation problem at hand and devise appropriate methodologies, preliminary evaluations were carried out to establish the actual numbers of design options that exist for problems of various sizes. This entailed determining the actual number of possible, feasible and practical treatment trains that could be generated using the information contained in the knowledge base and combining it with combinations of different number of potential users. A feasible treatment train is defined here as the treatment train that meets all assembly rules specified in the knowledge base. Similarly, a treatment train is considered practical if it produces the effluent quality required for a particular end-use.

The number of possible treatment train is influenced by the following factors: influent water quality, reclaimed water end-use, number of unit processes in the knowledge base and rules for assembling treatment trains. The first three factors have a similar effect, as they can reduce the number of processes considered and the resulting number of possible treatment trains. For example, if raw sewage is considered for influent, all processes in the knowledge base can be used to form a practical treatment train, whereas preliminary, primary and secondary processes would be excluded from consideration if the water reclamation facility was to receive secondary effluent from an existing WWTP. The water quality requirements vary between different end-use types, and lower reclaimed water quality can be produced by larger number of treatment trains, whereas the treatment trains capable of producing high quality effluent would have to include certain processes thus reducing the number of practical treatment trains (as defined in the previous paragraph). An increase in the number of treatment train assembly rules would also result in a reduced number of possible treatment trains. Impacts of all of the factors discussed above were quantified using the 44 unit processes included in the knowledge base prior to determining appropriate optimisation approach.

The first results of the enumeration of treatment trains shown in Table 4.1 point out the size of the potential search space without the rules for assembling treatment trains, and with the rules applied. The first number, indicating the total number of treatment trains, is computed simply as 2^n, where n=44 (number of unit processes in the knowledge base). The combined effects of the influent quality and treatment train assembly rules are evident in the second column. The number shown in Table 4.1 indicate that the number of feasible treatment trains is reduced by several orders of magnitude as a result of limiting the combinations of unit processes to logical combinations, even if the raw sewage is used as source water. The number of treatment trains meeting the rules is again substantially reduced for influents of higher quality, to the point where less than three thousand feasible treatment trains exist for treatment of secondary effluents. These reductions are attributed to the fact that a smaller number of processes are considered in the formation of treatment trains.

Table 4.1 Number of Possible and Feasible Treatment Trains

Source Level of Treatment	Total Possible	Feasible
None	1.76×10^{13}	2,942,221
Primary		47,765
Secondary		2,780

The number of practical treatment trains (those that are both feasible and meet the effluent quality criteria) was also computed for different types of end-users. Typical contaminant concentrations found in raw wastewater and water treated to primary and secondary levels were used as source, and the enumeration of treatment trains was repeated but this time counting only those treatment trains that met the specified quality criteria. The results of these computations, shown in Table 4.2, point out that the numbers of practical treatment alternatives represent a fraction of feasible treatment trains that varies between 5% and 20% depending on the source water and end-use considered.

Table 4.2 Number of Practical Treatment Trains

End-use	Source Level of Treatment		
	None	Primary	Secondary
Irrigation	308,904	4,892	190
Industrial	179,712	3,071	150
Groundwater	201,783	2,939	149
Environmental	282,465	3,734	178
Urban	545,097	8,283	318
Potable	201,783	2,939	149

Finally, the numbers of possible end-user combinations were calculated (again using 2^n where n indicates the number of potential end-users) and multiplied with the numbers possible treatment combinations to provide an indication on the number of possible design alternatives for integrated water reuse schemes of different size. The results of these computations, summarised in Figure 4.1, provide a clear indication that different integrated water reuse scheme optimisation methodologies are appropriate depending on the influent water quality and the number of end-users considered. If the secondary effluent is used as a source and only several potential end-users are considered, a simple enumeration approach could be used to identify the optimal alternatives. In the case of raw sewage influent, the large total number of design alternatives for any number of end-users requires that a robust optimisation methodology appropriate for large-scale problems be applied. The situation is similar if the primary effluent is used as a source in a system incorporating several potential end-users.

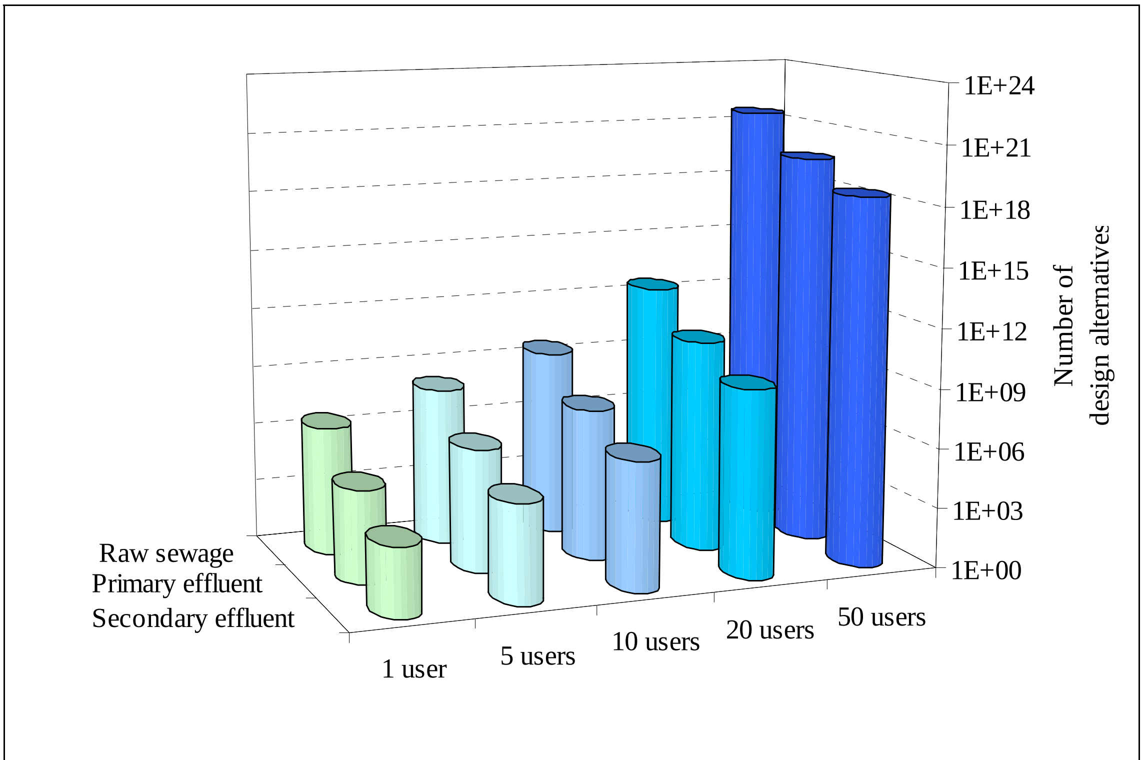


Figure 4.1 Number of Integrated Water Reuse Scheme Design Alternatives

In order to accommodate the wide range of the number of possible design alternatives, three methods are used in the DSS, whose application is summarised in Figure 4.2. If the secondary effluent is to be reclaimed and the number of potential customers is not large, enumeration is used to determine the best design alternatives for all combinations of potential end-users. If the secondary effluent is to be reclaimed for a large number of potential end-users, a simple GA is used for optimal user selection. Finally, if the source of water is raw sewage or primary effluent, the optimisation algorithm used is a GA with customised operators. The algorithm conducts a simultaneous search of least-cost design alternatives and the selection of customers. Optimisation methodologies used in the DSS, described in the remainder of this Chapter, are of progressing objective function complexity as well.

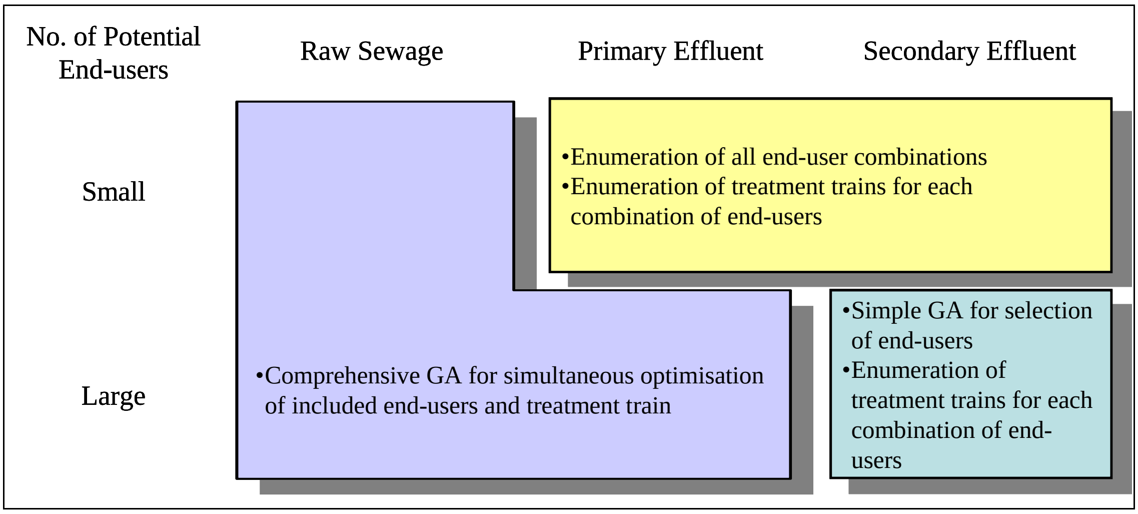


Figure 4.2 Use of Alternative DSS Optimisation Methods

4.3 Optimisation using Enumeration

Using enumeration, all possible design alternatives are explored, with respect to the potential end-users and alternative treatment trains that satisfy their requirements. The actual procedures followed are shown in Figure 4.3, and a further explanation of the methodology is provided below.

The procedure starts with the calculation of all possible combinations of potential end-users, and recording of combinations. For each combination of potential end-users, the distribution network is updated to eliminate the appropriate nodes, followed by the "pruning" of the distribution network that entails eliminating all unnecessary components (pipes, pumping stations and storage elements).

Once the distribution network is reduced to only necessary components, calculations are carried out to determine the updated average and peak flows, on which the sizing of the distribution network and the evaluation of treatment trains is based. Also, the required effluent quality is calculated as the combination of lowest allowable concentrations for each pollutant, for the remaining set of potential end-users. It is also noted that the treatment train is sized to meet the volume requirements of the remaining end-users, while excess is assumed to be discharged as before. This is a sensible assumption given that this particular methodology is applicable only to cases where upgrade of secondary effluent is considered, which result in sufficiently small number of design alternatives.

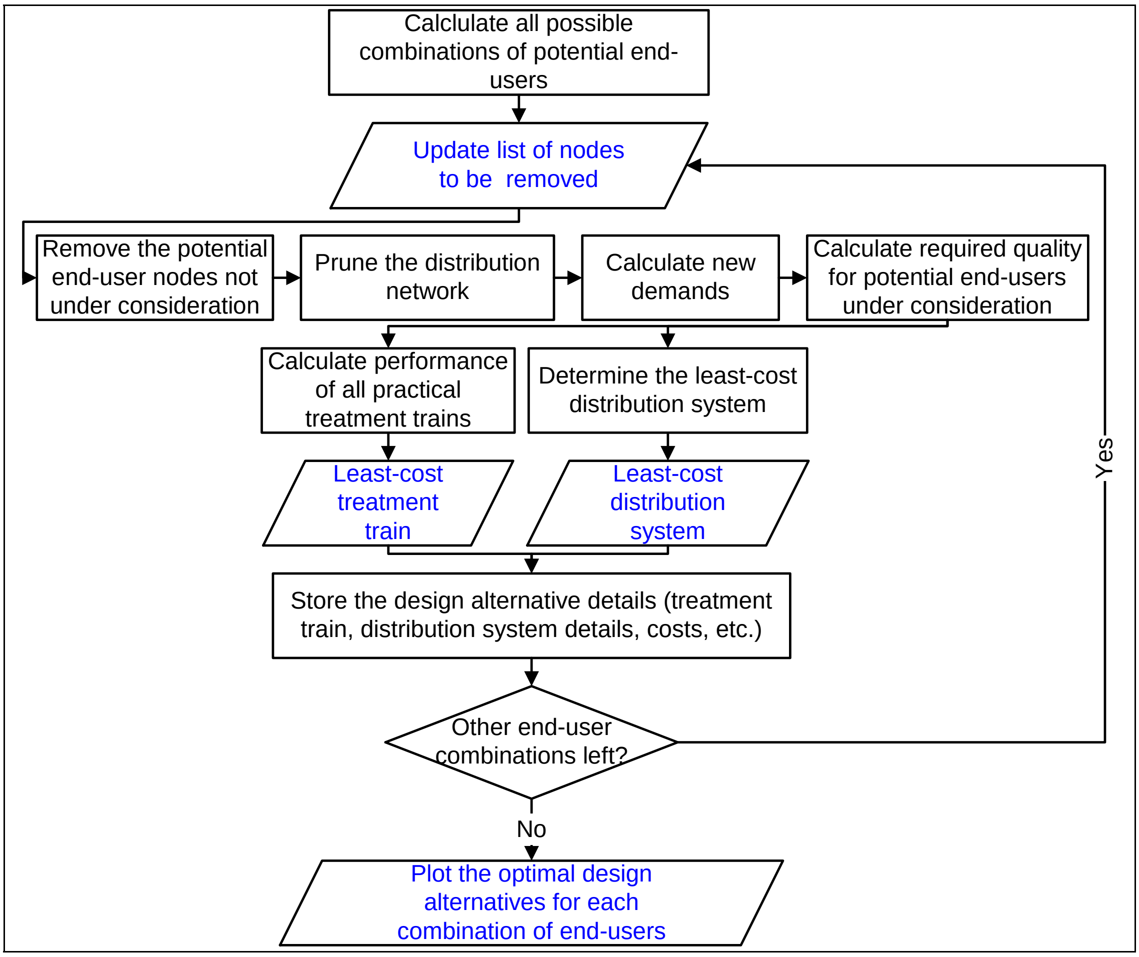


Figure 4.3 Enumeration of Design Alternatives

With the flow and quality requirements determined for the set of potential end-users under consideration, selection of the optimal treatment train is carried out. Beginning with each of the possible unit processes for the given inflow (secondary effluent), other unit processes are added sequentially according to the treatment train assembly rules, and the effluent quality is determined. The process of adding unit processes is stopped if: 1) the required effluent quality is reached (i.e. a practical treatment train, defined earlier, is found), or 2) the list of unit processes contained in the knowledge base is exhausted. In the first case, the assembled treatment train is evaluated in detail, and it is stored if its lifecycle cost is smaller than that of any treatment train stored previously. In the second case, the least-cost treatment train identified previously (beginning with a different unit process) is used for further comparisons. The end result of this process is the least-cost treatment train, which is then combined with the calculated least-cost distribution system to form the optimal design alternative for the set of potential end-users under consideration. Once all possible combinations of end-users have been explored, the output of the enumeration provides the least-cost design alternatives for each set of end-users.

4.4 Optimisation using GAs

As stated earlier, two GA-based methodologies are developed and used in the DSS. The simpler GA is used if secondary treated wastewater is used as a source, and a large number of potential end-users are involved. The comprehensive GA methodology is appropriate for use with water reuse systems using raw sewage or primary treated wastewater as source. A brief general overview of GAs is provided next, followed by a discussion on the search space definitions adopted in this research. Details of the optimisation methodologies used in the DSS are subsequently presented in the two sections that follow.

4.4.1 GA Basic Concepts

Genetic algorithms belong to a group of random search techniques which utilise principles of natural evolution for optimisation and search problems. First introduced by (Holland 1975) and expanded by (Goldberg 1989), GAs have been applied successfully to solve a large number of diverse optimisation problems in various fields. This includes optimising a wide range of water resources planning, design and management problems covering water distribution systems (Murphy and Simpson 1992; Ostfield 2005; Savic and Walters 1997), water and wastewater treatment (Dinesh and Dandy 2003; Loughlin et al. 2001), urban drainage systems (Yeh and Labadie 1997), reservoir systems (Labadie 2004), groundwater remediation (Hilton and Culver 1998) and river management (Cho et al. 2004; Vasquez et al. 2000).

The basic concept of GAs involves representing the search space of the optimisation problem (i.e. potential solutions) as a population of genetically coded individuals, and evolving that population over a number of generations to come up with fitter individuals by allowing the survival of the fittest. This progression is shown in Figure 4.4, which also identifies the fundamental processes used in the evolution process, further explained below.

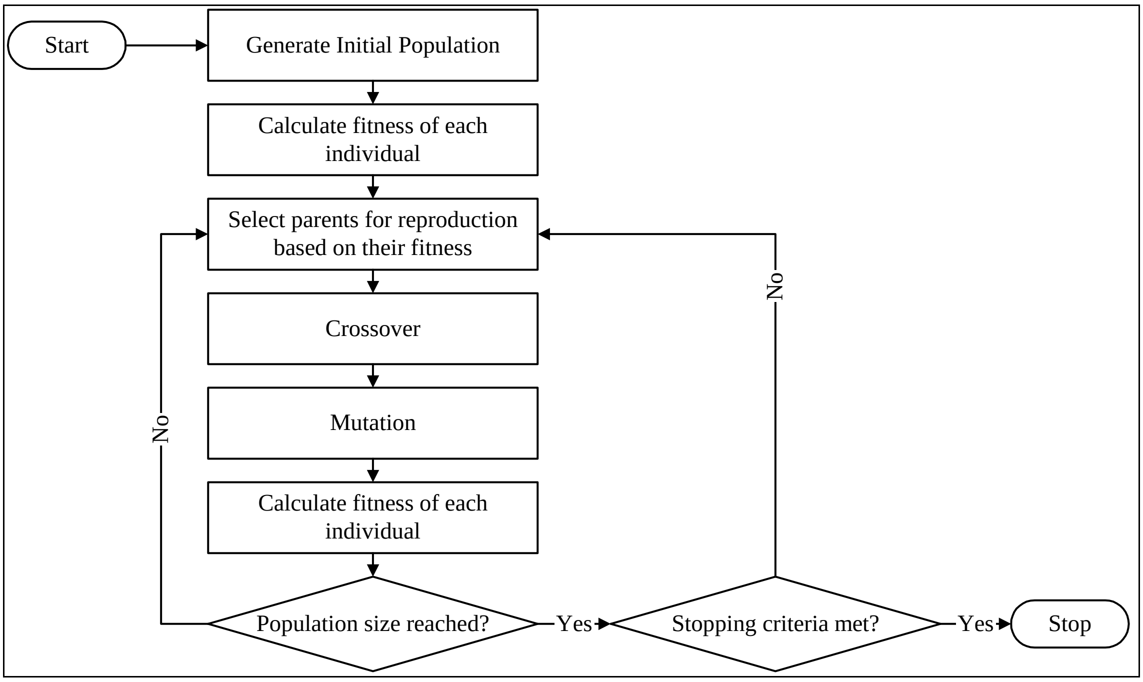


Figure 4.4 Basic Flowchart of Genetic Algorithms

The first component of a GA is the genetic representation of potential solutions using a coding scheme, which can either be binary, Gray, integer or real. In binary and Gray coding, 0 and 1 are the components of a chromosome (string) that represents an individual of the population. The difference between the two coding schemes is that Gray coding represents each number in the sequence of integers in an order such that the representation of adjacent integers differs in only one bit position, whereas several changes in the bit may be required for binary coding. In the integer and real coding schemes, the individual is represented as a chromosome containing integers and real (floating-point) numbers, respectively. Different coding schemes have been applied to a variety of optimisation problems, but an overall conclusion does not exist regarding which coding scheme is most appropriate for any particular type of problem. The coding scheme used in the DSS is discussed in Section 4.4.2.

The generation of the initial population can be done in three ways: randomly, heuristically, and by a combination of the two. The heuristics can be incorporated in the generation process, with some knowledge or past experience with the problem at hand. None of the methods have been shown to be universally better over the others. The initial population, however, should generally be well spread over the entire search space as much of as possible, without weighing too heavily on the solutions of poor quality, thus slowing down the search process, or those of good quality, leading to premature convergence and thus identification of suboptimal solutions. The generation of initial population is further discussed in Section 4.6.2, which also describes the implementation included in the DSS.

The fitness function is used to determine the relative strength of each individual. The individual fitness relative to all other individuals in a population thus determines its chances of survival and reproduction. In single-objective GAs, both the objectives and constraints are typically contained in the fitness function. Multiple objectives are typically aggregated, and the violation of constraints by the individuals is reflected in the fitness function through penalties, which are subtracted from or added to the value of the aggregated objective function for maximisation and minimisation problems, respectively. Alternatively, the objective function can be multiplied by a factor proportional to a constraint violation, as in the Multiplicative Penalty Method (MPM) used by Carlson (1995) and Hilton and Culver (1998). An exhaustive review of approaches to constructing penalty functions is presented by Michalewicz (1999). A multi-objective GA formulation allows exploration of tradeoffs between the multiple and often conflictive objectives. Both a single-objective and multi-objective formulations of the GA were developed and implemented within the DSS, as discussed in Sections 4.6.5 and 4.6.6.

Once the fitness of each individual in a population is calculated, the process of selection takes place according to Darwin's theory of evolution, whereby the fittest individuals survive to create new offspring. In the context of optimisation, the basic purpose of the selection process is to focus the search on most promising solutions in the search space. The selection of parents that can be used for reproduction may be performed in several different ways that have been classified into proportionate selection, ranking selection and tournament selection (Goldberg and Deb 1991). These and other authors, such as (Blickle and Thiele 1995) and (Zhang and Kim 2000), provide an overview and performance comparison of different selection schemes. The selection schemes implemented in the DSS are discussed in Sections 4.6.5 and 4.6.6.

The two remaining mechanisms that are used in a GA to create the child population from the genetic pool created through selection are the crossover and mutation. Crossover is used to conduct the partial exchange of the genetic material of parent chromosomes. Some of the crossover operators that have been used in the past include one-point, two-point, uniform, partially matched and cycle crossovers, whose descriptions can be found in (Goldberg 1989). The probability of crossover (P_C) defines the average frequency of crossover occurrence in a population. The mutation operator is used to enhance the population diversity and thus prevent convergence to local optima. It is implemented by altering the bits of randomly selected individual chromosomes with average frequency equal to the probability of mutation (P_M). The implementation of the crossover and mutation operators in the DSS is discussed in Sections 4.6.3 and 4.6.4, respectively.

Two additional terms also need to be defined that determine how the replacement of the parent population is carried out. With steady-state GAs, one (or several) individuals with low fitness are replaced with one (or several) higher fitness chromosomes that underwent the selection, crossover and mutation process. In generational GAs, on the other hand, the entire population is replaced, except in cases where elitism is used, where several individuals with highest fitness values are automatically carried out to the next generation.

4.4.2 Definition of Search Space and Feasible Alternatives

The primary reason for this section is to present the reasoning behind some of the decisions taken in the development of the optimisation methodology used in the DSS. The choices made here are based to a considerable extent on the detailed review of MOSTWATER implementation and results reported by Dinesh (2002), as it is the only GA-based tool developed specifically for optimal selection of technologies for wastewater reclamation and reuse.

The fundamental difference between the approach taken here and the one in MOSTWATER deals with the definition of feasible treatment alternatives, as illustrated in Figure 4.5, where the large rectangles represent all combinations of unit process in a knowledge base. The smaller rectangles, representing combinations of unit processes that meet treatment train assembly rules and the reuse water quality criteria, represent a fraction of all possible unit process combinations. These fractions are actually very small, as the calculations based on the knowledge base of this DSS showed in Section 4.2. In MOSTWAR, feasible treatment trains are defined in a classical way as those meeting both the reuse criteria and the rules for their assembly. These treatment trains were defined earlier in this work as practical treatment trains. In contrast, the approach taken here and shown on the right identifies all treatment trains meeting the generation rules as feasible.

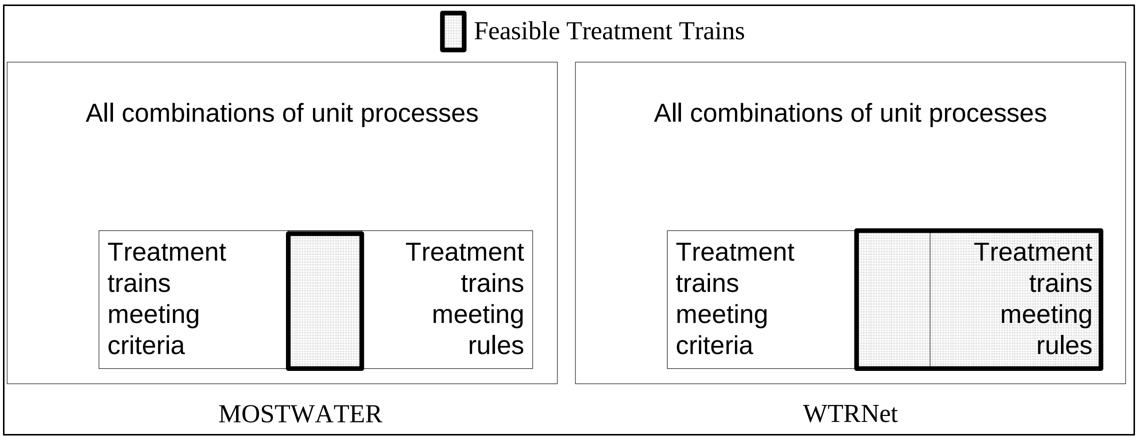


Figure 4.5 Illustration of Search Space Feasible Treatment Alternatives

The definition of feasible treatment trains has significant implications in the GA, in terms of scope and efficiency of the search process, since a narrower representation of the feasible solution space results in a larger number of infeasible options throughout the search process. This was evident in MOSTWATER, where significant effort was expensed on the modification of methods for generation of initial population, types of operators used and their values, and penalty calculation methods to reduce the number of infeasible solutions and the time in which good solutions were found.

A more efficient approach was sought in this thesis, which was considered to hinge on the definition of feasible treatment trains. Due to the demonstrated fact that feasible treatment trains, as defined in this work, represent such a small portion of the overall number of combinations of unit processes, the approach was aimed at reducing the GA search to that region only. In other words, the goal in the development of the optimisation methodology was to devise methods for coding of design alternatives, generation of initial population, and crossover and mutation operators that allowed exploration of feasible alternatives only. Such methodology would effectively account for the most complex constraints in the optimisation problem, the treatment train assembly rules, within the search process. The methods that were developed and implemented in the DSS are presented next.

4.5 Simple GA

A full description of a design alternative for integrated water reuse scheme includes the following details: composition of the treatment train, the potential end-users and distribution system details. The details of the distribution system, however, such as the size and layout, did not need to be specified here by the chromosome since they are determined by the selection of end-users (using the sequential approach for sizing and user-defined layout). Since enumeration is used to determine the least-cost treatment for any combination indicated in the GA, the chromosome includes only information on the potential end-users that are incorporated in the evaluation, using binary coding.

The procedure used by the simple GA for optimal selection of end-users is similar to the enumeration illustrated in Figure 4.3. Instead of evaluation of all end-user combinations, which may be computationally prohibitive, the GA is used to optimise the selection of customers. The GA is implemented using optiGA library (Salomons 2001), which includes a number of options for GA parameters. The implementation of the library in the DSS uses the tournament selection operator, one point crossover and the flip bit mutation operators. Since the simple GA approach is considered relatively trivial in the context of the overall DSS, it was not extensively tested or used in further analyses presented in this thesis.

4.6 Comprehensive GA

4.6.1 Coding of Design Alternatives

The simple GA, due to enumeration being used for treatment train evaluation, uses binary strings to represent only the selection of end-users. Contrary to this, the chromosome representation of design alternatives in the comprehensive GA needs to include both the unit processes forming the treatment train and the information on the selection of end-users.

The simplest way of describing the treatment train and inclusion of end-users in a water reuse scheme is using a binary string. The binary approach was adopted for representation of treatment trains in MOSTWATER. The treatment train chromosome used there consisted of genes that were assigned a value of 1 for unit processes included in the treatment train and 0 for those that were not included. The binary values of the string were ordered to correspond to identification numbers of unit processes contained in the knowledge base, and the maximum chromosome length was thus limited to its size (number of unit processes in the knowledge base). The representation of a treatment train with binary strings was deemed appropriate in MOSTWATER, since it allowed for a simple GA to be used in the optimisation by the author. However, the binary representation along with the definition of feasible alternatives discussed earlier resulted in significant computational cost of generating feasible treatment trains both during the generation of initial population and throughout the optimisation process.

The binary representation was initially considered in this research, but found to be inadequate because of incompatibility with the choices made with regards to the definition of feasible solutions and crossover and mutation operators. Instead, an integer representation of design alternatives illustrated in Figure 4.6 was developed, which offered significant advantages in later development and application of the GA.

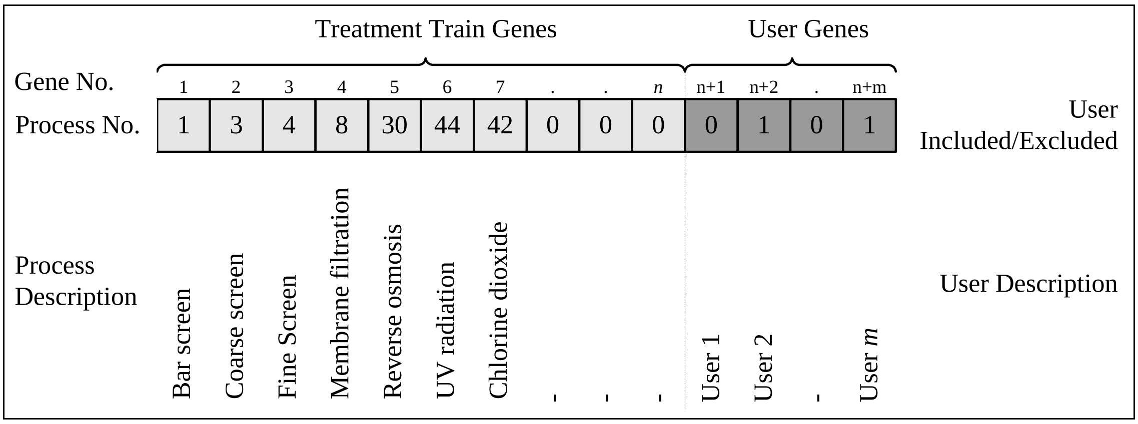


Figure 4.6 Chromosome Representation of Design Alternatives

The chromosome representing design alternatives consists of two parts. The first part represents the scheme treatment train, and it is composed of genes representing unit process serial numbers as they are ordered in the knowledge base. The maximum length of this part of the chromosome (*n*) is determined through experimentation and varies depending on the number of unit processes in the knowledge base, treatment train assembly rules and the quality of the influent. Using the information contained in the DSS, *n* was found to take values of 16, 10 and 7 if raw sewage, primary effluent and secondary effluent are used as source respectively. The second part of the chromosome represents the end-users that are included or excluded from the scheme using values of 1 and 0 respectively, and its maximum length (*m*) is equal to the number of potential end-users considered in the evaluation.

The chromosome representation of design alternatives used in the DSS has both advantages and drawbacks. The first advantage is that the overall length of the chromosome can be kept relatively low since it does not need to include a gene for each unit process added to the knowledge base. Secondly, it allows the order of unit processes in the treatment train to be specified explicitly, which was an important and necessary feature needed in the development of crossover and mutation operators. Finally, the representation exhibits good scaling properties, should the optimisation involve multiple treatment facilities (not addressed in this thesis) or large numbers of potential end-users. The drawback of the approach is that the exact length of the treatment train part of the chromosome has to be determined through experimentation (enumeration of unit process combinations) if any changes are made to the knowledge base related to the number of unit processes, starting processes or assembly rules.

4.6.2 Generation of Initial Population

The main issue that guided the development of the procedure for generating initial population of design alternatives was that all had to be feasible. This necessitated incorporating some of the input information into the procedure. The information used as input to the process is the water quality of the influent, from which the possible staring unit processes and the maximum length of the treatment train are calculated, and the number of potential end-users. These parameters determine the chromosome length, as described in the previous section.

The flowchart of the procedure developed for generation of the initial population of chromosome representing design alternatives is shown in Figure 4.7. The generation of the chromosome portion representing the treatment train is shown on left side. A starting unit process is randomly selected from the list of possible processes (SP_i), and added to a temporary treatment train (TT_{Temp}). A procedure is then employed to determine the unit processes ($Paths(TT_{Temp})$) that can be added to TT_{Temp} without violating the treatment train assembly rules contained in the knowledge base. If the number of unit processes that can be added (np) is zero, the process of formation of the treatment portion is finished and TT_{Temp} is included in the created chromosome. Otherwise, a zero is added to TT_{Temp} to allow the process to include the possibility of early ending of the formation procedure of the treatment train. The next unit process to be added to TT_{Temp} is selected randomly from the unit processes contained in $Paths(TT_{Temp})$, and the process for building of the treatment train is repeated until no further processes can be added or a zero is selected (randomly) from subsequent $Paths(TT_{Temp})$.

The process of representation of end-users included in a design alternative, shown on the right side of Figure 4.7, is less involved. A random number between 0 and 1 is selected m times, and the corresponding end-user is included in the design alternative being generated if the number generated is higher or equal than 0.5. If the random number is smaller than 0.5, the corresponding user is not included in the scheme. The end result of the process is a string of binary values of length m, which is then copied to form the end-user portion of the created chromosome.

The procedures for generating the treatment and end-user portions of the chromosome described above are repeated until the number of chromosomes created is equal to the set population size (*Pop.Size*). Its performance was thoroughly tested for a range of inflow conditions and the results of this testing are provided in Section 5.2.2.1.

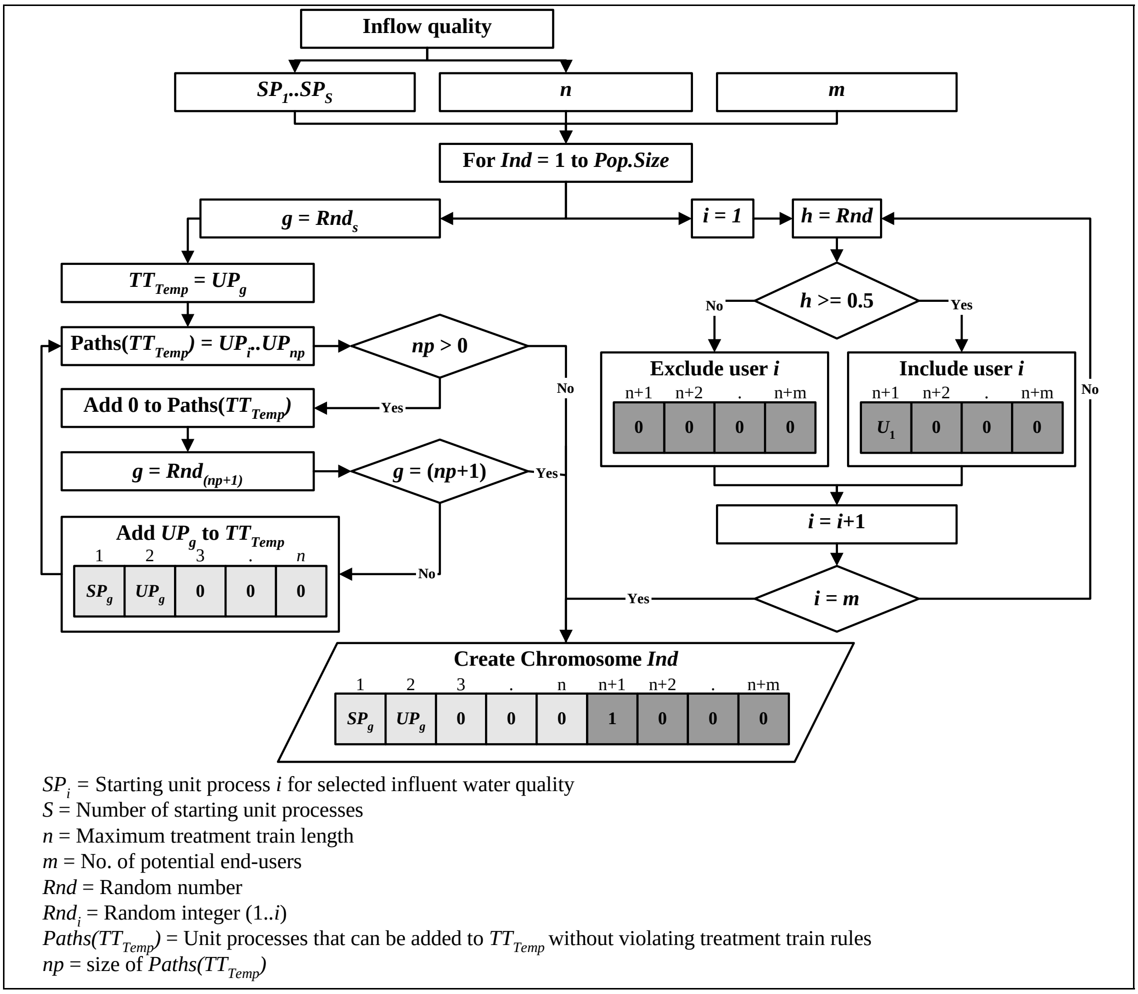


Figure 4.7 Initial Population Generation Flowchart

4.6.3 Crossover Operator

As with the generation of the initial population, the main objective in the development of the crossover operator was to avoid producing infeasible design alternatives, this time in child populations. The basic purpose of the crossover operator is to provide a mechanism for exchange of genetic material, or "building blocks", between parents in the hope that the created chromosome will be better if it inherits good characteristics from both parents. In this work this concept is interpreted as swapping portions of parent chromosomes dealing either with the treatment or the end-user selection aspect.

For the treatment part of the chromosome, the "building blocks" considered were processes belonging to the same category. This way, the unit processes belonging to the same category could be exchanged between the parent chromosomes, albeit not in a simple manner. The portion of the chromosome describing the inclusion of end-users in the reuse scheme was considered as the final "building block". The final formulation of the crossover operator developed and implemented in the DSS is showed in Figure 4.8. For improved clarity, parent chromosomes shown in the flowchart of the procedure include example unit process serial numbers. In addition, grey-scale is used in genome representation to indicate the category to which each unit process used in the example belongs.

Through testing of various approaches of performing the exchange of parent unit processes, a difficulty of performing the crossover was observed if processes were present in parent treatment trains that can exist in multiple positions related to other processes. These processes are distinguished in the treatment train rules matrix shown in Figure3.3 as having both "<<" and ">>" operators. For example, *MF* and *UF* can be used both as immediate pre-cursors and immediate post-cursors to *GAC*. Therefore the first procedure performed in the crossover is to examine the categories to which unit processes of each parent treatment train belong ($Cat(UP_i)$). This examination is performed by determining the categories of unit processes included in each parent chromosome sequentially. If a process encountered in this way is of a lower category than the preceding one for any of the parent chromosomes, the crossover procedure diverts to exchanging the complete portions of chromosomes describing parent treatment trains. The testing of the crossover operator, discussed in Section 5.2.2.2, indicated that the condition described above is encountered infrequently and thus does not influence the overall search process significantly.

If none of the two parent chromosomes contain treatment trains that include the "complicating" unit processes described above, the numbers of unit processes belonging to each category are determined for each parent ($P1.C_i$ and $P2.C_i$). If both parents have unit process belonging either to the same or adjacent categories, the category is designated as a potential mating category (PMC_i) and the number of potential mating categories (*NMC*) is increased by one. If no potential mating categories are identified, the crossover procedure again diverts to exchanging the complete treatment train portions of chromosomes. Otherwise, a mating category (*mc*) is set through random selection from the list of potential mating categories, and unit processes belonging to higher categories are temporarily swapped between the parent chromosomes to create potential offspring chromosomes (PO_i).

The treatment trains created in the potential offspring are checked against the treatment train assembly rules. If either treatment train fails to meet the rules, a new mating category is selected and the procedure of creating and checking the potential offspring is repeated, unless the list of potential mating categories has been exhausted. In that case, treatment train portions of parent chromosomes are swapped. If, on the other hand, a potential offspring is created that meets all treatment train assembly rules, the swap in unit processes belonging to categories higher than the current mating category is made permanent and the offspring is created.

The procedure described above is performed by selecting pairs of individuals from the parent population, and conducting the mating if a randomly generated number between 0 and 1 exceeds the specified probability of crossover. The efficiency of the operator, in the context of unit processes and treatment train assembly rules contained in the knowledge base, was examined through testing, and is discussed in Section 5.2.2.2

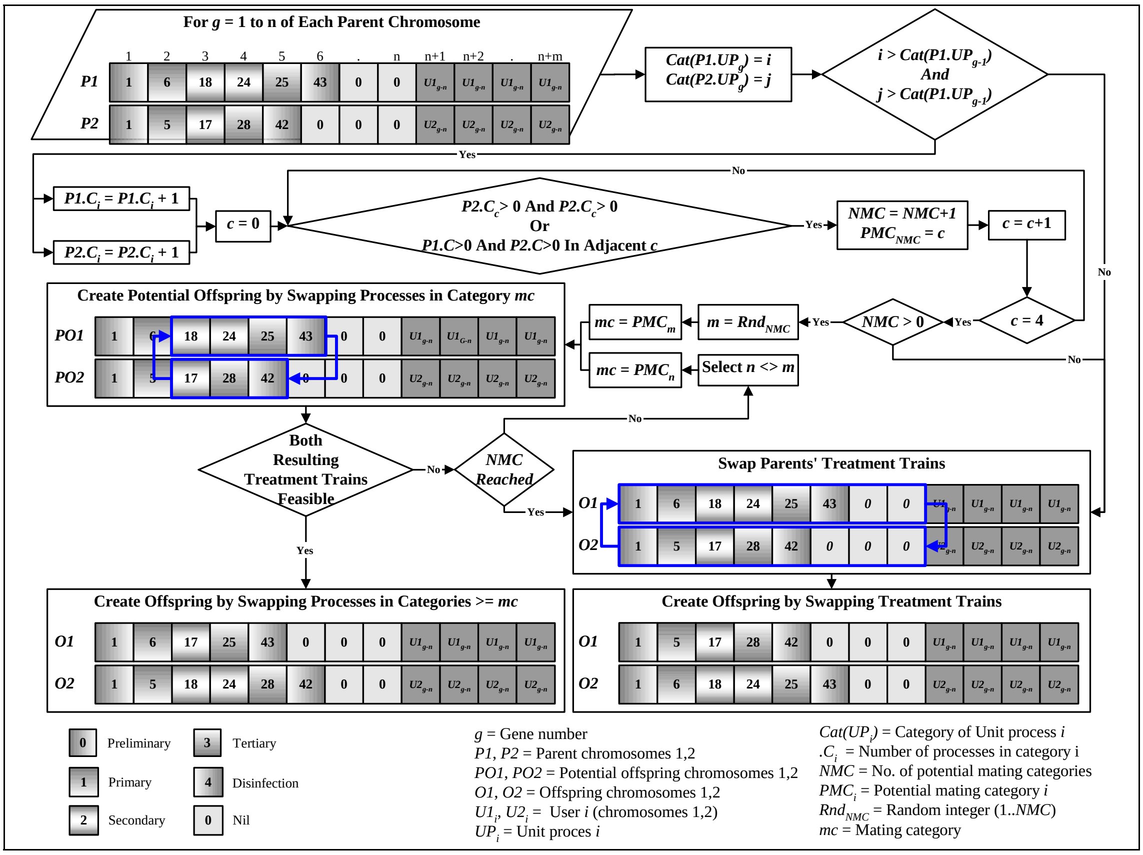


Figure 4.8 Crossover Operator Flowchart

4.6.4 Mutation Operator

The development of the mutation operator was also conducted under the overall objective that the procedure should minimise or possibly eliminate the generation of infeasible design alternatives. The mutation procedure, illustrated in Figure 4.9 is performed on the child population of chromosomes produced by the crossover procedure.

The procedure begins by randomly identifying the gene that will be mutated in the selected chromosome. If the gene selected for mutation is in the chromosome portion that deals with the end-user selection, the binary value is simply flipped to include/exclude the selected end-user. If a treatment train portion of the chromosome is to be modified by the mutation operator, the unit process corresponding to the gene identified is temporarily replaced with each of the unit processes included in the knowledge base. The resulting treatment train, which now includes the replacement process (RP_i), is checked against the treatment train assembly rules. If the treatment train passes the rules, the process used as replacement is identified as a possible replacement processes (PRP_i), and the procedure is repeated depending on the size of the knowledge base (N_{TPros} times). If no possible replacement processes are identified, a new gene is randomly selected and the while procedure is repeated. Otherwise, the unit process identified for mutation in the original chromosome is replaced with a unit process randomly selected from the list of possible replacement processes.

The procedure described above is invoked for each member of the child population, but it is executed in entirety only if a randomly generated number from 0 to 1 is smaller than the probability of mutation specified in advance. The mutation operator was also tested to better understand the practical aspects of its mechanism, and results of the testing are described in Section 5.2.2.3.

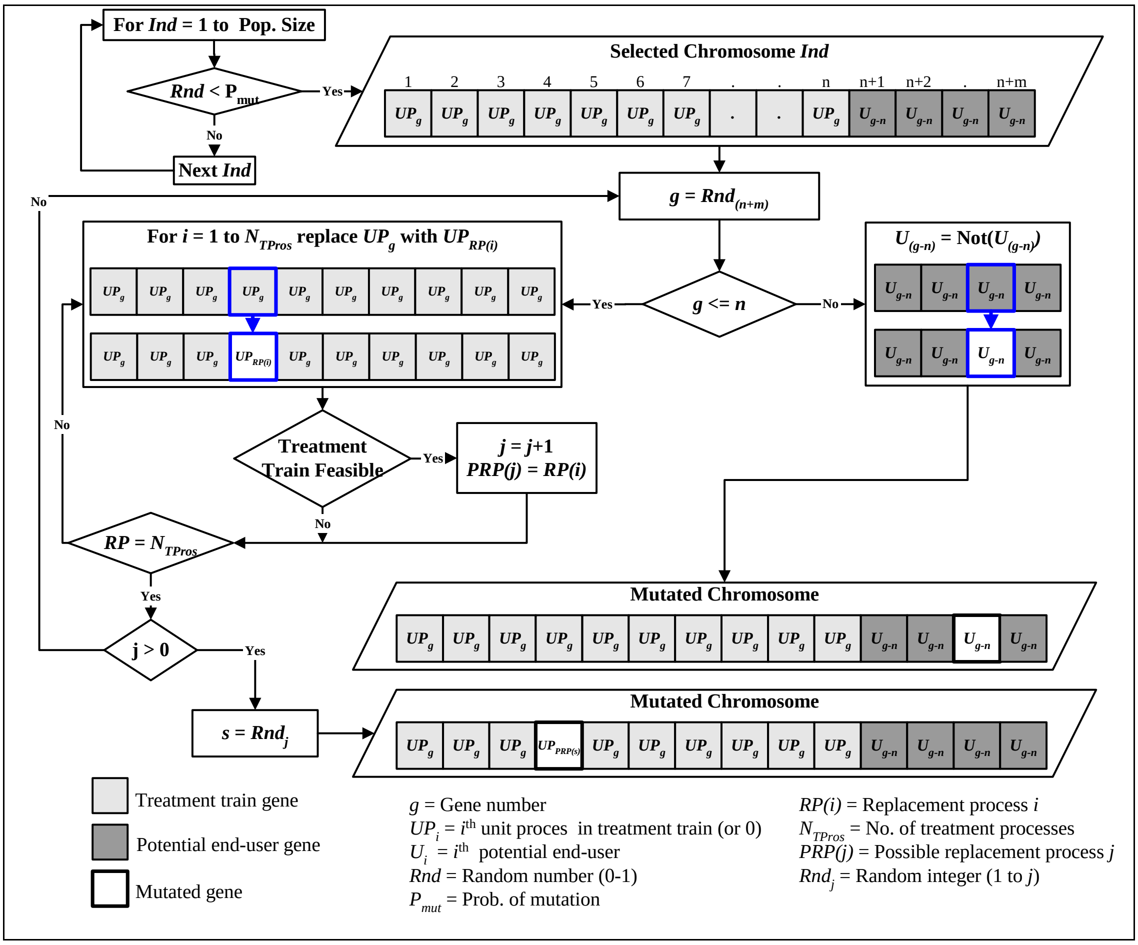


Figure 4.9 Mutation Operator Flowchart

4.6.5 Single-objective Optimisation

Evaluations of integrated water reuse schemes are conducted based on a number of criteria described previously. However, a single-objective formulation was considered useful for testing of the developed DSS methodology because of its simplicity and ease of use, and was thus implemented as described in this section.

A common way to incorporate multiple objectives into a single-objective framework is the weighing technique, whereby weights are assigned to individual objectives to express the importance or preference for one criterion over the others. This methodology was used to calculate the overall treatment train qualitative criteria score, as described in Section 3.3.2. Since the main use of the single-objective formulation was for testing of the developed GA, the fitness function is kept simple and does not include all results of the evaluation of integrated water reuse schemes. Instead, four major results are used in the fitness function (F_i) shown in Equation 4.1: scheme net present value, fraction of overall potential demand satisfied, treatment train qualitative criteria score and penalty for not meeting the required water quality.

$$F_i = \frac{NPV_i^{w_F} \cdot DS_i^{w_{DS}} \cdot QS_{TT_i}^{w_{QS}}}{w_P \cdot \left(1+P_i\right)} \tag{4.1}$$

Where:

w_F = User assigned weight for financial viability
NPV_i = Net present value of design alternative i
w_{DS} = User assigned weight for satisfying potential demand
DS_i = Fraction of potential end-user demand satisfied
w_{QS} = User assigned weight for qualitative criteria treatment train score
QS_{TT_i} = Overall qualitative criteria treatment train score
w_P = User assigned weight for water quality penalty
P_i = Penalty for inadequate water quality

The scheme net present value is calculated from annual costs associated with the design alternative and revenues generated from sale of reclaimed water to end-users included, as shown in Equation 4.2. The fraction of potential demand satisfied is calculated simply from the potential and satisfied end-user requirements, and the treatment train qualitative criteria score is calculated as described earlier using Equation 3.14. Weights are included in the formulation of fitness function to allow emphasising any of the criteria in the optimisation, which were kept equal to one for all three objectives in this work. The fitness function is subject to maximisation, and therefore these three objectives are included as the nominator.

$$NPV_i = \sum_{t=0}^{LC} \frac{AR_t^i - AC_t^i}{\left(1+r\right)^t} \tag{4.2}$$

Where:

LC_i = Lifecycle of design alternative i
r = Dicount rate
AR_t^i = Revenues from end-users included in design alternative i in year t
AC_t^i = Costs associated with design alternative i in year t

The penalty for inadequate water quality is included in the denominator of the fitness function, and it is calculated using Equation 4.3. This formulation of the penalty function captures how far the treatment train of the generated design alternative is from the region where it satisfies the quality requirements of the end-users served, or how far it is from being practical (as defined earlier). It aggregates the failures of meeting the water quality criteria for all pollutants under consideration. As such, it is a distance based static penalty function, which have been found to perform better than penalty functions based on the number of constraints violated (i.e. how many pollutant concentrations are higher than required) (Coit and Smith 1995).

$$P_i = \begin{cases} \sum_{n=1}^{NPols} \dfrac{CE_i^n - CR_i^n}{CR_i^n} & , CE_i^n > CR_i^n \\ 0 & , CE_i^n \leq CR_i^n \end{cases} \tag{4.3}$$

Where:

$NPols$ = Number of pollutants in the knowedge base

CR_i^n = Concentration of pollutant n required for scheme i

CE_i^n = Concentration of pollutant n produced in scheme i

Tournament selection is used to create the genetic pool from which the child population is created, in which a number of solutions from the parent population are chosen and the best are carried forward. The number of solutions chosen for comparison is defined as the tournament size, and the most common implementation (also used here) is to choose two individuals at a time. An additional term is defined called the selection probability, which determines the frequency with which the best of the solutions being compared is advanced. The purpose of this term is to control the selection pressure so to avoid premature convergence of the algorithm. A constant value of 0.9 was used in this work for selection probability.

4.6.6 Multi-objective Selection

While the single-objective optimisation was considered appropriate for testing of the developed methodology, a multi-objective formulation was also developed for application of the DSS. The importance of considering the optimisation of an integrated water reuse scheme as a multi-objective problem is evident in the objectives that it needs to achieve, since some of them may be conflicting. For example, satisfying a greater percentage of the potential end-user demand will most likely lead to systems of larger cost, and may lead to higher sludge production, energy consumption and land and labour requirements. While this may be true in most of the cases, the objectives may not necessarily conflict depending on the choice of treatment methods, as well as the locations and requirements of the end-users. Finally, a multi-objective approach was needed since one of the objectives of this thesis is to provide some potentially useful design principles for integrated water reuse systems based on multiple objectives.

The potential tradeoffs between objectives cannot be explored if the objectives are lumped into a fitness function, and require that a multi-objective algorithm be used for optimisation. More fundamentally, in multi-objective optimisation problems with conflicting objectives, a single optimal solution does not exist (Deb 2001). Instead, there exist a set of solutions that are referred to as non-dominated or Pareto-optimal solutions, which can be explored by the decision maker to make intentional and quantifiable trade-offs in satisfying different objectives . Other advantages of the multi-objective approach, argued by (Cohon 1978) and summarised in (Savic 2002), include:

- a wider range of alternatives are usually identified,
- roles of the analyst, who generate the solutions, and the decision maker, who uses the solutions to make informed decisions, are made more appropriate, and
- consideration of many objectives allows for more realistic models of a problem.

The objectives considered in the multi-objective optimisation incorporated in the DSS developed here are summarised in Table 4.3, which also provides the units used for each of the objectives and whether they are to be maximised on minimised in the optimisation.

Table 4.3 Objectives Considered in Multi-objective Optimisation

Objective	Units	Maximise	Minimise
Demand satisfied	%	✓	
Scheme lifecycle cost*	€		✓
Scheme net present value*	€	✓	
Qualitative criteria score	-	✓	
Land required (treatment)	ha		✓
Sludge production	Tonnes/year		✓
Concentrates production	m^3/year		✓
Energy consumption (treatment)	kWh/year		✓
Labour requirements (treatment)	Person.hours/month		✓
*Alternative objectives			

The percent of demand satisfied objective would be appropriate in situations where competing customers are being considered for a limited production of reclaimed water and there is a need, driven by political reasons perhaps, to maximise the number of customers serviced. Two of the objectives, lifecycle cost and net present value, are both financial and expressed in monetary terms so they can be used alternatively. The difference between the two is that the lifecycle cost does not account for any projected revenues from the project, which may be appropriate if rates that are charged for providing service to potential customers cannot be specified in advance. Also, minimisation is desired if the lifecycle cost is chosen as the objective, whereas the net present value of an integrated water reuse scheme should be maximised in the optimisation.

The treatment train qualitative criteria, presented in Table 2.1, are also considered albeit as a single objective, indicated by the overall treatment train qualitative criteria score to be maximised. Inclusion of individual criteria would result in what has been termed a Evolutionary Many-objective Optimisation (EMO) problem. Issues that arise with inclusion of many objectives in the optimisation using evolutionary algorithms, a review of remedial measures and a promising new approach were presented by di Pierro (2006), however, this is considered beyond the scope of this thesis. The remaining objectives are all to be minimised as they are essentially indicators for resources needed for the production of reclaimed water. While the land requirement is a completely separate criterion, the other four are actually accounted for in the cost of treatment. Nevertheless, they are included as separate objectives that may need to be explored in cases where resources are scarce or limited.

Several multi-objective GA algorithms have been developed over the past decade and a half and applied to a variety of optimisation problems. A detailed review of these approaches is beyond the scope of this thesis, and an interested reader is referred to an excellent overview presented by Deb (2001). An algorithm that has been used perhaps most often in optimisation of multi-objective engineering problems, which was also employed in this research, is the Fast Elitist Non-Dominated Sorting Genetic Algorithm (NSGA-II) (Deb et al. 2002). For full explanation of the algorithm, the reader is referred to the publication cited above, whilst the general concept is presented here. First, however, the dominance of a solution needs to be defined. The definition of dominance can be illustrated on a problem where all objectives considered are to be minimised as follows: if *A* and *B* are two different design alternatives with scores on the objectives denoted as $f_i(A)$ and $f_i(B)$, respectively, alternative *A* is said to dominate the other alternative if $f_i(A) \leq f_i(B)$ for $i = 1,\ldots,m$, and $f_i(A) < f_i(B)$ for at least one i, where m = number of objectives. More generally, this means that alternative *A* dominates alternative *B* if it satisfies all objectives at least equally to the level to which they are satisfied by alternative *B*, and at least one objective is better than that offered by alternative *B*. The NSGA-II algorithm follows all basic steps presented in previous sections, with the exception of the selection of children population and the calculation of individuals' fitness, both of which are described below.

In the reproduction process, the offspring population is first created using the tournament selection described earlier but using different information in the comparison of individuals. Since no unique fitness value is used in multi-objective GA, the individuals selected are compared on the basis of their rank or crowding distance, both of which are described below. Following this selection process, the parent and offspring population are grouped into a genetic pool, which is then sorted whereby non-dominated solutions are assigned the highest available rank and copied into the child population. These individuals are then taken out of the genetic pool and therefore not included in subsequent sorting operations. The process of determining the rank of the individuals remaining into the genetic pool and copying the ones of highest rank continues, until the number of individuals forming the highest current rank is greater than the number needed to fill the child population.

When the number of individuals in the currently highest rank is higher than needed, a crowding-distance is determined for each individual, which is an operation used to guide the selection process towards a uniform spread of solutions along the trade-off curve. The individuals with the highest current rank are then sorted according to their crowding distance in a decreasing order, and the individuals required to reach the set size of the child population are used, while all others are discarded. The whole procedure is illustrated in Figure 4.10, where current population is denoted by P_t, offspring is labelled Q_t, differently ranked individuals are denoted by F_i, and the child population (or the population of the next generation) is indicated as P_{t+1}.

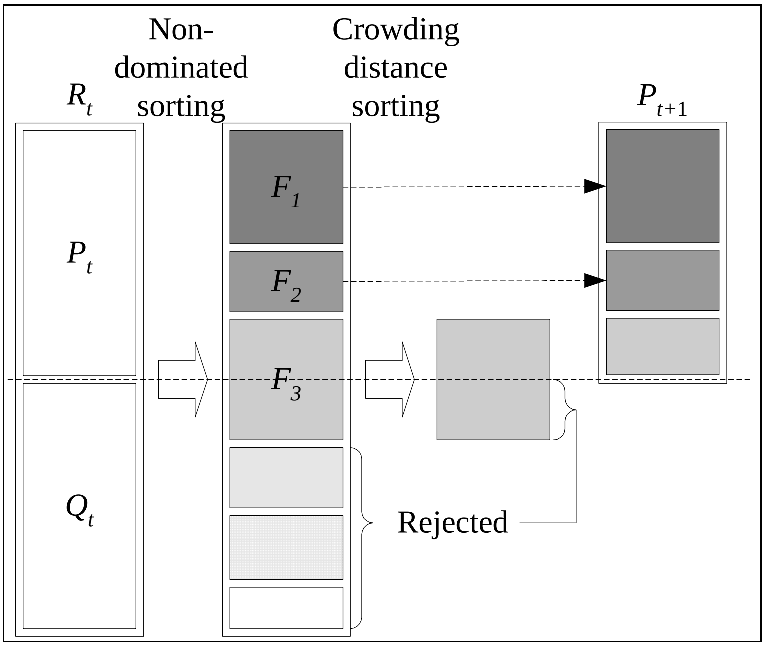


Figure 4.10 NSGA-II Procedure for Generation of Child Populations

An additional advantage of using NSGA-II is that design alternatives that violate certain constraints can be included in the optimisation. This is particularly relevant to the treatment part of the integrated water reuse scheme optimisation, since the user can provide a tolerance level for constraint violation (i.e. specific level of effluent quality violation that can be tolerated). Therefore, treatment trains that are close to meeting the required effluent quality are not discarded in the optimisation, and can be part of the optimal set of design alternatives. This is advantageous, since optimal solutions often lie on the boundary of feasible solution space (Siedlecki and Sklansky 1995), and since unit process efficiencies can vary from the values used in the knowledge base. The treatment train constraint violation is calculated by aggregating the failures on all water quality criteria considered using Equation 4.3. The sensitivity of optimisation results to different tolerance limits was explored and the results are reported in Section 5.3.4.

The output from the NSGA-II optimisation run obviously does not produce a single best solution. Instead, the output contains a number of non-dominated solutions with the lowest rank, which can then be compared prior to deciding on a single solution. In an integrated water reuse optimisation problem optimised here, the output could be a range of alternative designs satisfying different percentage of potential demand and each having its own lifecycle costs, if these were the two objectives selected for optimisation. If three or more objectives were selected for optimisation, the analyses of optimisation results become more complicated and further discussion on this is included in Section 5.3.4.

4.7 Summary and Conclusions

Building on the methodology for evaluation of treatment and distribution components used in the DSS and presented in the previous Chapter, this Chapter deals entirely with the problem of optimising integrated water reuse schemes. The hierarchical approach used in the DSS is described, where a smaller problem of optimal distribution system sizing for a given set of end-users is solved as part of the global optimisation of the water reuse scheme.

This approach to solving large, complex optimisation problems is not new and several examples of similar methodologies are presented in the introduction of this Chapter. The section that follows includes a discussion on the results of investigation into the number of alternative designs that can be generated for systems of different size according to the information currently contained in the knowledge base. The results provide a clear indication that different integrated water reuse scheme optimisation methodologies are appropriate depending on the influent water quality and the number of end-users considered.

Based on the findings of investigations into the optimisation problem size, a simple enumeration procedure is developed and described, which might be appropriate for smaller systems involving several users and with an existing secondary wastewater treatment facility. The procedure identifies potential end-user combinations, and evaluates all treatment trains that can be assembled according to the knowledge base rules to determine the optimal solution. Design alternatives are evaluated by computing their overall cost, and the least-cost alternative is defined as the optimal.

The same least-cost objective function is used in the second, GA-based optimisation methodology, which is deemed appropriate for projects that involve upgrading secondary effluent and providing it to a large number of potential customers. The methodology uses a simple binary GA formulation to represent the potential end-users, and is implemented in the DSS using a commercially available optiGA library. A number of options are available to the user in terms of standard GA operators. This methodology is relatively simple and was not further tested or explored in this research.

For larger and more complicated systems, a novel and potentially very effective GA-based optimisation approach was developed and incorporated in the DSS. Details of the approach covering the representation of design alternatives and special operators used are provided, as well as the single and multi-objective formulations that were later used for testing and evaluation of case studies. In the single-objective GA, the fitness function is defined by combining the project net present value, percent of demand satisfied, treatment qualitative criteria score and the effluent quality achieved. The multi-objective GA includes further objectives related to the utilisation of resources. The widely used NSGA-II algorithm is implemented in the DSS for multi-objective optimisation.