Modelling and Optimisation of Pressure Irrigation Systems

By

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To my family

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Statement of Originality

This work contains no material which has been accepted for the award of any other degree or diploma in any university or the tertiary institutions and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference is made in the text.

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ABSTRACT

Trickle and sprinkler irrigation systems together represent the broad class of pressurised irrigation methods in which water is carried through a pipe system and is distributed close to the plants. The major aim of this thesis is to develop a mathematical model for the optimum design of pressure irrigation systems and thus achieve major cost savings. Throughout the study outlined in this thesis two optimisation approaches (full enumeration and genetic algorithms) are utilised. As a result, this thesis is divided into two sections: Section I deals with models in which a fixed layout for the piping system is considered and the enumeration approach is utilised and Section II considers models in which the piping layout is not fixed and the genetic algorithm is utilised as a relatively new approach to optimisation problems.

In the first section (which covers chapters 1 to 5) a review of the design and optimisation of pipe networks is undertaken. This emphasises branched networks which are used in pressure irrigation systems. Then a simple model for a Subunit with one control head is developed. A number of factors affecting the least cost solutions including the geometry of field, irrigation interval and irrigation time, slope and the positions of the manifold and supply pipes are examined. The model is extended to more complicated cases dealing with multiple subunits in which the agronomical and the agrotechnical aspects of irrigation systems are taken into account. In this model, a field is divided into a variable number of subunits in the X and Y directions. At each iteration of division the optimisation process is carried out to find the least cost solution considering various combination of subunits being irrigated simultaneously.

In the second section, the optimum layout (connection between nodes) and also optimum component sizes considering capital and operating costs are investigated. In this section, a new method called genetic algorithms (Gas) is used for optimisation. A general model using Gas dealing with optimum layout, pipe sizes and pump selection for any branched pipe system is developed. This model may be applied to any branched pipe system supplying a drip or

sprinkler irrigation system or supplying just a number of hydrants. A sensitivity analysis of GA parameters in this section is performed. The effect of various values of probability of crossover, mutation and also using different population sizes and seed numbers which create different sequence of random numbers is examined. The optimisation procedures considered in this thesis demonstrate considerable potential to produce cost savings in the design of pressure irrigation systems.

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Chapter 1

Introduction

1.1 PRESSURE IRRIGATION SYSTEMS AND OPTIMISATION

As water becomes scarce and more costly to provide and agricultural labour costs increase, there is a general move towards pressurised methods of applying water to the soil. Sprinkler and trickle irrigation systems together represent a broad class of pressurised irrigation methods in which water is carried out through a pipe system and then is distributed close to the plant. Trickle and sprinkler irrigation methods in various forms and configurations have been devised which overcome the problems of variations in topography and soil type, and allow excellent control over the amount, distribution uniformity and frequency of irrigation. This sort of system is designed to respond to plant water requirements and permit water application at a defined rate.

In trickle irrigation, the objective is to provide each plant with a continuous readily-available supply of soil moisture which is sufficient to meet the evapotranspiration demand. It offers unique agronomical, agrotechnical and economical advantages for different uses of water. The main disadvantages of trickle irrigation systems are sensitivity to clogging and salinity (which lead to a poor soil moisture distribution) and the high capital cost.

There is scope for major savings through the optimum design of these systems. In the design of an economical irrigation system, engineering and agronomical factors such as uniformity of the distribution of water along the crop rows, the infiltration rate of soil, topography, the configuration of pipes and operational limitations such as irrigation times, irrigation intervals, number of shifts, available discharge at the source and labour requirements must be taken into account.

Optimum design of water distribution systems generally implies finding the lowest cost alternatives that will satisfy the required hydraulic characteristics in the network (Karmeli et al, 1985). To achieve a least cost solution the designer of a pipe network system has to make decisions about some or all of the following factors:

- The pipe diameters and locations (layout);
- Pump locations and sizes;
- Valve and regulator locations;
- Tank locations and sizes.

In this study, two optimisation techniques are utilised. Firstly, the complete enumeration method which simulates every possible alternative solution and selects the cheapest one (global optimum). Secondly, genetic algorithms (GAs) which are based on natural selection and the mechanisms of population genetics (Holland, 1975, Goldberg, 1989). GAs have a high capability to produce a number of good solutions very close to the global optimum.

1.2 THE OBJECTIVE AND SCOPE OF THIS THESIS

A major purpose of the study outlined in this thesis is to develop mathematical models for the optimum design of pressure irrigation systems focusing on drip/trickle systems. In most previous studies on the optimum design of water distribution pipe networks, particularly in irrigation systems, a fixed layout is considered and optimum size of pipes and other components are investigated. In this study, the main effort has been attempted to develop models which consider both layout and component sizes as decision variables. In the first section of the study, an investigation is carried out to identify the best dimensions of subunits and also the best combination of subunits to be irrigated simultaneously in a multiple subunit system. In this part of the thesis a fixed configuration of layout for main and submain and also micropipes is considered and a number of other factors affecting the optimum solution are taken into account as decision variables. In the second part of study the layout and the component sizes are investigated by applying genetic algorithms as a new optimisation approach.

To summarise, the following are the objectives of this thesis in relation to drip irrigation systems:

- To investigate the use of optimisation techniques for the design of pressure irrigation systems;
- To evaluate the effect of field dimensions on the system cost for a known configuration of micropipe layouts;
- To investigate the effect of slope as an important factor affecting the uniformity of distribution and to identify the optimum position of manifold and supply pipes as they are affected by changes of slope;
- To develop a model for the division of a field into various subunits in order to identify the least cost division and also find the best combination of subunits to be irrigated simultaneously, considering the capital cost and the present value of operating cost of the system;
- To develop a model for the least cost connection of demand nodes to the source node (optimum layout) and optimum component sizes (pipe and pump) for a regular piping system;
- Finally to develop a general optimisation model for any branched piping system consisting of a source node and a number of demand nodes with any configuration considering the layout, component sizes and pump selection.

1.3 THE THESIS STRUCTURE

Throughout the study outlined in this thesis, two optimum approaches are utilised: Complete enumeration and genetic algorithms (GAs).

The thesis is structured in a manner that can be divided into two sections:

Section I deals with models in which a fixed layout for the pipe system (micro pipes and major pipes) is considered and the enumeration approach is utilised and;

Section II concerns models in which the piping layout is not fixed and genetic algorithm as a new approach to optimisation problems are utilised.

The following shows a brief outline of the remaining 9 chapters of this thesis:

Chapter 2 discusses the review of water distribution pipe networks with special emphasis on drip irrigation systems. In this review, different design approaches and various optimisation methods utilised in the design and operation of drip irrigation systems are discussed.

In Chapter 3 a simple model for the optimum design of drip irrigation system on flat terrain is introduced. A subunit with a control head and a known piping configuration is examined. The length of two given pipe sizes of each lateral and the size of manifold and supply pipes are considered as decision variables. In this model, the optimum field geometry and also the choice of various irrigation intervals and irrigation times (duration) are investigated.

The extension of the above model considering the effect of field slope is discussed in Chapter 4. In this part of the work the optimum position of the manifold and supply pipes against the slope considering a desirable distribution uniformity is investigated. This may be achieved by undertaking a tradeoff between the slope and the size of pipes and also the position of the manifold and supply pipes. This is verified by imposing different loading cases which are yielded by implementing various irrigation intervals and irrigation times.

In addition to decision variables considered in the previous model, the position of the manifold and supply pipes and also the lengths of two segments of manifold on the up and down slopes are considered. The objective function which is to be minimised consists of the capital cost (pipe, pump and accessory costs) and the present value of the annual operating cost.

Chapter 5 deals with development of an optimisation model for design of multiple subunit drip irrigation systems. The analysis is based upon dividing a field into subunits, evaluating various irrigation shift patterns with the corresponding pipe and pump sizes in order to identify a minimum cost solution. The decision variables considered are the lengths of two given pipe sizes for the laterals, the diameters of all other pipes, the size of the pump, the dimensions of the subunits, the shift patterns and the irrigation time for each shift. In this part of the research a field with known dimensions is divided into various subunits with different dimensions in the X and Y directions. For each iteration which yields a number of subunits various combinations of subunits to be irrigated simultaneously is investigated to find the best combination of subunits for an optimum operating schedule.

The second section of the study is focused on developing models to find the optimum layout and also the component sizes of pressurised irrigation systems utilising genetic algorithms. Accordingly, Chapter 6 of this section introduces the theory behind genetic algorithms (GAs) and explains the principles of this technique. The main operators of simple (standard) genetic algorithms (SGAs) are discussed. In this chapter, a brief review of the application of GAs to water distribution pipe networks is presented.

The application of GAs to pressure irrigation systems is demonstrated in Chapters 7, 8 and 9. In Chapter 7, a model for the optimal selection of the layout and connectivity of a branched pipe network is introduced and the methodology for optimising the layout using genetic algorithms is presented. The model deals with a pipe network consisting a number of demand nodes located in a rectangular pattern. Each demand node may be a control head (a supply valve of a subunit), a hydrant or a sprinkler of a pressure irrigation system. The system cost in this model consists of the major pipes supplying the demand nodes and pump costs. As the diameter of pipes is unknown the cost of the pipes is modelled as a function of their flow. The model assumes that the layout is selected from a directed base graph which reduces the size of the search space. Directed base graph leads to the use of less computer time and memory. In this model, each string as a trial solution is represented by a string of binary numbers indicating the existence of link(s) directed to corresponding nodes.

In Chapter 8, the model outlined in Chapter 7 is extended to a full optimisation method considering the layout and also component sizes of a branched piping system. A model, "OPDESGA" is developed for the optimal layout and design of a multiple subunit pressure irrigation system. Genetic algorithms (GAs) are utilised as the main optimisation technique for the layout and sizing of the main and submain pipes. However, the optimum design of subunits on the basis of a maximum pressure variation of 20% in the manifold and laterals was carried out using an enumeration approach. The value of the demands at nodes representing the irrigation requirements of subunits is calculated from the soil and crop characteristics using equations suggested by Karmeli et al (1985). In the GA process the length of strings each representing a trial solution was made up of binary and integer numbers for the layout and pipe sizes respectively. This model minimises the sum of costs including: micropipes within the subunits, main and submain pipes, pumping system and the present value of operating cost.

The optimisation models discussed in Chapters 7 and 8 deal with piping systems in which demand nodes are located in a regular pattern. However, from practical point of view, it is more desirable to generalise the model to irregular networks. Consequently, the model discussed in Chapter 8 is modified and extended to a general model which is outlined in Chapter 9.

Chapter 9 concerns a general optimisation model which investigates an optimum layout as well as component sizes of an irregular pipe network. The formulation of this model is also based on genetic algorithms. In contrast to strings developed for the previous model, each string in this model consists of three segments with integer numbers. The first and the second segments represent the layout and corresponding pipe sizes, while the third segment with only one bit represents the selection of an appropriate pump sizes.

In this part of the work, a sensitively analysis is performed to evaluate the GA parameters used in Chapters 7, 8 and 9. The effect of various values of probability of crossover, mutation, different population sizes and seed numbers which create different sequences of random numbers are examined.

The summary and conclusions are presented in Chapter 10. These incorporate the effect of the various parameters involved in the optimisation models developed in the thesis. Finally, the possible future extensions and work that could be carried out to achieve a comprehensive optimisation model for pressure irrigation systems are discussed.

Chapter 2

Review of Water Distribution Pipe Networks with Emphasis on Pressure Irrigation Systems

2.1 INTRODUCTION

Irrigation systems can be broadly classified as being either gravity flow or pressurised. Gravity flow systems are characterised by water flow in channels across the field. A channel may be a furrow between crop rows, a strip of land bordered by low dykes, or an entire field. The amount and uniformity of water infiltration for gravity flow systems are largely functions of the soil characteristics. Pressurised systems deliver water under pressure through pipes and release it from sprinkler nozzles or small orifices or tubes. In principle, pressurised systems have the advantage of greater control on the application amount and location of water, and therefore allow, the potential for greater uniformity of water application compared with gravity systems.

Modern irrigation equipment and technology has the capability of applying water both accurately and uniformly. Therefore, there has been a gradual shift from highly labour-intensive irrigation systems to those which require additional energy and capital cost but less labour. The efficient utilisation of energy in irrigation will become more important in the future. Three percent of the whole energy usage (in the world) is devoted to agriculture. Of this, 23% is used for pumping and delivering water to irrigation systems (Gilley, 1983).

Irrigation systems for agricultural crops should be properly designed, installed and managed to achieve high efficiencies. Irrigation scheduling, a key element of proper management is the accurate forecasting of water application (in timing and amount) for optimal crop production. An irrigation system can only be efficient when it is both scheduled properly and operated to apply the desired amount of water efficiently. These conditions are necessary to avoid poor management (Heermann et al, 1990).

Pressurised irrigation systems provide better control on the amount of applied water and, in most cases, better irrigation uniformity than gravity flow systems (Letey et al, 1990). The drip (or trickle) irrigation system is the newest of all commercial methods of water application. It is described as the frequent, slow application of water to the soil through mechanical devices called emitters. In other words "trickle irrigation is the slow application of water on, above, or beneath the soil by surface trickle, sub-surface trickle, bubblers, mechanical-move, and pulse system" (Bucks and Davis, 1986). Water is applied as discrete or continuous drops, through emitters or applicators placed along a water delivery line near the plant. In most cases the emitters are placed on the ground, but they can also be buried. The emitted water moves within the soil system largely by unsaturated flow. Since the area wetted by each emitter is a function of the soil's hydraulic properties, one or more emission points per plant may be necessary. Trickle irrigation, like other irrigation methods will not suit every agricultural, or land situation. However, trickle irrigation does offer many unique agronomic, agro-technical, and economic advantages for present and future irrigation technologies.

Presently trickle irrigation has the greatest potential where water is expensive or scarce; soils are sandy, rocky, or difficult to level and high-value crops are produced. The main agricultural crops under trickle irrigation are avocados, citrus, stone fruits, grapes, strawberries, sugarcane and tomatoes. This method of irrigation continues to be important in the greenhouse production of tomatoes, cucumbers and flowers. Trickle irrigation is also used for landscaping of parks, highway verges, commercial developments and residences. As water, labour and land preparation costs increase, more trickle systems will be substituted for conventional irrigation methods (Bucks et al, 1982).

Nowadays, drip irrigation systems are used for various crops and fruits, in particular for areas with hilly topography, poor soils and water shortages (Holzapfel et al, 1990). Along with improvements in technology and increasing world population, irrigation networks have become more complex and more sensitive. Therefore, the optimum design of drip irrigation systems has become more important than before and recently much research has been carried out in this area.

2.1.1 Disciplinary Involvement

Trickle irrigation involves participation by agricultural and hydraulic engineers, as well as soil and plant scientists. The design engineers may look at the system in terms of the hydraulics, water distribution flow patterns, the soil scientists in terms of water and salt distribution, and the plant scientist in terms of water and nutrient use and crop behaviour. Special problems, such as emitter clogging require the involvement of chemists and microbiologists.

2.1.2 A Short Review of Drip Irrigation History

Historical and archaeological findings show that irrigation has played a major role in the development of ancient civilisations. The oldest civilisations with irrigation developed along the Nile, Tigris, Euphrates, Indus and Yellow rivers. For example, gravity irrigation began along the Nile about 6,000 B.C. The dominant methods of irrigation from these early times have been surface or gravity and sprinkler irrigation. Trickle irrigation is a considerably new approach compared to these methods and developed from subirrigation where irrigation water is applied by raising the water table (Bucks and Davis, 1986).

A long time ago, ***Coozehi*** irrigation was used in Persia (Iran) where the available water was scarce. This is a basic but very efficient irrigation method (similar to drip irrigation) in which a clay pot was installed just beside each tree within the root zone. Each pot was filled with water manually whenever water was used by plant. It appears that the idea of drip irrigation comes from such method.

As the literature show, drip irrigation was developed as a method of sub-surface irrigation applying water beneath the soil surface (Davis, 1974). In 1869 the first experiments began in Germany, when clay pipes were used in a combination of irrigation and drainage systems. The first reported work in the U.S.A. was carried out by House in Colorado in 1913 (Davis, 1974). He indicated that the concept was too expensive for practical use. An important breakthrough was made around 1920 in Germany when perforated pipe was introduced. Since then, various experiments have been carried out in relation to the development of drip systems, usually perforated pipe made from various materials. With the development of plastics during and after World War II, the idea of using plastic pipe for irrigation became feasible.

In 1962, K. Dorter and others in Germany began extensive work on sub-surface (underground) irrigation. Over 100 publications were listed on the concept of underground irrigation before 1962. The idea of using the soil as a storage reservoir was discarded or minimised and replaced with the concept of irrigation keeping up with evapotranspiration on a daily basis. The availability of low-cost plastic pipes for water delivery lines helped to speed the use of trickle irrigation systems.

Publications on the modern-day surface trickle system began in Israel in 1963 and the U.S.A. in 1964. From Israel the drip irrigation concept spread to Australia, North America, and South Africa by the 1960's, and finally throughout the world. Now many thousands of hectares are drip irrigated in different states of the United States of America, Australia, New Zealand, Israel, South Africa, Canada, Germany, and other countries (Bucks and Davis, 1986).

2.1.3 Expansion in Land Area

In 1977, the Food and Agriculture Organisation of the United Nations (FAO) estimated that the total global irrigated area was 223 million ha (1977) and that this would increase to about 273 million ha by 1990. The 1978 census of agriculture indicated that there were 20.3 million ha of irrigated land in the United States. Of these about 12.6 million ha were irrigated by gravity irrigation, 7.4 million ha by sprinkler irrigation, and 0.2 million ha (1.1%) by trickle or subirrigation (Bucks and Davis, 1986). A recent survey conducted by the International Commission on Irrigation and Drainage (ICID) indicated that about 417,000 ha were under trickle irrigation throughout the world. The major use of trickle irrigation was in the U.S.A. where the area has expanded from approximately 4,000 ha in 1972, to 185,300 ha in 1982. Table 2.1 shows the area under trickle irrigation and major crops for the countries with over 1000ha. Figure 2.1 shows the relative area of trickle irrigation comparing with furrow and overhead irrigation. Figure 2.2 shows the growth in the area under trickle irrigation in the U.S.A.

TABLE 2.1 Trickle irrigation land area and principal crops throughout the world in 1982 (Nakayama and Bucks, 1986)

Country	Land area (ha)	Principal crops
United States (U.S.A),	85,300	orchard, vine, vegetable, sugarcane
Israel,	81,700	citrus, cotton, fruit
South Africa,	40,000	vine, orchard
France,	22,000	orchard, vine, glasshouse, vegetable
Australia,	20,050	orchard, vine, vegetable,....
Soviet Union	11,200	orchard, glasshouse, vine, tea
Italy,	10,300	orchard, vegetable, glasshouse,
China,	8,040	orchard
Cyprus,	6,600	orchard, glasshouse, vine
Mexico,	5,500	
Canada,	4,985	orchard, berry, glasshouse
Morocco,	3,600	orchard
United Kingdom,	3,150	orchard, glasshouse
Hungary,	2,500	orchard, vine
Brazil,	2,000	orchard, vine, nursery
Jordan,	1,020	glasshouse, fruit
New Zealand,	1,000	
others,	3,715	orchard, vine, vegetable, nursery
Total	**416,660 ha**	

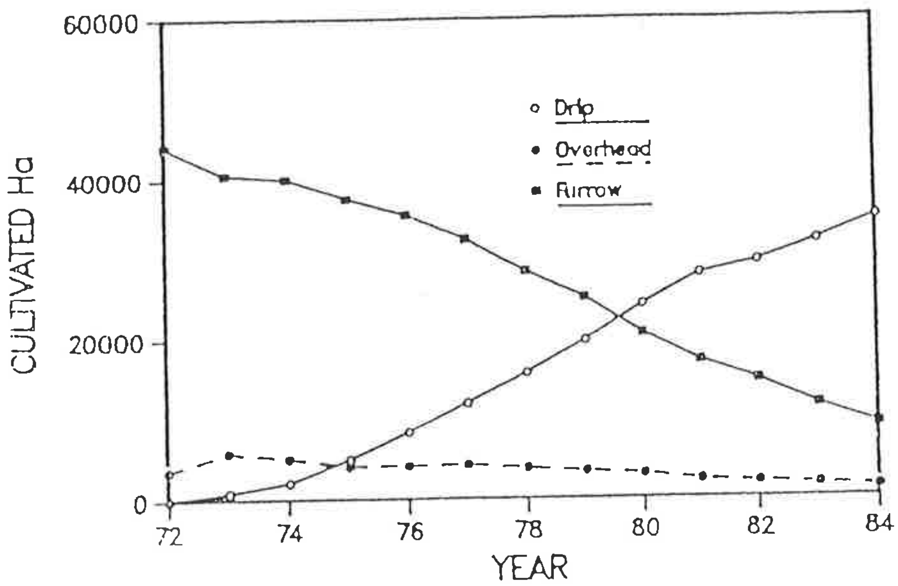


Fig. 2.1 Irrigation area under different methods of irrigation 1972-1982, in Hawaii (International Drip Irrigation Congress, 1985)

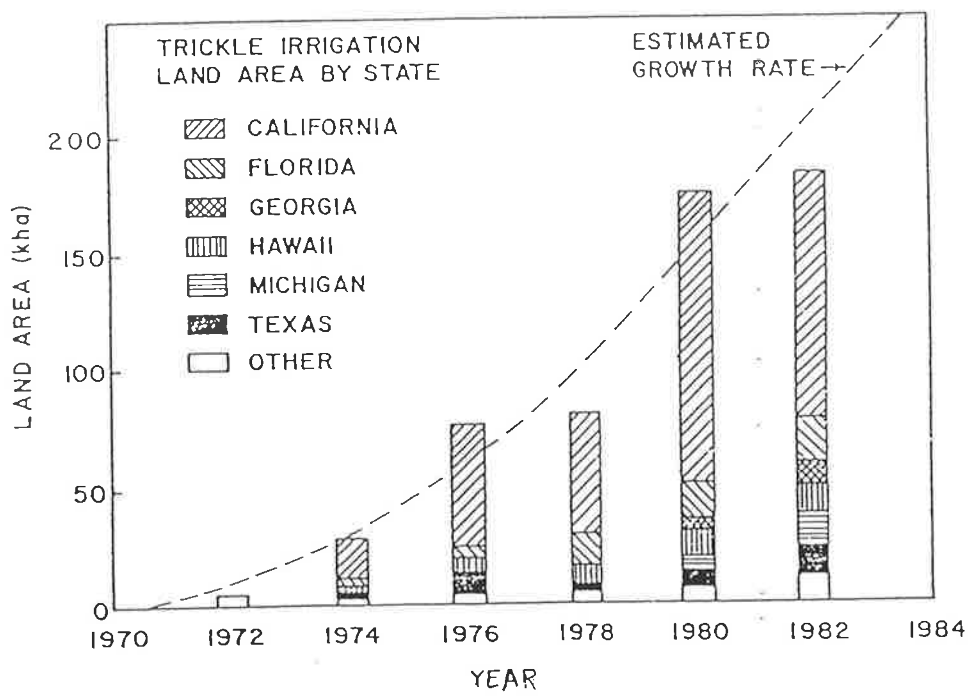


Fig. 2.2 Trickle irrigation land area distribution in the United States from 1970 to 1982 (Nakayama and Bucks, 1986)

A great deal of research has been carried out into the development of improved irrigation technologies. In the future, resources must be directed towards optimising the use of natural precipitation and reducing the demand for water where irrigation is practised so that supplies are less likely to fail, enabling greater areas of crops to be grown (Wiener, 1972).

2.1.4 Advantages of Drip Irrigation Systems

Trickle irrigation can reduce water loss and operational costs for a number of reasons. Ideally, the actual amount of water applied to the plants just equals their requirement. Labour costs are reduced due to the use of automatic equipment and reduction in weed growth. Fertilisers and pesticides can be injected into the irrigation water to reduce labour costs. All irrigation water contains some dissolved salts, which are usually pushed toward the fringes of the wetted soil during the irrigation season. By applying more water than the plants consume, most of the salts can be pushed or leached below the root zone, but it is impossible to avoid having some areas of salt accumulation. Frequent irrigation maintains a stable soil moisture condition that keeps the salts in the soil water more dilute, thus it is possible to irrigate with water of high salinity. Goldberg et al (1976) gave a full description of several experiments which showed that when using saline water, trickle irrigation produced better yield than spray or furrow irrigation. Probably the main advantage of drip irrigation was that, by applying the water frequently, soil suction was kept low, so that the reduction in osmotic potential due to the salt had little overall effect on the availability of water to the plants. Trickle leaches salts from previous irrigation away from the roots to the edge of the wetted zone, and also avoids putting saline water on the leaves of the plant, so eliminating uptake of chloride by the leaves. As noted by Turner (1984) trickle irrigation will allow the use of water with salinity up to about 11250 mg/L.

A drip system enables the soil moisture tension to be kept low. This overcomes one of the problems of using saline water in conventional methods which is applied with low frequency. Tickle irrigation systems can be designed to operate efficiently on almost any topography. Energy costs for pumping may be reduced with trickle irrigation, since the operating pressure is lower than with other types of pressurised systems. The common operating pressure is about 10 m (98 Kpa) and many commercial outlets have designed discharge at this pressure. This level of pressure is low enough to allow the use of cheap, low density pipes, simple pressure fit connections, reasonable outlet orifices, and a low operating cost (Turner, 1984). Cultivator operation such as spraying, weeding, thinning and harvesting are possible on a continuous basis without interrupting the normal irrigation cycle. The water saving can be achieved by irrigating smaller portion of the soil volume, which decreases surface evapotranspiration, reduces irrigation run off from the field. Direct evapotranspiration from the soil surface and water uptake by weeds are reduced by not wetting the entire soil surface between rows or trees. Trickle irrigation can prevent run off even for contour farming practices on steep hills. The development of surface crust and disturbing of surface soil structure can be avoided, whereas water infiltration into the soil can be improved by using a low-application rate trickle system. Deep Percolation losses can be controlled especially on sandy soils (Bucks et al, 1982).

2.1.5 Disadvantages of Drip Irrigation Systems

Trickle irrigation systems are expensive due to the high initial investment cost. Clogging due to mineral or organic materials in the water may cause problems which can reduce the uniformity of water distribution. In steep slopes, emitter discharge during irrigation may differ from the design discharge and water may drain through lower emitters after the water is shut off. The accumulation of salt at the soil surface is another disadvantage of this method. Since roots extract nutrients and water only from a relatively small volume of wetted soil, uncontrolled events which interrupt irrigation may cause crop damage. Normally, regular maintenance is required in trickle irrigation which increases maintenance costs.

2.2 ANALYSIS OF WATER PIPE NETWORKS

A water distribution system usually includes: pipes, valves, hydrants, pumping station(s), reservoir(s) or tank(s) and all other parts of the conveyancing system after the water leaves the main pumping station or the main distribution reservoir. In general, all water distribution systems which are designed for domestic, industrial or irrigation use are either branched (tree, open) or closed (loop) networks. Open or tree networks are a type of distribution system which do not contain any loops. There is only one flow path from a source to any particular point of supply. Closed networks contain loops, in which there may be more than one path from a source to any particular supply point. In most irrigation systems branched networks are used. The basic hydraulic equations that link the flows to the pressure heads are the head loss and continuity equations. The relationship between head loss and flow rate in pipes is non linear, consequently, in the analysis of a water distribution, a system of non linear equations is to be solved. There are two sets of governing equations for flow and pressure in a network of pipes: continuity at each node and conservation of energy for each loop in the network. Most solution techniques to these sets of equations begin with assumed flows that satisfy the continuity equations (Wood and Charles, 1972) as discussed below:

2.2.1 Continuity Equations

Conservation of mass at nodes or junctions in a network yields a set of linear algebraic equations in terms of discharges. Flow continuity at each junction can be expressed as:

$$\text{Flow in} = \text{Flow out} + \text{Demand at node} \tag{2.1}$$

A general mathematical expression for the continuity of flow at node i (flow away from i is positive) with an offtake or demand D_i is given as:

$$\sum_{j=1}^{NPJ} Q_j + D_i = 0 \qquad \text{for all nodes } i \tag{2.2}$$

where Q_j is the flow in each of the pipes attached to node i (flow away from i is positive); NPJ is the number of pipes attached to node i, and D_i is the demand at node i. For a network with NJ nodes where all external flows or demands are known there are $NJ-1$ independent continuity equations.

2.2.2 Energy Equations

Energy Equations around loops in a network are non-linear. The head loss in a pipe in the network can be computed from a number of empirically obtained equations. Two commonly used equations are the Darcy-Wiesbach head loss equation and the Hazen-Williams equation. The general form of the head loss equation of pipe j between nodes i and k is given as:

$$H_i - H_k = hf_j = r_j Q_j |Q_j|^{n-1} \tag{2.3}$$

In Equation 2.3, H_i, $\left(= Z_i + \frac{P_i}{\gamma}\right)$ is the total head (elevation plus pressure head) at node i ; H_k is the total head at node k, $\left(= Z_k + \frac{P_k}{\gamma}\right)$; hf_j is the head loss in pipe j; r_j is the resistance for pipe j which depends on the form of the head loss equation, Q_j is the flow rate, and n is an exponent dependent on the form of the head loss equation; $n = 2$ for the Darcy-Wiesbach equation and 1.852 for the Hazen-Williams equation. Z_i is elevation of the centre of the pipe at node i with respect to a datum, P_i is pressure in the pipe at node i , γ is specific weight of water (typically 9800 N/m^3 for water at 20 C$^\circ$).

The sum of the head losses around each loop must be zero ($\sum hf_j = 0$).

2.3 DRIP IRRIGATION DESIGN

A drip irrigation system is a type of pipe system, including main and sub-main lines, laterals, and a number of emitters which is designed to deliver water directly to the root system of plants. Water is applied as discrete or continuous drops with a low pressure delivery system (Bucks and Davis, 1986). It is usually operated at a pressure less than 15 psi (100 kPa) (Wu et al, 1973). The best irrigation design is one in which all outlets (emitters) deliver the same flow rate. This is necessary to ensure a uniform growth rate of the crop over the whole field. From a practical point of view, it is impossible to have such an idealised case, because the emitter's flow is affected by variation in water pressure and manufacturing characteristics. The water pressure variation can be controlled by hydraulic design, and the manufacturing characteristics, can be corrected by improved quality control in manufacturing.

However, in general practice, the emitter characteristics are usually kept fixed, and discharge uniformity is achieved by controlling the pressure variations (Wu et al, 1986). Wu et al (1986) show that because of emitter discharges from laterals and outflows from submains and mainlines, the energy gradient line in all pipes will not be a straight line but an exponential type curve. The shape of the energy gradient line for level irrigation lines is shown in Figure 2.3.

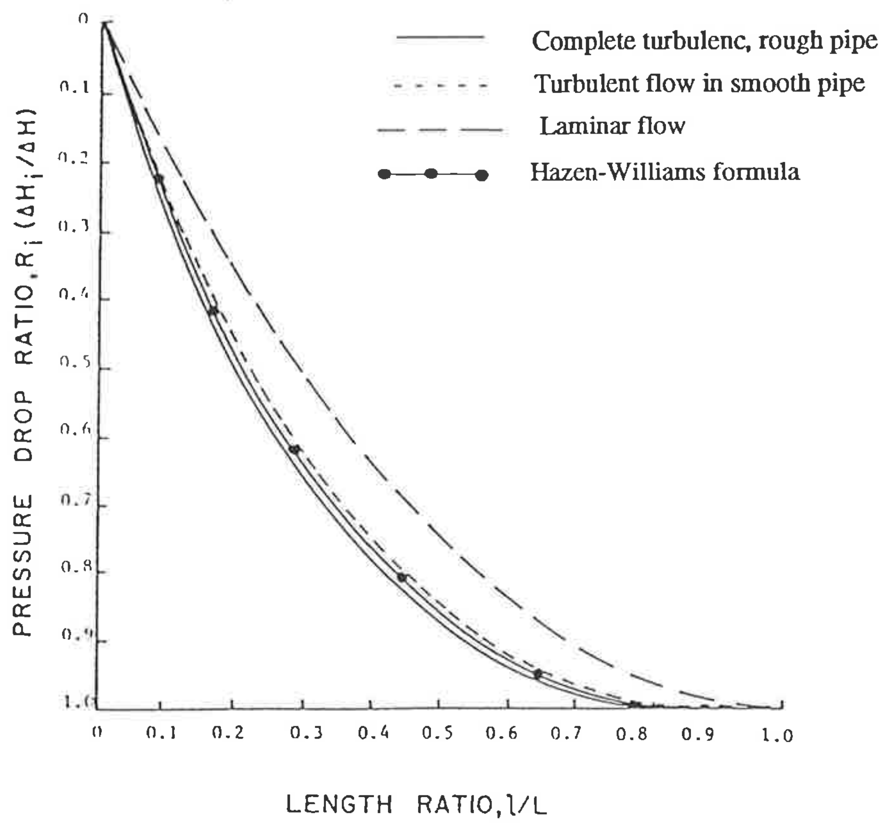


Fig. 2.3 Dimensionless curves showing the friction head drop caused by laminar and turbulent flow in the lateral lines (Nakayama and Bucks, 1986)

The pressure drop ratio may be formulated as (Nakayama and Bucks, 1986):

$$R_i = 1 - (1 - i)^{n+1} \tag{2.4}$$

or

$$R_i = \frac{\Delta H_i}{\Delta H} \tag{2.5}$$

where R_i = pressure drop ratio at i ; n = exponent of the flow rate in the friction equation; ΔH_i = pressure drop (head loss) expressed in metres at the length ratio $i = \left(\frac{l}{L}\right)$; ΔH =

total pressure drop (total head loss) at the end of the line (m); L= total length of the line (m); l = length, measured from the head of the line (m).

When the Hazen-Williams formula is used for the pipe flow, the dimensionless energy gradient line can be expressed as:

$$R_i = 1-(1-i)^{2.852} \tag{2.6}$$

Wu et al (1986) explain that the design of the main line is based on output energy (from a reservoir or a pumping station), slope, required operating pressure for irrigation, and the energy gradient which will give a total energy higher than that required at any submain for irrigation. The design parameters of the main line are: allowable energy drop for each main line section; and the main line size selected from the allowable friction drop. The design procedure can be very simple if the main line supplies water to only a single field (one submain).

This main line design can be carried out by considering a pipe flow condition in which the pipe size can be determined by the allowable energy drop, ΔH, total required discharge, Q, and the main line length, L. When a main line system is supplying water to a series of fields, the main line flow capacity changes with respect to length.

There will be different discharges in different main line sections. This design requires the estimation of the energy gradient curve so that the energy drop for each section can be determined. The main line design is a series of pipe flow designs. Once the field layout is set, the required discharge rate in each section can be determined. It is common practice for the Hazen-Williams formula to be used to determine pipe sizes. The energy slope, or the slope of the energy gradient line, should be selected so that the energy gradient line is above the required water pressure along the line as shown in Figure 2.4. As long as the total energy is greater than the required operating pressure, the design is hydraulically sound. If an available inlet pressure at point A is determined, and point B indicates the pressure required at the downstream end, a straight line and curves will connect A to B. The straight energy gradient AB is one solution, and all the curves connecting A and B are the other possible solutions. Each solution will result in a different main line design.

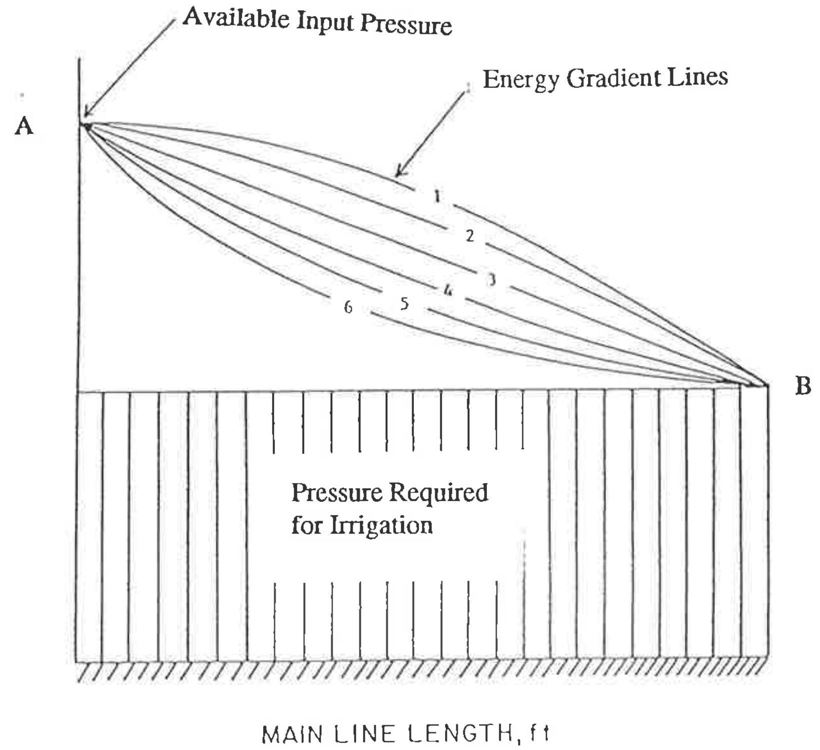


Fig. 2.4 Main line profile and energy gradient lines (Nakayams and Bucks, 1986)

Myers and Bucks (1972) proposed a graphical solution by using a multiple emitter size system to obtain uniform emitter discharge. They argued that a good emitter discharge uniformity can be obtained in a low-pressure trickle system by using simple emitters of different diameters to compensate for pressure changes along the lateral pipes.

Wu and Gitlin (1973) proposed a simple way of estimating the pressure distribution along a drip line and presented possible ways in which emitters can be arranged or adjusted for uniform discharge. They show that the pressure gradient line is not a straight line, since the upstream sections will have more friction drop due to the large amount of discharge. The pressure gradient line is a curve of an exponential type. They claim the calculation of pressure gradient can be simplified if the average discharge is used. They developed a computer program on the basis of the assumption of an equal discharge from each outlet, dividing the pipeline into many sections and using the average discharge. They outlined that if only two segments are used, the errors will be reduced to about 5%, and if three or four segments are used, the error will be reduced to about 2% or 1%. They concluded, dividing the total length of the line into three or four segments will yield a good estimate of the energy gradient line.

Uniformity and efficiency of water application are two major factors for any irrigation systems. Since emission uniformity in drip irrigation is very important, considerable work has been carried out to reach a desired uniformity. Keller and Karmeli (1974) suggest two parameters to define the uniformity of a drip irrigation systems. The first one is the emission uniformity (EU) which involves the minimum and average emitter discharge rate. This parameter is defined as $EU = 100\frac{q_n}{q_a}$ in which q_n is the average of the lowest quarter (1/4) of the emitter flow rates (litres per hour) and q_a is the average of all the emitter flow rates (litres per hour). They use EU in the design procedure for computing the gross irrigation depth, irrigation interval and required system capacity. They recommend that the desirable emission uniformity (EU) should be 94% or more and should never be less than 90%. Increasing the number of emitters per plant should improve EU considerably.

The second uniformity parameter suggested is the absolute emission uniformity (EU_a) which involves the relationship between the maximum and minimum emitter discharge rates to the average emitter flow rate. EU_a is defined as:

$$EU_a = 100\frac{1}{2}\left(\frac{q_n}{q_a} + \frac{q_a}{q_x}\right) \qquad (2.7)$$

in which q_x is the average of the highest eighth (1/8) of the emitter flow rates (litres per hour).

Yitayew and Warrick (1988) believe that emission uniformity or absolute emission uniformity represented by Keller and Karmeli has been useful to express uniformity of emitter discharge throughout a system. However, they used the Christiansen uniformity coefficient (UC) for evaluating the uniformity of discharge. This parameter (UC) proposed by Christiansen (1942) for evaluating the uniformity of sprinkler irrigation is based on sum of the absolute deviations of each observed emitter discharge from the mean discharge (Yitayew and Warrick, 1988).

Howell and Hiller (1974) developed a set of equations and graphs which enable the length of the laterals to be determined with respect to desirable discharge uniformity. In addition, the emitter spacing can be found using their proposed equations.

Solomon and Keller (1978) derived an expression for linear head loss for laterals, tapered manifolds (changing to a smaller size as flow decreases), and pressure distributions within a system of subunits. These could be used for evaluating the effect of emitter characteristics, variability in manufacturing, and, aging of emitters and frictional head losses. They provide histograms showing the pressure distribution and subunit flow variation in a typical trickle irrigation system. They concluded that manufacturing variation has considerable effect on the head loss and emission uniformity can be improved by 2.0% by doubling the number of emitters. Finally, they argue that Keller and Karmelis' work on the emission uniformity (EU) and the absolute emission uniformity (EU_a) are also applicable to sloping terrain.

Wu and Gitlin (1977a) developed a general design chart and a set of simplified submain design charts for designing the different submain sizes. They also present a design procedure for irregular shaped fields by introducing adjusted discharge and shape coefficients. Work on design criteria for multiple outlet irrigation lateral pipes were extended by Peroid (1977). He also recommended the use of the average absolute deviation from the mean outflow $|\delta|$, instead of the Christiansen coefficient (UC) for flow variation. Wu and Gitlin show that the energy gradient line of a submain or lateral with varying pipe sizes in drip irrigation is close to the slope of the submain or lateral if the slope of each section is balanced by the friction loss of that section. They develop a general formula to determine the energy loss ratio for different length ratios of a lateral or submain with varying pipe size. They plot this for a lateral with single, two and four equal sections with different pipe sizes. They conclude if the number of equal sections with different pipe sizes is increased the maximum pressure variation will be reduced, and by using several sections the energy gradient lines can be brought close to a straight line. For example, if two different pipe sizes are used, the maximum pressure variation will be reduced to 0.18 ΔH which occurs around the middle of the second section. For four equal sections with four different pipe sizes, the maximum pressure variation will be reduced to 0.09 ΔH, which occurs at the middle of the last section. While the maximum pressure variation for a pipe with one size when $\Delta H = \Delta H'$, is equal to 0.36 ΔH (see; Figure 2.5). They argued that this approach can also be used for non uniform slopes, However, all slopes have to be down slopes ($\Delta H'$ is the total energy gain by slope).

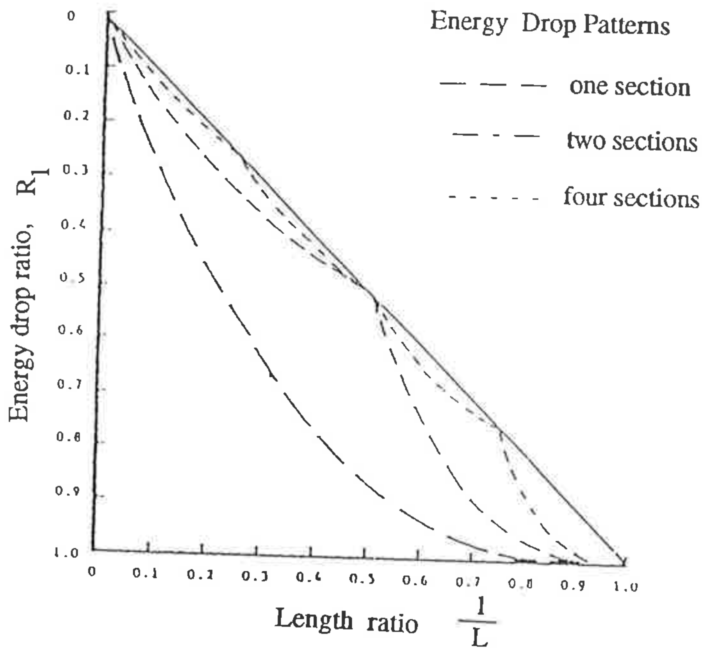


Fig. 2.5 Dimensionless energy gradient lines for the irrigation lines with varying sizes (Wu and Gitlin, 1977a)

Wu and Gitlin (1977b) modified their previous work on pressure variation so that it can be used for lateral line design on different non uniform slopes. They also presented three methods for lateral line design on non uniform slopes which include developing a dimensionless non uniform slope design chart, a modified polyplot (graphical method for the hydraulic design of multi-emitter irrigation systems) and isograph, and a simple down slope design with variable pipe size.

Wu and Gitlin (1980) classified the pressure profiles along a lateral line on uniform slope into five types based on the dimensionless ratio $\frac{\Delta H'}{\Delta H}$. They showed that among the five pressure profiles, the one which produces the minimum difference from the optimum desired profile, occurs when $\frac{\Delta H'}{\Delta H}=1$. They argued that the optimal pressure profile cannot be achieved for a single inlet system, when the lateral line slope is zero or up slope. Wu and Gitlin proposed that the double-inlet system and an inflow-outflow system can produce the best shape of the pressure profile for achieving discharge uniformity.

Wu and Gitlin (1982) present a set of mathematical derivations and design charts for the double-inlet and the inflow-outflow systems. They concluded that much better water pressure uniformity along the lateral lines can be obtained by the double-inlet and the inflow-outflow systems, rather than by the single inlet systems.

The pressure difference of a double-inlet or inflow-outflow lateral line system is about one-third to one-fifth of the pressure difference caused by the single inlet system. The major design criteria for a drip irrigation system is the minimisation of the emitter flow variation along either laterals or submain lines (Gillespile et al, 1979).

Gillespile et al (1979) analysed and presented five different types of pressure profiles along lateral or submain lines. Their study was only for uniform slopes. They also derived the mathematical expression for laterals or submain lines which simplify the design technique for finding the lengths of the lines by a specific pressure variation along the lines for a desired flow variation.

2.4 OPTIMISATION

Optimisation is a mathematical procedure for finding the best decisions. Although the economical design of hydraulic networks has long been an area of interest for researchers in hydraulics, the subject has received particular emphasis since the 1960's because of the access to digital computers (Perez et al, 1993; Goulter, 1990). Millions of dollars are spent each year on water distribution infrastructure. As Dandy et al (1993) explain the use of optimisation techniques provides an opportunity for potential saving in costs for water supply authorities. Over the last two decades a number of different techniques such as linear programming (Alperovits and Shamir, 1977, Quindry et al, 1981); non-linear programming (El-Bahrawy and Smith, 1985); dynamic programming (Perez et al, 1993;

Walters and Lohbeck, 1993); partial enumeration approach (Gessler, 1982; Loubser and Gessler, 1990; Hassanli and Dandy, 1995a); and recently genetic algorithms search (Murphy and Simpson, 1992, Dandy et al, 1993; Simpson et al, 1994, Hassanli and Dandy, 1995b, 1996) have been applied to the optimisation of water distribution networks.

Most irrigation systems are branched networks. As linear programming is an appropriate technique for this class of systems, in this part of the review, papers which have dealt with linear programming will be considered. Genetic algorithms provide a new and effective set of techniques which is employed in this area of research. It will be discussed in detail in Chapter 6. The literature concerning the optimal layout and the full design of pipe networks using genetic algorithms are reviewed in that chapter.

2.4.1 Optimisation of Water Distribution Networks Using Linear and Non-Linear Programming

Linear programming is a powerful optimisation technique, but it is restricted to problems where the relationship between variables is linear (Stephenson, 1984). However, linear programming can be used for optimising the design of looped networks by using successive approximations. The LP approach assumes that pipes are available in discrete sizes, and that the cost per unit length depends only on the pipe diameter. These are good assumptions for most pressurised pipe networks (Godfrey et al, 1993).

Karmeli et al (1968) present a method for the design of water distribution networks which is only applicable to branched networks. The model is suitable either for the case in which the water pressure at the source is to be selected or for the case where the pressure is given. Since the flow in each branch is known, the effect of changing the pipe diameter on head loss can be computed directly. The authors selected the length of the pipe segments with a given diameter as the decision variables. Since the head loss and cost are linear functions of the pipe length, the optimum design can be found using linear programming. For computational feasibility, they used a set of admissible diameters for each section. The initial cost including pump, pipes and the annual operating cost were considered in the objective function for this work.

Schaake and Lai (1969) developed a linear programming formulation for use in planning a major addition to the New York water supply system. Before the linear programming model is formulated, they specify a certain head at every node in the system. In order to formulate a linear program, they define a new variable X_{ij} as $X_{ij} = d_{ij}^{2.63}$ then substitute this variable in the Hazen-Williams equation:

$$q_{ij} = \alpha C_{ij}^{0.54} l_{ij}^{-0.54} h_{ij}^{0.54} d_{ij}^{2.63} \tag{2.8}$$

to give:

$$q_{ij} = \alpha C_{ij}^{0.54} l_{ij}^{-0.54} h_{ij}^{0.54} X_{ij} \tag{2.9}$$

where α= A constant depending on the units; l_{ij}= the length of pipe between nodes i and j; C_{ij} = The Hazen-Williams coefficient; h_{ij} =The head loss from node i to node j; d_{ij}= The pipe diameter between nodes i and j; q_{ij} = The discharge through the pipe from nodes i to j.

When h_{ij} is positive, flow occurs from i to j and q_{ij} is also positive. In their approach the discharge from each node j must equal the algebraic sum of the flow in all pipes connected to node j.

They formulated a linear program by assuming a certain value for the head at each node, and defined β_{ij} as the cost per unit length per unit of X_{ij} for pipe ij.

If Z is the objective function then:

$$Z = \sum_{i=1} \beta_{ij} \ell_{ij} X_{ij} \tag{2.10}$$

In their method, any objective function which can be expressed as a linear function of the X_{ij} variable can be used (or linearisation of the objective function may be required). Using the node and head equations and linear programming, the least cost pipe sizes can be obtained. In this approach the need to specify the head at each node in advance is a disadvantage. If pipe length and Hazen-Williams coefficients are known an assumed value of the head at each node will result in a linear relationship between q_{ij} and the X_{ij} variables, (Equation 2.9). These equations can then be used as the constraint set in a linear program.

Karmeli et al, 1968; Gupta, 1969; and Gupta et al, 1972; dealt with the optimal design of branching networks. When demand is known, the flow in each link of the branching networks is known. Since the pipe cost and head loss are both linear functions of pipe length, if one selects pipe lengths as decision variables, the optimisation can be evaluated as a linear program. Thus the lengths of the segments of pipe in each link were considered as decision variables and a set of diameters for each link was selected in advance. These works considered only the initial cost in the objective function but it is not difficult to include operating cost (Shamir, 1974).

Kally (1972) extends the above method to looped networks using the same decision variables (length of known pipe diameter in the link) and the same objective function. He used an iteration process by a method of approximation.

Alperovits and Shamir (1977) present a method for designing branching networks using the linear programming gradient (LPG) method. In this technique, pipe flows are assumed before the linear program can be formulated. Furthermore, it is assumed that the layout of the network is given. The head at each node is between a given maximum and minimum value and the decision variables are the lengths of the segments of pipe of known diameter. In their formulation the objective function includes only the cost of the pipe lines. In addition, Alperovits and Shamir developed the basic LPG method for a looped pipeline network operating under gravity for a single loading condition. It was extended to cover multiple loadings and pumps, valves and reservoirs.

Behave (1979) developed a method based on the critical path concept to select the optimal sets of pipe sizes for optimisation of branched networks using linear programming. He obtained two optimal sets of pipe sizes which have friction slopes which are immediately lower and higher than the values obtained by the critical path concept. He claimed that these optimal sets reduce the size of the L.P. model and finally give a solution which can be considered as a global optimum for all practical purposes. However, it is necessary to select four or even more pipe sizes for the optimal sets when the interval in consecutive pipe size is small or when one wants to make sure that a global optimum solution is obtained. This is a disadvantage of Behave's method.

Quindry et al (1981) add a gradient step to the Schaake and Lai formulation in order to simplify it from a computational point of view. Quindry et al claim that the linear programming formulation which was used by Schaake and Lai was limited by the fact that the head at each node must be specified in advance. Their gradient search technique can overcome this limitation and a local optimal solution with respect to nodal heads can be obtained. Quindry et al, introduced a dual variable, $\lambda_j = \frac{d(\cos t)}{dQ}$. It is possible to calculate the gradient terms $\frac{\partial(\cos t)}{\partial H_j}$ for each node in the network by using the values of nodal heads, dual variables and pipe flows. If the sign and magnitude of each gradient term is known, changes can be made in the nodal heads. If the process is repeated, then a local optimum with respect to node heads will be approached.

Stephenson (1984) developed a computer code for optimising a simple closed network using linear programming. He reduced the network to a branched system and then selected the optimum pipe diameters. He used the simplex method to find the trunk main diameters. Taejin (1993) presents an optimisation model to minimise the total cost of a pipe network which includes the cost of pipes, pumps, storage tanks and pump operation under multiple loading constraints. He applies his model to a paddy irrigation system and New York city, water supply system. In this work the layout of system is assumed to be fixed.

2.4.2 Optimisation of Drip Irrigation Systems

The first drip irrigation systems were simple and were mainly composed of laterals and mechanical operating valves (Oron, 1982). These systems were originally designed for highly productive crops, where water was expensive or scarce. Nowadays, drip irrigation systems are used for various crops and fruits, in particular for areas with hilly topography, poor soils and water shortages (Holzapfel et al, 1990).

Bagley and Linsley (1961) developed a set of graphs for determining the economical size of commonly used aluminium and steel pipes for water conveyance. In their analysis, a set of mathematical expressions were developed for fixed and variable costs, and the total annual cost was differentiated with respect to pipe diameter. In their nomograph the use of ratio of cost to diameter based on current price information allows greater flexibility in the use of the chart for a wide variety of pipe types. According to Bagley and Linsely some variables such as costs of trenching, labour, welding, engineering pumping plant, etc. will have some effect on the selection of economic pipe size, but in normal cases, their influence on pipe sizes is not much considerable. Furthermore, costs of some of the above items are essentially independent of the pipe line so that they can be safely ignored.

Wu et al (1986) showed that there is an optimum shape of the energy gradient line which will produce the minimum cost for trickle irrigation lines. They explain that the energy gradient line can be a straight line or any one of a set of curves, between two points of pipe line. The optimum shape of the energy gradient line is a curve just slightly below the straight line as shown in Figure 2.6. Among fifteen energy gradient curves which are plotted dimensionlessly: No. 8 is a straight line, and No. 11 is the optimal energy gradient. However, examination of the optimal energy gradient line indicates that the difference in cost between the optimal shape and a straight line is only 2%. This provides a very fast and convenient method of design. When the main line or submain line profile, discharge, inlet pressure and required operating pressure are known, the straight energy gradient line can be used to design the main lines.

Oron and Walker (1981) developed an optimisation model for sprinkler irrigation systems. In this model, the objective function minimises the sum of the initial investment costs and the annual operating cost. The constraints include: water distribution uniformity; efficiency of control units; management considerations; number of outlets operating simultaneously in terms of available discharge; and the application rate relative to the infiltration rate for avoiding run off. Their optimisation procedure involves a non linear mixed integer programming approach. They examined a number of fields in terms of their geometrise to evaluate the effect of size and geometry on the system cost. They concluded that the cost per unit area increases as the total area increases, and the minimum cost is obtained when the field geometry is close to square or the ratio of width to length is close to 1 (between 0.4 and 1.25). In their model, as the available pressure head decreases the total number of subunits under operation increases. A decrease in the operating pressure and outlet discharge brings about a significant reduction in the cost. As a result, trickle irrigation may be more an economical system compared with the other pressure irrigation systems. Although this model has been developed for sprinkler irrigation with some additional constraints it can be extended to drip irrigation systems.

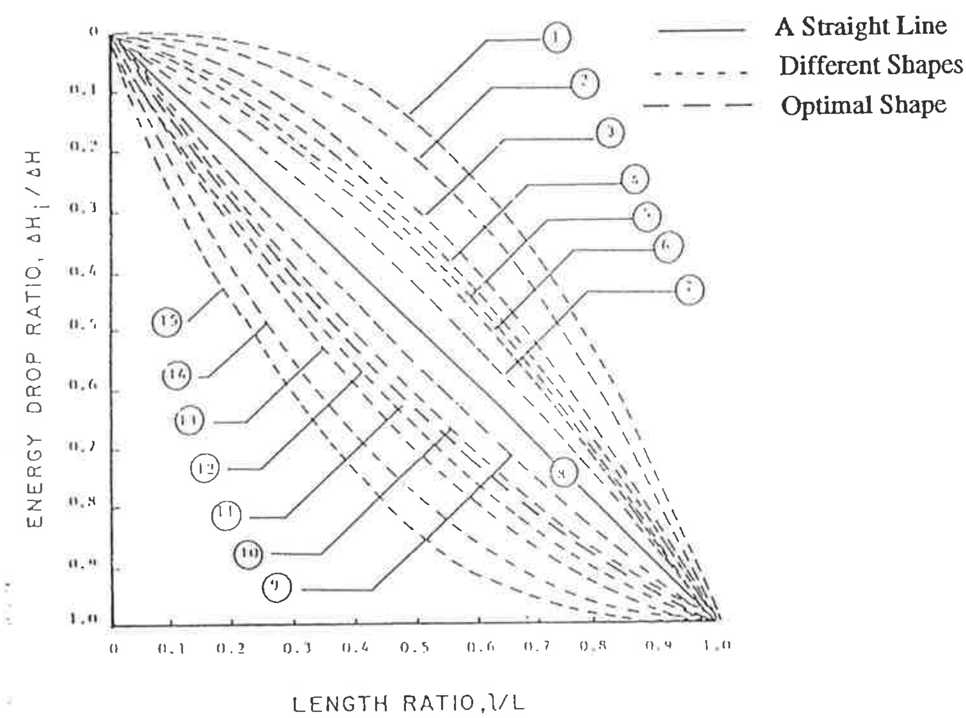


Fig. 2.6 Dimensionless energy gradient lines for the main pipes in a drip irrigation system (Nakayama and Bucks, 1986)

Oron (1982) indicated that, in terms of engineering and economic considerations, agricultural irrigation fields should be divided into subunits. He showed that each subunit is characterised by the length of laterals and axillary pipes as well as the number of outlets which are located on them. Oron also presented some theoretical possibilities for different subunits and concluded that the main differences are in the length of auxiliary and supply pipes which influences the selection of the appropriate diameters. Oron pointed out that in multiple subunit systems one can irrigate part of the field at a time, achieve a more uniform emitter discharge, select smaller pipe sizes, and increase flexibility in the irrigation practice.

Hassanli and Dandy (1993) examined the influence of various field dimension ratios of a constant field area on the system cost. They concluded that the optimum length/width ratio lies between 1.04 and 1.5. They also examined the influence of various irrigation intervals and irrigation times for various combinations of field dimensions on the system cost.

Pleban et al (1984) presented a design procedure for minimising the capital cost of pipe lines with multiple-outlets and different diameters throughout the length. They used the Lagrange Multiplier method to select the combination of pipe lengths of various diameters in order to reach the minimum cost. Much research has been carried out on pipe diameter segment combinations by using trial and error methods to check the outlet discharge uniformity, rather than minimising the total cost of pipe lines. In contrast, Pleban et al's study concentrates on the optimum cost. Their design procedure is applicable to both dripper and sprinkler lateral and feeder pipe lines. However, it is restricted to some assumptions such as equal outlet spacing, and equal outlet flow rate over a given range of pressure and uniform slopes.

Labye et al (1988) suggested three simple graphical methods for minimising the cost of pipe lines in branching networks. The first method uses the shortest path without intermediate junction, (proximity layout). In the second method (120° layout rule) a node is introduced somewhere between hydrants such that its distance from the hydrants is a minimum. In the third method the aim is to reduce the total cost of the network by shortening the lengths of the larger diameter pipes, and increasing the length of the smaller diameter pipes (least cost layout). However, when the number of hydrants increases, these graphical methods become complicated. Since the model is based on a geometric scheme the method would be inefficient and converge slowly for large networks. In practice it is easier to achieve a solution by computer methods.

Hassanli and Dandy (1994) develop a mathematical model to minimise the cost of pipe lines in branching networks. The formulation of this model is based on an effective search method to find the optimum layout of pipes where the position of hydrants (nodes) are known. This model is formulated using the genetic algorithm approach and overcomes the problem stated in the methods were suggested by Labye et al, 1988.

Perez et al (1993) considered the effect of the pipe thickness on the system cost instead of pipe sizes in the irrigation systems. They explained that the higher pressure implies thicker, and consequently, more expensive pipes. As a result, reduction in the static pressure can lead to savings in the piping costs, for this reason, they proposed the use of pressure reducing valves (PRVs) in the system. Their method was based on a dynamic programming formulation. Although this method may reduce the system cost significantly, it is not generally applicable for most drip irrigation systems except for those on hilly topography areas.

Holzapfel et al (1990) developed a non linear optimisation model for the design and management of trickle irrigation systems. They analysed the benefits of the yield obtained from a drip irrigation in terms of water application, whereas all other required resources were at the optimum level. In this model, as in Oron (1982) and Oron and Walker (1981) multiple subunit systems are considered, but their model is more elaborate. The main reason for this complexity is due to the maximising the net benefits instead of minimising the system cost. In their non-linear model the objective function was constructed using the benefits from production and the cost of the drip system due to pipes, valves, emitters, filters, the pump system, accessories and energy. In order to reduce the initial investment they increase the number of emitters during root development and the growth period. The constraints were based on hydraulic losses, water and time availability, and management conditions. Holzapfel et al, considered the following as design variables: pipe diameters, pipe lengths and number of emitters. Their analysis shows that the cost of the system and its operation were relatively small in comparison with the benefits gained from it. For example, a 50% increase in pipe cost, reduces the profit by 0.7%. According to their sensitivity analysis, changing the price of the product from 40.0 to 10.0 $ch [$ch being Chilean pesos; 1 $US=216.0$ch] reduces the profit by 77.6%, and a 100% increase in the cost of energy reduces the profit by 1.4%. They claim their developed model can be used for design and management of drip irrigation systems in flat, hilly and sloping areas. For hilly and sloping areas, it is necessary to use the procedure given by Keller and Rodrigo (1979).

Oron and Karmeli (1979) applied generalised geometric programming (GGP) and the branch and bound (B&B) technique as two basic algorithms to develop an optimisation method for non linear and mixed integer constrained problems in pipe network systems.

They combined these two techniques in order to find the optimal values and to include integer variables in their work. They used B&B to present some variables such as the number of outlets per section in an integer form. Finally, they applied the developed optimisation procedure to an irrigation system. In their example the cost of the pipe was expressed by a concave expression, as follows:

$$C = aD^2 + bD + e \tag{2.11}$$

where C = cost of pipe per unit length ($); D = nominal pipe diameter (m); a, b and e are constants.

In their model, the layout, incoming flow rate, working pressure between laterals and sprinklers or emitters and the diameter of pipes are known. The objective was to find the number of outlets either on the manifold or on the laterals. Consequently, the final lengths of laterals and manifold were calculated.

Calhoun (1970) applied linear programming to optimise irrigation networks. He argued that several types of pipe networks are broken into two general cases, the gravity case and pumped case. In the first case, the head at the source is known while in the second case it is not known. In the gravity case, the objective function expresses the total cost of a gravity pipe distribution system. In this work, the cost per unit length of any diameter of pipe in the system is known, and decision variable is the length of each known pipe size. The objective function is minimised subject to constraints consisting of the summation of the coefficient of head loss per unit length of each pipe times the corresponding pipe length.

In the pumped case the objective function expresses the total cost of a pumped distribution system as the sum of the cost of every section of pipe plus the capitalised cost of pumping. The objective function is minimised subject to constraints consisting of coefficients which evaluate the unknown pump head in terms of the friction slope per unit length of each pipe.

2.4.3 A Short Review on Optimisation of Irrigation Scheduling and Management Strategies

The review of optimisation of irrigation systems described in the previous sections was mainly focussed on design problems. However, some literature is available which considers the yield production function and the water distribution function in addition to the system cost. Some researchers have attempted to combine the yield production function, water distribution function and the system cost function in the objective function to optimise irrigation systems. Since pipe network design problems and irrigation operating aspects have been investigated in this study, combining the yield and distribution functions in the objective function are the beyond its scope. However, since it is a full and comprehensive attempt to achieve a completely optimised irrigation plan the following short review on this matter is presented.

Seginer (1987) outlines the following three functions that playing a major role in the optimisation of an irrigation system:

- The water distribution function, describing the water distribution over the field, $f\{w\}$;
- The crop response to water or the yield production function, $y\{w\}$;
- The irrigation system cost, as a function of the water distribution function, $c\{f\{w\}\}$.

In recent years several studies attempted to optimise the design and operation of irrigation systems emphasising the impact of water distribution uniformity. Seginer (1987) found optimal seasonal water application as a function of water cost and uniformity; Hill and Keller (1980) considered a dependence of the system cost on uniformity, Hart et al (1980) evaluated the same problem considering a drainage system; Chen and Wallender (1984) developed a method to optimise simultaneously the distribution uniformity and the seasonal water application.

Seginer (1987) believes that a number of the existing methods are essentially the same, and the basic general objective function of different methods may be formulated as follows:

$$Z = h\overline{Y} - C - p\overline{W} \qquad (2.12)$$

where Z is the objective function (per unit area) to be maximised; $\overline{Y}$ is the mean yield; $\overline{W}$ is the seasonal mean water application depth; C is the cost of the irrigation system (per unit area and season); h is the unit price of the yield while p shows the unit price of water. In his proposed objective function the constant costs and the revenues are not included. Assuming that both the mean yield and the irrigation system cost are a function of the mean water application and uniformity. Thus Equation 2.12 may be modified as follows:

$$Z = h\overline{Y}\{\overline{W},U\} - C\{\overline{W},U\} - p\overline{W} \tag{2.13}$$

where U is the uniformity of water distribution.

Apart from the effect of distribution uniformity on the maximum profit of an irrigation project, a number of authors have also attempted to analyse the effect of crop yield as a function of the mean depth of water applied to find the maximum net profit.

Stegman et al (1983) identify that the optimum economic level of production for an unlimited water supply is estimated by equating the marginal value of yield improvement with the marginal cost of the further water application as follows:

$$\frac{dY}{dI} = \frac{p(I)}{p(Y)} \tag{2.14}$$

where Y= is the yield; I is the applied irrigation depth; p(I) = variable cost of water application, and p(Y) is the crop price.

They added that the maximum profit level for a limited irrigation water supply can also be estimated from a function of yield versus applied irrigation depth. Y vs I functions typically assume other production inputs are at some fixed level. Usually in this type of analysis, the nutrient level is assumed to be near optimal.

Jensen and Sletten (1965) describe experiments which show that there is a relationship between fertiliser and water input to crop production. As a result, two-variable production functions can be presented in the yield function (Egli, 1971; English and Dvoskin, 1977).

An economic optimum of this production function is achieved by maximising the following objective function:

$$Z = Y.p(Y) - F.p(F) - I.p(I) \qquad (2.15)$$

where F is the amount of fertiliser and p(F) is its unit price.

As additional variable inputs are considered, the profit maximisation becomes more complex. However, the use of marginal cost analysis for yield functions provides only general guidelines for water management. These guidelines are most applicable to average or normal climatic conditions in a given region and, therefore may not apply to specific sites or specific years (Jensen and Sletten, 1965).

In recent years, a number of simulation models have been developed to maximise the profit due to yield. Optimisation techniques such as dynamic programming are frequently utilised to show how optimal water scheduling can be derived under conditions of stochastic inputs. Mathematical modelling for optimising of on farm water management, have been developed by Trava et al, 1976; Mapp et al, 1975; Howell et al, 1975; Stapleton et al, 1973; Yaron and Strateener, 1973; Dudley et al, 1971 and Hall and Butcher, 1968.

The electronic computer with its rapid data processing capabilities makes large scale irrigation scheduling possible. However, because of some problems in details (related to the soil, water and plants) and economic justifications, these complex simulation models are not yet available widely. In addition, a second major problem commonly cited by most model developers is the need for more detailed production functions. These inputs may be due to the plant growth modelling activities that are designed to evaluate water management strategy effects at any crop growth stage (Curry, 1971; Barfield et al, 1977; Splinter, 1974; Childs et al, 1977).

Trava et al (1977) and Pleban et al (1983) developed optimisation programs that minimise the cost of labour within the constraints of the irrigation system and available water. The optimisation programs were constrained so that water never limited crop production. An irrigation scheduling program was used to predict the depth of water required to refill the profiles for each day within a forecast period. An irrigation system has a minimum application depth that can be efficiently applied and was a constraint on the earliest irrigation date to prevent unintentional leaching. The maximum allowed depletion is the most water a crop can extract from the soil before stress occurs and usually determines the latest date for irrigation. In their optimisation model, they developed a schedule to irrigate all fields within the timing constraints and to minimise the labour costs for implementing the schedule.

According to Heermann et al (1990) several individuals have used simulation programs to develop irrigation schedules and management strategies. Simulation models can be integrated with crop production schedules.

Recently, simulation models have been combined with optimisation programs to provide schedules that include the physical constraints of the irrigation system and crop (Martin, 1984). Thus analyses beyond conventional scheduling are required to develop management techniques for water-limiting conditions. Heermann et al (1990) explained that land is the limiting resource and the economic objective is to irrigate until the marginal net return from applying a unit of water equals the marginal cost of applying that unit. The optimal irrigation depth for land-limited irrigation is often near the depth which gives the maximum yield. Therefore, traditional scheduling procedures have been very useful when water is generally available. The economic criteria for optimising the use of a limited water supply is different than that for scheduling an unlimited source. When water is limiting, the economic criteria is to maximise the average net return per unit of water used.

Planning for the optimal irrigated area also depends upon the factors considering in the net return calculation. Martin et al (1990) developed a general method to predict the optimal irrigated area and depth of irrigation. They showed that several parameters are involved in the decision, including: the efficiency of the irrigation system, the cost of preparing land for irrigation , the cost of water and the yield response and price expected for the irrigated and dry land crops. Their results showed that the optimal policy varies from irrigating for maximum yield on a small area, to spreading the available water over the entire irrigable area (for larger areas).

2.5 SUMMARY

As noted in this review, few studies have been carried out on the optimisation of drip irrigation systems based on the partitioning the field into subunits. Also optimising the layout and component sizes of a branched pipe system (simultaneously) for irrigation systems was not considered in literature. Although some literature is available on multiple subunit systems, no information is available for partitioning a field into optimum subunit sizes and identifying the optimum irrigation shift patterns via the best combination of subunits to be irrigated simultaneously.

The primary purpose of the first part of this study is to investigate both the optimum size and optimum dimensions of subunits in a multiple subunit system. Also examine the effect of a number of possible shift patterns for irrigation of an optimum combination of subunits at any one time. In the second part, the research will be focussed on developing optimisation models to find the optimum layout (connection between nodes) and also the optimum component sizes of a branched pipe network by employing GAs. Selection of an appropriate pump for the system will also be considered as decision variable.

Chapter 3

Optimisation of a Drip Irrigation Systems with One Control Head

3.1 INTRODUCTION

"A subunit is an individually irrigated area consisting of a manifold and lateral system" (Oron and Walker, 1981). It can be one part of a larger field or one independent field. Normally it consists of at least one control head, one supply pipe, one manifold, several laterals, and several emitters (see Figures 3.1 and 3.2). In this part of the research, an optimisation model is developed for a field with one control head on flat terrain. In the following two chapters, this work is extended to sloping lands and also to fields with multiple subunit systems in which a number of factors affecting the minimum system cost are examined. The model which produces the minimum total cost including pipe, emitter and accessory costs has been identified for various field dimensions under different irrigation times and irrigation intervals. The optimisation procedure uses a complete enumeration approach.

Complete enumeration is one approach for the optimisation of pipe networks. In this method every possible combination of specified discrete pipe sizes is evaluated. Among the various possible solutions the minimum cost solution which meets the minimum operating pressure is selected as the global optimum (Dandy et al, 1993). A disadvantage of complete enumeration compared to other optimisation methods is the large amount of computer time that is required to investigate every possible combination of pipe sizes or connectivity (in unknown layout) problems.

3.2 CHARACTERISTICS OF THE MODEL

This model has been developed for a rectangular flat field of known area. The objective function is to minimise the total cost of the system, by choosing the optimum lengths of two given sizes for the laterals and the optimum sizes for the manifold and supply pipes for various irrigation intervals and irrigation times. In this model, only the capital cost of pipes, emitters and accessories in control head has been considered while the cost of pumping system and the annual operating cost has been taken into account in Chapters 4 and 5.

The analysis is based on a subunit or a field with one control head consisting of a supply pipe, a manifold, several laterals, several emitters, and, at least one control valve for pressure and discharge regulation. Water flows from the supply pipe through the manifold and then is distributed through the laterals and runs out slowly from the emitters (Figure 3.2). In the proposed scheme, water which runs through the manifold and laterals is divided into two equal flows with opposite direction. The pipe characteristics of both laterals and manifold are the same and the pipes are laid over a flat surface. As a result, only half the length of the manifold with half the discharge was taken into account for the pressure variation analysis. The diameter of manifold pipe and the length of both segments of laterals are selected subject to a limited head loss. This ensures to have an acceptable distribution of irrigation water across the field.

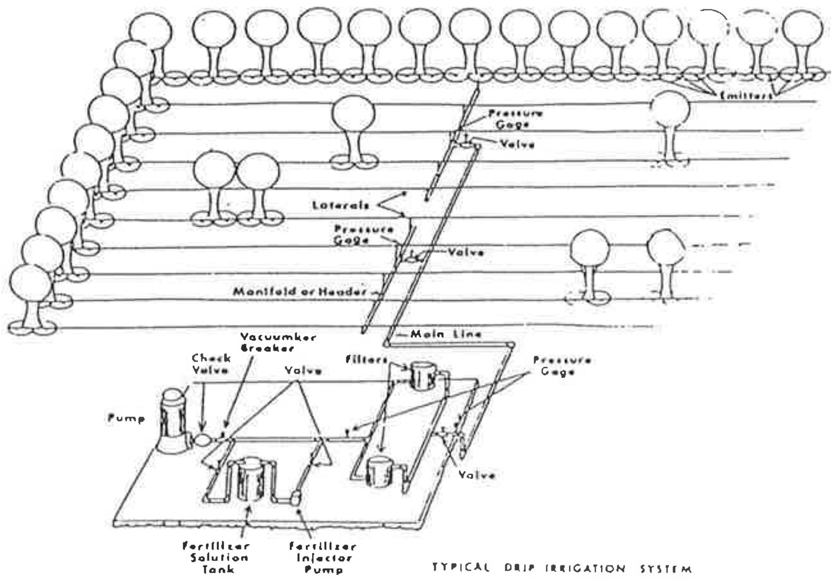


Fig. 3.1 An example of a basic drip irrigation system (Jensen, 1983)

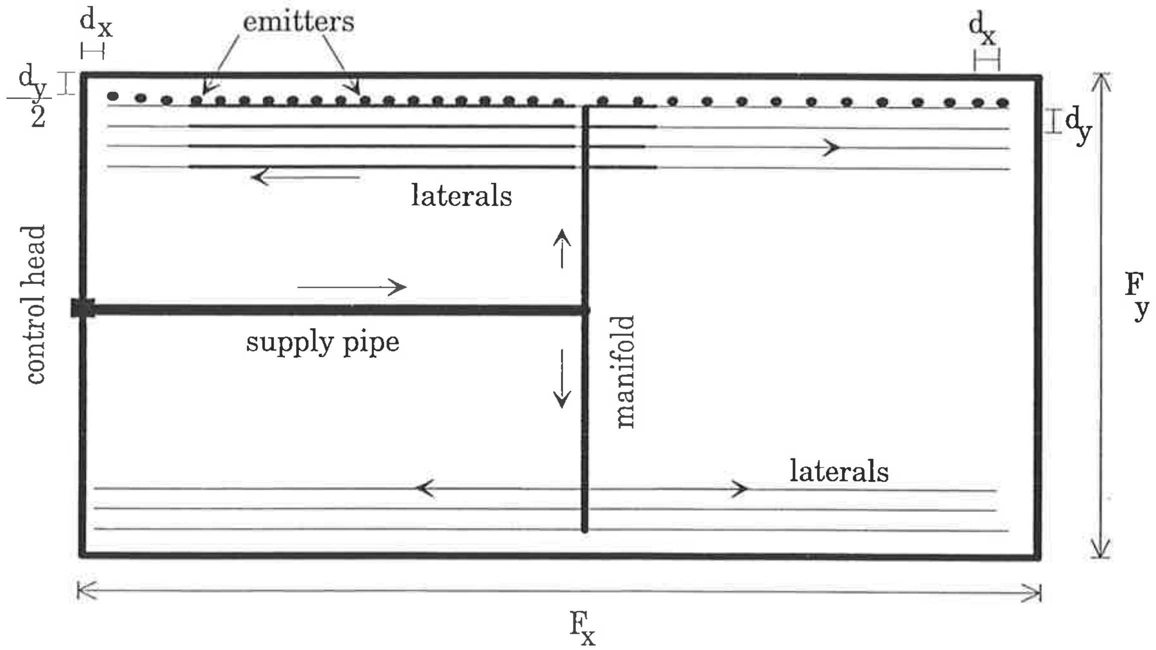


Fig. 3.2 A typical layout of a drip irrigation system with one control head

3.3 CROP WATER REQUIREMENTS

The crop water requirement depends directly on the potential evapotranspiration and crop coefficient. The crop water requirement is given by:

$$ET_C = K_C ET_0 \tag{3.1}$$

where ET_C= crop water requirement (mm/day); K_C is a factor, often called the "crop coefficient" which reflects the physiological aspects of the particular plant and relates the actual rate at which a crop uses water; ET_0= potential evapotranspiration (mm/day). K_C varies with stage of growth and season and is also sensitive to the adequacy of the water supply. In practice K_C is determined experimentally for each crop. Since a well designed and operated trickle system offers small quantities of water at frequent intervals, a low and constant soil water tension will exist around the root zone. Frequent trickle irrigations would improve plant growth and increase yields, assuming that no problems occur such as those related to soil aeration, plant disease, or restricted plant rooting (Bucks and Davis, 1986).

The crop water requirement will be reduced for young orchards with widely spaced crops. Up to a 60% reduction in ET_C has been observed for young orchards with about 30% ground cover on light, sandy soil and under high evaporation conditions (FAO, 1984). For mature and closely spaced crops under drip irrigation, the value of ET_c can be calculated from Equation 3.1. However, for design purposes the worst condition will be considered, i.e. K_C=1 (Jobling, 1974). Design flow rate depends upon the operating schedule which may be adjusted by changing the irrigation interval and irrigation time. This issue is discussed in the following section.

3.4 IRRIGATION INTERVALS AND IRRIGATION TIMES

The irrigation requirement or field flow rate (design flow) is based not only on the crop evapotranspiration but also on the irrigation interval and irrigation time. In this analysis the irrigation is assumed to be carried out in a discontinuous manner. The irrigation interval is the time in days between the commencement of two irrigation events and the irrigation time is the length of an irrigation event (hours). An irrigation event is the period during which water is being released from a particular set of emitters. A graphical representation of irrigation interval and irrigation time is given in Chapter 5 (Fig. 5.2). Very little research work on optimum intervals between irrigation events is available (Jobling, 1974). It can be said that the best approach is to apply water whenever the soil suction reaches a critical level which is a function of transpiration rate and a particular plant. As a general guide it can be said that short intervals are necessary where transpiration rates are high, soils have poor water holding-capacity, water is saline, or plants are shallow rooted. Longer intervals are allowable where transpiration rates are low, the soil has good water-holding capacity, water is of high quality, or plants are deep rooted (Jobling, 1974).

In this model, the system cost (field cost) is examined under various irrigation intervals and irrigation times. It is apparent that as the irrigation interval increases and the irrigation time decreases, the rate of application of water to the plants will need to be increased to satisfy the water requirement. As a result, the piping system including delivery pipe sizes must be enlarged. This will increase the system cost.

3.5 FORMULATION OF MODEL

The total system cost includes: the cost of three types of pipe (supply, manifold, and laterals), emitters, and accessories in the control head. The cost of the pump, and annual operation are not considered in this simple model. The decision variables are the lengths of two known lateral pipe sizes, the diameter of manifold and supply pipes. These variables are identified for the most appropriate field dimensions and irrigation time and interval. All costs in this study are based on the Australian market price in 1990 and also expressed in terms of Australian dollars (A$ 1 equals US$ 0.75 approximately). The objective function may be formulated as follows:

$$Z = N_{lm}\left(C_{bs} \times L_{bs} + C_{ss} \times L_{ss}\right) + \left(C_m + C_s + C_{em} + C_{acc}\right) \qquad (3.2)$$

where Z= objective function, comprising the total system cost ($); N_{lm}= number of laterals on the manifold; C_{bs} = cost per unit length of the larger size of laterals ($); C_{ss}= cost per unit length of the smaller size of laterals ($); L_{bs}, L_{ss}= length of larger and smaller size of lateral segments (m) respectively; C_m, C_s, C_{em}, C_{acc}= cost of manifold, supply pipe, emitters and accessories ($) respectively.

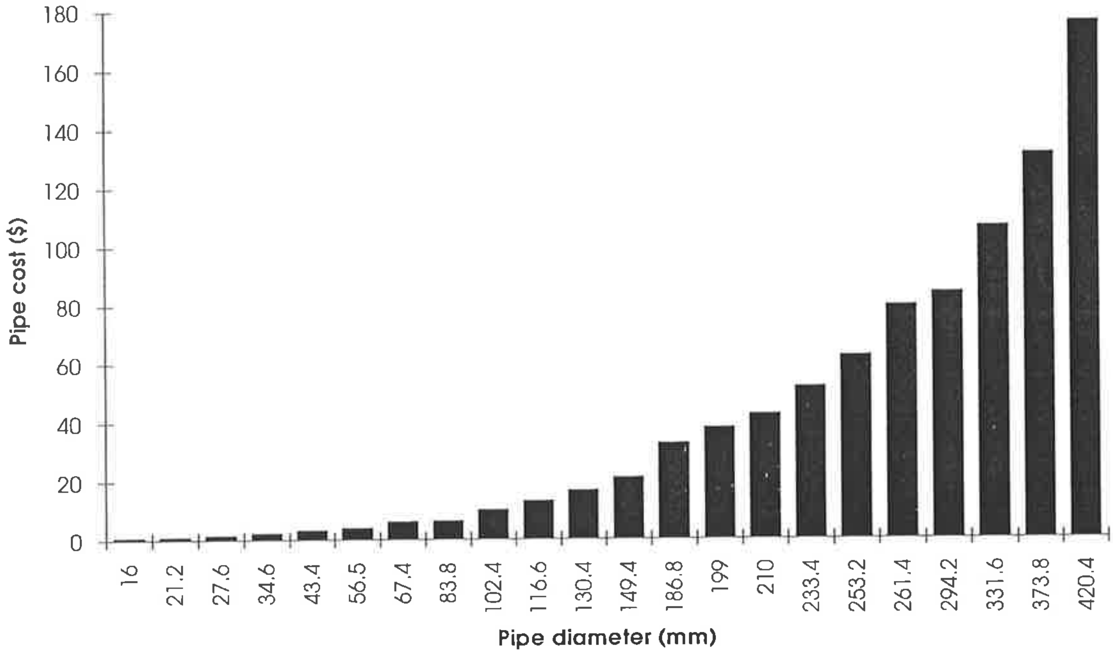


Fig. 3.3 Variation of pipe cost in terms of pipe diameters

The cost per unit length of manifold and supply pipe may be expressed by the following non-linear equation (Oron and Karmeli, 1979):

$$C = a D^2 + b D + e \qquad (3.3)$$

where C = cost of pipe per unit length ($/m); D= diameter of pipe (mm); a, b, e = constant parameters.

The least squares method was used to identify the constants in Equation 3.3. The pipes are Hardi tube industrial polyethylene pipe. The values obtained for three constant parameters in the pipe cost equation using least squares analysis are shown in the following equation:

$$C = 0.00096\,D^2 + 0.0061\,D\ + 0.18 \tag{3.4}$$

The above equation only represents the pipe purchase price. The installation cost has not been taken into account in this case. The variation of pipe prices in terms of internal diameter is shown in Figure 3.3. This figure shows that the slope of the pipe cost curve increases sharply as the internal diameter increases.

3.5.1 Pipe Lengths

The length of pipes within the subunit according to the piping configuration shown in Figure 3.2 considering a space for borders, are calculated as follows:

$$L_l = \frac{F_x}{2} - d_x \tag{3.5}$$

$$L_m = F_y - d_y \tag{3.6}$$

$$L_s = \frac{F_x}{2} \tag{3.7}$$

$$N_{el} = \frac{L_l}{d_x} + 1 \tag{3.8}$$

$$N_{lm} = 2\left(\frac{L_m}{d_y} + 1\right) \tag{3.9}$$

$$T_{ll} = L_l \times N_{lm} \tag{3.10}$$

where F_x = field length in the X direction (m); F_y = field width in the Y direction (m); d_x = emitter spacing on laterals (m); d_y = lateral spacing on manifold (m); L_l = length of lateral pipe (m); L_m = length of manifold pipe (m); L_s = length of supply pipe (m); N_{el} = number of emitters on each lateral; T_{ll} = total lateral lengths (m).

The laterals are located in either side of the manifold with a uniform spacing.

3.5.2 Cost of Pipes

The cost of each pipe is proportional to its length and changes with changes in the field dimensions. The cost of lateral depends on the length of the larger and smaller size segments as follows:

$$C_l = (C_{bs} \times L_{bs} + C_{ss} \times L_{ss})N_{lm} \tag{3.11}$$

The cost of manifold and supply pipes are computed using Equation 3.4 as follows:

$$C_m = (0.00096\, D_m^2 + 0.0061\, D_m + 0.18)L_m \tag{3.12}$$

$$C_s = (0.00096\, D_s^2 + 0.0061\, D_s + 0.18)L_s \tag{3.13}$$

The cost of emitters obtained considering the unit cost and the number of units was used as below:

$$C_{em} = N_{lm} \times N_{el} \times CU_{em} \tag{3.14}$$

where C_l, C_m, C_s = cost of lateral, manifold and supply pipes respectively (\$); D_m, D_s = diameter of manifold and supply pipes, respectively (mm); CU_{em} = cost of each emitters (\$); C_{em}= total cost of emitters(\$).

It is assumed that all pipes will be laid on the ground, hence installation cost is not taken into account.

3.6 CONSTRAINTS

The objective function is to minimise the system cost, subject to constraints which limit the hydraulic pressure losses through the manifold and laterals. The variation of discharge from the nearest emitter to the most distant emitter from the manifold is required to be less than an acceptable value. This is to ensure a limited variation in the discharge from the emitters over the entire field. The details are discussed in the following sections.

3.6.1 Hydraulic Constraints

Hydraulic constraints are important, because the pressure variation along the laterals and manifold must be restricted so that a good uniformity of water distribution is achieved. On the other hand, the total head losses plus minimum required pressure at the emitters must be less than or equal to the pressure at the source. In this model, the head loss of the multiple outlet pipes in the system is determined by applying the Hazen-Williams formula as below:

$$HL = K \frac{Q^{1.852} L}{C^{1.852} D^{4.87}} F(n_0) \tag{3.15}$$

where HL = friction head loss (m); Q = pipe discharge (m^3/s); L = pipe length (m); D = pipe diameter (m); C = Hazen-Williams roughness coefficient (usually between 130 to 150 for polyethylene and PVC pipes); $F(n_0)$= correction function to account for the variation in discharge through pipes with multiple outlets; n_0 = number of outlets on the pipe; K = a constant (10.68 for metric system).

As water flows through multiple outlet pipes the discharge reduces along the pipe. This affects the head loss along the pipes. According to Oron and Walker (1981), the discharge correction function for multiple outlet pipes may be expressed as:

$$F(n_0) = 0.6387 n_0^{-1.8916} + 0.35929 \tag{3.16}$$

In a well designed drip irrigation system, the variation of discharge along the laterals and the manifold should not be more than 10%. According to Wu et al (1986), in most normal emitters (orifice types) to limit the variation of discharge within this range the pressure variation along the laterals and the manifold should not exceed 20%. On the basis of a working pressure of 10m this gives 2m allowable head loss within the manifold and laterals. Therefore, the head loss in the manifold may be expressed as follows:

$$HL_m = 2.0 - HL_l \tag{3.17}$$

where HL_m = head loss of manifold in one segment (m); HL_l = head loss in larger and smaller segments of laterals (m).

Although through a simulation analysis it would be possible to specify what portion of the total allowable head loss within the manifold and laterals should be allocated to each pipe, in this work up to 1.80m was allowed to occur in the laterals and the balance in the manifold. This is reasonable as laterals make up a great portion of total length of micropipes in the system.

The constraint of the total system head implies that the total head loss plus the minimum required working pressure should be less than or equal to the total pressure at the source. i.e:

$$H_w + HL_l + HL_m + HL_s + HL_{acc} \leq TH \tag{3.18}$$

where H_w= design working pressure on emitters (m); HL_s= head loss in supply pipe (m); HL_{acc}= head loss in accessories (m); TH = total head at the source (m).

Equations 3.16 is used to compute the head loss in each segment of the laterals and manifold to consider the effect of reducing the discharge along the multiple outlet pipes. Then:

$$HL_l = HL_{bs} + HL_{ss} \tag{3.19}$$

The length of smaller size segment in each lateral depends on the length of the larger size segment and also field dimensions. It is obtained as below:

$$L_{ss} = L_l - L_{bs} \tag{3.20}$$

where HL_{bs}, HL_{ss} = head losses in larger size and smaller size of laterals (m); L_{bs}, L_{ss} = lengths of larger size and smaller size of laterals respectively (m).

The diameter of the manifold as one of the decision variables is obtained on the basis of the head loss, the length and the discharge in one segment (half length) as follows:

$$D_m = \left[\frac{\left[10.68F(N_{lm})\left(\frac{L_m}{2}\right)\right]^{\frac{1}{4.87}}\left(\frac{Q_m}{2}\right)^{\frac{1.852}{4.87}}}{(CHM)^{\frac{1.852}{4.87}}(HL_m)^{\frac{1}{4.87}}} \right] \tag{3.21}$$

where D_m = diameter of manifold (m); $F(N_{lm})$ = manifold correction discharge function; Q_m = manifold discharge (m^3/s); CHM = Hazen-Williams coefficient of manifold.

In this model, D_m obtained from the above equation is rounded up to the next available discrete diameter. Since the whole discharge of the field passes through the supply pipe, the final supply pipe size is assumed to be equal to one discrete size larger than D_m.

The head loss in the supply pipe then is computed as follows:

$$HL_s = \frac{10.68 L_s Q_s^{1.852}}{D_s^{4.87} CHS^{1.852}} \tag{3.22}$$

where: HL_s = head loss of supply pipe (m); Q_s = supply pipe discharge (m^3/s); D_s = supply pipe diameter (m); CHS = Hazen-Williams coefficient for the supply pipe.

3.6.2 Discharge Constraints

The discharge of a subunit depends on the plant water requirement which is directly affected by evapotranspiration. It also varies for different combinations of irrigation intervals and irrigation times. In this and the following chapter a simple agronomic model is used and ignores the final storage capacity of the soil and the percentage of wetted area. A more detailed model is developed in Chapter 5.

Discharge in emitters is estimated as follows

$$Q_T = ET_c d_x d_y \tag{3.23}$$

$$Q_E = \frac{F(I)\, Q_T}{T(K)\, E_a n} \tag{3.24}$$

where Q_T = plant water requirement (L/day); Q_E = emitter discharge (L/hr); E_a = irrigation application efficiency; n = number of emitters around each plant; $F(I)$= irrigation interval (from 1 to 6 days); $T(K)$ = irrigation time (from 2 to 20 hours).

In the design of a drip irrigation system it is necessary to make sure that an appropriate size is chosen for emitters. Very large emitters with a high flow rate may exceed the infiltration capacity of the soil and consequently, cause water loss and soil erosion, while very small size emitters are susceptible to clogging or build up of chemical deposit.

In this model, the emitter flow is limited as follows:

$$2 \le Q_E \le 24 \text{ L/hr} \tag{3.25}$$

This wide range of emitter discharges in the present model allows the designer to select one of the feasible solutions among the various alternatives that is compatible with the existing conditions of soil infiltration rate and the operating point of view.

Discharge of the laterals, manifold and supply pipes are computed considering the number of emitters allocated to each lateral and laterals located on the manifold as follows:

$$Q_l = N_{el}\, Q_E \tag{3.26}$$

$$Q_m = N_{lm}\, Q_l \tag{3.27}$$

$$Q_s = Q_m = Q_f \tag{3.28}$$

where Q_l, Q_m, Q_s, Q_f discharge of lateral, manifold, supply pipe and the field respectively (m^3/s).

3.7 OPTIMISATION PROCEDURE

A constant area of 6 hectares is assumed for the field with rectangular form. The field length, F_x is allowed to vary from 50 to 900 m, irrigation intervals, from 1 to 6 days, and irrigation times, from 2 to 20 hours. D_{bs} and D_{ss}, are assumed to be the two smallest available pipe sizes for each segment of laterals. First the model examines the least cost solutions by the use of smallest available diameter for all micropipes (laterals, manifold and supply pipe) unless they violate the pressure constraints. This process is repeated for various emitter discharges which are provided by implementing different combinations of irrigation interval and irrigation time. Finally for each combination of irrigation interval and irrigation time considering the pressure and discharge constraints the least cost solution is identified. Complete enumeration is employed as the optimisation approach and proceeds in the following steps:

1 Let $F_x = 50$ m;

2 $F_y = \dfrac{A}{F_x}$ (A is a constant area of 60,000 m^2);

3 Let F(I) = 1 day;

4 Let T(K) = 2 hours;

5 Calculate Q_E, Q_l, Q_m and Q_s using Equations 3.24, 3.26, 3.27 and 3.28 respectively;

6 Let $L_{bs} = 0$;

7 Find L_{ss} considering L_l and L_{bs} using Equation 3.20;

8 Find HL_l from Equation 3.19;

9 If HL_l greater than 1.8 m go to step 14;

10 Find HL_m from Equation 3.17;

11 Find D_m from Equation 3.21 and round it up to the next available diameter;

12 Let D_s equal the next size larger than D_m;

13 Calculate cost of the system using Equation 3.2;

14 Increment L_{bs} by 10% of L_l and return to step 7;

15 If L_{bs} equals L_l, increase $T(K)$ by 2 hours. If $T(K) \leq 20$ hours, go to step 5, otherwise go to step 16;

16 Increase $F(I)$ by 1 day. If $F(I) \leq 6$ days go to step 5 otherwise go to step 17;

17 Increase F_x by 50 m. If $F_x \leq 900$ m go to step 2, otherwise stop.

The above optimisation procedure is illustrated as a flow chart in Figure 3.4.

The model is written in FORTRAN code and was run on a mainframe machine on Unix system.

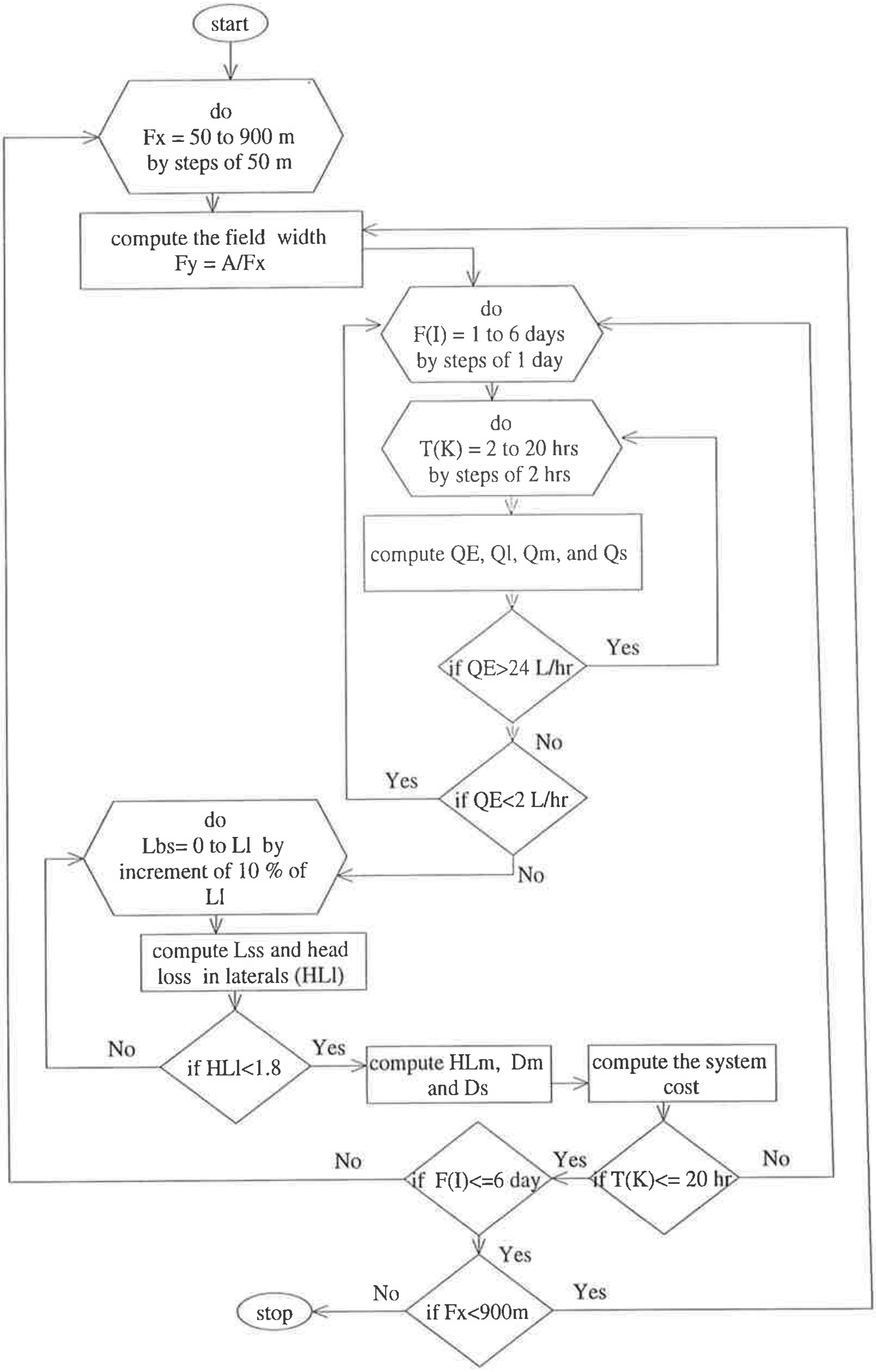


Fig. 3.4 Flow chart of optimisation process

3.8 RESULTS AND DISCUSSION

As explained previously, in the current simple model, the effect of different irrigation intervals and irrigation times and also the effect of field geometry which resulted from changing the length and width of field on the system cost are examined. The model examination is carried out by a case study with the input data tabulated in Table 3.1.

TABLE 3.1 Input data used in the case study

Parameters	Values	Parameters	Values
A	60,000 m^2	ET_0	6 mm/day
D_{bs}	19 mm	K_c	1
D_{ss}	13 mm	d_x	2 m
irrigation interval	1,2,3 4,5,6 days	d_y	3 m
irrigation time	2 to 20 steps of 2 hr	C_{bs}	0.85 \$/m
CHL	130	Css	0.42 \$/m
CHM	140	CU_{em}	0.90 \$/emitter
CHS	150	$Cval$	\$50
n	1	C_{fil}	\$100
E_a	95 %	C_{fer}	\$100
H_w	10 m		

3.8.1 Minimum System Cost for a Field with Fixed Dimensions

In this work, the variation of the cost of a drip irrigation system for a field of constant area and varying dimensions under different feasible irrigation intervals and irrigation times has been evaluated. To demonstrate the effect of irrigation interval and irrigation time on the system cost, the variation of the minimum system cost for fixed dimensions (i.e. F_x = 250 m, Fy = 240 m) is plotted in Figure 3.5. Each individual line represents the variation of the minimum system cost with respect to different irrigation times for a fixed irrigation interval. The lower lines showing the lower cost correspond to the smaller irrigation intervals. Each individual line also shows that as the irrigation time increases the minimum system cost decreases. Another finding shown in Figure 3.5, is that for larger irrigation intervals, the number of feasible irrigation times decreases. For example, for the highest irrigation interval (6 days), the only feasible irrigation time is 20 hours, whereas for the lower irrigation intervals more irrigation times are feasible. i.e. for a 1 day of irrigation interval, the feasible irrigation times vary from 4 hours to 18 hours. The feasible limited irrigation times for higher irrigation intervals are due to the limited range of emission are for the emitters or due to the head loss constraints in the laterals and the manifold. i.e. infrequent watering implies high flow rates and hence high head loss in the laterals and as a result, it violates the allowable pressure variation requirements. In the same way, using small irrigation times necessitates application of water with a high flow rate. This again violates the constraints. The details showing the feasible irrigation intervals and corresponding information for a field with dimensions of 250 and 240 m are summarised in Table 3.2.

In a similar way, the results of the same procedure but for another set of field dimensions (F_x= 50 m, F_y = 1200 m) are represented in Figure 3.7. This case corresponds to the field with the shortest laterals and the longest manifold illustrated in Figure 3.8c. In this iteration, due to the short length of laterals, the head loss in the laterals is small, consequently, a large head loss is permitted in the manifold (Equation 3.17). On the other hand, the manifold does not have the same restriction in diameter as the laterals, consequently, lower irrigation times even causing a higher flow rate become feasible in comparison with the case in which F_x is 250 and F_y is 240 m. The global minimum cost in this case is obtained at an irrigation interval of 1 day and irrigation times of 12 to 18 hours.

As illustrated in Figure 3.6 in some cases, the minimum system cost for two different irrigation intervals with the same irrigation time are identical (where the corresponding lines cross). Also in some cases the minimum system cost is identical for different irrigation times under a fixed irrigation interval. This could be due to the use of short length of laterals with one diameter only and also the discrete sizes of manifold which do not change when the change of discharge is not considerable.

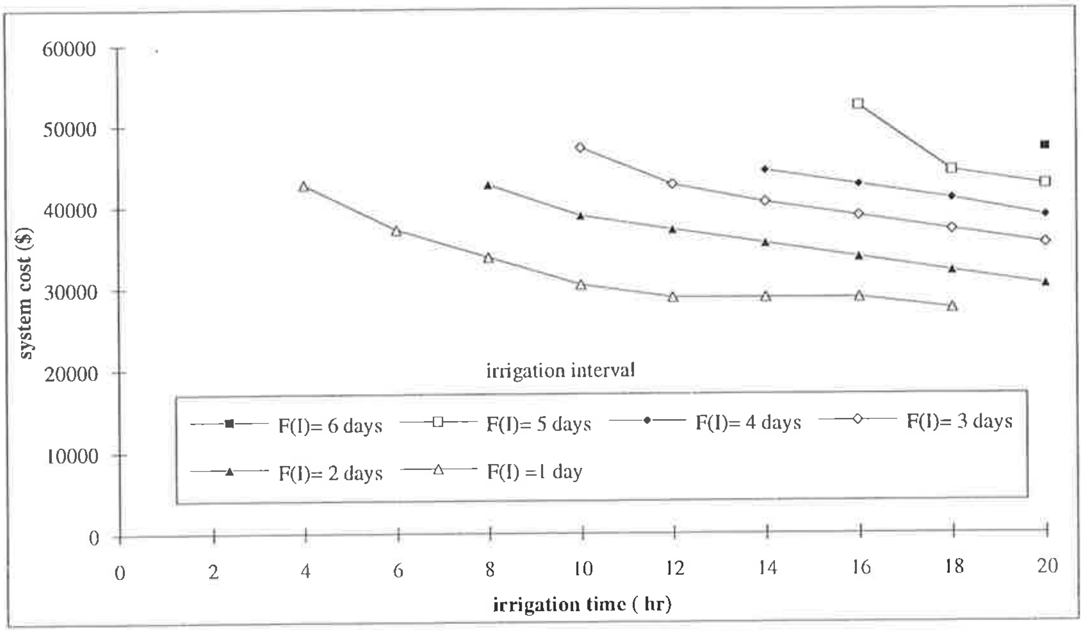


Fig. 3.5 Variation of minimum system cost for different combinations of irrigation time and irrigation interval (F_x= 250 m, F_y= 240 m)

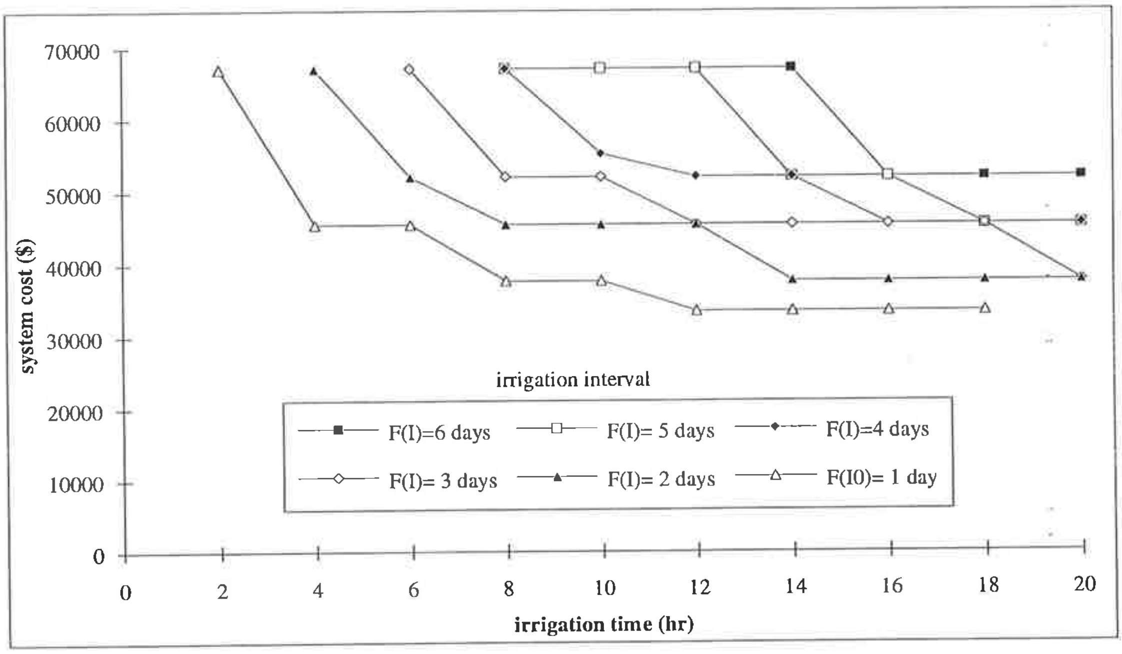


Fig. 3.6 Variation of minimum system cost for different combinations of irrigation time and irrigation interval (F_x= 50 m, F_y= 1200 m)

TABLE 3.2 Total minimum system cost for various irrigation intervals and irrigation times for fixed dimensions of field

field length (m)	field width (m)	irrigation interval (day)	irrigation time (hr)	emitter discharge (L/hr)	T(K)/F(I) (hr/day)	system cost ($)
250	240	1	4	9.47	4.00	42,735
250	240	1	6	6.32	6.00	37,182
250	240	1	8	4.74	8.00	33,797
250	240	1	10	3.79	10.00	30,412
250	240	1	12	3.16	12.00	28,816
250	240	1	14	2.71	14.00	28,816
250	240	1	16	2.37	16.00	28,816
250	**240**	**1**	**18**	**2.11**	**18.00**	**27,489**
250	240	2	8	9.47	4.00	42,735
250	240	2	10	7.58	5.00	38,874
250	240	2	12	6.32	6.00	37,182
250	240	2	14	5.41	7.00	35,489
250	240	2	16	4.74	8.00	33,795
250	240	2	18	4.21	9.00	32,100
250	240	2	20	3.79	10.00	30,412
250	240	3	10	11.37	3.33	47,221
250	240	3	12	9.47	4.00	42,735
250	240	3	14	8.12	4.67	40,567
250	240	3	16	7.11	5.33	38,874
250	240	3	18	6.32	6.00	37,182
250	240	3	20	5.68	6.67	35,489
250	240	4	14	10.83	3.50	44,427
250	240	4	16	9.47	4.00	42,735
250	240	4	18	8.42	4.50	41,043
250	240	4	20	7.58	5.00	38,874
250	240	5	16	11.84	3.20	52,438
250	240	5	18	10.53	3.60	44,427
250	240	5	20	9.47	4.00	42,735
250	240	6	20	11.37	3.33	47,221

The effect of the ratio of irrigation time to irrigation interval ($\frac{T(K)}{F(I)}$) for the fixed dimensions (F_x = 250 m, F_y= 240 m) is given in Table 3.2 and Figure 3.7. As shown in Figure 3.7, as long as this ratio increases the minimum cost decreases.

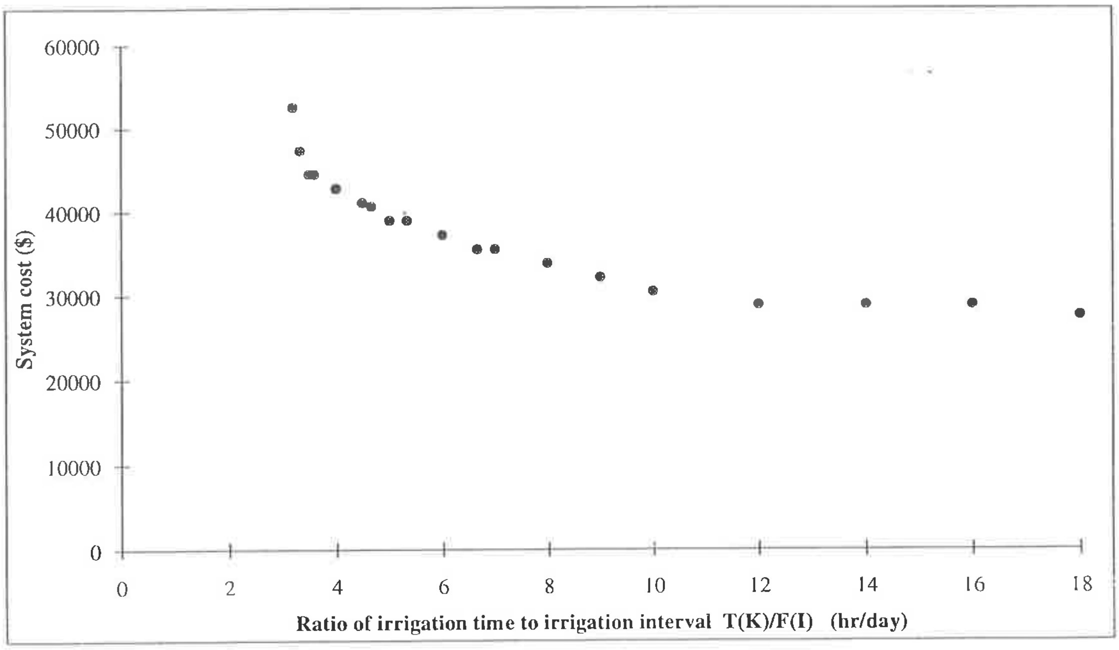


Fig. 3.7 Total minimum system cost versus the ratio of irrigation time to irrigation interval for fixed field dimensions (F_x = 250 m, F_y= 240 m)

The rate of change of cost for lower ratios is much more than for the higher ratios. The feasible ratio of irrigation time to irrigation interval varies from 3.33 to 18 hr/day which correspond to 10 hours per 3 days or 20 hours per 6 days and 18 hours per 1 day respectively. As shown in Figure 3.7, the global minimum cost which is $27489 occurs at the maximum ratio of irrigation time to irrigation interval. The findings indicate that the minimum cost may be obtained for a ratio less than the maximum value for another field geometry. For example, for F_x = 50, F_y= 1200 and F_x = 100, F_y= 600 m the minimum cost solution occurs at a ratio of 12 and 10 respectively. Table 3.2 demonstrates that different irrigation intervals and irrigation times with the same $\frac{T(K)}{F(I)}$ ratio lead to the same minimum system cost.

Another part of the analysis is focused on the local optima for each set of feasible dimensions. The total minimum cost or local optima is evaluated for each set of feasible field dimensions. These values together with the corresponding irrigation interval and irrigation time are shown in Table 3.3. The results indicate that only an irrigation interval of 1 day with the highest feasible irrigation time (18 hr) lead to the minimum cost for each set of dimensions. However, for the fields with dimensions of (50m, 1200m) and (100m, 600m) in which the lateral lengths are long, the lower irrigation times (12 and 10 hours) lead to the minimum cost.

3.8.2 Variation of System Cost for Different Field Dimensions

In the first stage of evaluation an analysis is carried out for a field with fixed dimensions under various feasible irrigation intervals and irrigation times. In the second stage, the main purpose is to evaluate the variation of the system cost with respect to the various field dimensions for a range of values of irrigation interval and irrigation time. As an example, a field with constant area and 5 different geometrise including two extreme cases are shown in Figure 3.8. The results showing the minimum system costs corresponding to the optimum irrigation interval and irrigation time for different field dimensions are summarised in Table 3.3. It is clear from this table that the global minimum cost occurs at dimensions of 250 and 240 m with irrigation interval of 1 day and irrigation time of 18 hours. An irrigation time of 20 hours is also considered but it is infeasible because it has an emitter discharge lower than the minimum allowable value.

The total length of the manifold and laterals are greatly affected by changing the field dimensions. Hence the influence of the field length and field width on the system cost is also examined. The minimum system cost variation in terms of field length is represented in Figure 3.9. Although field lengths of up to 900 m were examined, the results show that the maximum feasible length is 750 m. The reason for this is that as the length of field increases the length of laterals increases as well, consequently, the head loss in laterals increases. A head loss higher than a certain value violates the maximum allowable pressure variation in laterals. The minimum system cost and also minimum pipe costs for each value of field length at the optimum values of irrigation time and irrigation interval are shown in Figure 3.9. It is clear that there is a decline in cost from the lowest field length toward the higher lengths. The system cost reaches a minimum of $27489 (Fx= 250 m) and remains almost constant for lengths between 250 to 300 m, then increases smoothly and reaches a maximum at a length of 750 m.

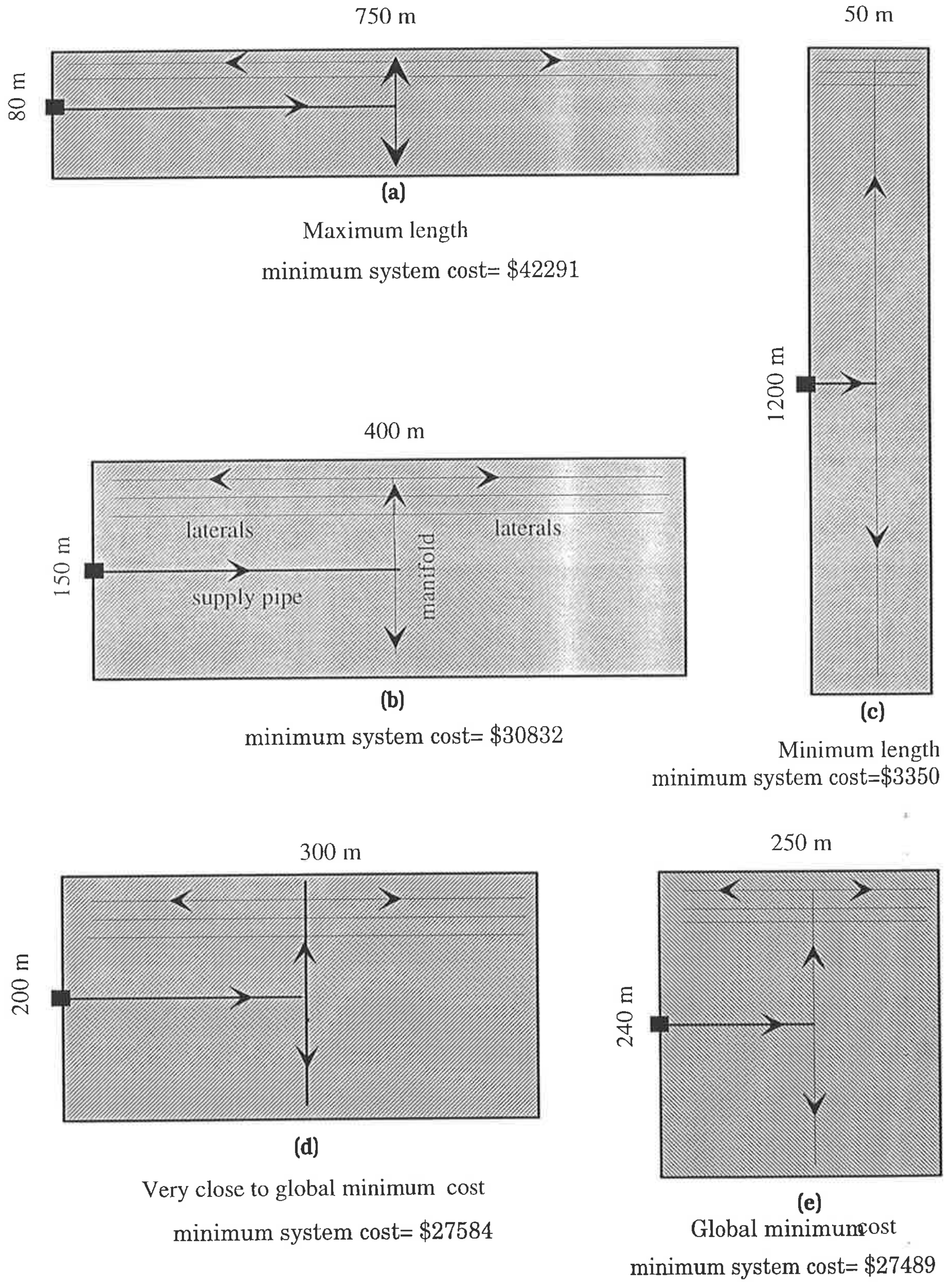


Fig. 3.8 An example of fields with constant area of 6 ha, different dimensions and corresponding minimum costs

TABLE 3.3 Global optimum cost and pipe cost for various field dimensions

field length (m)	field width (m)	irrigation interval (day)	irrigation time (hr)	emitter discharge (L/s)	T(K)/F(I) (hr/day)	pipe cost ($)	system cost ($)
50	1200	1	12	3.16	12	24,612	33,502
100	600	1	10	3.79	10	21,099	30,349
150	400	1	18	2.11	18	19,965	29,073
200	300	1	18	2.11	18	18,274	27,524
250	**240**	**1**	**18**	**2.11**	**18**	**18,311**	**27,489**
300	200	1	18	2.11	18	18,356	27,584
350	171.4	1	16	2.37	16	19,703	28,880
400	150	1	18	2.11	18	21,582	30,832
450	133.3	1	18	2.11	18	23,253	32,373
500	120	1	18	2.11	18	25,348	34,598
550	109.1	1	18	2.11	18	27,025	36,152
550	109.1	1	18	2.11	18	27,025	36,152
600	100	1	18	2.11	18	27,554	36,714
650	92.3	1	18	2.11	18	29,189	38,333
700	85.7	1	18	2.11	18	31,285	40,512
750	80	1	18	2.11	18	33,121	42,291

Figure 3.9 shows that among the local optima the largest one occurs for the largest field length and the lowest optimum cost occurs when the width and length are close to each other, or when the geometry of the field is close to a square. The reason for the highest optimum cost at largest length is that for long pipes the head loss in the laterals is high. As a result, the larger size of laterals is used more than the smaller size. On the other hand, the remaining head loss for the manifold is reduced (see Equation 3.17). Therefore, a larger size of manifold is used as well. In addition, as shown in Figure 3.2 for a long field length the supply pipe which has a larger size than the manifold also has its longest length.

The results represent the another fact that for the large field lengths most of the operating schedules given in this model are infeasible. The infeasible solutions mainly correspond to the low ratios of irrigation time to irrigation interval. Irrigation operation with high irrigation intervals and short duration (low irrigation times) necessitates the use of high flow rates in emitters and also in the pipes. This increases the head loss and violates the pressure constraints.

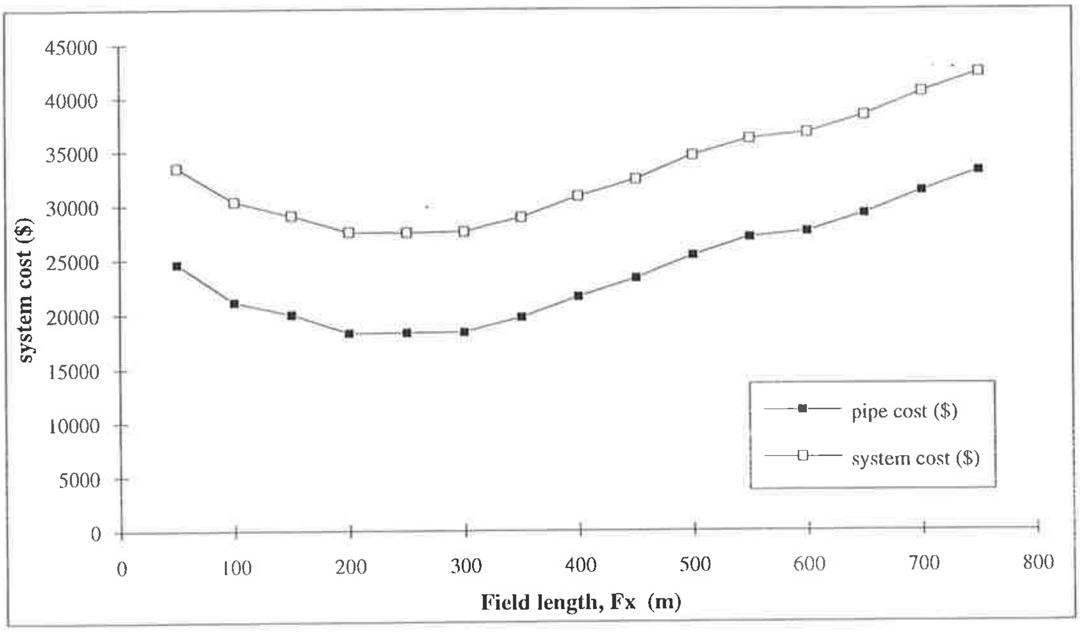


Fig. 3.9 Total minimum system cost and the minimum cost of pipes versus field length for the optimum operation

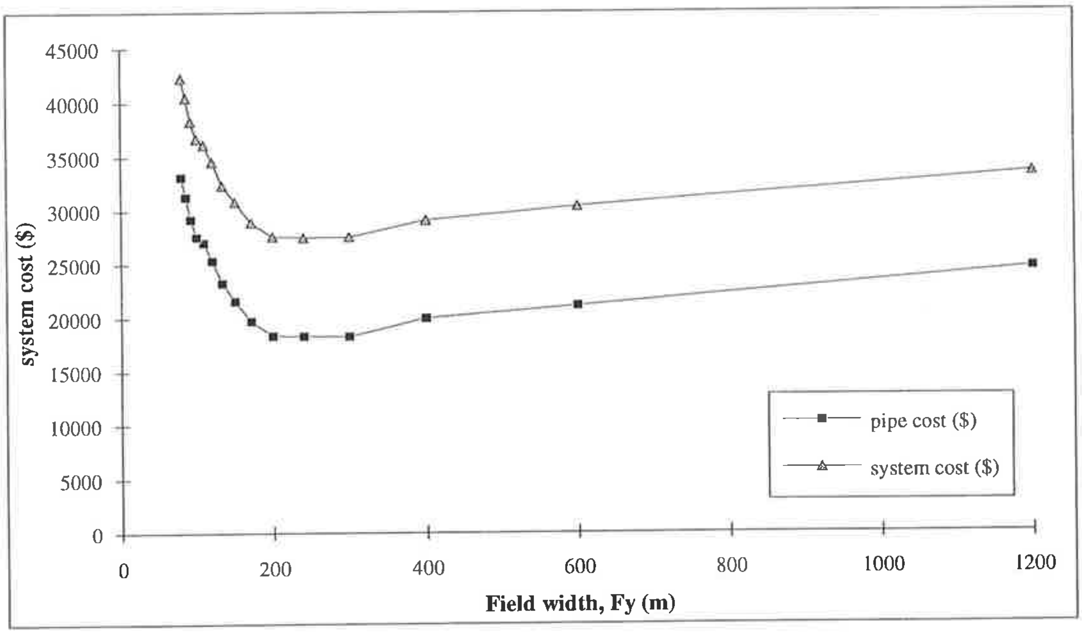


Fig. 3.10 Total minimum system cost and minimum cost of pipes versus field width for the optimum operation

In Figure 3.9 there are two similar graphs, the lower one shows the cost of pipes while the upper one represents the cost of the whole system. The difference between those two parallel graphs shows the constant cost of emitters and the accessories (in this model the cost of emitters and the cost of accessories are assumed constant). In a similar way, the variation of the minimum system cost for each set of field dimensions but against the field widths is shown in Figure 3.10. In this graph the system cost drops from its maximum value which is at the minimum field width with a sharp slope until it reaches the global optimum of $ 27489 at the width of 240 m. It then rises smoothly for greater widths until it reaches a cost of $33502 at the maximum feasible length of 1200 m.

The system cost at the global optimum is $4581 per ha, while for the highest cost (which corresponds to the F_x=750) is $7048 per ha. This represents a 35% saving cost. Although both costs correspond to the local minimum cost the difference shows the considerable reduction in system cost due to employing the optimisation process. The details of system cost with the corresponding designs for the optimum field geometry and two other extreme cases are summarised in Tables 3.4 to 3.6.

TABLE 3.4 Cost of different parts of system at the global optimum (F_x=250m, F_y=240m, $F(I)$=1 day, $T(K)$=18 hr)

	Length (m)	Diameter (mm)	Discharge (L/s)	Head loss (m)	Min. cost ($)
Field	F_x=250 F_y= 240		5.8		**27489**
Larger size lateral	0.00	19	0.00	0.00	0.00
Smaller size lateral	123	13	0.036	0.51	16531
Manifold pipe	118.5	56	2.9	1.49	852
Supply pipe	125	83.8	5.8	1.58	928
Emitters			2.1 L/hr		8928
Accessories				3.2	250

TABLE 3.5 Cost of different parts of system when the length of field is a maximum (F_x= 750 m and F_y = 80 m, $F(I)$=1 day, $T(K)$=18 hr)

	Length (m)	Diameter (mm)	Discharge (L/s)	Head loss (m)	Min. cost ($)
Field	F_x=750 F_y=80		5.8		**42291**
Larger size lateral	261	19	0.109	1.36	22297
Smaller size lateral	112	13	0.033	0.40	6209
Manifold pipe	38.5	83	2.9	0.24	573
Supply pipe	375	102	5.8	1.81	4042
Emitters			2.1 L/hr		8920
Accessories				3.2	250

TABLE 3.6 Cost of different parts of system when the length of field is a minimum (F_x= 50 m and F_y = 1200 m, $F(I)$=1 day, $T(K)$=12 hr)

	Length (m)	Diameter (mm)	Discharge (L/s)	Head loss (m)	Min. cost ($)
Field	F_x=50 F_y=1200		8.42		**33502**
Larger size lateral	0.00	19	0.00	0.00	0.00
Smaller size lateral	23	13	0.011	0.009	15456
Manifold pipe	598.5	83.8	4.21	1.99	8887
Supply pipe	25	102	8.4	0.24	269
Emitters			3.16 L/hr		8640
Accessories				3.2	250

3.9 SUMMARY AND CONCLUSION

In this part of work, a simple drip irrigation system consisting of a control head a supply pipe, a manifold pipe and a number of laterals was modelled and optimised. It will be extended to the more complicated cases in the next chapters. In this work the effect of three main factors including: field geometry, irrigation interval and irrigation time on the minimum cost of a drip irrigation system with one control head was analysed by employing a complete enumeration approach. As discussed previously, the minimum system cost at each field geometry was expected to be obtained at the minimum possible irrigation interval with the longest irrigation time. As a result of employing the higher irrigation times and the lower irrigation intervals the design flow rate is reduced. Consequently, the distribution of irrigation water may be carried out using smaller pipe sizes which leads to the lower system cost.

In this study, in order to avoid either possible clogging of emitters (low flow rate) or soil erosion problem (high flow rate), the allowable flow rate of emitters was limited to change within a limited range between 2 and 24 L/hr which is associated with the irrigation application rate. Therefore for the minimum irrigation interval of 1 day, the highest feasible irrigation time leading to the minimum system cost was 18 hours (Fx = 250 m, Fy =240 m). If the clogging problem was not serious and smaller emitters were allowed to be used a lower system cost could be achieved. Similarly, for the highest irrigation interval of 6 days the minimum feasible irrigation time should be 10 hours to satisfy the emitter flow rate constraint. However, the results indicate that the minimum feasible irrigation time for 6 days was 20 hours. This is due to the pressure variation constraints to limit the pressure drop for achieving an acceptable water distribution uniformity.

The field geometry is the another factor considered in this part of work. The effect of varying the dimensions of the field shows that, the minimum system cost is achieved when the geometry of the field is close to square. The most economical field shape is a rectangle very close to square (F_x= 250, F_y= 240 m). The results indicate that there are other solutions very close to the optimal one in which the dimensions parallel to the laterals (x) are between 0.66 and 1.50 times the perpendicular dimensions (i.e. F_x= 200, F_y= 300m and F_x= 300, F_y=200m). The details in Tables 3.5 and 3.6 show that, for two extreme cases (F_x=50, F_y=1200 m and F_x=750, F_y=80m) the corresponding minimum system costs are \$1002 and \$2466 per ha higher than the global optimum cost which occurred at F_x=250 and F_y=240 m respectively (F_x being the dimension parallel to the laterals). The system cost for the highest field length (F_x = 750 m) is much more than for the highest feasible field widths. This is due to the using the larger size of laterals more than the smaller size and also using the larger size of the manifold and delivery pipe. The results show that for large field lengths most of the operating programs those in which the irrigation time interval ratio ($\frac{T(K)}{F(I)}$) is low are not feasible. In other words for such cases it would not be possible to irrigate infrequently because the high irrigation intervals and the low irrigation times cause the use of high flow rate that violates the pressure constraints. In the next two chapters (4 and 5) the system cost will be examined more comprehensively considering the pumping system and operating costs.

Chapter 4

Optimal Design and Operation of Drip Irrigation Systems on Sloping Lands

4.1 INTRODUCTION

In drip irrigation the objective is to produce each plant with a continuous readily available supply of soil moisture which is sufficient to meet transpiration demand. The ideal drip irrigation system is one in which all emitters deliver the same flow rate in a given irrigation time so that each plant would receive the same quantity of water in an irrigation interval. However, the uniformity of emitter discharge is affected by pressure variations and manufacturing characteristics (Wu et al, 1986). In this study, manufacturing characteristics and the clogging of emitters are assumed to be negligible, while the discharge uniformity of emitters is controlled by hydraulic design. Two basic factors which affect the hydraulic pressure of drip irrigation lines on sloping lands are (i) friction loss due to pipe roughness; and (ii) the energy gain or energy loss due to land slope which is linearly proportional to the slope and length of the lines. The lines which are laid on the down slope gain energy while the lines on the up slope lose energy. In either case the energy is proportional to the length of the pipe. To minimise the pipe sizes, both the laterals and manifold are assumed to be laid on the up and down slopes so that the irrigation water is divided into two opposite directions.

First, the position of the manifold and supply pipes is assumed to be at the centre of the field as shown in Figure 4.1a. The analysis is then carried out on the basis of finding the optimum position of manifold and supply pipe as shown in Figure 4.1b. In both cases the system cost is evaluated by identifying the optimum size for both segments of manifold, supply pipe, the optimum length of two given sizes of laterals and the optimum value for the present value of operating cost and an appropriate size for the pump. The system cost is also evaluated by considering different irrigation intervals, and irrigation times, different slopes in two directions and variations in the ground water level. In addition, the optimum design is examined while a number of different allowable working pressures are considered along the laterals.

4.2 PIPE CONFIGURATION

The configuration of the piping system is similar to the piping system described in Chapter 3. The water is pumped from the source to the supply pipe and then to the manifold. In order to minimise the friction loss, the water is divided into two parts, one part flowing through the pipe on the up slope and the other part through the pipe on the down slope. Both segments of the manifold feed the corresponding laterals. The laterals distribute the water through the emitters at a slow rate of application. In the analysis of the optimum design of drip systems on sloping lands two alternatives are considered. In the first alternative, the position of the manifold and the supply pipe is fixed while in the second alternative, the model identifies the optimum position of manifold and supply pipes. As a result, the length of laterals and the manifold on the up slope and down slope are not the same for a constant discharge. In fact, the position of manifold and supply pipes is affected by the slope as shown in Figure 4.1b.

4.3 SYSTEM COMPONENTS

As it is clear from Figures 4.1a and 4.1b, the piping system and the pump are the main components of the system. The piping system consists of a supply pipe, a manifold and a set of laterals and emitters. The emitters which dissipate pressure and also discharge water to the soil are normally installed on laterals with the equal spacing. The pump system consists of a turbine pump with a series of centrifugal impellers located below the water

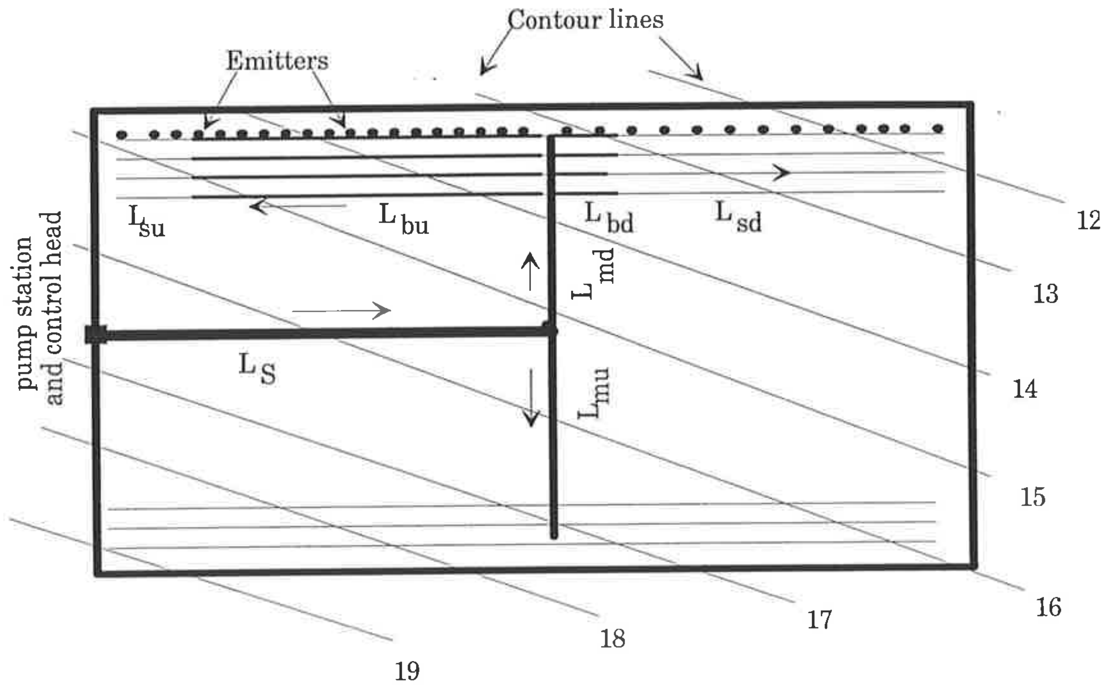


Fig. 4.1a Pipe configuration when the manifold and supply pipes are located at the centre of field

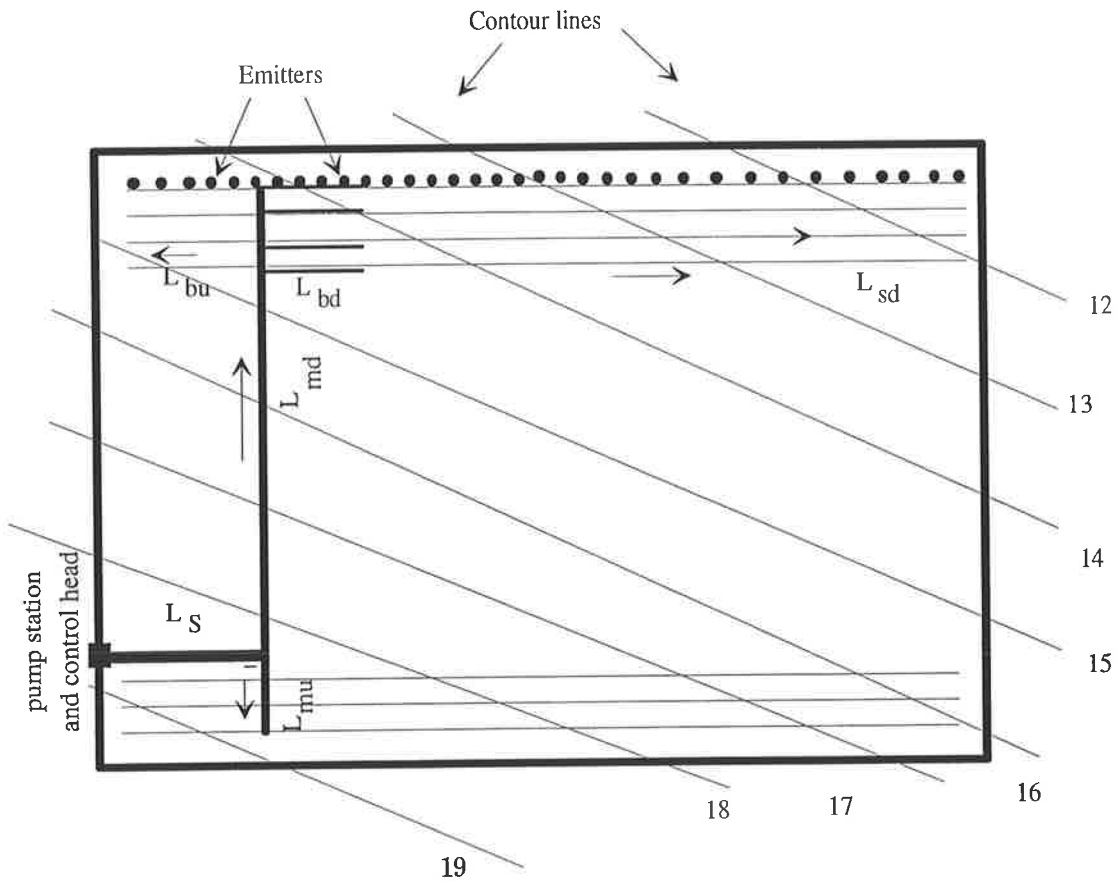


Fig. 4.1b Pipe configuration when the manifold and supply pipes are located at the optimum positions

table and connected to a vertical shaft which extends through a discharge tee, or head, at the surface. The shaft, is rotated by a vertical shaft electric motor, or a vertical belt drive from engine or motor. More details relating to the pumping system are given in Chapter 5. The auxiliary components including a filter, a fertiliser injector, a pressure regulator, an on-off valve and a flow meter are essential for the operating purposes and increasing the system reliability. An example of a drip irrigation system with the auxiliary components is illustrated in Figure 4.2.

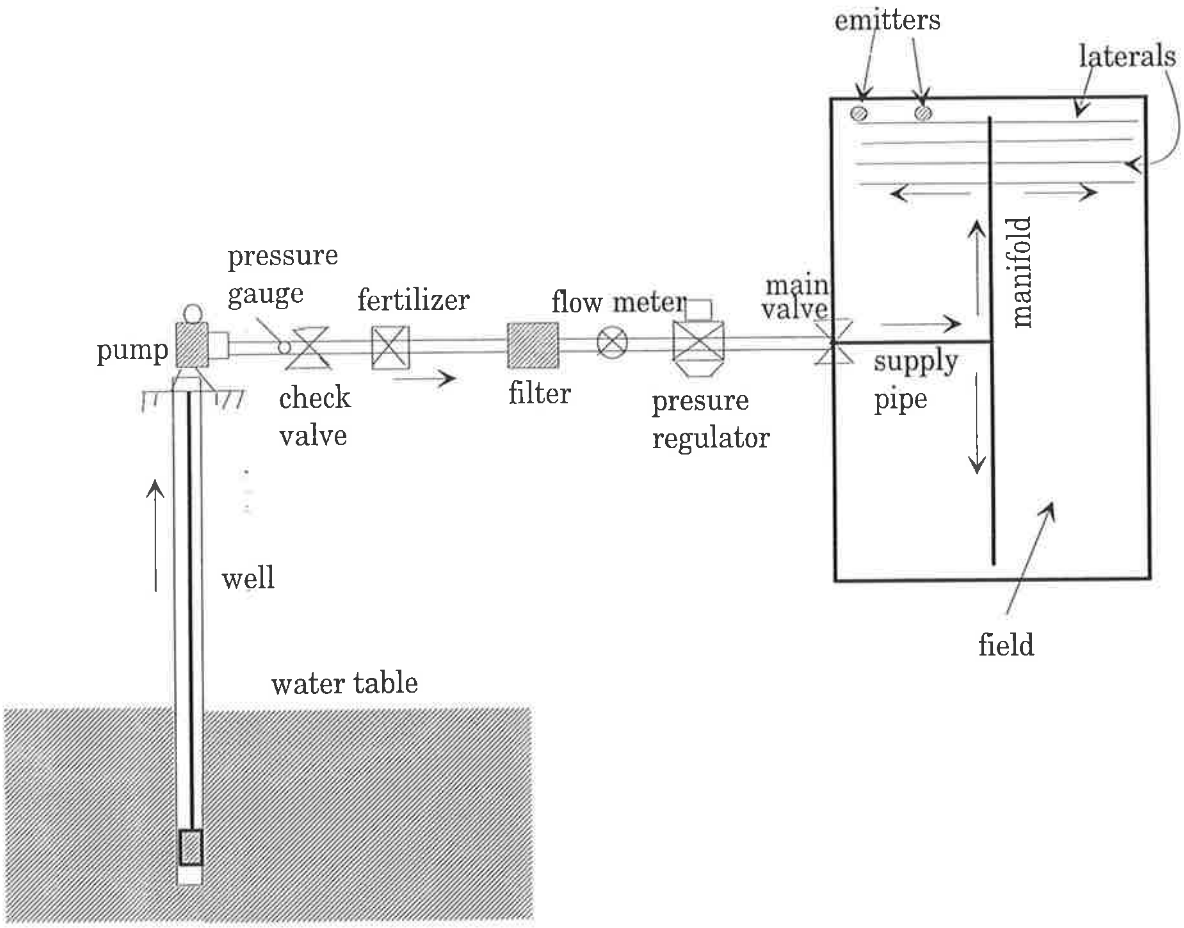


Fig. 4.2 An example of a drip irrigation system with associated components

4.4 MODEL ASSUMPTIONS

The optimum solution considering the minimum cost of different parts of system is obtained under various possible conditions. These conditions are based on the following assumptions:

- **Operating Conditions**

The model is evaluated for different irrigation intervals from 0.5 to 3 days with an increment of 0.5 day (0.5, 1.0, 1.5, 2.0, 2.5, 3.0), and different irrigation times from 4 to 12 hours with an increments of 2 hours (4, 6, 8, 10, 12). Each feasible combination of intervals and times yielded to a new design discharge (loading case) which may change the system cost.

- **Pipe Sizes**

Two small available sizes are assumed for the laterals (19 and 28 mm) and also 14 discrete sizes are assumed for both segments of the manifold and for the supply pipe. The discharge uniformity within the laterals is controlled by selecting the appropriate lengths for two given sizes and also within the manifold it is also controlled by the appropriate size for each segment among the given range.

- **Slopes**

Two different uniform slope patterns are considered in the X and Y directions. The slope of the field along the supply and lateral lines (X direction) is assumed to vary from 0.0 to 1.0 m per 100 m and along the manifold from 0.0 to 2.0 m per 100 m. The system cost is examined for any possible combination of slopes in both directions. Obviously, for the steep slopes even for some above slopes the uniformity constraint does not meet the requirements.

- **Position of the Manifold and Supply Pipes**

For each particular slope in the X and Y directions it is possible to identify the optimum position for the manifold and supply pipes which is affected by the slope, length of pipes, discharge, and the allowable pressure variation within the system. The position of the supply pipe is allowed to vary from the centre of the field with steps of 8 m in the Y direction, while the position of the manifold is allowed to vary from the centre with steps of 12 m in the X direction (Figures 4.1a and 4.1b).

- **Pressure Variation**

As explained previously in this study, the discharge uniformity is controlled by pressure variation along multiple outlet pipes (manufacturing variation is assumed to be negligible).

As the allowable pressure variation decreases, a higher discharge uniformity is achieved; however the system cost may be increased due to the use of larger pipe sizes.

- **Different Delivery Pipe Sizes**

The system cost is evaluated for a given range of discrete pipe sizes for the supply pipe which delivers water from the source node to the manifold. The effect of delivery pipe sizes is significant for the systems in which the source node is far from the distribution valve.

- **Ground Water Level**

Normally, in an irrigation area the groundwater level could vary over a wide range. In this study, it is assumed to vary from 0.0 to 100 m with a step of 10 m and the total system cost is evaluated for this range.

4.5 HYDRAULICS OF DRIP IRRIGATION SYSTEMS ON SLOPING LANDS

"Flow in the drip irrigation lines is hydraulically steady, spatially varied flow" (Jensen, 1983; Wu et al, 1986). As can be seen in Figure 4.1a the total discharge in the manifold and laterals decreases along their length. Both the manifold and laterals as multiple outlet pipes are considered to have similar hydraulic characteristics (i.e. smooth plastic pipe, mild slope, controlled pressure, similar patterns for pressure variation along the pipes and flow variation from the outlets). The Hazen-Williams or the Darcy-Weishbach equation may be used to determine the head loss in all pipes in the system.

Although a discharge correction factor for the multiple outlet pipes is recommended by some authors (Oron and walker, 1981, James, 1988) in this study, a set of equations for the head loss, pressure and also the position of the minimum and the maximum pressure for the multiple outlet pipes are developed (Section 4.7). The head loss in the emitter connections and pipe fittings are not included in this analysis. However, since the outlets are spaced closely on the laterals and manifold, the Hazen-Williams roughness coefficient for these pipes are considered to be 130 and 140 respectively. This might compensate for the minor losses compared to 150 that is assumed for the supply pipe. According to Jensen (1983) for a maximum velocity of 1.5 m/s in plastic pipes, the value of Hazen-Williams roughness coefficient depends on pipe sizes. He recommends using a Hazen-Williams roughness coefficient of 130 for 13-15 mm diameter, 140 for 18-19 mm and 150 for 25-27 mm pipe.

4.6 HYDRAULICS OF EMITTERS

Emitters are the last component of an trickle irrigation system and are usually located on the laterals. In drip lines water flows out through the emitters at a slow application rate. In the design of a drip irrigation system, it is necessary to make sure that an appropriate size is chosen for the emitters. Very large emitters with a high flow rate may exceed the infiltration capacity of the soil and subsequently cause water loss and soil erosion, while very small size emitters are susceptible to clogging or build up of chemical deposits. As previously mentioned, the two main factors which affect the emitter discharge uniformity are manufacturing and hydraulic variations. The emitter coefficient of the manufacturing variation is identified by the manufacturer, or it can be estimated from the measured discharge of a sample set of emitters operated at a constant pressure head. This coefficient can be determined using the following equation (National Engineering Hand book, 1984):

$$\mu = \frac{\sqrt{\dfrac{q_1^2 + q_2^2 + \ldots\ldots q_n^2 - n\bar{q}^2}{n-1}}}{\bar{q}} \tag{4.1}$$

where μ = emitter coefficient of manufacturing variation; $q_1, q_2 \ldots\ldots q_n$ = individual emitter discharge rate (L/hr); n = number of emitters in sample; $\bar{q}$ = average flow rate of the emitters in the sample (L/hr).

The second factor represents the sensitivity of emitters to pressure variation and is the most important characteristic of emitters. It indicates a relation between pressure and discharge as illustrated in Figure 4.3 which displays the sensitivity for different emitters. Most emitters can be hydraulically classified as orifice, long path, vortex or pressure compensating emitters. The hydraulic characteristics of each emitter are identified by the type of flow inside the emitters as characterised by the Reynold number(R_e). Reynold number is directly related to the diameter of emitter (D); velocity of flow (V) and

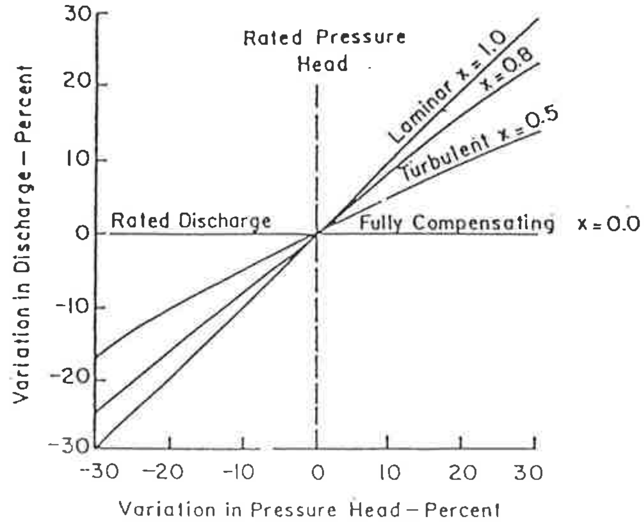


Fig. 4.3 Variation in emitter flow rate resulting from variation in the pressure head for different flow regimes (National Engineering Book, 1984)

inversely related to the kinematic viscosity (ν), ($R_e = \frac{DV}{\nu}$). These flow regimes are usually characterised as laminar flow in which $R_e \leq 2,000$; unstable flow $2,000 \leq R_e \leq 4,000$; partially turbulent flow $4,000 \leq R_e \leq 10,000$; and fully turbulent flow in which $R_e \geq 10,000$.

Orifice type emitters are assumed to be used in this study. The flow regime in an orifice emitter is usually fully turbulent (Jensen, 1983). The general flow equation for this type of emitter in fully turbulent conditions can be expressed as :

$$q = AK\sqrt{2gh} \tag{4.2}$$

where q= emitter discharge (m3/s); A= orifice cross section area (m2); K= orifice coefficient, which depends on the characteristics of the nozzle and ranges from 0.6 to 1.0 (National Engineering Hand Book, 1984); h= pressure head at the orifice (m); g =

acceleration due to gravity (m/s^2). The flow rate from the orifice emitters depends on the geometry of the material and the operating pressure considering different flow regimes. Usually it is determined by empirical methods as a function of operating pressure as follows:

$$q = ch^{\alpha} \tag{4.3}$$

where $c =$ discharge coefficient that characterises emitter dimensions for a lateral (c is generally considered as a constant); $h =$ the pressure head in the lateral pipes at the emitter under consideration; $\alpha =$ emitter discharge exponent which measures the shape of the discharge pressure curve. As Keller and Karmeli (1974) explain the value of α characterises the flow regime of emitters. $\alpha = 0.5$ for fully turbulent flow, $0.5 \le \alpha \le 0.7$ for partially turbulent flow, $0.7 \le \alpha \le 1.0$ for unstable flow regime and $\alpha = 1.0$ for laminar flow.

The coefficients c and α may be determined by plotting q versus h on a log-log plot. As shown in Figure 4.3 for turbulent-flow emitters, the discharge varies with the square root of the pressure, while in laminar-flow emitters it changes linearly, i.e. doubling the pressure will double the discharge. Hence in order to keep the discharge variation less than 5%, the allowable variation in the operating pressure head within the system is often kept less than 5% for laminar-flow emitters and 10% for turbulent-flow emitters (National Engineering Hand Book, 1984). In this work, the pressure variation along the laterals on both the up and down slopes is allowed to change less than 10%. Under such pressure variation for the orifice type emitters the discharge variation will be kept less than 5%.

4.7 HYDRAULICS OF MULTIPLE OUTLET PIPES

Irrigation systems are designed to give a reasonably uniform water distribution over the irrigation area. The multiple outlet pipes within the system are designed so that the variations in outflow between the individual outlets should not be excessive. This can be achieved by applying a limited allowable pressure variation along the multiple outlet pipes. Two basic factors affecting the pressure head are head loss due to pipe roughness and gravity due to the land slope.

Usually the outlets along the pipes are spaced uniformly and the discharge flowing from each outlet as shown in Fig. 4.4 is assumed to be identical. Therefore the flow along the distributor pipe decreases. This may be approximated as a linear decrease along the length of the pipe. In the following sections the equations governing head loss, the pressure head and the position of maximum and minimum pressure in multiple outlet pipes are developed.

4.7.1 Head Loss and Pressure Head in Single Size Pipe with Zero Discharge at the End

The minimum pressure on up slope simply occurs at the end of the pipe, but on the down slope it is not always so obvious. Its position depends on the slope and the head loss. The maximum pressure on an up slope always occurs at the beginning the pipe and on a down slope it occurs either at the beginning or at the end of the pipe. The general form of the Hazen-Williams equation used in hydraulic analysis is shown as Equation 3.15 with $F(n_0)=1$ if there are no outlets from the pipe.

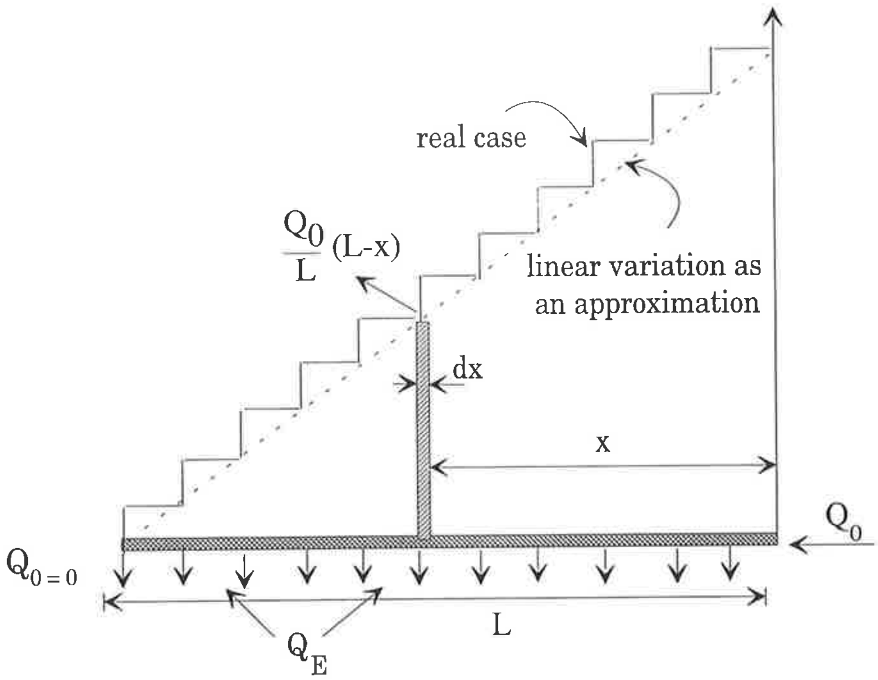


Fig. 4.4 Flow in the multiple outlet pipes decreases linearly with respect to the pipe length

The head loss in a small section of pipe of length dx as shown in Figure 4.4, may be calculated using Equation 3.15 as follows:

$$hl_{dx} = \frac{10.68dx\left(\frac{Q_0}{L}(L-x)\right)^{1.852}}{C^{1.852}D^{4.87}} \tag{4.4}$$

where hl_{dx} = head loss for length dx (m); dx = small length of pipe (m); Q_0 = initial flow rate in pipe (m^3/s); x = distance from the head end (m).

The head loss over the length of x can be obtained by integrating Equation 4.4 through the process shown in Appendix A. The final simplified equation may be expressed as:

$$hl_x = 3.745\frac{Q_0{}^{1.852}L}{C^{1.852}D^{4.87}}\left[1-\left(\frac{L-x}{L}\right)^{2.852}\right] \tag{4.5}$$

where hl_x = head loss for length x (m).

Equation 4.5 represents the head loss at any point of a single size multiple outlet pipe with x measured from the upstream end of the pipe.

Due to the effect of gravity on down slopes the minimum pressure will occur somewhere between the beginning and the end of pipe, while the maximum pressure will occur either at the beginning or at the end of pipe. The equation determining the position of the minimum pressure is developed when the derivative of the pressure equation equals zero. The pressure head equation along the pipe on sloping lands (as shown in Figure 4.5) may be represented as below:

$$H_x = H_0 + S.x - hl_x \tag{4.6}$$

where H_x = pressure head at distance x along the pipe (m); H_0 = input pressure at the beginning of the pipe (m); S = slope of the pipe (m/m) with negative sign for an up slope and positive for a down slope (the variation of pressure head on the basis of the Equation 4.6 is shown in Figure 4.5).

Considering Equation 4.5 representing the head loss at distance x, and Equation 4.6 representing the pressure head at distance x, the equation for the position of the minimum pressure for a down slope may be developed as follows:

$$H_x = H_0 + S.x - 3.745\frac{Q_0^{1.852}L}{C^{1.852}D^{4.87}}\left[\frac{1-(L-x)^{2.852}}{L^{2.852}}\right] \quad (4.7)$$

$$\frac{d(H_x)}{d_x} = +S - 3.745\frac{Q_0^{1.852}L}{C^{1.852}D^{4.87}}\left[-2.852\left(-\frac{1}{L}\right)\left(\frac{L-x}{L}\right)^{1.852}\right] \quad (4.8)$$

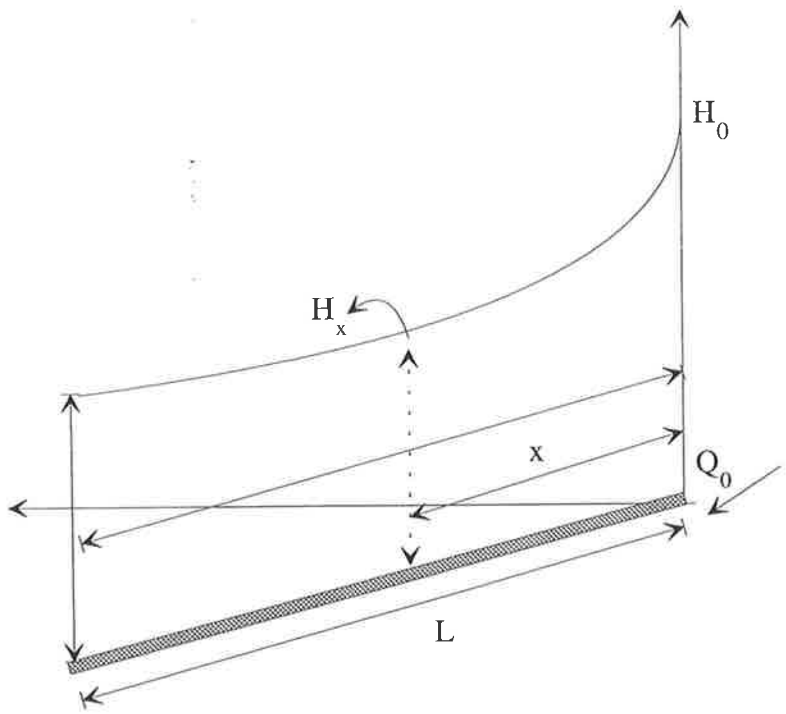


Fig. 4.5 Variation of pressure head in multiple outlet pipes on down slopes

$$\frac{d(H_x)}{d_x} = +S - \frac{10.685Q_0^{1.852}}{C^{1.852}D^{4.87}}\left[\left(\frac{L-x}{L}\right)^{1.852}\right] = 0 \quad (4.9)$$

$$x = L - \left[\frac{+SC^{1.852} D^{4.87} L^{1.852}}{10.68 Q_0^{1.852}} \right]^{\frac{1}{1.852}} \tag{4.10}$$

Equation 4.10 represents the position of the minimum pressure head along a multiple outlet pipe with a down slope.

4.7.2 Head Loss and Pressure Head in Multiple Outlet Pipes with Continuous Flow at the End

As previously mentioned, two pipe sizes are assumed for each lateral, and the optimum length of each size is one of the decision variables. Clearly, for two-size pipes the flow will not be zero at the end of the first segment. Considering the continuous flow rate at the end of a pipe as shown in Figure 4.6 the head loss at x is given below:

$$hl_x = \frac{3.745L}{C^{1.852} D^{4.87} (Q_0 - Q_{ss})} \left[Q_0^{2.852} - \left(Q_{ss} + \frac{Q_0 - Q_{ss}}{L}(L - x) \right)^{2.852} \right] \tag{4.11}$$

where Q_{ss} = discharge at the start of the smaller size segment of laterals (m^3/s).

Development of Equation 4.11 showing the head loss at distance x for a multiple outlet pipe with continuous flow at the end is carried out by integration of the head loss over distance x. This process with the corresponding mathematical expressions is given in Appendix B.

Clearly, for $x = L$, the head loss in the full length of pipe is expressed as below:

$$hl_L = 3.745 \frac{L}{C^{1.852} D^{4.87} (Q_0 - Q_{ss})} \left[Q_0^{2.852} - Q_{ss}^{2.852} \right] \tag{4.12}$$

Equation 4.12 is used to find the head loss in the first segment of 2-size laterals in this model.

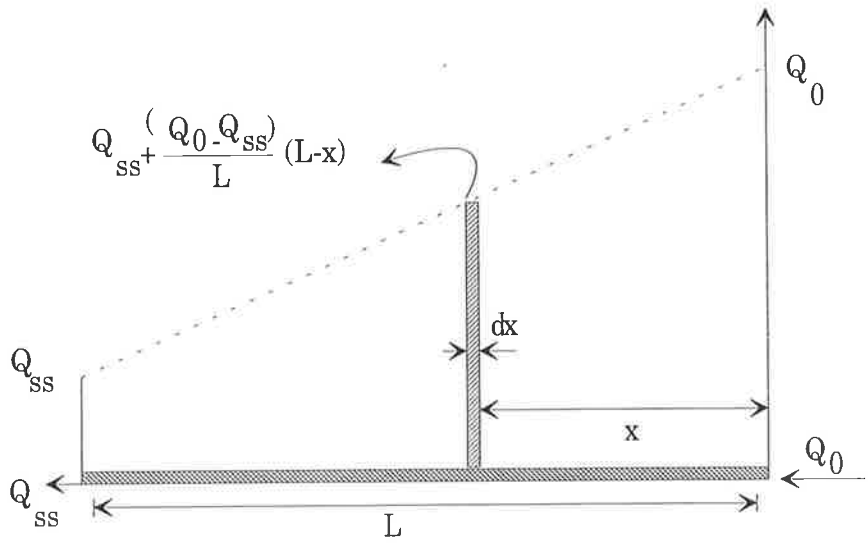


Fig. 4.6 Variation of discharge in the multiple outlet pipes with continuous flow rate at the end

Considering the head loss at x, (hl_x) the pressure head at any distance of multiple outlet pipes with continuous discharge at the end can be calculated as follows:

$$H_x = H_0 + S.x - \frac{3.745\,L}{C^{1.852} D^{4.87} \left(Q_0 - Q_{ss}\right)} \left[Q_0^{2.852} - \left(Q_{ss} + \frac{(Q_0 - Q_{ss})}{L}(L-x) \right)^{2.852} \right]$$

(4.13)

For $x = L$ the pressure head at the end of the first segment (larger size segment) shown in Fig. 4.6 may be expressed as follows:

$$H_L = H_0 + SL - \frac{3.745L}{C^{1.852} D^{4.87} \left(Q_0 - Q_{ss}\right)} \left[Q_0^{2.852} - Q_{ss}^{2.852} \right] \quad (4.14$$

4.7.3 Minimum and Maximum Pressure Head along Multiple Outlet Pipes with Two Sizes

The minimum pressure on the down slope may occur either in the smaller size or in the larger size segment. The corresponding equation for both cases may be developed as follows:

Case 1: When the minimum pressure occurs in the second segment

As shown in Figure 4.7 the pressure head at distance x, (H_x) is given by :

$$H_x = H_0 + S.x - hl_x \tag{4.15}$$

$$hl_x = T_{hl} - hl_{L-x} \tag{4.16}$$

$$T_{hl} = hl_{bs} + hl_{ss} \tag{4.17}$$

where T_{hl} = total head loss in both segments (m); hl_{L-x} = head loss in length ($L-x$); hl_{bs}, hl_{ss}= head loss in the larger and smaller size of the laterals respectively (m).

Head loss for the larger size of laterals was already developed for pipes with continuous flow at the end (Equation 4.12) this may be written as:

$$hl_{bs} = \frac{3.745 L_{bs}}{C^{1.852} D_{bs}^{4.87} (Q_0 - Q_{ss})} \left[Q_0^{2.852} - Q_{ss}^{2.852} \right] \tag{4.18}$$

Using Equations 4.15 and 4.16 the pressure along the pipe will be expressed as:

$$H_x = H_0 + S.x - \frac{3.745}{C^{1.852}} \left[\frac{L_{bs}\left(Q_0^{2.852} - Q_{ss}^{2.852}\right)}{D_{bs}^{4.87}\left(Q_0 - Q_{ss}\right)} + \frac{L_{ss}\left(\frac{Q_0}{L} L_{ss}\right)^{1.852}}{D_{ss}^{4.87}} - \frac{(L-x)\left(\frac{Q_0}{L}(L-x)\right)^{1.852}}{D_{ss}^{4.87}} \right] \tag{4.19}$$

Note that the flow in the pipe at the junction of the larger and smaller sizes is $(\frac{Q_0 L_{ss}}{L})$.

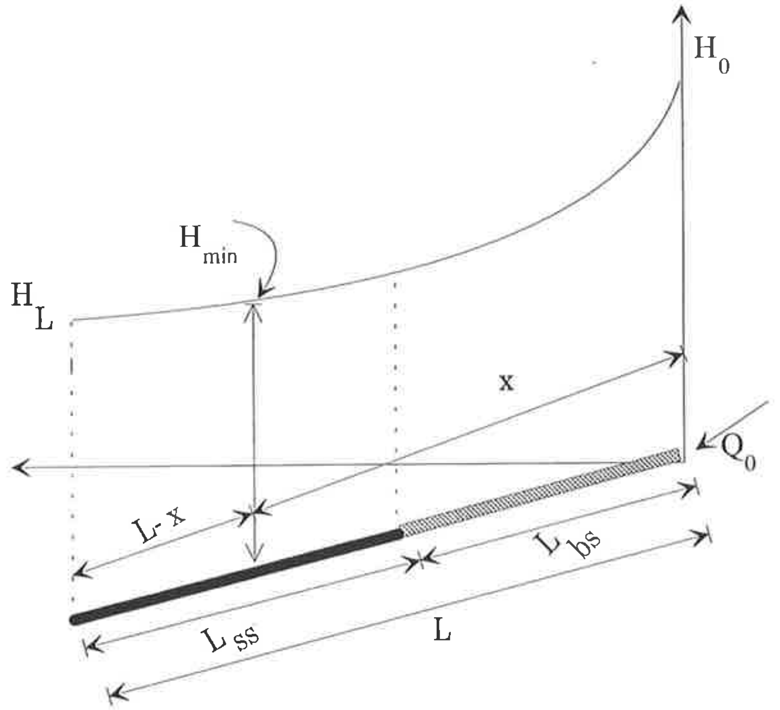


Fig. 4.7 Pressure head for 2-size multiple outlet pipes when the minimum pressure occurs in the second segment

The position of the minimum pressure may be obtained by assuming the derivative of the pressure equation (Equation 4.19) equals zero as follows:

$$\frac{\partial H_x}{\partial x} = S - \left[\frac{10.68\left(\frac{Q_0}{L}\right)^{1.852}}{C^{1.852} D_{ss}^{4.87}} (L-x)^{1.852} \right] = 0 \tag{4.20}$$

$$x = L - \left[\frac{S C^{1.852} D_{ss}^{4.87} L^{1.852}}{10.68 Q_0^{1.852}} \right]^{1/1.852} \tag{4.21}$$

In this equation x represents the position at which the minimum pressure occurs, (within the smaller pipe size segment). In some cases, the maximum pressure may occur either at the beginning or at the end of pipe.

Case 2: When the minimum pressure occurs in the first segment

In this case (Figure 4.8) the equation representing the position of the minimum pressure in two-size multiple outlet pipes on down slopes is developed. Equation 4.11 will be used. It represents the head loss in pipes with continuous flow rate (in the larger size segment).

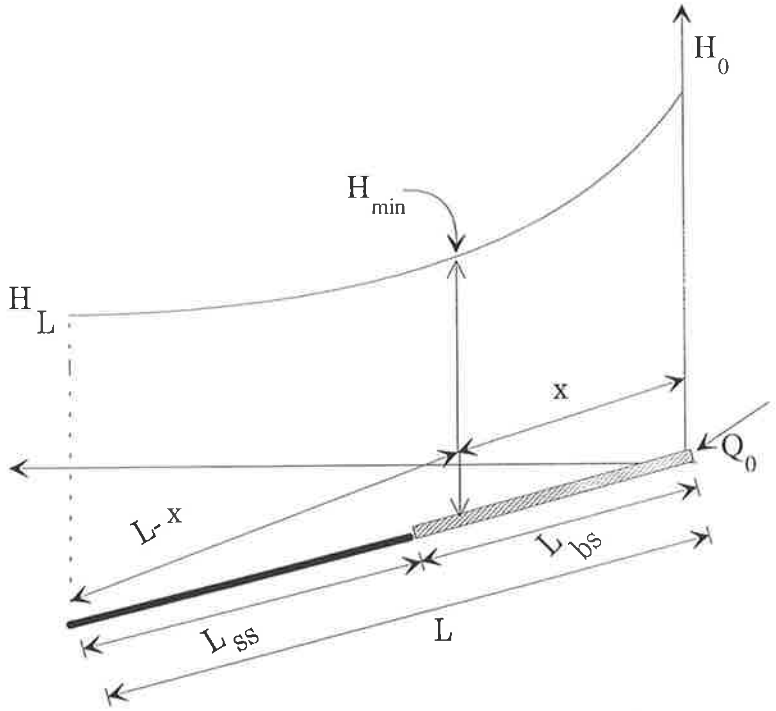


Fig. 4.8 Pressure head at 2-size multiple outlet pipes when the minimum pressure occurs in the first segment

After simplification, the following equation is obtained.

$$hl_x = \frac{3.745 Q_0^{1.852}}{C^{1.852} D_{bs}^{4.87} L^{1.852}} \left[L^{2.852} - (L-x)^{2.852} \right] \qquad (4.22)$$

the pressure head at the x then may be as follows:

$$H_x = H_0 + S.x - 3.745 \frac{Q_0^{1.852}}{C^{1.852} D_{bs}^{4.87} L^{1.852}} \left[L^{2.852} - (L-x)^{2.852} \right] \qquad (4.23)$$

In the similar way to the previous case, the position of the minimum pressure in the larger size of laterals on the down slope will be as follows:

$$x = L - \left[\frac{SC^{1.852} D^{4.87} L^{1.852}}{10.68 Q_0^{1.852}} \right]^{0.54} \tag{4.24}$$

4.8 FORMULATION OF THE MODEL

As previously discussed a well designed drip irrigation system should provide a reasonably uniform distribution of water throughout the field. Apart from the manufacturing characteristics of emitters and clogging problems, two main factors affecting the uniformity of water distribution are head loss and the slope of lines. In Chapter 3, hydraulic analyses in pipes were carried out for flat terrain and only the effect of head loss was considered. However, in this model, the same configuration of a piping system within the subunit on uniform sloping land is analysed.

4.8.1 Energy Gradient Line for the Multiple Outlet Pipes

The total energy at any section of a trickle line is given by the energy equation as:

$$H_0 = Z + H + \frac{V^2}{2g} \tag{4.25}$$

where H_0= total energy at the upstream end of the line (m); Z = elevation or potential energy (m); H = pressure head (m); $\frac{V^2}{2g}$ = velocity head (m); (V = velocity of water in the pipe (m/s); g = acceleration due to gravity (m/s^2).

As shown in Figure 4.9, the energy gradient line in multiple outlet pipes (laterals and manifold) is not a straight line but an exponential type curve resulted from combination of energy gain or loss due to the slope and energy loss due to the head loss as indicated by Equation 4.6.

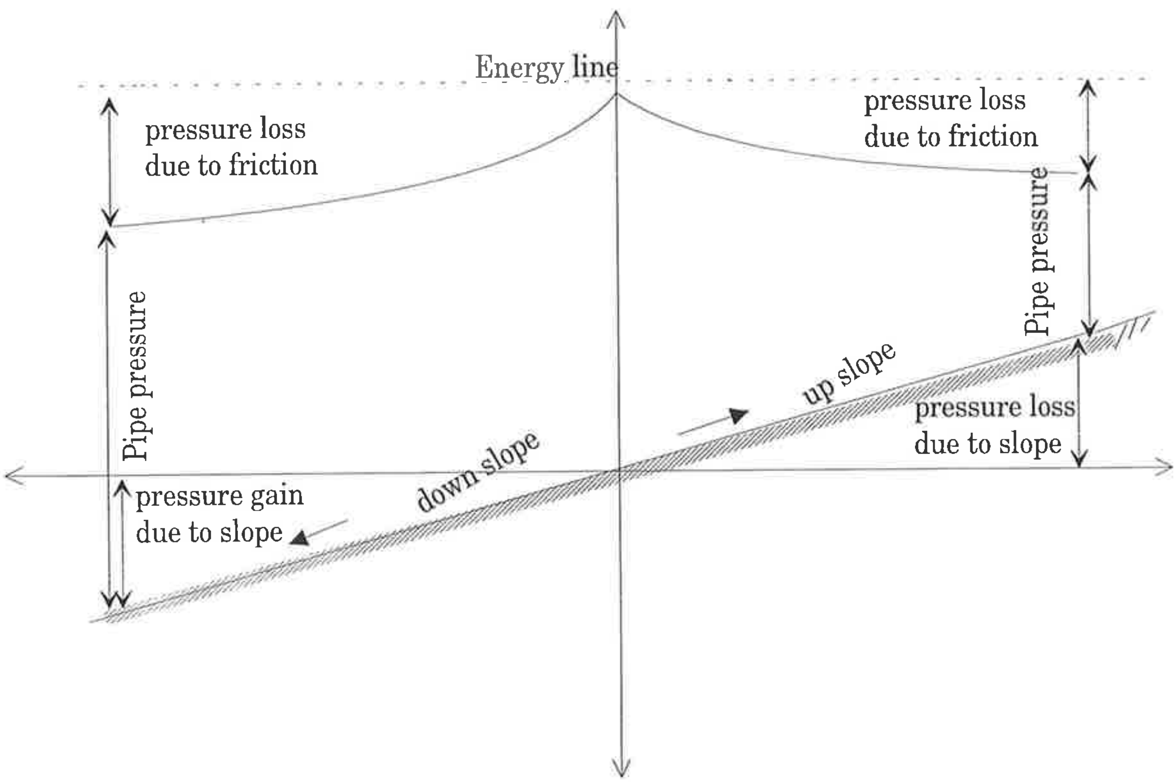


Fig. 4.9 The energy gradient line in multiple outlet pipes on up and down slopes

In this study, the Hazen-Williams equation is utilised to calculate the pressure drop and consequently the pressure head in the pipes. The Hazen-Williams equation (Equation 4.4) should be modified for multiple outlet pipes and pipes with more than one size which are used in drip irrigation systems. The full mathematical process leading to a set of modified equations for the head loss and the pressure head in multiple outlet pipes was presented in Section 4.7.

4.8.2 Pressure in Laterals

The analysis of pressure head along the laterals and the evaluation of the optimum lengths of two given sizes are carried out for two cases. First, for a constant position of the manifold at the centre of field supplying two symmetrical laterals with the equal lengths on up and down slopes (Figure 4.1a), then for a variable position of the manifold supplying a set of symmetrical laterals with different lengths on either side of manifold as shown in Figure 4.1b.

The uniformity of discharge along the laterals is controlled by the pressure head. As shown in Figure 4.9 and Equation 4.6, the maximum pressure in laterals occurs either at the split point (very close to manifold) or at the end of the pipe on the down slope, but the variation of pressure is not similar on either side of the manifold. With symmetrical laterals on either side of manifold pipe friction causes a drop in the pressure on both sides while gravity causes an increase in the pressure on the down slope and a decrease in pressure on the up slope (see Figure 4.9).

Since the total length and the given sizes of laterals are constant the pressure is controlled by selecting an appropriate length for each given size. Clearly, the length of the larger size causing less head loss is expected to be increased on the up slope and the length of the smaller size causing more head loss on the down slope will likewise be greater (as shown in Figure 8.10). This depends on the slope, discharge, pipe roughness, the allowable pressure variation and the total length of pipe. Since one of the primary aims is to achieve an optimum solution with a desired discharge uniformity the length of the smaller and larger sizes can also be varied by changing the allowable pressure variation. The same analysis is carried out assuming different positions for the manifold which affect the lengths of laterals in either side of manifold and consequently, the optimum length of segments of the laterals. In the current model, the overall pressure variation is assumed to be less than 25% within the field.

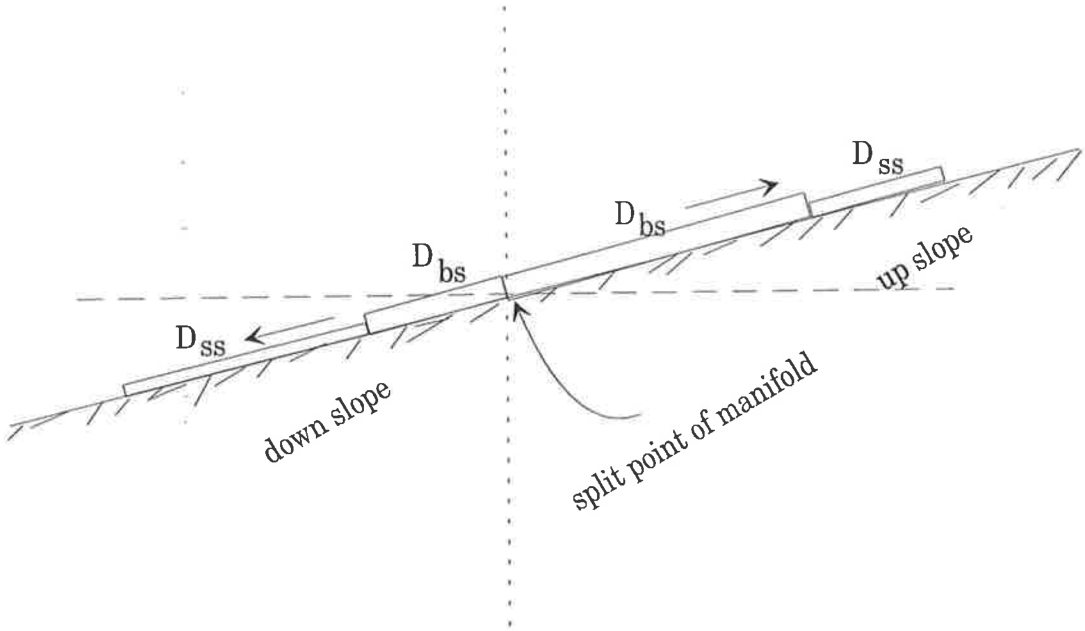


Fig. 4.10 A schematic diagram of 2 given sizes for laterals laying up and down slopes

4.8.3 Pressure Variation for Multiple Outlet Pipes on Sloping Lands

The change in energy with respect to the length of pipe may be expressed by taking the derivative of Equation 4.25 with respect to length as follows:

$$\frac{\partial H_0}{\partial L} = \frac{\partial Z}{\partial L} + \frac{\partial H}{\partial L} + \frac{\partial\left(\frac{V^2}{2g}\right)}{\partial L} \tag{4.26}$$

where $\frac{\partial H_0}{\partial L}$ = the slope of the energy line; $\frac{\partial Z}{\partial L}$ = the slope of the pipe; $\frac{\partial H}{\partial L}$ = the pressure variation with respect to the length and $\frac{\partial(\frac{V^2}{2g})}{\partial L}$ = the change in velocity head with respect to length.

The energy line with corresponding components at two sections of a pipe (Sections 1 and 2) is shown in Figure 4.11. The change in velocity head due the low outflow from the emitters may be ignored.

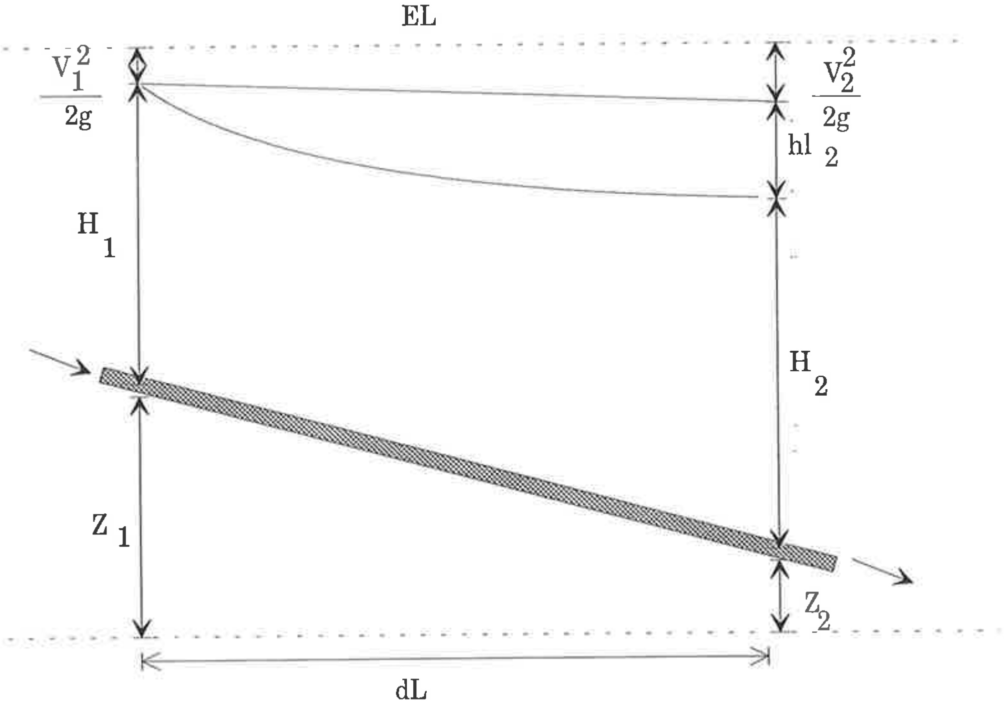


Fig. 4.11 Energy line within a pipe laying on sloping lands

As a result, the change in energy shown in Equation 4.26 can be expressed as below:

$$\frac{\partial H_0}{\partial L} = \frac{\partial Z}{\partial L} + \frac{\partial H}{\partial L} \tag{4.27}$$

The slope of the energy line may be expressed as $\frac{\partial H_0}{\partial L} = -S_e$; the minus sign indicates the fact that the energy in the pipe decreases as the length of pipe increases (Figure 4.11). Similarly, the slope of the pipe (gravity slope) may be expressed as $\frac{\partial Z}{\partial L} = S_0$; where S_0 is positive for an up slope and negative for a down slope. Consequently, the pressure variation from Equation 4.27 can be written as follows:

$$\frac{\partial H}{\partial L} = -S_0 - S_e \tag{4.28}$$

As shown above, the change in pressure in a pipe is a linear combination of the pipe slope and the energy slope. Equation 4.28 is one of the basic equations used to identify the pressure head including the maximum and the minimum pressure in the lateral and manifold pipes in this model. In order to determine the pressure along the pipes, the length of each pipe is divided into 20 equal divisions each one being 5% of the total length. The pressure at each division is then identified by determining the head loss due to the friction and the energy gain or loss due to the slope as follows:

$$H_i = H_0 - \Delta H_i - \Delta H_i' \tag{4.29}$$

where H_i = pressure head at any given length ratio i (m), i = length ratio ($i = \frac{l}{L}$); H_0= input pressure (m); ΔH_i = total head loss at i, (m); $\Delta H_i'$ = potential energy gain (with minus sign) or potential energy loss (with plus sign) at i (m). Equation 4.29 may be expressed in terms of the head loss ratio and energy gain or energy loss ratio at a given i, as follows:

$$H_i = H_0 - \frac{\Delta H_i}{\Delta H}\Delta H - \frac{\Delta H_i'}{\Delta H'}\Delta H' \tag{4.30}$$

where $\frac{\Delta H_i}{\Delta H}$ and $\frac{\Delta H_i'}{\Delta H'}$ indicate the head loss ratio of the pipe and the gravity energy ratio at the length ratio i respectively.

As shown in Figure 4.9, the head loss ratio along the pipes does not vary linearly. Its variation depends on the length ratio and the exponent of the flow rate in the head loss equation. According to Wu et al (1986) when the Hazen-Williams equation is used the head loss ratio in the laterals can be expressed as:

$$hl_i = \frac{\Delta H_i}{\Delta H} = 1 - (1+i)^{2.852} \tag{4.31}$$

However, the energy gain or energy loss ratio due to the gravity at any point for uniform slopes are equal to the length ratio at that point. The pressure equation therefore may be written as follows:

$$H_i = H_0 - hl_i . hl_l - i(L.S_0) \tag{4.32}$$

where hl_i = head loss ratio in laterals at length ratio i ; hl_l = head loss at distance l (m).

In this model, the pressure heads at 20 different points with equal spacing are determined. The maximum and minimum pressures are then identified to determine the pressure variation. In order to estimate the expected minimum and maximum pressure along the laterals the input pressure (H_0) for the laterals up the slope and down the slope is assumed to equal the minimum and maximum pressures in the manifold respectively. The corresponding equations are expressed as following:

$$H_{id} = HM_{\max} - hl_i . hl_{ld} + i(L_{ld} S_l) \tag{4.33}$$

$$H_{iu} = HM_{\min} - hl_i.hl_{lu} - i(L_{lu}S_l) \tag{4.34}$$

where H_{id}, H_{iu} = the pressure head in laterals on the down slope and the up slope at length ratio i (m); hl_{ld}, hl_{lu}= the head loss in laterals on the down and the up slope respectively (m); L_{ld}, L_{lu} = the length of laterals on the up and the down slope (m); S_l= the slope of laterals (with positive sign); $HM_{\max}$, $HM_{\min}$= the input pressure in laterals on the down slope (maximum pressure in the manifold) and the up slope (minimum pressure in manifold) respectively (m).

Equations 4.33 and 4.34 related to the worst case in which the maximum inlet pressure is considered for laterals on the down slope and the minimum inlet pressure is considered for laterals on the up slope. The head loss and the pressure head at any section of the manifold (at distance x) is computed using the following equations:

$$hl_{mx} = 3.745\frac{10.68Q_m^{1.852}L_m}{C^{1.852}D_m^{4.87}}\left[1-\left(\frac{L_m - x}{L_m}\right)^{2.852}\right] \tag{4.35}$$

$$H_{mdx} = H_0 - hl_{mx} + xS_m \tag{4.36}$$

$$H_{mux} = H_0 - hl_{mx} - xS_m \qquad \text{4.37)}$$

where x= distance from inlet to the manifold (m); hl_{mx} = head loss at x in the manifold (m); H_{mdx}, H_{mux}= the pressure head in the manifold on down slope and up slope at x (m); S_m= the slope of manifold (with positive sign).

The head loss in the manifold is examined for a number of given discrete sizes (from small to large sizes) to verify that the pressure variation (for the minimum cost pipe) satisfies the required discharge uniformity. The maximum and minimum pressure in the manifold are found by evaluating Equations 4.36 and 4.37 at a number of locations.

4.8.4 Supply Pipe

As shown in Figures 4.1a and 4.1b the supply pipe delivers the whole irrigation requirement of a subunit from the source to the split point in the manifold (where a valve is located). The head loss in this pipe is also determined using the Hazen-Williams equation for a discharge equal to the design discharge and for the same range of pipe sizes as examined for the manifold. The optimum size for the supply pipe is selected among the given sizes in conjunction with the cost of energy.

4.8.5 Pump Power and Annual Energy Requirement

The annual energy required to satisfy the operating schedule depends upon the required pump power and the annual operating hours. Operating hours are expressed as the total number of hours the pump operates to provide the annual irrigation requirement. The pump power is directly proportional to the design discharge and the total dynamic pumping head as follows:

$$P_m = \frac{\gamma \, Q_{pu} H_{pu}}{\eta_m \, \eta_p} \qquad (4.38)$$

where P_m= electric motor power (kW); $\gamma = \rho g$; ρ= density of water (1000 Kg/m^3); Q_{pu}= pump discharge (m^3/s); H_{pu}= total design head provided by the pump (m); η_m= electric motor efficiency; η_p= pump efficiency.

4.8.5.1 Derivation of Pump Cost Equation

The cost of the pumping system is assumed to be a function of its head and discharge (Holzapfel et al, 1990) as follows:

$$C_{pu} = K \, Q_{pu}^a \, H_{pu}^b \qquad (4.39)$$

where C_{pu} = the cost of pump system; K , a and b= constant coefficients.

In order to fit a pump cost equation and identify the constant coefficients, (K, a, b) different alternatives of pump sizes in terms of head, discharge and shaft diameter were evaluated. A combination of depth of water table from 5 m up to 40 m in 5 m steps and total pressure head including: working pressure and head losses from 14 m up to 28 m in 2 m steps as well as a set of discharges between 20 and 55 litters per second for three different shaft sizes were examined.

For each case, the head loss of the foot valve, pipe and shaft combination of pump system were considered as well. Then, by using pump characteristic curves for each discharge and corresponding total head, the number of pump stages, the pump efficiency and the required pump power were determined. Consequently, by using these data as well as the list of pump prices (issued by Southern Cross) the corresponding price for different parts of the pump system including: the turbine pump, the electric motor, the column and shafts were found. As a result, for each particular assumed head and discharge, the total cost of the pump system was used to fit the pump price equation (Equation 4.39) to the data. In this study, a vertical shaft electric motor drive was assumed to be used. This type of electric motor drive head consists of the standard solid shaft drip proof type. In Appendix C a typical turbine pump with a vertical hollow shaft electric motor drive and also a set of characteristic curves for a LAJ pump type are shown. The LINEST routine for regression analysis from the Microsoft EXCEL package was used to fit Equation 4.39. Consequently, the pump cost parameters (K , a, b) were found (see Table 4.1b).

The estimated volume of annual irrigation requirements was obtained by considering the depth of an assumed annual irrigation, irrigation area (m^2) and application efficiency.

4.8.5.2 Annual Operation Cost

The cost of operation and maintenance (O&M) is one of the significant costs of an irrigation project. An efficient operation program provides potential savings in the project costs. In this model, a semi-automatic system is proposed. Therefore, the labour cost is considered to be small compared to the capital and energy costs. In the analysis of annual operating cost, only energy cost is considered. It would be a good idea to consider the management cost as well, but as this model has been focused on efficient design from an engineering point of view, only the system cost in terms of technical design criteria is considered. The cost of energy to operate the system on an annual basis can be represented by the unit cost of energy multiplied by the total energy required over the operating season. The annual energy requirement in turn, may be obtained using the annual irrigation requirements, the power of the motor needed to drive the pump, and the total operating hours of pump as follows:

$$A_{en} = P_m A_{ir} \left(\frac{E_a \; T(K)}{K_c \; ET_0 \; \mathrm{F(I)}} \right) \tag{4.40}$$

where A_{en} = annual energy requirement (kWh); A_{ir} = annual (seasonal) irrigation requirement (mm). The term in brackets is the reciprocal of the depth of application per unit time (mm/hr). This is developed from Equations 3.23 and 3.24.

Annual operating cost during the expected life of project considering a discount rate is converted to present value as follows:

$$C_{op} = A_{en} C_{en} \left[\frac{1-(1+i)^{-n}}{i} \right] \tag{4.41}$$

where C_{op} present value of operating cost ($); C_{en} = cost of energy ($/kWh); i = discount rate; n = project life (years).

4.8.6 Discharge in Emitters, Pipes and Pump

Wu et al (1986) indicate that the emitters in trickle irrigation systems can be designed as a point source or line source to supply water into the plant root zone. This depends on the cropping system. As described in Chapter 3, the basic data used for optimum design of drip irrigation in this study are more compatible to low density tree planting system. The emitters therefore are designed as a point source emitters. The expected flow running out from each emitter depends on the plant water requirement, irrigation interval, irrigation time, irrigation application efficiency and the number of emitters allocated to each plant.

In this study, a cost analysis is carried out with different operating conditions. Different combinations of irrigation interval and irrigation time yield different design discharges which allow the designer to select different sizes of emitters. However, the potential evapotranspiration, the irrigation application efficiency and the number of emitters for each plant are assumed to be constant. Obviously, they may be changed under different circumstances. The discharge equations in the emitters, laterals, manifold and supply pipe are given in Chapter 3 (Equations 3.24, 3.26, 3.27 and 3.28 respectively). As shown in Equation 3.24 the discharge in the emitters and consequently in the piping system and the total design discharge is not constant due to using different irrigation intervals and irrigation times. In fact, the design discharge is the required discharge that should be provided by the pump under a known irrigation interval and time to meet the irrigation requirements.

4.8.7 Constraints

The objective function is to minimise the system cost subject to a number of constraints. The constraints are necessary not only to ensure a reasonable uniformity in water distribution but also to prevent soil erosion, water loss and clogging problem during the life expectation of the project. More details regarding the different types of constraints are discussed in the following sections.

4.8.7.1 Hydraulic Constraints

In this study, as explained in Chapter 3, the manufacturing emitter characteristics affecting the emitter discharge are assumed to be constant. Hence the uniformity of discharge in emitters is controlled by hydraulic pressure. In the present model, a sub program is developed and embedded into the main program to calculate the Christiansen coefficient (UC) in each optimum solution to evaluate the degree of discharge uniformity in the laterals. The values of UC for each optimum solution will be discussed later.

As shown in Equation 4.3 the discharge in orifice type emitters for turbulent flow regimes is a function of pressure head (H) with a power of $\alpha = 0.5$. For a constant c the discharge of such emitters varies in proportion to H^{α}. If the pressure varies within a limited range then the discharge will also vary in a limited range. For example, for the $\alpha = 0.5$ the variation of pressure (H) is almost twice the variation of discharge (q) as indicated in Equation 4.43.

The pressure head variation may be formulated as below:

$$H_{\text{var}} = \frac{H_{\max} - H_{\min}}{H_{\max}} \tag{4.42}$$

where H_{var} = pressure variation (dimensionless); $H_{\max}$, $H_{\min}$ = maximum and minimum pressure within the multiple outlet pipes respectively (m).

The discharge variation is affected by the pressure variation and may be obtained using the following equation (Wu et al, 1986):

$$q_{\text{var}} = 1 - \left(1 - H_{\text{var}}\right)^{\alpha} \tag{4.43}$$

where q_{var} = variation of emitter discharge along the laterals (dimensionless).

In this model, the pressure and discharge variation in both symmetrical laterals and both segments of the manifold are controlled independently on the basis of the Equations 4.42 and 4.43. The constraints for pressure variation in the manifold are given below:

$$HMU_{\text{var}} \le 10\% \tag{4.44}$$

$$HMD_{\text{var}} \le 10\% \tag{4.45}$$

$$HM_{\text{var}} \le 10\% \tag{4.46}$$

where HMU_{var}, HMD_{var}, HM_{var} = pressure variation within the manifold segments laying up slope, down slope and along the whole length respectively (dimensionless). Similarly, the pressure variation for laterals is assumed to be limited as below:

$$HLU_{\text{var}} \leq 15\% \tag{4.47}$$

$$HLD_{\text{var}} \leq 15\% \tag{4.48}$$

where HLU_{var}, HLD_{var} = the pressure variation within the laterals laying up and down slope respectively. As shown in Equation 4.43 for the orifice emitters with α=0.5 the pressure variation presented above may lead to a discharge variation of 7.5 % in laterals. This shows a good agreement with the original assumption that discharge uniformity should be high for a well designed system. The overall pressure variation within the subunit is limited to less than 25% in this analysis.

The total dynamic pumping head including the pressure head (H_0) required at the split point (distribution valve) in the manifold may be written as follows (see Figure 4.12):

$$TH = H_0 + HL_s + HL_{acc} + H_{wt} \pm L_s.S_l \tag{4.49}$$

where TH = total design head (m); HL_{ac} = the head loss in accessories including: the head loss in filter, fertiliser, valves and the pump shaft (m); H_{wt}= the depth of groundwater (m). The term "$L_s.S_l$" represents the difference in the elevations along the supply pipe (m). If the source node is located in a lower elevation than the distribution valve the positive sign is used otherwise the negative sign is used in Equation 4.49.

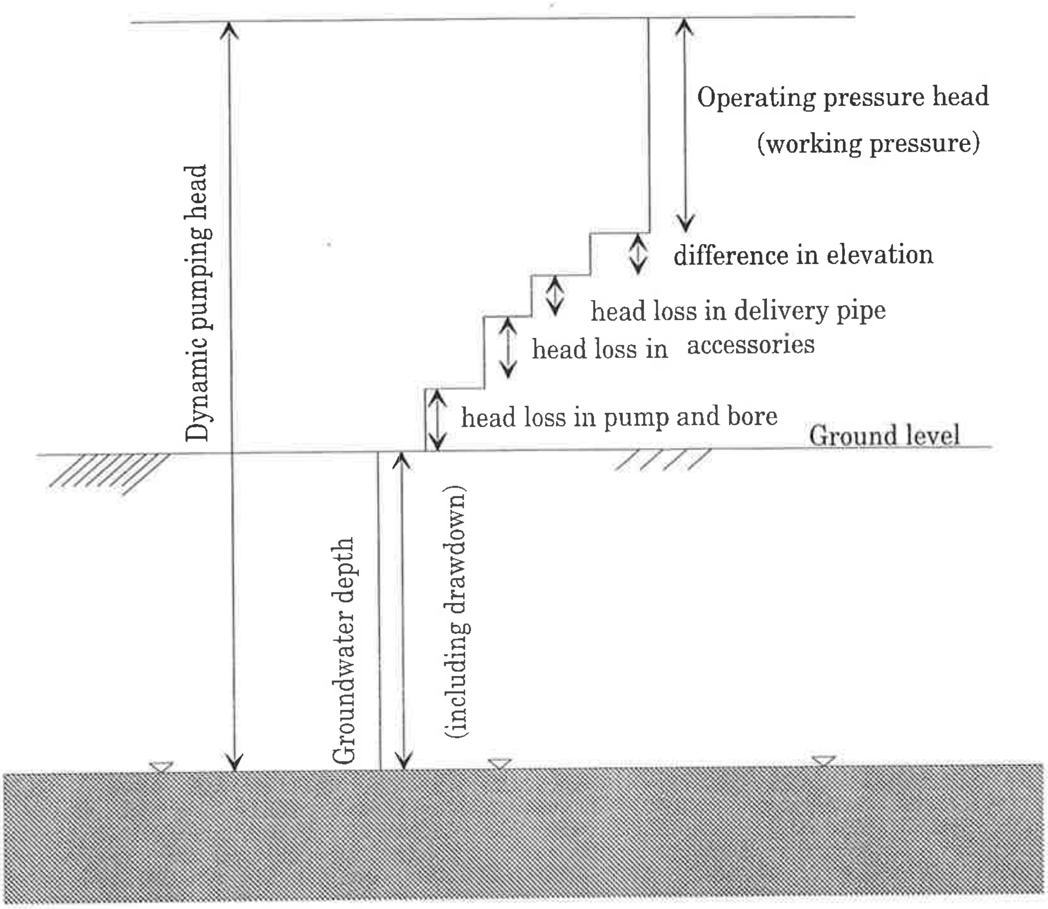


Fig. 4.12 Dynamic pressure head (design head) to be provided by the pump

4.8.7.2 Length Constraints

For a constant position of manifold and supply pipes the length constraints may be formulated as:

$$L_{bu} + L_{su} = L_{bd} + L_{sd} = \frac{F_x}{2} - d_x \tag{4.50}$$

$$L_{mu} + L_{md} = F_y - d_y \tag{4.51}$$

$$L_s = \frac{F_x}{2} \tag{4.52}$$

where L_{bu}, L_{su}, L_{bd}, L_{sd} = the length of larger and smaller size of laterals on up and down slope respectively (m); L_{mu}, L_{md}, L_s = the length of manifold on up and down slope and the length of supply pipe respectively (m).

When the position of the manifold and the supply pipe is not fixed and the model examines different positions of those pipes to reach an optimum solution, the corresponding length constraints may be formulated as below:

$$L_{bu} + L_{su} = L_{lu} \leq \frac{F_x}{2} - d_x \tag{4.53}$$

$$L_{bd} + L_{sd} = L_{ld} \geq \frac{F_x}{2} - d_x \tag{4.54}$$

$$L_{ld} + L_{lu} = F_x - 2d_x \tag{4.55}$$

$$L_{mu} \leq \frac{F_y - d_y}{2} \tag{4.56}$$

$$L_{md} \geq \frac{F_y - d_y}{2} \tag{4.57}$$

where L_{ld}, L_{lu} = the lateral length on the up and down slope respectively (m).

The discharge constraints are similar to the discharge constraints described in Chapter 3 (Equations 3.23 to 3.28).

4.9 OBJECTIVE FUNCTION

The objective function, including the cost of the piping system, pumping plant, emitters, accessories and the present value of the annual operating cost, is minimised subject to the constraints described above. The mathematical expression of the objective function may be formulated as below:

$$Z = C_p + C_{pu} + C_{em} + C_{acc} + C_{op} \tag{4.58}$$

where Z = objective function to be minimised ($).

4.9.1 System Cost

The total system cost includes the cost of piping system, the cost of pump, the present value of the annual operating cost, the cost of emitters and the cost of accessories. The details of the piping and the emitter costs within a subunit have been explained in Chapter 3 (Equations 3.11 to 3.14). The details of pump and the annual operating costs are discussed in Sections 4.8.5.1 and 4.8.5.2. In this study, a reasonable constant cost is considered for accessories including a filter, a fertiliser, a pressure regulator, and valves.

4.10 OPTIMISATION PROCEDURE

A field with an area of 6 ha and with a fixed piping layout and a uniform slope with a number of patterns in the X and Y directions is assumed. The optimum length of laterals with two different sizes on the up and down slopes, the length and the size of manifold on the same slope, as well as the optimum size of the supply pipe are taken into account as decision variables. In the modified version of the model the optimum position of the manifold and supply pipes is also investigated. Furthermore, the decision variables are searched for various working pressures and variations of the groundwater level. The procedure for an optimum design of the system in which the positions of the manifold and the supply pipe are at the centre may be summarised as follows:

1 Find the length of laterals using $L_l = \dfrac{F_x}{2} - d_x$;

2 Find the length of manifold L_m and supply pipe L_s using Equations $L_m = F_y - dy$ and $L_s = F_{x/2}$ respectively;

3 Let irrigation interval F(I)= 0.5 day ;

4 Let irrigation time T(K)= 4 hours;

5 Find the discharge of emitters Q_E, laterals Q_l, manifold Q_m and supply pipe Q_s, using Equations 3.24, 3.26, 3.27, 3.28 respectively;

6 Find the irrigation application rate I_{app} ($I_{app} = \dfrac{Q_E}{d_x d_y}$) if it is greater than soil infiltration rate (I_{soil} mm/hr) go to step 4;

7 Let slope in the X direction $S_l = 0.00$ and slope in the Y direction $S_m = 0.00$;

8 Set the manifold diameter on up slope equal to the smallest available diameter from the given discrete sizes;

9 Find the maximum and minimum pressure and the pressure variation within the manifold segment on the up slope;

10 If the pressure variation in the manifold (up slope segment) is greater than 10% go to step 27;

11 Repeat the same procedure (from steps 8 to 10) for the manifold on a down slope. If the pressure variation (in down slope segment) is greater than 10% go to step 28;

12 Find the pressure variation in both segments of manifold; if it is greater than 10% go to 27;

13 Let the length of the larger size of laterals laying down the slope L_{bd}, equal zero;

14 Find the length of the smaller size of laterals L_{sd}, using $L_{sd} = L_l - L_{bd}$;

15 Calculate the corresponding head loss in laterals;

16 Find the pressure variation within the laterals using Equation 4.42;

17 Find the average pressure, and use it to calculate the discharge coefficient of emitters K_d, using Equation $K_d = \dfrac{2.78 \times 10^{-7} Q_E}{(H_{av})^{0.5}}$;

18 Use K_d, $H_{\max}, H_{\min}$ to calculate the value of average discharge of emitters, q_{av} ;

19 If the pressure variation in the laterals on the down slope is greater than 15%, go to step 29, otherwise go to subroutine to find the Christiansen coefficient UC;

20 Repeat the same procedure from steps 13 to 16, for the laterals laying up the slope, if the corresponding pressure variation is greater than 15% go to step 30;

21 Set the supply pipe diameter D_s, equal to the smallest available size using the given discrete sizes;

22 Find the head loss in the supply pipe using the Hazen-Williams equation;

23 Find the total dynamic head TH, provided by the pump, using Equation 4.49;

24 Find the pump power, p_{pu} and pump cost, C_{pu} using Equations 4.38 and 4.39;

25 Find the present value of the annual operating cost using Equation 4.41;

26 Find the cost of piping system and emitters using Equations 3.11 to 3.14 and the objective function, using Equation 4.58;

27 Increment the size of the manifold on up slope. If it is less than or equal to the maximum available diameter go to step 9, otherwise go to step 11;

28 Increase the size of the manifold on down slope. If it is less than or equal to the maximum available diameter, go to step 11, otherwise go to step 12;

29 Increment the length of the larger size of laterals on down slope L_{bd}, by 10% of lateral length L_l, and return to step 14;

30 Increment the length of the larger size of lateral on up slope by 10% of L_l, and length of smaller size and return to step 15;

31 Find the size of the supply pipe, D_s, from the available pipe sizes, start from the smallest size and calculate the corresponding head loss;

32 Increase the slope of field in the X direction S_l, by the increment of 0.1 %, if it is less than or equal to 1 % go to step 8, otherwise go to step 34;

33 Increase the slope of field in the Y direction S_m, by the increment of 0.2 %, if it is less than or equal to 2.0 % go to step 8, otherwise go to step 34;

34 Increase the irrigation time by increment of 2 hours, if it is less than 12 hours go to step 4, otherwise go to stop 35;

35 Increase the irrigation interval by increment of 0.5 day, if it is less than or equal to 3 days go to step 3, otherwise stop.

The above procedure is carried out to find the optimum solution under different operating conditions and different slope patterns, but for only a fixed position of manifold and supply pipes. However, changing the position of the manifold and supply pipes from the centre toward the higher elevation could be helpful to reduce the length of the larger size in the up slope and increase the length of smaller size in the down slope.

The same model with a similar optimisation procedure, is modified to identify the minimum system cost for different positions of manifold and supply pipes. Clearly, a change in positions of the manifold and supply pipe yields two different lengths for laterals and also two different lengths for the manifold segments. As a result, the following modifications in steps (1) and (2) are carried out. In step (1), the length of laterals in either side of manifold may be calculated using the following equations:

$$L_{lu} = \left(\frac{F_x}{2} - d_x \right) - I \tag{4.59}$$

$$L_{ld} = \left(\frac{F_x}{2} - d_x \right) + I \tag{4.60}$$

where I indicates the size of increment for changing the manifold position (m) and changes from 0 (centre) up to $\frac{F_x}{2} - d_x$. Similarly, in step (2) the length of both segments of manifold in the up and down slope may be expressed as:

$$L_{mu} = \frac{F_y - d_y}{2} - J \tag{4.61}$$

$$L_{md} = \frac{F_y - d_y}{2} + J \tag{4.62}$$

In which J indicates the size of increment for the change in position of the supply pipe and changes from 0 up to $\frac{F_y - d_y}{2}$.

The flow chart for the optimum solution based on the described procedure for the fixed position of the manifold and supply pipes is illustrated in Figure 4.13.

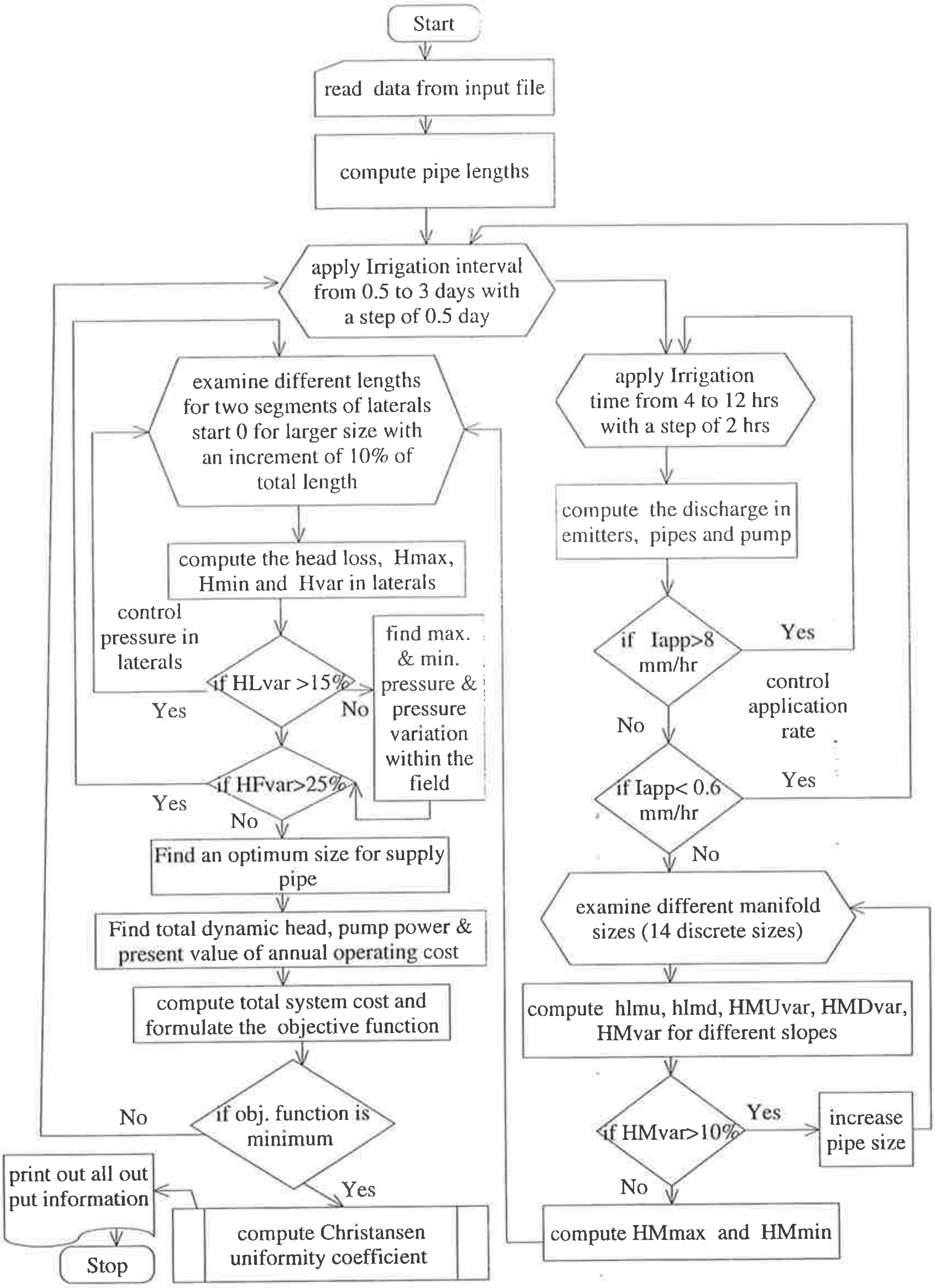


Fig. 4.13 Flow chart of optimum solution process for a drip irrigation system

4.11 RESULTS AND DISCUSSIONS

As explained previously, to achieve an optimum solution for a drip irrigation system, several factors need to be considered. In the present model, the optimum solution of a drip irrigation system with one control head on a sloping land is investigated. The minimum system cost is examined for a number of different positions of manifold and supply pipes. The optimum position of the manifold and supply pipes are identified by imposing various loading cases. Each loading case is achieved by using different irrigation intervals and irrigation times. During the investigation the impact of a number of factors including the working pressure, the slope in the X and Y directions, the groundwater level and delivery pipe sizes on the system cost is examined. The model is tested by a case study consisted of a network with a known layout and the input data tabulated in Tables 4.1a and 4.1b.

TABLE 4.1a The input data used in the case study

	Length (m)	Diameter (mm)	Cost ($)/m	Slope (%)	allowable pressure variation (%)
Field	300 200	— —	-	-	25
Lateral pipes	148	19 28	0.85 1.10	0 % to 1% with a step of 0.1%	15
Manifold pipe	98.5	28,35,43, 57, 83, 102,130,139,187, 199, 120, 233, 253	Eq. 3.12	0 % to 2% with a step of 0.2%	10
Supply pipe	150	same as manifold	Eq. 3.13	same as laterals	no limit

TABLE 4.1b The input data used in the case study

F_x	300 m	CH_l	130	η_p	0.72	HL_f	2.0 m
F_y	200 m	CH_m	140	η_m	0.92	HL_{fer}	1.5 m
d_x	2 m	CH_s	150	C_{em}	\$ 0.9	HL_v	0.7 m
d_y	3 m	g	9.81m/s^2	CU_{em}	.09 \$/kwh	HL_{sh}	3 m
A_{ir}	800 mm	I_{soil}	8 mm/hr	C_{fil}	\$ 100	T(K)	4 to 12 hrs step of 2 hrs
ET_0	6 mm/day	H_0	12 m	C_{fer}	\$ 100	F(I)	.5 to 3 days step of .5 day
Kc	1	i	10 %	C_{val}	\$ 80	K	1000.0
H_{wt}	.0 to100 m step of 10m	n	12 years	C_{preg}	\$ 90	a b	0.2305 0.9038

4.11.1 Optimum Cost for Different Operating Conditions

Providing a sufficient quantity of water under tight control to meet the full water requirements of the plants is one of the advantages of a well designed drip irrigation system. To schedule an appropriate irrigation operating program, not only the plant water requirements and soil water-holding capacity but also economic considerations should be taken into account. As the plant transpiration rate decreases or the soil has a higher water-holding capacity, it is recommended that irrigation be carried out with a greater volume of application but less frequently. This can be fulfilled by selecting an appropriate irrigation interval and irrigation time. In this part of study, the optimum design is investigated applying different irrigation intervals and irrigation times. In fact, the emitter flow rate and, consequently, the selection of emitters is significantly affected by operating schedule which should be finally adjusted with the soil characteristics.

In Chapter 3, it was shown that as the irrigation interval increases and irrigation time decreases the application rate and also the emitter flow rate increase. Consequently, the design discharge and the discharge flowing through the piping system is increased. This leads to a considerable increase in the system cost. However, some combinations of the given irrigation intervals and times are not feasible due to causing a pressure variation higher than the maximum allowable value. The results indicate that achieving an optimum design is not possible for all the given irrigation intervals and irrigation times. The infeasible solutions usually occur under high irrigation intervals (low frequent) with the low irrigation times. For example, the present model is examined for different irrigation intervals from 0.5 to 3 days with a step of 0.5 day (0.5, 1, 1.5, 2, 2.5, 3) each with 5 different irrigation times from 4 to 12 hours with a step of 2 hours (4, 6, 8, 10, 12). It is assumed that slopes in the X and Y directions are 0.5% and 0.8% respectively. Table 4.2 summarises the results for the various operating conditions when manifold and supply pipes are located at the centre.

TABLE 4.2 The least cost solutions for various operating conditions (slope=0.5% in the X and 0.8% in the Y directions)

irrigation interval (day)	irrigation time (hr)	emitter discharge (L/hr)	design discharge (m3/s)	design head (m)	pipe cost ($)	pump cost ($)	annual op cost ($)	total cost ($)
0.5	4	4.7	0.0132	43	20262	13943	5042	48662
0.5	6	3.2	0.0088	41.2	19562	12227	4835	46039
0.5	8	2.4	0.0066	40.6	19562	11279	4758	45014
0.5	**10**	**1.9**	**0.0053**	**41.2**	**19064**	**10869**	**4835**	**44182**
0.5	12	1.6	0.0044	40.8	19064	10318	4782	43578
1	4	9.5	0.0265	41.6	23947	15864	4873	54100
1	6	6.3	0.0176	41.4	21470	14399	4855	50139
1	8	4.7	0.0132	43	20262	13943	5042	48662
1	10	3.8	0.0106	41.9	20889	12929	4909	48142
1	**12**	**3.2**	**0.0088**	**41.2**	**19562**	**12227**	**4835**	**46039**
1.5	4	14.2	0.0397	41.9	26829	17549	4914	58707
1.5	6	9.5	0.0265	41.6	23947	15864	4873	54100
1.5	8	7.1	0.0199	41.8	22346	14933	4905	51598
1.5	10	5.7	0.0159	41.1	21222	13958	4818	49414
1.5	12	4.7	0.0132	43	20262	13943	5042	48662
2	6	12.6	0.0353	42.9	25449	17445	5030	57340
2	8	9.5	0.0265	41.6	23947	15864	4873	54100
2	10	7.6	0.0212	42.1	22346	15247	4937	51944
2	12	6.3	0.0176	41.4	21470	14399	4855	50139
2.5	6	15.8	0.0441	42.4	28778	18168	4970	61332
2.5	8	11.8	0.0331	42.5	24939	17055	4988	56397
2.5	10	9.5	0.0265	41.6	23947	15864	4873	54100
2.5	12	7.9	0.0221	42.3	22593	15455	4959	52422
3	8	14.2	0.0397	41.9	26829	17549	4914	58707
3	10	11.4	0.0318	42.3	24691	16820	4963	55889
3	12	9.5	0.0265	41.6	23947	15864	4873	54100

Within the feasible solutions illustrated in Table 4.2, the minimum cost occurs for an irrigation interval of 0.5 day and irrigation time of 12 hours. However, this case is rejected because it leads to a continuous irrigation with a low emitter flow rate. The subsequent minimum cost as expected is obtained with an irrigation interval of 0.5 day and irrigation time of 10 hours with a cost of $44182. Low flow rate emitters are very susceptible to clogging and siltation and also may limit the wetted area; on the other hand, high flow rate emitters (higher than a certain value) violate the constraints. Table 4.2 and Figure 4.14 indicate that for high irrigation intervals and low irrigation times there are no feasible solutions (i.e. irrigation intervals of 2, 2.5 and 3 days with irrigation times of 4 and 6 hours respectively).

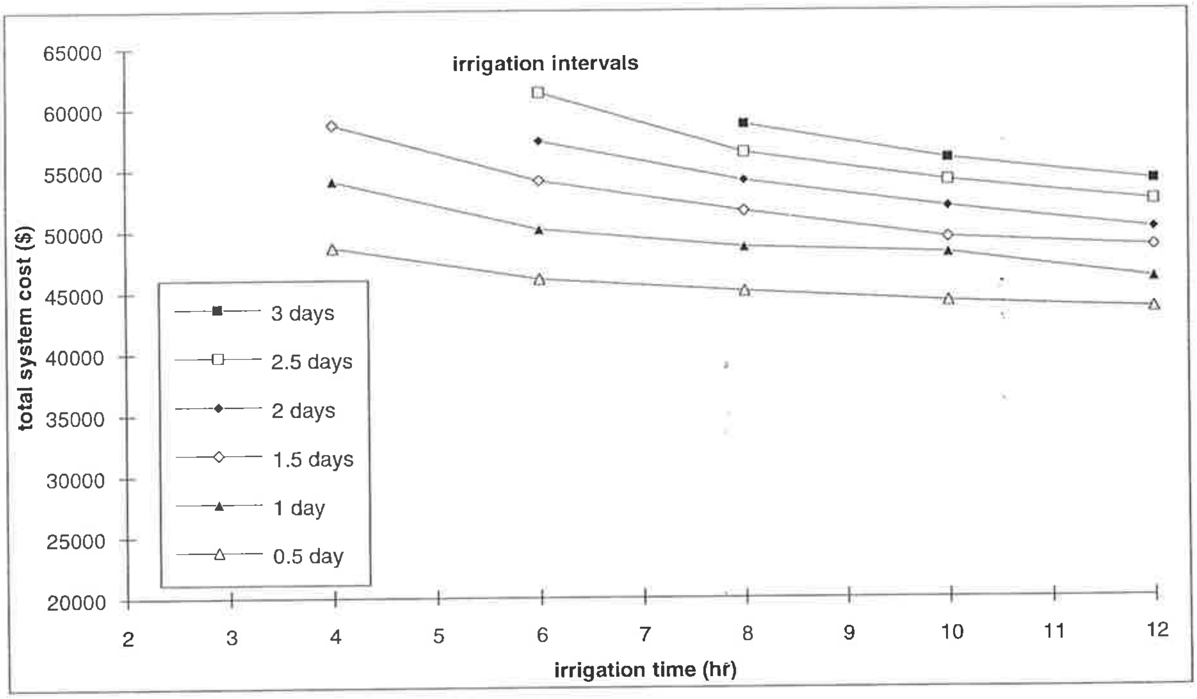


Fig. 4.14 Variation of system cost for different operating conditions

However, to select high flow rate emitters in order to reduce possible clogging, the following changes may be considered in the model:

- decrease the length of laterals and the manifold (field area will be decreased);
- increase the allowable pressure variation (this will lead to a lower distribution uniformity);
- increase the size of the laterals and manifold (this will lead to a higher system cost); Consider a number of pressure reducing valves in the system.

As shown in Figure 4.14 when the irrigation interval increases and the irrigation time decreases the system cost increases. In addition, the slope of the system cost for each irrigation interval is steeper for lower irrigation times than for higher irrigation times. Clearly, the global optimum occurs at the highest irrigation time and the lowest irrigation interval as expected. It corresponds to an interval of 0.5 day and an irrigation time of 10 hours. The pressure variation corresponding to this optimal case within the subunit is 19%. More details corresponding to minimum cost solution are given in the Table 4.3.

A comparison between four main components of the system cost including: pipes, pump, accessories and the present value of annual operating cost for five different irrigation times and an irrigation interval of 1 day is presented in Figure 4.15. According to this figure, the operating cost does not show a significant difference for given irrigation times.

TABLE 4.3 Minimum cost design and associated decision variables (slope: 0.5% and 0.8 % in the X and Y directions; irrigation interval of: 0.5 day and irrigation time of 10 hrs)

Items	Length (m)		Diameter (mm)		Head loss (m)	Discharge (L/s)	Cost ($)	Percent of total cost (%)
	L_{bs}	L_{ss}	D_{bs}	D_{ss}				
Lateral up slope	0.0	148.0	-	19	0.11	0.039	16857	38.15
Lateral down slope	0.0	148.0	-	19	0.11	0.039		
Manifold up slope	-	98.50	-	84	0.12	2.64	1088	2.46
Manifold down slope	-	98.50	-	57	0.75	2.64		
Supply pipe	-	150.0	-	84	1.58	5.29	1119	2.53
Pump	-	-		-	3	5.29	10869	24.6
Emitters	-	-	-	-	-	1.89 L /hr	9045	20.48
P.V.of operating cost	-	-	-	-	-	-	4834	10.94
Accessories		-	-	-	5.4	5.29	370	0.84
Total	-	-	-	-	**10.97**	**5.29**	**44182**	**100**

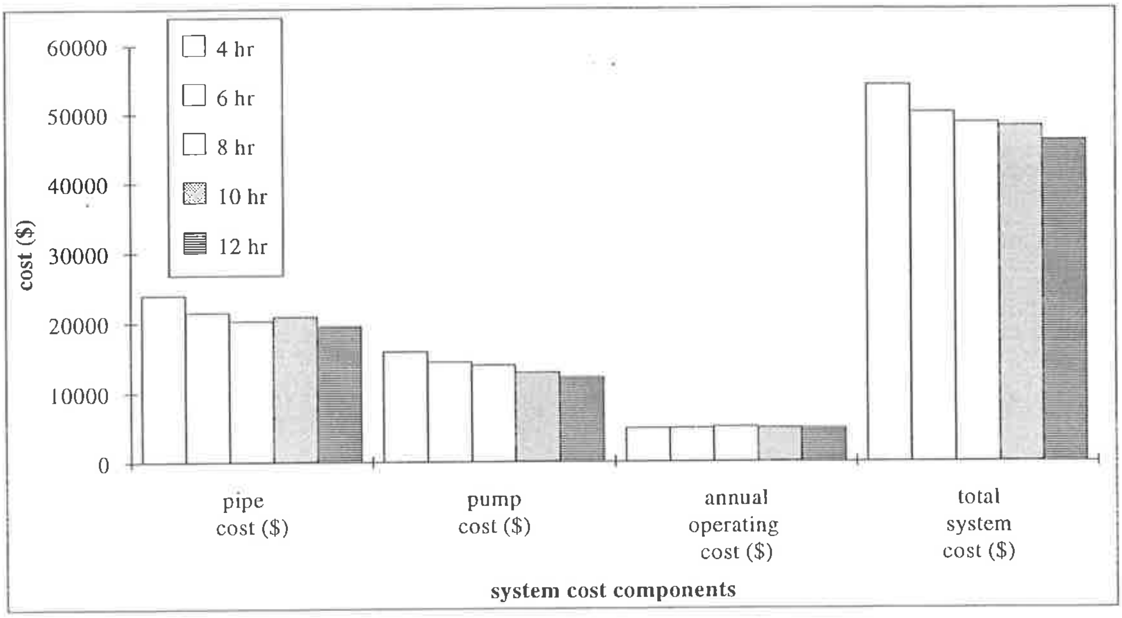


Fig 4.15 Total system cost and associated component costs for five different irrigation times and an irrigation interval of one day

This could be due to the fact that pressure variations within a specified range do not affect the total dynamic head and also the delivery pipe is not long enough to create high head loss. Moreover, there would be no significant difference in flow rate for given irrigation times. However, the cost of pipes and pump is affected. To assess the influence of various factors on the system cost, the model is examined for a number of different cases including: changes of slope in the X and Y directions and changes in the positions of the manifold and supply pipes.

4.11.2 Slope Variation

As shown in Figure 4.9 and Equation 4.6, the slope plays a significant role in the pressure variation within the pipe networks. According to the piping system displayed in Figures 4.1a and 4.1b, the irrigation water flows from the source to the manifold through the supply pipe. It then is divided into two parts. Each part flows through one of the segments of the manifold in two opposite directions. The irrigation water is distributed across the field by the laterals which are located on either side of the manifold, laying up and down slope. The discharge uniformity which is significantly important in the pressure irrigation systems depends upon the head loss and the slope along the multiple outlet pipes. As the slope increases, the pressure (H_i) along the pipes increases for a down slope and decreases for an up slope. As shown in Equation 4.6, for a known pipe size and a known input pressure on steep slopes a change in the slope may cause a considerable change in the pressure variation within the multiple outlet pipes laying on either side of the manifold and supply pipes. This is due to the inverse effects of slope on the pressure of pipes on the up and the down slope.

In the current model, two sets of slope patterns in the X (parallel to the laterals and supply pipe) and Y direction are considered. The slope in both directions is firstly assumed to be 0.00, in which the pressure variation is only affected by the head loss. Secondly, a set of uniform slopes of 0.1% to 1% with an increment of 0.1% in the X direction and also 0.2% to 2% with an increment of 0.2% in the Y direction are examined. The system cost corresponding to the optimum solution for each combination of the given slopes is plotted in Figure 16. As this Figure illustrates, each individual line corresponds to a specific slope in the X direction with respect to the various slopes in the Y direction. The results indicate a good consistency with our original assumption that, as the slope increases the pressure variation increases, and also the maximum allowable pressure variation is limited within the multiple outlet pipes. Otherwise any combination of slopes considered in the model could cause a feasible solution.

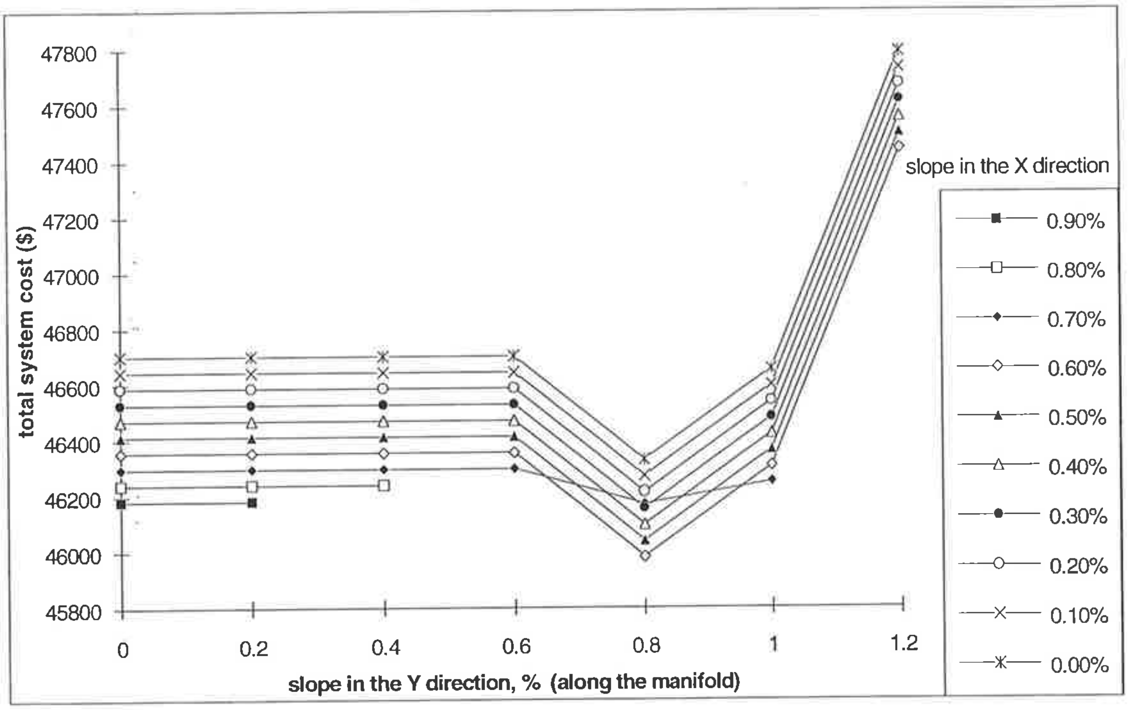


Fig. 4.16 System cost against the variation of slopes in the X and Y directions (irrigation interval of 1 day and irrigation time of 12 hr)

In general, as the slope increases the difference between the maximum and minimum pressure in two symmetrical pipes increases. However, the model attempts to find the optimum solution, thus it searches and examines different pipe sizes and lengths of lateral segments for each slope. For the up slope as the slope increases the model attempts to reduce the head loss in order to keep the pressure variation lower than the maximum allowable variation; hence the size of manifold and the length of the larger size of laterals is increased. However, on the down slope, as the slope increases the model attempts to increase the head loss in order to reduce the pressure variation, so the size of manifold and the length of larger size of lateral are decreased.

For a constant slope in the X direction, any change of slope in the Y direction may affect the size of manifold in one or both segments. As long as the pressure variation due to the changes in the slope in the manifold segment remains less than 10% the minimum possible diameters are selected for both segments to keep the cost of the manifold at the minimum level. As the slope increases from 0.00 with an increment of 0.2%, the pressure variation increases within the allowable variation. If it is still less than 10%, the diameters of both segments remain constant at their optimum level. Therefore, the system cost remains constant as well. However, for the pressure variation greater than 10%, the diameter in up slope is increased to reduce the head loss, but as the diameter increases for the higher slopes the effect on head loss is not that significant. Consequently, for those slopes (higher than 1.2% in the Y direction) there are no feasible solutions while for the pressure between 0.8% to 1.2 %, the cost increases due to the use of larger pipe sizes to keep the pressure variation less than or equal to the 10 %.

The results show that for any slope in the X direction, the cost of all components are almost the same for different slopes in the Y direction, except for the slopes equal or greater than 0.8%. When the slope in the Y direction is 0.8% only the cost of manifold drops. Since a lower pipe size is used for the manifold segment in the down slope (i.e. a 57 mm pipe is swapped with a 84 mm). However, for the slopes higher than 0.8% a larger size is selected for the up slope to reduce the effect of the up slope and keep the pressure variation within the allowable range (i.e. a 102 mm pipe is swapped with a 84 mm). For the same logic when the slope is higher than 0.1% the next larger size is selected for the up slope segment (i.e. 149 mm). This is the reason that the total system cost increases. The reason for the different rate of increase is due to the difference in the size of the available discrete pipe sizes which are assumed in the model.

The minimum cost of laterals is obtained when the whole length of laterals on either side of the manifold can have the smaller size. However, using the smaller size with a constant length and constant discharge may cause a high head loss and consequently, violate the allowable pressure variation. In order to reduce the head loss to keep the pressure variation lower than the maximum allowable variation, the length of the larger size has to be increased. As a result, the system cost is increased. For any given slope in the X direction the model attempts to use the greatest possible length for the smaller size unless the pressure variation requirements are violated. As shown in Fig. 4.16 for any slope in the Y direction, as the slope in the X direction increases the system cost decreases. The results show that the component costs are almost the same except the cost of operating and the cost of pump. This is due to the total head as shown in Equation 4.49. On the basis of the Equation 4.49 for a higher slope in the X direction (parallel to the supply pipe) the total system head decreases. Thus it directly affects the capital and operating cost of the pump.

However, for the slopes which cause a big difference in elevation, the model attempts to minimise the effect of head loss for the up slope segments while the down slope segments still would have their lowest size. This may cause an increase in the system cost. Nevertheless, for slopes higher than 0.9 % in the X direction since changing the length of limited sizes on head loss is not as significant as the effect of changes in slope, the pressure variation will no longer be below the allowable level. Consequently, there are no feasible solutions for slopes higher than 0.9 %. This concept is illustrated in Figure 4.16.

The global minimum cost with a value of $45,981 (7,663 $/ha) occurred at a slope of 0.6 % in the X and 0.8 % in the Y directions for an irrigation interval of 1 day and irrigation time of 12 hours . This case corresponds to an overall pressure variation of 23% within the field. Details of decision variables associated with the global minimum cost design are given in Table 4.4. More details regarding the minimum cost solutions for the all possible combinations of different slopes for a 1 day irrigation interval and 12 hours of irrigation time are listed in Appendix D. Note: applying an irrigation interval of 0.5 day leads to a lower cost.

TABLE 4.4 Global minimum cost with associated variable values (slope: 0.6% in the X and 0.8% in the Y directions; irrigation interval of 1 day and irrigation time of 12 hr)

Items	Length (m)		Diameter (mm)		Head loss (m)	Discharge (L/s)	Cost ($)	Percent of total cost (%)
	L_{bs}	L_{ss}	D_{bs}	D_{ss}				
Lateral up slope	0.0	148.0	-	19	0.29	0.066	16857	36.66
Lateral down slope	0.0	148.0	-	19	0.29	0.066		
Manifold up slope	-	98.50	-	84	0.29	4.41	1088	2.37
Manifold down slope	-	98.50	-	57	1.93	4.41		
Supply pipe	-	150.0	-	102	1.57	8.82	1617	3.52
Pump	-	-		-	3	8.82	12187	26.50
Emitters	-	-		-	-	3.16 L/hr	9045	19.67
P.V.of operating cost	-	-	-	-	-	-	4817	10.48
Accessories	-	-	-	-	5.4	8.82	370	0.8
Total	**-**	**-**	**-**	**-**	**12.77**	**8.82**	**45981**	**100**

4.11.3 Variation of the Manifold and Supply Pipe Positions

The known layout of the piping system is one of the basic characteristics of the present model as shown in Figures 4.1a and 4.1b. In Chapter 3, an investigation was carried out to find an optimum design of a drip irrigation system on flat terrain. However, in this part of the study a model is developed to assess the optimum design of a drip irrigation system on sloping lands with different uniform slope patterns in two directions. The optimum design for a level area is achieved when the manifold and supply pipes are located in the middle of the field. In the previous sections, we discussed how an optimum solution may be obtained under the influence of various slope patterns and various operating conditions. For this analysis, the position of the manifold and supply pipes were assumed to be fixed and located at the centre of the field. Nevertheless, because of the different effects of slope on the pressure of symmetrical pipes on up and down slopes, the system cost might be decreased if the position of the manifold and supply pipes can be changed. Hence a new version of the present model is therefore developed to allow for this possibility.

In a similar way to the previous sections, the pressure variation along the pipes is constrained to be less than a specified value in order to achieve an acceptable distribution uniformity. As shown in Figure 4.1b a change in the position of the supply pipe brings about two unequal manifold segments laying up and down slope to be created. Similarly, changing the position of the manifold leads to the creation of two unequal segments of laterals laying up and down slope. The analysis associated with different positions is carried out for a constant slope of 0.5% in the X and 0.8% in the Y directions. The process of identifying the optimum position of the manifold and supply pipes is considered from the middle of the field for both pipes. Clearly, in the first iteration the position of the manifold and supply pipes is at the centre and there is no change in position. This step has already been discussed comprehensively in the previous sections.

The position of the supply pipe then is allowed to change along the up slope from 0.00 up to $\frac{L_m - dy}{2}$ m from the centre with an increment of 8m. Similarly, the position of the manifold is allowed to change along the up slope from 0.00 up to $L_l - d_x$ m with an increment of 12 m.

The model attempts to find the least cost solution for different positions of the manifold and supply pipes under various irrigation intervals and irrigation times. At each trial solution for the new position of the manifold and supply pipes, the lengths of the two segments of the manifold and laterals are found. First, the model searches to select an optimum size for each trial length of manifold laying up and down slope. It then examines the various lengths of the two laterals. The investigation is continued to find the optimum lengths for both given sizes of laterals laying up and down slope. The same process is repeated for each new position of supply pipe and manifold until the whole process is performed and an optimum solution is achieved. The whole process is repeated for a new design discharge resulting from a new set of irrigation intervals and irrigation times.

Table 4.5 summarises the results corresponding to the optimum positions of the manifold and supply pipes using various irrigation intervals and irrigation times. The findings indicate that the minimum system cost for an irrigation interval of 0.5 day and irrigation time of 10 hours occurs when the supply pipe is located 58.5 m from the centre of field in the Y direction on the up slope and the manifold is located 144 m from the centre of field in the X direction on the up slope (see Table 4.5 and Figure 4.17).

In other words, the global minimum cost is achieved when the length of the manifold and laterals on the up slope are 58.5 and 4m respectively as shown in Figure 4.17. The value of the global minimum cost for this case is $41961 (irrigation interval of 0.5 day with 12 hours is ignored because it leads to a continuous irrigation). The new minimum system cost achieved by the modified version is $2221 (370 $/ha) less than the previous one which was found under the same conditions but for a constant position of the manifold and supply pipes. In fact, the improved design shows a 5% saving cost . This new achievement is consistent with the original assumption that on sloping lands the optimum position of the manifold and supply pipes will be somewhere other than the centre of the field towards the higher elevations (see Figure 4.17). Figure 4.18 represents the variation of the minimum system cost for both cases and compares the effect of position of pipes on the system cost. This reduction in the system cost might be due to the effect of down slope which causes the use of the smaller pipe sizes. Details regarding the global minimum system cost for the optimum position of manifold and supply pipes and an irrigation interval of 0.5 day and irrigation time of 10 hours are given in Table 4.6.

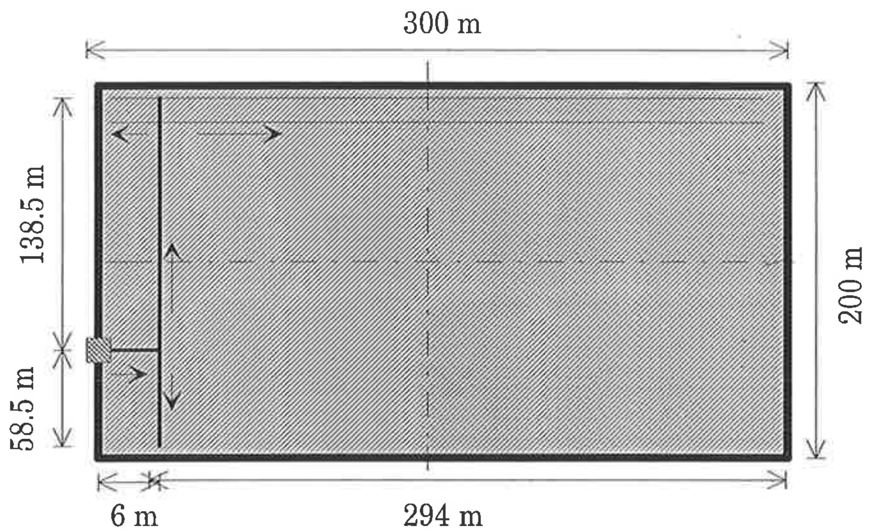


Fig 4.17 The optimum position of the manifold and the supply pipe

TABLE 4.5 The minimum system cost for the optimum position of manifold and supply pipes for different operating conditions (slope: 0.5% and 0.8%)

irrigation interval (day)	irrigation time (hr)	manifold length up (m)	lateral length up (m)	pipe cost ($)	pump cost ($)	annual op. cost ($)	total cost ($)
0.5	4	50.5	52	18464	13375	4842	45991
0.5	6	98.5	4	17764	11953	4741	43768
0.5	8	74.5	4	17388	11180	4738	42616
0.5	**10**	**58.5**	**4**	**17284**	**10626**	**4741**	**41961**
0.5	12	66.5	4	17273	10184	4739	41506
1	4	58.5	52	21757	15521	4783	51371
1	6	74.5	52	19808	14119	4777	48014
1	8	50.5	52	18464	13375	4842	45991
1	10	58.5	4	17916	12473	4744	44444
1	**12**	**98.5**	**4**	**17764**	**11953**	**4741**	**43768**
1.5	4	50.5	76	23787	17509	4928	55534
1.5	6	58.5	52	21757	15521	4783	51371
1.5	8	82.5	16	20506	14452	4756	49025
1.5	10	82.5	28	19475	13717	4752	47254
1.5	12	50.5	52	18464	13375	4842	45991
2	4	74.5	112	26568	18951	4999	59828
2	6	90.5	76	23233	16895	4882	54320
2	8	58.5	52	21757	15521	4783	51371
2	10	90.5	28	20660	14721	4775	49466
2	12	74.5	52	19808	14119	4777	48014
2	12	74.5	52	19808	14119	4777	48014
2.5	6	42.5	88	25014	17746	4870	56940
2.5	8	82.5	64	23032	16521	4842	53705
2.5	10	58.5	52	21757	15521	4783	51371
2.5	12	90.5	64	20658	15014	4830	49812
2.5	12	90.5	64	20658	15014	4830	49812
3	6	74.5	112	26568	18951	4999	59828
3	8	50.5	76	23787	17509	4928	55534
3	10	82.5	76	22819	16387	4848	53364
3	12	58.5	52	21757	15521	4783	51371

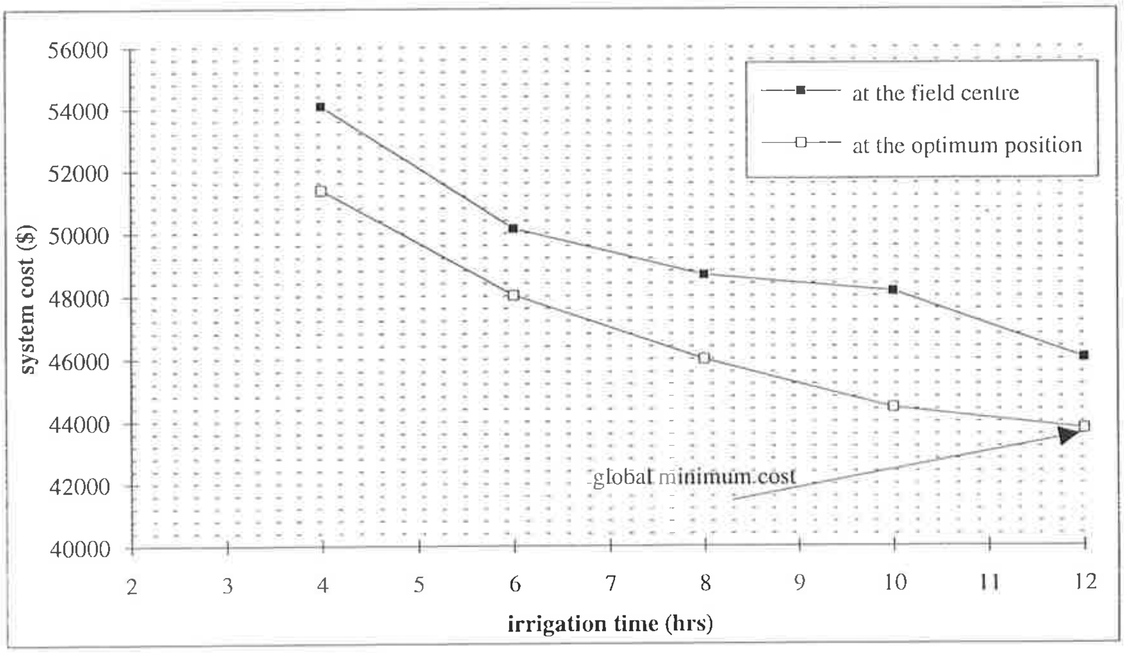


Fig. 4.18 **Minimum system costs when the manifold and supply pipes are at the centre and at the optimum position (for 0.5 day and 10 hr)**

TABLE 4.6 Global minimum cost and associated decision variables for optimum position of the manifold and supply pipes (irrigation interval of 0.5 day and irrigation time of 10 hr)

Items	Length (m)		Diameter (mm)		Head loss (m)	Discharge (L/s)	Cost ($)	percent of total cost (%)
	L_{bs}	L_{ss}	D_{bs}	D_{ss}				
Lateral up slope	0.0	4.0	-	19	≈ 0.00	0.001		
Lateral down slope	0.0	292.0	-	19	0.78	0.077	16606	39.57
Manifold up slope		58.5		43	0.65	1.54		
Manifold down slope		138.5		57	1.92	3.64	634	1.51
Supply		6.0		84	0.06	5.18	45	0.11
Pump		-		-	3	5.18	10626	25.32
Emitters		-		-	-	1.89 L/hr	8940	21.31
P.V. of operating cost		-		-	-	-	4741	11.30
Accessories		-		-	5.4	5.18	370	0.88
Total		**-**		**-**	**11.81**	**5.18**	**41962**	**100**

4.11.4 Working Pressure

In pressure irrigation systems the working pressure varies for different types of outlets. In this model which deals with drip system a working pressure of 12 m is assumed. This is the pressure that is expected to be at the split point of water in the manifold. As the working pressure increases the more pressure head should be provided by the pumping system. To evaluate the effect of working pressure, the model was examined for a number of working pressures from 10 to 30 m with an increment of 2m. The variation of the system cost versus the working pressure is illustrated in Figure 4.19. As can be seen from Figure 4.19, the system cost increases almost linearly with the increase of working pressure for any of the loading case.

According to Equations 4.42 and 4.44-4.48, the allowable pressure variation depends on the maximum pressure in the system. As it decreases, the corresponding allowable pressure variation increases. Thus the model attempts to maintain the pressure variation below the maximum allowable variation. Hence the larger pipe size will be used which causes an increase in the system cost. This can be seen from the graphs shown in Figure 4.19 for the low working pressure (i.e. 10 m) particularly for 8 hours irrigation time (the use of discrete pipe sizes also causes a different change in the slope of system cost). This is why a slight change in the slope of system cost for the low working pressure and also an increase in the system cost for irrigation time of 8 hours have occurred. (It can be concluded that the use of working pressure from this point of view is limited). The graphs shown in Figure 4.19 correspond to the case that position of the manifold and the supply pipes are at centre and the slopes are 0.5% and 0.8% in the X and Y directions.

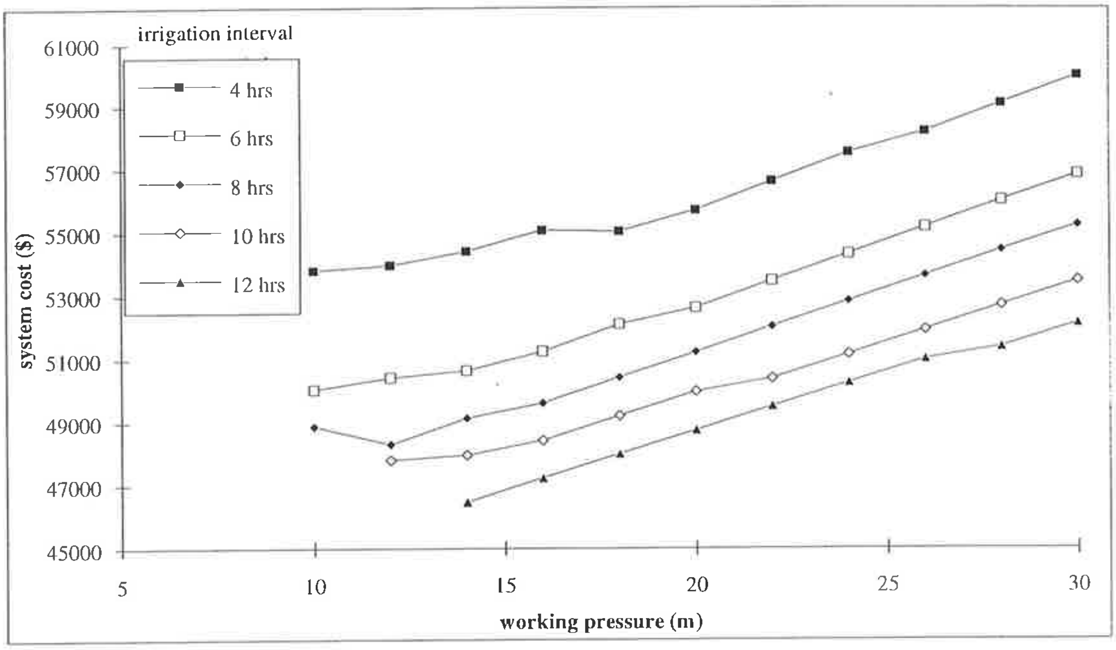


Fig. 4.19 Minimum system cost against the working pressure for a number of loading cases (irrigation interval = 1 day; irrigation time =4 to 12 hrs)

4.11.5 Variations of Groundwater Level

Although the main body of an optimum design of a drip irrigation system may not be directly affected by the variation of groundwater level, a lower groundwater level however may lead to use of a larger pump and also have a higher energy consumption. A pump in a drip irrigation system should provide an adequate pressure for operation and also for compensation of head losses within the system. When the water table is lower than the normal ground surface, extra energy should be provided by the pump to extract the required water from the groundwater and deliver it to the laterals and emitters. The system cost for a number of different groundwater levels from 0.0 to 100 m with the increment of 10 m is examined. As is expected, with the drop of the groundwater level the system cost is increased. The variation of system cost for different discharge rates (resulted from different irrigation times) against the groundwater level is illustrated in Fig 4. 20.

As pointed out earlier, for lower irrigation times which cause a higher design discharge the system cost increases. As Figure 4.20 displays the minimum system cost occurs at the lowest groundwater level and rises linearly as the groundwater level drops. It also indicates that from the five different lines each corresponding to a certain irrigation time, the lower one representing the lower system cost associates with the higher irrigation time (lower discharge) and vice versa. This is again consistent with the earlier findings that the system cost decreased as the irrigation time increased, and also the fact that as the irrigation time decreased the rate of increase in the system cost increased. This can be seen from the distance between the straight lines showing the system cost for different irrigation times (Figure 4.20). In this case also it was assumed that positions of the manifold and supply pipes are at the centre and slopes in the X and Y directions are 0.5% and 0.8% respectively.

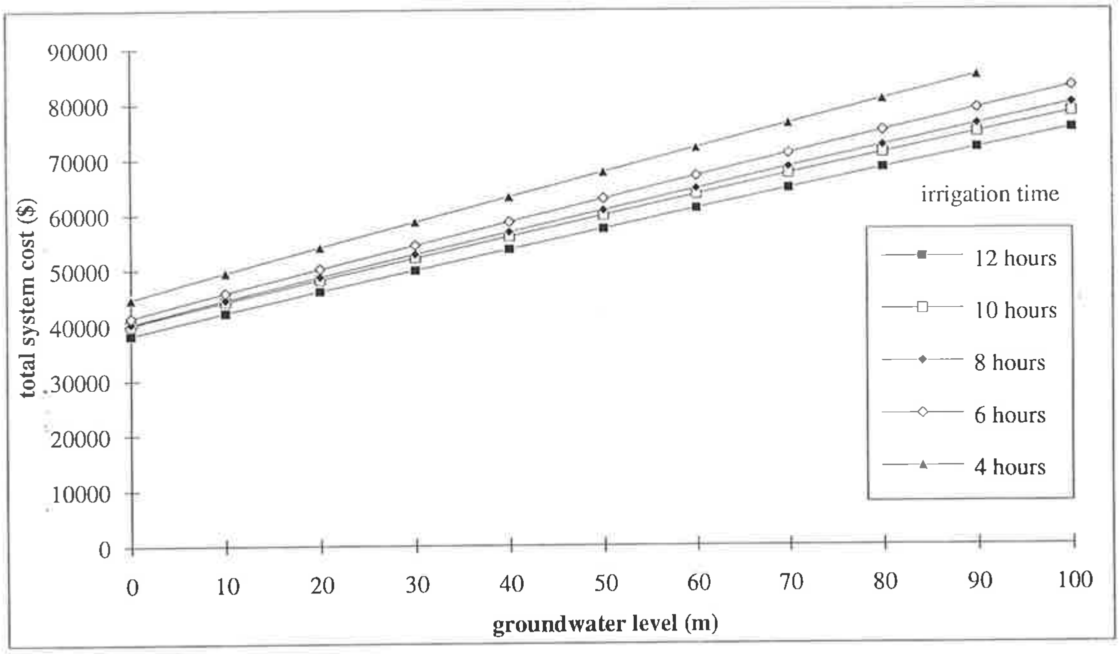


Fig. 4.20 The influence of groundwater level on the system cost (irrigation interval of 1 day with various irrigation times)

4.11.6 The Effect of Delivery Pipe Sizes on the System Cost

On the basis of the piping configuration shown in Figures 4.1a and 4.1b, the supply pipe delivers the whole irrigation water from the source to the manifold. Despite the fact that keeping a limited pressure variation in the multiple outlet pipes is extremely important in order to increase the distribution uniformity, it is not important for the delivery pipes. In order to examine the influence of different sizes of supply pipe on the system cost, a number of discrete sizes are considered. The examination of different supply pipe sizes is carried out independent of pressure variation within the multiple outlet pipes. In other words, the model attempts to find a minimum system cost for each size of supply pipe while also searching the minimum cost for the manifold and laterals on the basis of the procedure explained in the previous sections. Figure 4.21 displays the variation of system cost and the other cost components against the different sizes of supply pipe. The optimum system cost is obtained when a size of 102 mm is used.

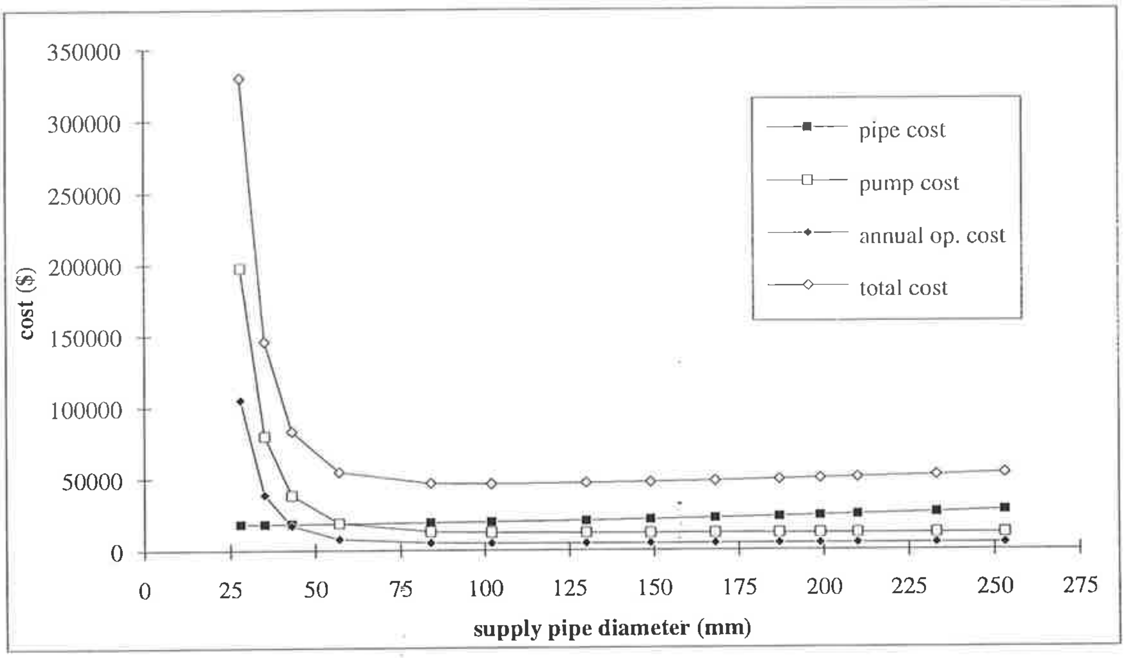


Fig. 4.21 The variation of system cost and component costs for different sizes of supply pipe

For very small sizes of the supply pipe the system cost increases very rapidly. This is due to the high head loss caused by using the small pipe sizes for a specific discharge. In such case, the size of required pump possibly should be enlarged to provide the required pressure for the system operation. Hence, the capital cost of pump and also the operating cost are increased. However, as the size of delivery pipe increases, the systems cost decreases until the optimum size is reached; it then remains relatively constant for a number of sizes larger than the optimum size. But again the system cost increases with a smooth rate for the larger sizes because of pipe cost. As Figure 4.21 indicates, using the large pipe sizes (a few sizes larger than optimum size) does not affect the cost of pump and operation because of a relatively small head loss. These findings provide evidence that the use of larger size of delivery pipes regardless of higher cost would be more economical than using very small sizes even its material cost is lower. The trend of variation depends on the length of delivery pipe and the configuration of the whole system. More details of pump, pipe, present value of annual operating and the system costs with respect to different supply pipe sizes under an irrigation interval of 1 day and irrigation time of 12 hours are given in Table 4.7.

TABLE 4.7 The influence of delivery pipe sizes on the system cost

pipe diameter (mm)	design head (m)	pump power (Kw)	pipe cost ($)	pump cost ($)	annual op. cost ($)	total cost ($)
28	895	113.3	18110	197417	104955	329897
35	328.2	41.5	18180	79724	38485	145804
43	145.5	18.4	18277	38230	17066	82988
57	66.5	8.4	18491	18832	7797	54535
84	43.7	5.5	19064	12891	5126	46495
102	41.2	5.2	19562	12227	4835	46039
130	40.1	5.1	20523	11934	4706	46578
149	39.9	5	21303	11871	4679	47267
168	39.8	5	22187	11841	4666	48109
187	39.7	5	23176	11826	4659	49076
199	39.7	5	23854	11820	4657	49745
210	39.7	5	24511	11816	4655	50398
233	39.7	5	25999	11811	4653	51878
253	39.7	5	27417	11809	4652	53292

4.12 DISCHARGE UNIFORMITY

As pointed out previously, considering a constant value for emitter manufacturing characteristics, the discharge uniformity within the laterals and manifold is controlled by hydraulic pressure. The emission of water from emitters located on the laterals is controlled by the equation $q = ch^{\alpha}$, in which c and α are constant for each emitter with a known flow regime. The emitter discharge is greatly affected by h. This varies along the pipe. For example, a dimensionless curve showing the friction drop pattern given by complete turbulent flow in multiple outlet pipes is shown in Figure 4.22. This curve is obtained from the model and corresponds to the pressure variation within the laterals laying down slope for the irrigation interval of 1 day and irrigation time of 10 hours. The limit for manifold pressure loss depends on the topography, pressure requirement in laterals, and total allowable pressure variation.

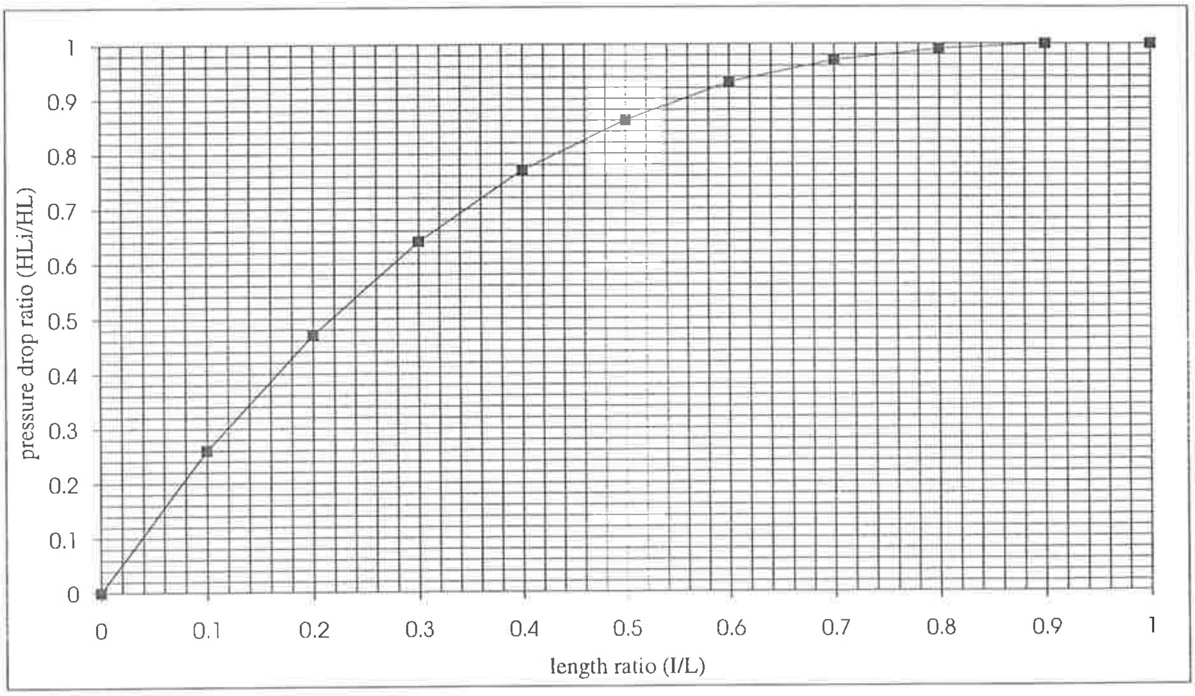


Fig. 4.22 Dimensionless curve showing the friction drop pattern in the laterals (irrigation interval= 1 day; irrigation time=10 hr)

On steep slopes it is very hard to control the pressure variation to meet the constraints therefore, one or more pressure regulating or flow-regulating points may normally be needed for the steep fields. One regulating point may serve one to five laterals (Engineering Handbook, 1984). However, for the case study used in this model, the slope is not steep enough to use more than one pressure regulator for the system.

The pressure variation pattern for a number of different flow rates in the laterals laying down slope is displayed in Figure 4.23. For the larger irrigation times leading to a lower discharge, the pressure drop is much less than for the lower irrigation time leading to a higher discharge. Moreover, as it is shown, the minimum pressure because of the effect of the down slope occurs somewhere between two ends of the multiple outlet pipes, while for the pipes laying up slope the minimum pressure always occurs at the end of pipe.

In the current model, the Christiansen (1932) uniformity coefficient (UC) was calculated for laterals laying down slope for each feasible solution. Since all feasible solutions are obtained considering a good pressure uniformity (overall pressure variation less than 25%) it is expected to achieve a high UC for each case. This uniformity coefficient is based on the sum of the absolute deviations of each observed emitter discharge from the mean discharge (Yitayew and Warrick, 1988) as follows:

$$UC = 1 - \left[\frac{1}{nq_{av}}\right]\sum_{i=1}^{n}\left|q_i - q_{av}\right| \tag{4.63}$$

in which n = number of emitters on each lateral; q_{av} and q_i = average discharge and discharge of $i\,th$ emitter respectively (L/hr). In this study, q_{av} and q_i were found by calculating the average pressure and pressure at i in the lateral pipes and then by using equation 4.3.

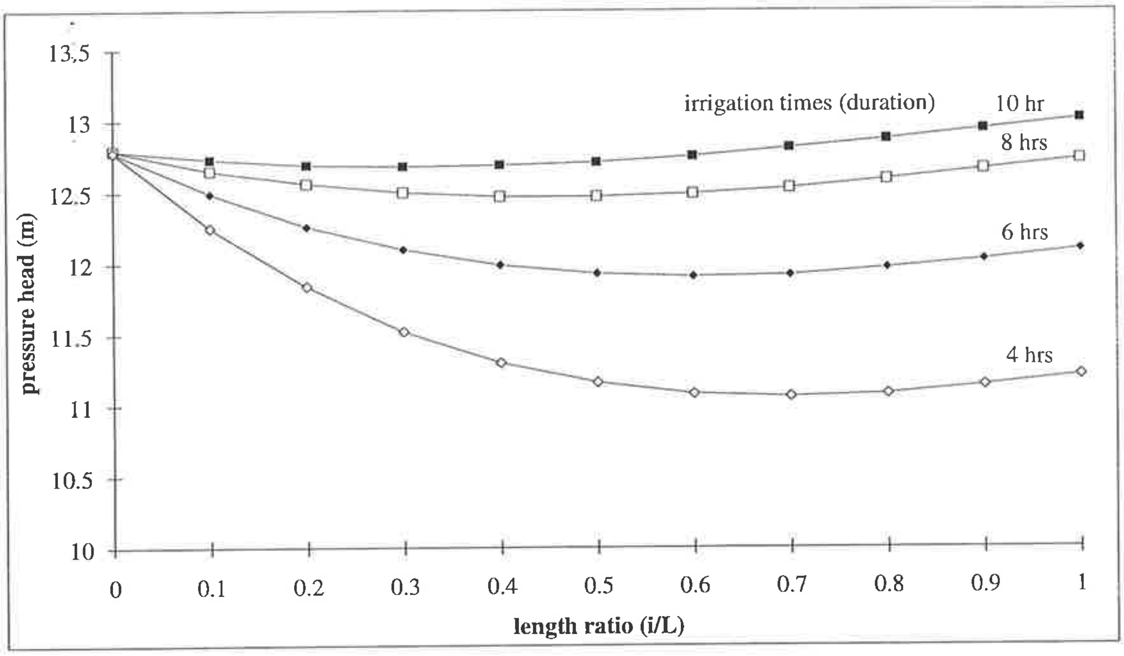


Fig. 4.23 Pressure head in laterals for various loading cases using 4 different irrigation times with irrigation interval of 1 day

However, in practice it is recommended that q_{av} should be calculated by measuring the discharge at each emitter (q_i) individually in the field. The pressure along the laterals is calculated by applying Equation 4.6 which is obtained considering 10 equal sections along the laterals. The values of UC based on Equation 4.63 for various loading cases and a fixed position of manifold and supply pipe for the slope of 0.5% and 0.8% are shown in Table 4.8. The values of UC indicate that there is a good uniformity within the laterals.

TABLE 4.8 The uniformity coefficient (Christiansen) for various loading cases in laterals

irrigation interval (day)	irrigation time (hr)	pump discharge (m3/s)	design head (m)	uniformity coefficient	pipe cost ($)	pump cost ($)	annual op. cost ($)	total cost ($)
0.5	4	0.01314	42.95	**0.996**	13906	43272	5036	48309
0.5	8	0.00657	40.56	**0.994**	11257	39920	4757	44677
0.5	**10**	**0.00525**	**41.2**	**0.993**	**10845**	**39010**	**4832**	**43842**
0.5	12	0.00438	40.76	**0.993**	10297	38462	4780	43242
1	4	0.02627	41.53	**0.972**	15828	48870	4870	53740
1	6	0.01751	41.37	**0.986**	14367	45569	4852	50420
1	8	0.01314	42.95	**0.996**	13906	43272	5036	48309
1	10	0.01051	41.83	**0.996**	12898	42896	4905	47801
1.5	4	0.03941	43.63	**0.962**	18172	54159	5117	59276
1.5	6	0.02627	41.53	**0.959**	15828	48870	4870	53740
1.5	8	0.0197	41.79	**0.988**	14897	46344	4901	51245
1.5	10	0.01576	41.07	**0.995**	13928	44500	4816	49316
1.5	12	0.01314	42.95	**0.996**	13906	43272	5036	48309
2	6	0.03503	42.85	**0.958**	17399	51686	5025	56711
2	8	0.02627	41.53	**0.962**	15828	48870	4870	53740
2	10	0.02102	42.07	**0.979**	15210	46656	4933	51589
2	12	0.01751	41.37	**0.988**	14367	45569	4852	50420

The results of this work and those given by Helmi et al, 1993; and Warrick and Yitayew, 1988; are very close and for some loading cases are the same. A common value of the Christiansen coefficient is 0.97 (Helmi et al, 1993) and the results of this work ranges from 0.958 to 0.996 (Table 4.8). Nevertheless, the value of UC obtained by the above process is not recommended. To reach a reliable UC it is recommended the average discharge and the discharge of each emitter be measured directly from the field. In this way, all parameters affecting the discharge emission including the manufacturing characteristics will be taken into account.

4.13 SUMMARY

An optimisation model for a drip irrigation system on sloping lands with one control head was developed. The model evaluates the main factors affecting the optimum design of a drip system with a known piping configuration. Although a discharge correction factor for multiple outlet pipes is recommended by some authors, in this study a set of equations for the head loss, pressure head and also the position of the minimum and the maximum pressure head for the multiple outlet pipes were developed (Section 4.7).

Achieving a desirable uniformity of discharge considering the effect of slope patterns makes the model more elaborate compared with the same piping system on flat terrain. In this model, not only the position of the supply and the manifold pipes but also different slope patterns were examined. In addition, the model evaluates the effect of variations of groundwater level and delivery pipe (supply pipe) sizes under different operating schedules.

The decision variables are the optimum position and the size of two segments of manifold and supply pipes, the optimum length of laterals in either side of manifold and also the optimum length of two given sizes under different operating conditions. The system cost includes the cost of the piping system, the pump, the present value of annual operating cost and the accessory costs. The findings indicate that there is a 5.0% saving in the system cost when the manifold and supply pipes are located at the optimum position compared to a location in the middle of the field. This might be due to the effect of the down slope which leads to the use of smaller sizes for laterals and manifold throughout the system. The findings showed that as the slope becomes steeper the optimum position of both pipes (manifold and supply) moves toward the upper border. For a slope of 0.5 % and 0.8 % in the X and Y directions and irrigation interval of 0.5 day and irrigation time of 10 hours the optimum positions of manifold and supply pipes were 4 m and 58.5 m from the upper end of the corresponding pipes (L_{lu}=4m; L_{mu}= 58.5m, see Fig. 4.17).

As shown in Figure 4.16 the system cost for a fixed slope in the X direction is constant for mild slopes in the Y direction until 0.6% is reached, then it drops at slope of 0.8% and again increases for the higher slopes. On the other hand for a fixed slope in the Y direction as slope in the X direction increases the system cost decreases. The global minimum cost obtained at the slope of 0.6% in the X and 0.8% in the Y directions. For slopes higher than 0.9% in the X and 1.2% in the Y direction due to the pressure variation constraint (for the case study examined) there are no any feasible solutions for the range of pipe sizes considered.

The findings from the implementation of different groundwater levels indicate that the system cost increases almost linearly with the increase of depth of groundwater level and the rate of increase for the lower irrigation times (higher flow rate) at each level is higher than that for the higher irrigation times. The results obtained from employing different delivery pipe sizes (supply pipe) showed that the optimum solution is obtained when a size of 102 mm is used. As Figure 4.21 indicates the use of very small sizes brings about a considerable increase in the system cost, while using the large sizes (larger than the optimum size) causes an increase but with a very smooth trend.

As the slope in either direction or design discharge increase and the allowable pressure variation decreases, the number of feasible solutions decreases. The Christiansen uniformity coefficient of emitters on laterals is calculated for various loading cases. The range of calculated uniformity coefficients varies from 0.958 to 0.996 and indicates that discharge uniformity within the laterals is quite desirable. It is very close to that given by Warrick and Yitayew, 1988; and also Helmi et al, 1993. The overall results indicate that to achieve a desirable distribution uniformity the use of pressure regulator(s) for the steep slopes is essential and; as the slope increases the position of manifold and supply pipes should be moved towards the up slope.

Chapter 5

Optimum Design of Multiple Subunit Drip Irrigation Systems

5.1 INTRODUCTION

This chapter outlines a model which can be used to optimise the design and operation of a drip irrigation system for a flat rectangular field divided into subunits. The advent of microprocessor control allows considerable flexibility in the layout and watering patterns used to irrigate any particular field. In this model, the designer is free to choose the following variables: (1) the dimensions of rectangular subunits into which the field will be sub divided; (2) the number of irrigation shifts (ie. one shift operation involves watering the whole field simultaneously, two shifts involve watering half the field simultaneously, etc.); (3) the shift pattern (i.e. different combination of subunits are irrigated simultaneously); (4) the lengths of two sizes of lateral pipes; (5) the size of the manifold, supply line, submain and mainline pipes (the layout of these pipes is assumed to follow a fixed pattern); and (6) the pressure head and hence the power required by the pump. Other decision variables such as the irrigation time (duration), irrigation interval and discharge rate of the emitters are determined by agronomic requirements. The model described in this chapter, identifies values for the above variables so as to minimise the present value of the capital and operating costs of the system.

Constraints on the system include: (1) ensuring that the crop water requirements are met; (2) ensuring that the rate of application does not exceed the infiltration capacity of the soil; (3) ensuring that the uniformity coefficient of emitters within each subarea lies within acceptable bounds; (4) ensuring that the depth of water applied during an irrigation cycle does not exceed the storage capacity of the soil; (5) ensuring that the percentage wetted area of the crop lies within acceptable bounds. The model represents a considerable extension of the work of Oron and Walker (1981) which only considered one shift operation and assumed that the pressure head at the pump is known.

The minimum cost of an irrigation system is not always found during the traditional design process, because of the difficulties in selecting the most economic layout among the many alternatives (Oron, 1982). Oron (1982) examined the alternative layouts of a sprinkler irrigation system. Oron noted that due to the difference in the size of subdivisions of two similar field areas, there was a trade off among the cost of system components for each particular layout. He added that differences in system cost occur due to changing the percentage of different pipe lengths in each particular layout.

The purpose of this part of the study outlined in this chapter is to investigate both the optimum size and optimum dimensions of subunits in a multiple subunit system, while also examining the effect of the number of possible shifts and shift patterns on the system cost. This is carried out by selecting the optimum diameters for all pipes via a trade off between the cost of pipes and required energy. The analysis is performed by developing an optimisation model based on the complete enumeration approach for drip irrigation in a multiple subunit system.

5.2 SYSTEM LAYOUT AND IRRIGATION PARAMETERS

The model developed in this study will identify the optimum drip irrigation system for a flat rectangular field. The assumed layout of the irrigation system is shown in Figure 5.1. It consists of a pump, filter and fertiliser units at the centre of the field (assumed location of bore). There is one pressure regulator for each subunit and one on-off valve for each main and submain pipe. It is assumed that the field is supplied from groundwater. The pipe system consists of two main pipes which deliver the water from the pump to the submain pipes and a set of multiple outlet pipes within the subunits. (micro pipes including: laterals, manifold and supply pipes for a subunit are shown in the top left side of Figure 5.1). The multiple outlet pipes receive water from the submains via the supply pipes and distribute it through the emitters with a slow rate of application.

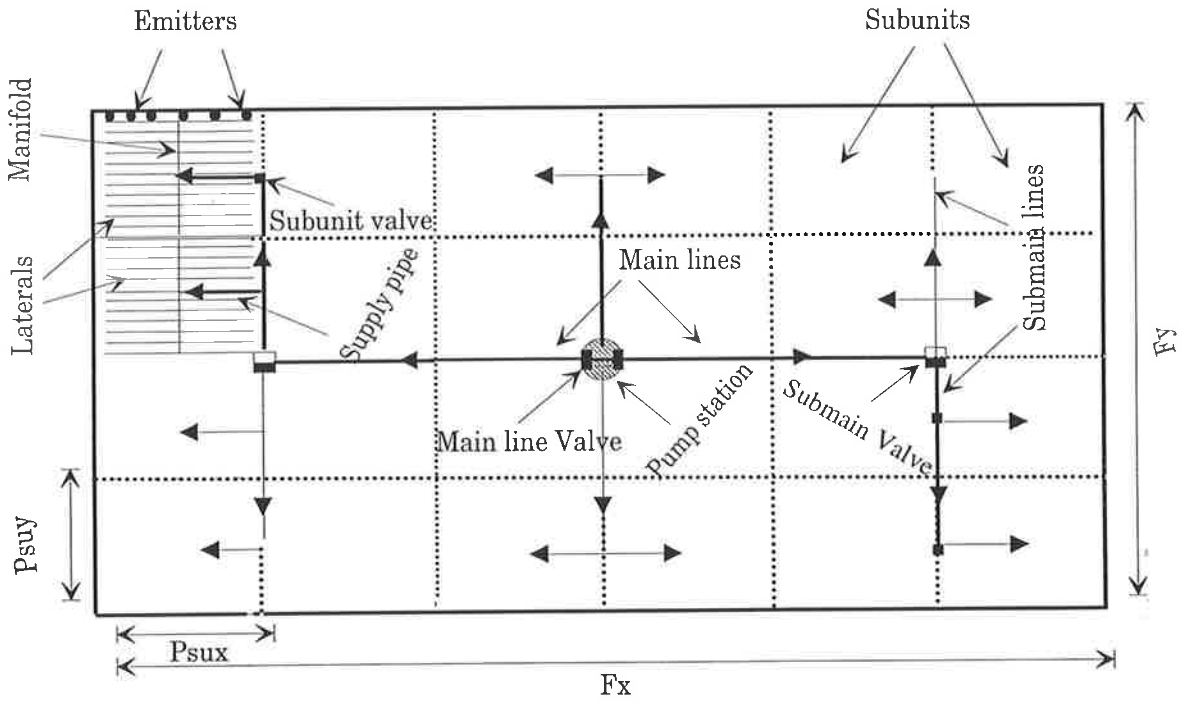


Fig. 5.1 Layout of a multiple subunit drip irrigation system with 24 subunits

The concepts of irrigation interval (F) and irrigation time (T) for one cycle of irrigation are illustrated in Figure 5.2. The irrigation interval is the time in days between the commencement of one irrigation cycle and the next. The irrigation time (duration) is the length of an irrigation event. That is, the period during which water is being released from the emitters for one particular irrigation shift (considering a 4-hour off per 24 hours). The number of irrigation shifts (N_{sh}) refers to schedule of irrigation with different irrigation times and flow rates from the emitters. For example, one shift operation involves watering the entire field simultaneously. Two shift operation involves irrigating half the field for half the time and then other half for the remaining time and so on.

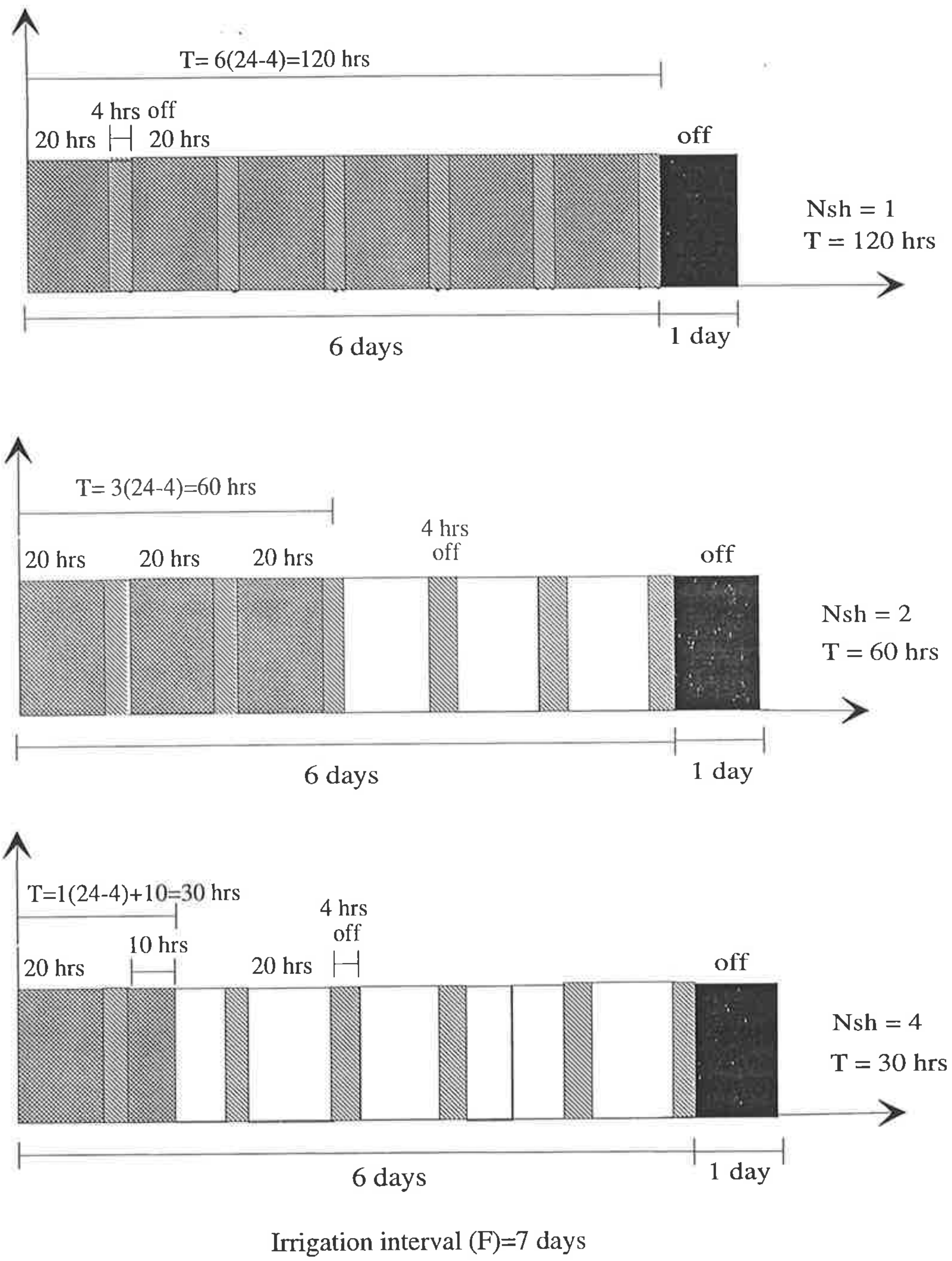


Fig. 5.2 An example of irrigation interval and irrigation time for three different numbers of irrigation shifts

The relationship between F, T and N_{sh} may be defined as follows:

$$T = \frac{D_h\left(F - N_f\right)}{N_{sh}} \tag{5.1}$$

where T = irrigation time (duration) for each shift (hr); N_f = number of days free of irrigation per irrigation cycle (day); D_h = time available per day for irrigation (hr).

The shift pattern in this study refers to the combination of subunits being irrigated simultaneously (This illustrated in Figure 5.3). In Figure 5.3 II, JJ = the number of subunits being irrigated simultaneously in the X and the Y directions (respectively). Both II and JJ are factors of the number of subunits in the X and Y directions (respectively).

5.3 NUMBER OF IRRIGATION SHIFTS

In this model, irrigation shift refers to the number of sets of subunits which are to be irrigated simultaneously during a specified irrigation interval. In each shift the number of irrigated subunits consists of the product of a number of subunits in the X-direction (II) and a number of subunits in the Y-direction (JJ). As the number of shifts increases the number of subunits which must be irrigated simultaneously decreases, as a result, the irrigation time for simultaneous irrigated subunits decreases. Since the whole system is scheduled to be irrigated during a specified irrigation interval and the design flow rate (Q_{pu}) is constant under any operating condition, any decrease in irrigation time leads to an increase in pipe and emitter flow rates, as a result, the head loss of corresponding pipes is increased. Therefore, only a limited number of shifts are feasible. This is due to possible violation of the head loss constraints. In the present model, only three cases (1, 2 and 4 shifts) are considered for cost analysis.

5.4 ADVANTAGE OF PARTITIONING A FIELD INTO SUBUNITS

For drip irrigation the advantages of partitioning a field into subunits are as follows:

• In the case of limited available water, the irrigation system may be designed in such a way that the field will be irrigated unit by unit with a desirable control.

• Large fields have long lengths of pipe and have higher head losses. The subunits allow shorter lengths of pipe to be used.

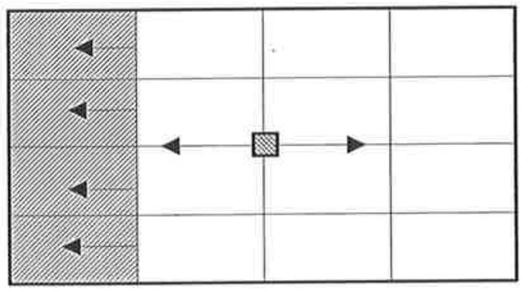
• When a field is divided into smaller units the sizes of the pipes and control unit including: the valves, pressure regulators, discharge regulators, etc. can be reduced. According to Oron and Walker (1982) *irrigation systems which consist of a relatively large number of small diameter control units are probably more flexible in operation, although their cost might be higher as compared to a system with a smaller number of control units.*

• Dividing the whole field into subunits with proper dimensions leads to more effective control of the irrigation systems and enhances the reliability of the system.

• By partitioning a field into subunits different set of subunits may be irrigated separately, a part of the field may be kept dry and agricultural activities such as: fertilisation, plowing, fruit picking, spraying, and other soil treatments can be carried out more easily and efficiently.

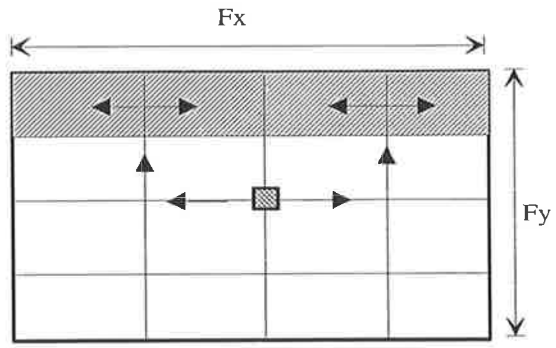
5.5 CHARACTERISTIC OF THE MODEL

The model has been developed for fields with known dimensions on flat terrain. The water source is assumed to be groundwater provided by a pump located at the centre of field. All submain pipes that feed the subunits via supply pipes are perpendicular to the mainlines and are fed from both sides of the mainlines (see Figure 5.1). All pipes are made from polyethylene, and emitters are fixed on the laterals at a fixed spacing. Each supply, submain and mainline pipe is controlled by one independent valve, which is located just at the beginning of the corresponding pipe. One filter unit is assumed to be located just after



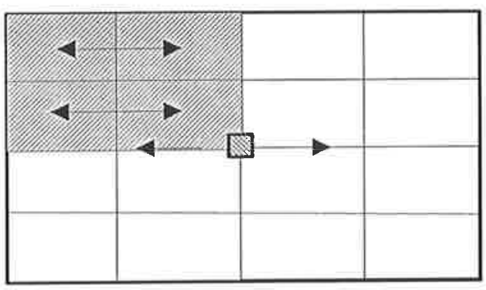
Nsh=4 II=1 JJ=4

Pattern No. 1



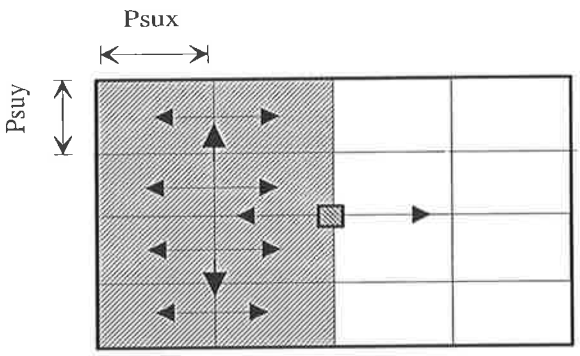
Nsh = 4 II=4 JJ=1

Pattern No. 2



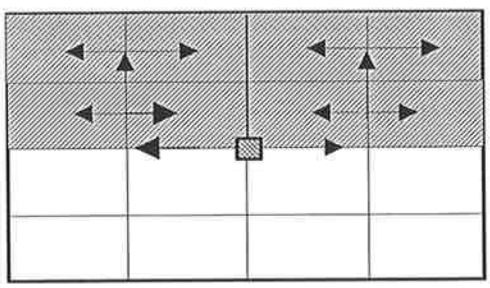
Nsh=4 II=2 JJ=2

Pattern No. 3



Nsh=2 II=2 JJ=4

Pattern No. 4



Nsh=2 II=4 JJ=2

Pattern No. 5

Fig. 5.3 An example of different shift numbers with associated shift patterns in a multiple subunit drip irrigation system (with 16 subunits)

the pump. Water is assumed to be extracted from groundwater by means of a turbine pump system. The main and submain pipes are buried while subunit pipes (laterals, manifold, supply) are laid on the ground. Total system cost consists of capital and installation costs plus the present value of the operating costs over the expected life of the project.

5.6 FORMULATION OF MODEL

5.6.1 Objective Function

The drip irrigation design model described in this paper consists of an objective function that minimises the sum of the capital cost and present value of operating cost subject to appropriate constraints. The system is assumed to be permanent with semi-automation, thus labor cost is considered to be small compared to the capital and energy costs. The main components of cost of a drip irrigation system equals the cost of pipes, pump, emitters, accessories, and energy.

The objective function is defined as follows:

$$Z = C_p + C_{pu} + C_{em} + C_{ac} + C_{op} \qquad (5.2)$$

The cost of pipes can be expressed as:

$$C_p = C_l + C_m + C_s + C_{sm} + C_{ml} \qquad (5.3)$$

where C_{sm}, C_{ml} = cost of submain and mainline pipes (respectively). A typical pipe configuration and the other accessories are shown in Figure 5.1.

The cost per unit length of pipes (other than laterals) including submain and mainline pipes are expressed by Equation 3.3 (Chapter 3). Least squares was used to identify the constants in Equation 3.3 from pipe cost data as shown in Equation 3.4. In this model, submain and mainline pipes are assumed to be buried, hence installation cost needs to be added to Equation 3.4 for submain and mainline pipes.

5.6.2 Subunit Dimensions and Pipe Lengths

According to the piping configuration shown in Figure 5.1 the length of different pipes and subunit dimensions are obtained using the following equations:

$$P_{sux} = \frac{F_x}{N_x} \tag{5.4}$$

where P_{sux}= the length of subunits in the X-direction (m); F_x= the length of field in the X-direction (m); N_x= the number of subunits in the X-direction (assumed to be even).

$$P_{suy} = \frac{F_y}{N_y} \tag{5.5}$$

where P_{suy}= the length of subunits in the Y-direction (m); F_y= the length of field in the Y-direction (m); N_y= the number of subunits in the Y-direction (assumed to be even). The higher numbers of N_x and N_y provide more subunits with smaller area within the field. In this model up to 10 divisions in the two directions are considered. The length of different pipes based on subunit dimensions are computed as follows:

$$L_l = \frac{P_{sux}}{2} - d_x \tag{5.6}$$

$$L_m = P_{suy} - d_y \tag{5.7}$$

$$L_s = \frac{P_{sux}}{2} \tag{5.8}$$

$$L_{sm} = \frac{F_y}{2} - \frac{P_{suy}}{2} \tag{5.9}$$

$$L_{ml} = \frac{F_x}{2} - P_{sux} \tag{5.10}$$

where L_{sm}, L_{ml} are the length of submain and mainline pipes respectively (m).

5.6.3 Number of Different Components in the System

The number of subunits created in each iteration of field division depends on the length and the width of subunits as below:

$$N_{su} = \frac{F_x}{P_{sux}} \cdot \frac{F_y}{P_{suy}} \tag{5.11}$$

where N_{su} = the number of subunits within the field.

Also a number of some other system components are computed as follows:

$$N_{el} = \frac{P_{sux}}{2 d_x} \tag{5.12}$$

$$N_{lm} = 2 \frac{P_{suy}}{d_y} \tag{5.13}$$

$$N_{ems} = \frac{P_{sux}}{d_x} \cdot \frac{P_{suy}}{d_y} \tag{5.14}$$

$$N_{em} = \frac{P_{sux}}{d_x} \cdot \frac{P_{suy}}{d_y} N_{su} \tag{5.15}$$

where N_{ems} and N_{em} are the number of emitters in each subunit and in the system (respectively).

$$N_l = 2 N_{su} \frac{P_{suy}}{d_y} \tag{5.16}$$

$$N_m = N_{su} \tag{5.17}$$

$$N_s = N_{su} \tag{5.18}$$

$$N_{sm} = \frac{F_x}{P_{sux}} \tag{5.19}$$

where N_l, N_m, N_s N_{sm} are the number of laterals, manifold, supply and submain pipes in the system (respectively).

According to the piping configuration shown in Figure 5.1, in addition to the laterals and emitters, there is one manifold, one supply pipe, and one volumetric valve within each subunit. The valve is located just at the beginning of supply pipe to control the flow into each subunit. The number of mainlines = 2 if N_x>2. For each submain and mainline pipe, one volumetric valve is considered which is located at the beginning of the corresponding pipe.

5.6.4 Cost of System

Cost of Pipes

The details of system cost are as follows:
Similar to the previous models outlined in Chapters 3 and 4, two available discrete pipe sizes are assumed for laterals. The corresponding expression is given in Equation 3.11. The cost of other pipes within each subunit may be expressed as:

$$C_m = \left(K_1 D_m^2 + K_2 D_m + K_3\right)\left(P_{suy} - d_y\right) N_{su} \tag{5.20}$$

K_1, K_2, K_3 = constant for the pipe cost equation.

$$C_S = \left(K1D_s^2 + K2D_s + K3\right)\left(\frac{P_{sux}}{2}\right)N_{su} \tag{5.21}$$

For a pump system located at the centre of the field and the pipe configuration shown in Figure 5.1, the costs of submain and mainline pipes are

$$C_{sm} = \left(K1D_{sm}^2 + K2D_{sm} + K3 + \phi\right)\left(\frac{F_y}{2} - \frac{P_{suy}}{2}\right)N_{sm} \tag{5.22}$$

D_{sm} = diameter of submain pipes (mm); ϕ = the cost of installation per unit length ($/m).

$$C_{ml} = \left(K1D_{ml}^2 + K2D_{ml} + K3 + \phi\right)\left(\frac{F_x}{2} - P_{sux}\right)N_{ml} \tag{5.23}$$

D_{ml} =diameter of mainline pipes (mm); N_{ml} = the number of mainline pipes.

Cost of Pumping System

The cost of the pumping system (pump, electric motor and corresponding accessories) and the procedure for developing the pump cost equation is discussed in Chapter 4. The final pump cost equation used in this study is shown in Section 4.8.5.1. In Equation 4.39 the dynamic pumping head, H_{pu}, including all head losses within the system and the required pressure for system operation as well as the depth of groundwater may be formulated as below:

$$H_{pu} = \sum HL + H_w + H_{wt} \tag{5.24}$$

where $\sum HL$= the sum of all head losses from the pump to most distant emitter (m); H_w = minimum working pressure head on emitters (m); H_{wt} = depth to water table (m).

Cost of Control Head

The control head in this multiple subunit system which serves all subunits consists of: the volumetric valves, the fertiliser and chemical tank equipment as well as the filtering equipment. An example of a typical control head with corresponding equipment is shown in Figure 5.4. The main components of control head are defined in more details as follows:

Fertilizer and Chemical Injection Equipment

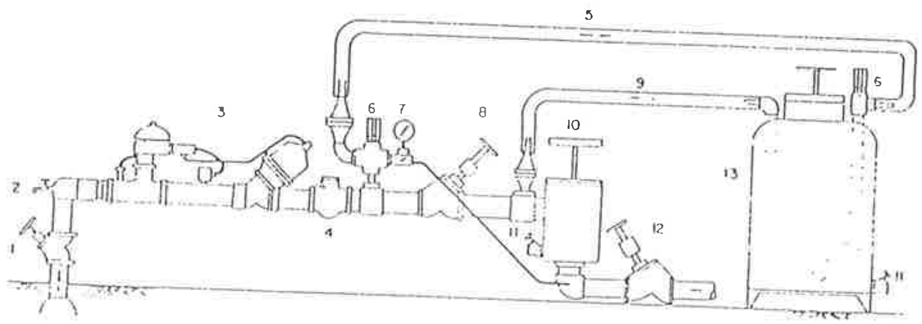
Injectors may be used to apply fertilisers or other chemicals directly into the drip irrigation systems. Correct application of fertiliser or chemical equipment is essential if higher yields are to be obtained in drip irrigation systems. A fertiliser tank, in which the required fertiliser or other chemicals are dissolved in water may be connected to the main system by means of two small pipes. This forces the water to flow through the inlet small pipe (tubing) into the tank and pushes the fertiliser solution through the outlet pipe back into the system (see Figures 5.4 and 5.5).

Filtering Equipment

Filters in the drip irrigation system are essential in order to reduce the risk of blockage or clogging in emitters due to soil particles and organic materials suspended in the water. This type of filter commonly has either a single or double screen. Head loss data for a clean filter are supplied by the manufacturers and should be taken into account in the design of drip irrigation systems. Sometimes in addition to the filters in the control head, small screen filters can be installed at the inlet to laterals. These extra filters are useful when laterals are portable or when the water has a high level of suspended materials. The type, size and the number of required filters depend on the quantity of the water and the discharge in the control head. The filtration system sometimes comprises several filters. However, filters do not overcome the problems of precipitation of calcium carbonate deposit. To solve this problem the system must be flushed periodically with a solution of hydrochloric acid, and then with compressed air under high pressure.

Valves and Controller Unit

Figure 5.4 displays the function of various type of valves used at the head of a typical drip irrigation system. The automatic metering valves are set in the system to allow the passage of a given volume of water, after which it shuts down automatically. Some valves also function as water meters.



1	Main valve	8	Pressure reducing valve
2	Tap	9	Fertiliser input pipe
3	Water meter & volumetric valve	10	Water filter
4	Non return valve	11	flushing valve
5	Inlet pipe to fertiliser applicator	12	Field supply valve
6	Air valve	13	Fertiliser application tank
7	Pressure gag		

Fig. 5.4 Components of a typical control head for a drip irrigating system

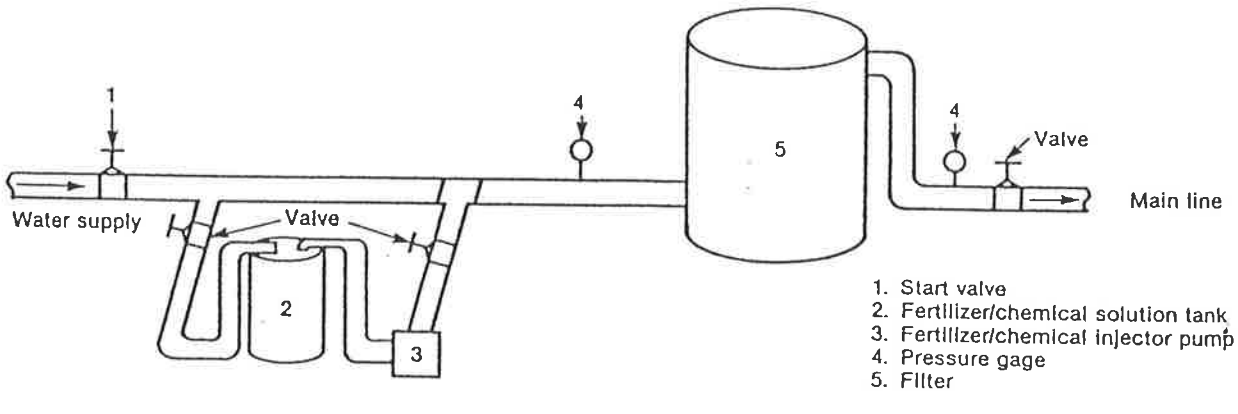


Fig. 5.5 Location of fertiliser, chemical solution tank and various valves in a typical irrigation system

Several valves can be hydraulically operated in sequence, thus minimising labor requirements while increasing the efficiency of water application. An automatic metering valve is selected on the basis of the required volume of water and the design flow-rate. The amount of head loss for any type of valve is normally specified by manufacturers. In this model for each mainline, submain and supply pipe one automatic metering valve has been considered. In the present model, for each set of 8 subunits one controller unit has been recommended. The recommended type of controller is powered by electricity supplied by a battery or other sources. It controls and adjusts the pressure as well as the flow rate of subunits.

The cost of the control head considering all accessories in the system may be expressed as follows:

$$C_{acc} = C_{fil} + C_{fer} + C_{con} + C_{val} \tag{5.25}$$

C_{fil} = the cost of the filter ($): C_{fer} = the cost of the fertiliser ($): C_{con} = the cost of the controller ($); C_{val} = the cost of the valves ($).

Although the cost of the above accessories could be presented as a function of the diameter, in this analysis a constant appropriate size with known cost and reasonable friction head loss are assumed for each one.

Emitters

The emitters are installed on the laterals beside the plants. In some drip irrigation systems the number of emitters around each plant is increased in order to supply the required volume of irrigation water to match the different stages of plant growth or plant activities. (Holzapfel et al, 1990). The total cost of emitters is based on the number and the unit cost as below:

$$C_{em} = N_{su}\left(\frac{P_{sux}}{d_x} \cdot \frac{P_{suy}}{d_y}\right) CU_{em} \tag{5.26}$$

Annual Operation Cost

The cost of energy to operate the system on an annual basis can be represented by the unit cost of energy multiplied by the total energy required over the operating season. The annual energy requirement depends on the annual irrigation requirement and the power of the pump providing the water. More details including the present value of annual operation cost (Equation 4.41) are given in Section 4.8.5.2.

The annual energy requirement may be obtained using the annual irrigation requirement, power of pump and the annual hours of pump operating as follows:

$$A_{e_n} = P_m A_{ir} \left(\frac{E_a \, T \, N_{sh}}{K_c \, ET_0 \, F} \right) \tag{5.27}$$

Total Cost

The main aim of the study outlined in this chapter, is the evaluation of a multiple subunit drip irrigation with respect to various subunit sizes under a specified number of irrigation shifts, and achieving an optimum solution among various alternatives. The total system cost consists of the capital and the present value of annual operating cost (electric energy). The capital cost includes: the cost of emitters, laterals, manifold and supply pipes within the subunits, the cost of submain and mainlines, the cost of the pump system (turbine pump, vertical hollow shaft electric motor) and the cost of the control head (filter, fertiliser tank, volumetric valves, controller units).

5.6.5 Discharge

In order to identify the size of pump and piping system, it is essential to determine the design discharge and the discharge in each individual pipe in each shift pattern. In the following sections the discharge of different pipes associated with the different irrigation shifts and shift patterns are presented.

Subunit Discharge

The flow rate of different elements of a drip irrigation system is based on crop transpiration and the operating conditions. In practice, an operating schedule based on the available time, the soil holding capacity and the other constraints is developed. Thus the amount of water which should be stored in the soil to meet the plant water requirement is directly affected by transpiration rate and soil characteristics. Although the transpiration rate under trickle irrigation is a function of the conventionally computed consumptive use rate and the extent of the plant canopy, in this study the crop coefficient factor and potential evapotranspiration are considered for the consumptive use as shown in Equation 3.1 and 3.23.

The discharge of emitters and the other elements is affected by the irrigation interval, the number of shifts, the irrigation duration and the application efficiency as follows:

$$N_{sh} = \frac{N_{sux}\ N_{suy}}{II\ JJ} \tag{5.28}$$

$$T = \frac{D_h \left(F - N_f\right)}{N_{sh}} \tag{5.29}$$

$$Q_E = \frac{K_c ET_0 F}{E_a\ T} d_x d_y \tag{5.30}$$

The irrigation interval (F) is estimated using Equation 5.63 in which the water that can be stored in the root zone and average daily transpiration for peak-use period are taken into account. The discharge of subunits (Q_E) is the total discharge of emitters working in each subunit as given below:

$$Q_{su} = Q_m = Q_s = 2.78 \times 10^{-7} \left(Q_E \frac{P_{sux}}{d_x} \cdot \frac{P_{suy}}{d_y} \right) \tag{5.31}$$

where N_{sux}, N_{suy} = the number of subunits in the X and Y directions respectively; Q_{su} =discharge in the subunits (m^3 /s).

Submain Line Discharge

The discharge of submain pipes varies not only with the number and the size of subunits supplied by those lines, but also varies with different number of irrigation shifts and shift patterns. As a result, the size and the number of subunits that are created by the field divisions as well as the number of applied shifts with the corresponding shift patterns are the main factors which affect the discharge of submain lines. For example, the discharge of the submain lines for one solution in which N_x=6 and N_y varies from 2 to 10, under 3 applied operating programs with 1, 2 and 4-shift may be formulated as follows:

For N_x=6

when N_y =2

$$Q_{sm}=2Q_{su} \qquad \text{if } N_{sh}=1, 2 \text{ or } 4 \tag{5.32}$$

when N_y=4 (see Fig. 5.6)

$$Q_{sm}=4Q_{su} \qquad \text{if } N_{sh}=1, 2 \tag{5.33}$$

$$Q_{sm}=4Q_{su} \qquad \text{if } N_{sh}=4 \text{ and II=3, JJ=2} \tag{5.34}$$

$$Q_{sm}=2Q_{su} \qquad \text{if } N_{sh}=4, \text{ and II=6, JJ=1} \tag{5.35}$$

when N_y=6

$$Q_{sm}=6Q_{su} \qquad \text{if } N_{sh}=1, 2 \text{ or } 4 \tag{5.36}$$

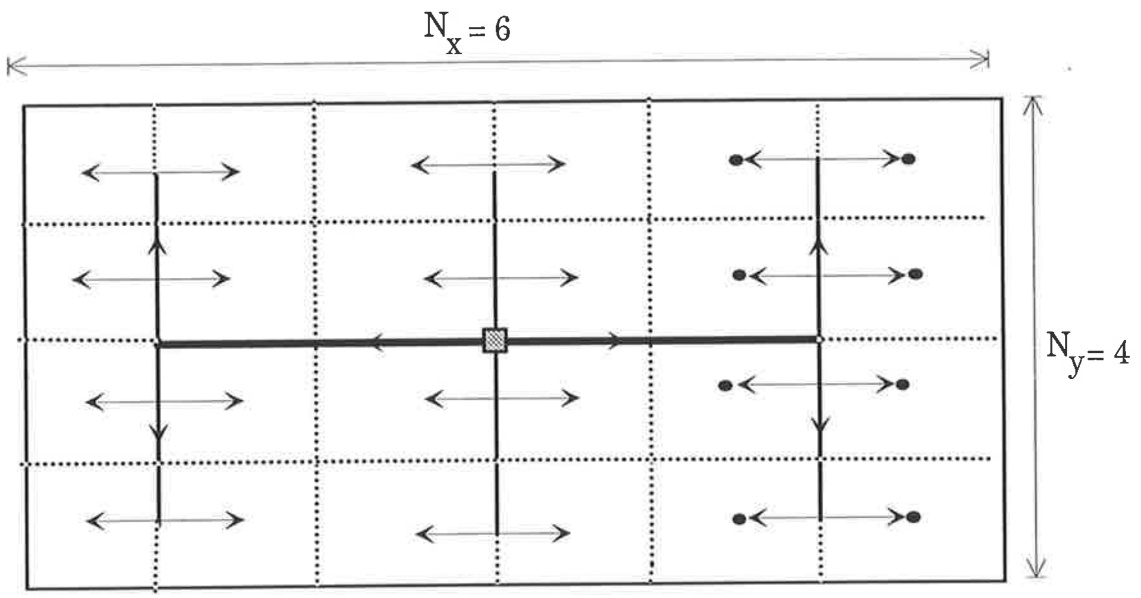


Fig. 5.6 An example of multiple subunit system with 24 subunits (4 subunits supplied by each submain line)

when N_y=8 (see Fig. 5.7)

$$Q_{sm}=8Q_{su} \qquad \text{if } N_{sh}=1,\ 2 \tag{5.37}$$

$$Q_{sm}=8Q_{su} \qquad \text{if } N_{sh}=4, \text{ and II=3, JJ=4} \tag{5.38}$$

$$Q_{sm}=4Q_{su} \qquad \text{if } N_{sh}=4, \text{ and II=6, JJ=2} \tag{5.39}$$

when N_y=10

$$Q_{sm}=10Q_{su} \qquad \text{if } N_{sh}=1,\ 2 \text{ or } 4 \tag{5.40}$$

Mainline Discharge

As shown in Figures 5.1,and 5.6 there are 2 mainlines in the distribution system, which deliver the irrigation water from the source to the submain lines. However, when the division of the field in the X-direction is 2 (N_x=2), mainlines are not required and water is delivered only by submain lines. The mainline discharge depends on the number of connected submain lines supplied by each mainline in each shift, the size and the number of subunits connected to the submain lines, the number of shifts as well as the shift patterns. For example, the discharges of the mainlines for the previous case discussed for submain line discharges (N_x=6 for different N_y) may be formulated as follows:

For N_x=6

when N_y=2

$$Q_{ml}=4Q_{su} \quad \text{if } N_{sh}=1 \tag{5.41}$$

$$Q_{ml}=2Q_{su} \quad \text{if } N_{sh}=2 \text{ and II=6, JJ=1} \tag{5.42}$$

$$Q_{ml}=4Q_{su} \quad \text{if } N_{sh}=2 \text{ and II=3, JJ=2} \tag{5.43}$$

$$Q_{ml}=2Q_{su} \quad \text{if } N_{sh}=4 \tag{5.44}$$

when N_y=4 (see Fig. 5.6)

$$Q_{ml}=8Q_{su} \quad \text{if } N_{sh}=1 \tag{5.45}$$

$$Q_{ml}=4Q_{su} \quad \text{if } N_{sh}=2 \text{ and II=6, JJ=2} \tag{5.46}$$

$$Q_{ml}=8Q_{su} \quad \text{if } N_{sh}=2 \text{ and II=3 and JJ=4} \tag{5.47}$$

$$Q_{ml}=4Q_{su} \quad \text{if } N_{sh}=4 \tag{5.48}$$

when N_y=6

$$Q_{ml}=12Q_{su} \qquad \text{if } N_{sh}=1 \tag{5.49}$$

$$Q_{ml}=6Q_{su} \qquad \text{if } N_{sh}=2 \text{ and II=6, JJ=3} \tag{5.50}$$

$$Q_{ml}=12Q_{su} \qquad \text{if } N_{sh}=2 \text{ and II=3 and JJ=6} \tag{5.51}$$

$$Q_{ml}=6Q_{su} \qquad \text{if } N_{sh}=4 \tag{5.52}$$

when N_y=8 (see Fig. 5.7)

$$Q_{ml}=16Q_{su} \qquad \text{if } N_{sh}=1 \tag{5.53}$$

$$Q_{ml}=8Q_{su} \qquad \text{if } N_{sh}=2 \text{ and II=6, JJ=4} \tag{5.54}$$

$$Q_{ml}=16Q_{su} \qquad \text{if } N_{sh}=2 \text{ and II=3, JJ=8} \tag{5.55}$$

$$Q_{ml}=4Q_{su} \qquad \text{if } N_{sh}=4 \text{ and II=6, JJ=2} \tag{5.56}$$

$$Q_{ml}=8Q_{su} \qquad \text{if } N_{sh}=4 \text{ and II=3, JJ=4} \tag{5.57}$$

when N_y=10

$$Q_{ml}=20Q_{su} \qquad \text{if } N_{sh}=1 \tag{5.58}$$

$$Q_{ml}=10Q_{su} \qquad \text{if } N_{sh}=2 \text{ and II=6, JJ=5} \tag{5.59}$$

$$Q_{ml}=20Q_{su} \qquad \text{if } N_{sh}=2 \text{ and II=3, JJ=10} \tag{5.60}$$

$$Q_{ml}=10Q_{su} \qquad \text{if } N_{sh}=4 \text{ and II=3, JJ=5} \tag{5.61}$$

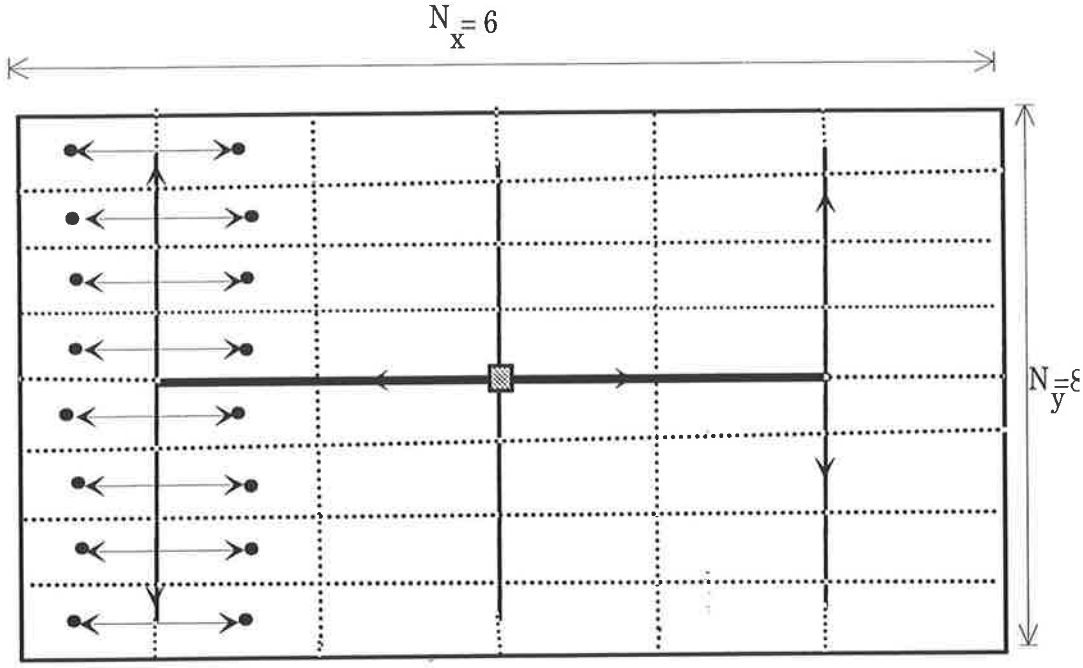


Fig. 5.7 An example of multiple subunit system with 48 subunits (8 subunits supplied by each submain)

5.7 CONSTRAINTS

On the basis of both the characteristics of the model, and the performance of the system, the objective function is minimised subject to a number of constraints as discussed below:

5.7.1 Net Depth per Irrigation Event

Normally, trickle irrigation wets only part of the soil area. Therefore, the equations for determining the desirable depth or volume of application per irrigation cycle and the maximum irrigation interval must be adjusted accordingly. The soil moisture deficit at each irrigation that is considered depends on the soil, the crop, and water-yield- economic factors (Keller and Bliesner, 1990).

The maximum net depth of water per irrigation event (d) is the depth of water that will replace the soil moisture deficit for a desirable level of deficit. d is computed as a depth over the whole crop area not just the wetted area, however, the percentage wetted area (P_w) must be taken into account. Thus, for trickle irrigation the net depth of water to be applied per irrigation cycle may be estimated as follows(Papazafirious, 1980; Keller and Bliesner, 1990):

$$d = \left(\frac{(FC - PWP)}{100}\right) R.\gamma_s.f.\left(\frac{P_w}{100}\right) \tag{5.62}$$

where d= depth of water which can be stored in the root zone (mm); FC=Field capacity (% by weight); PWP= permanent wilting point (% by weight); R=depth of crop root zone (mm); γ_s=specific gravity of soil (dimensionless); f=fraction of available moisture depletion allowed; P_w= percentage of wetted area.

An estimated value of P_w within a reasonable range is considered. Once the emitter discharge (Q_E) and hence wetted diameter (W_d) have been determined the new P_w may be estimated using Equation 5.65 to be consistent with the emitter flow rates.

The maximum irrigation interval (F) can be estimated using the amount of water which can be stored in the soil and the average daily transpiration during peak-use period (Keller and Bliesner, 1990) as follows:

$$F \leq \frac{d}{K_c ET_0} \qquad (5.63)$$

This equation ensures that the consumptive use of water in one irrigation cycle will be equal or less than the depth of water which can be stored in the root zone. Although the transpiration rate under drip irrigation is a function of the conventionally computed consumptive use rate and the extent of the plant canopy (Keller and Bliesner, 1990) in this study, the crop coefficient and the potential evapotranspiration rate are considered for consumptive use. Since the design should meet the peak irrigation requirement, K_C is taken equal to 1, this corresponds to a dry area with light to moderate wind and large mature citrus trees. It includes different free ground cover with clean cultivation and no weed control, (FAO, 1984). The effect of crop characteristics on the relationship between the crop evapotranspiration (ET_c) and the potential evapotranspiration (ET_0) for a number of crops is shown in Figure 5.8. The wide variation between the major group of crops is largely due to the resistance to transpiration of different plants. Total crop water requirements during a year or growth season is not constant.

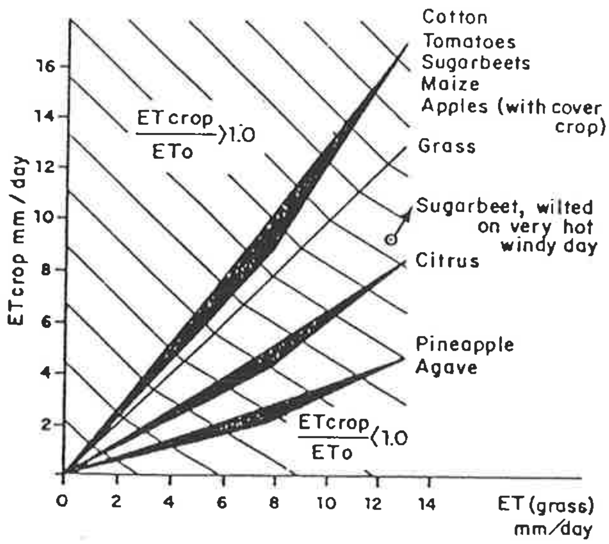


Fig. 5.8 ET_0 **as compared to** ET_c **for different crops (FAO, 1984)**

Factors affecting the values of the crop coefficient (K_C) are the crop characteristics, crop planting or sowing date, rate of crop development, length of growing season and the climatic conditions. K_C varies in terms of plant activities or stage of growth and rate of evapotranspiration. As an example, Figure 5.9 shows the relationship of crop water coefficient (K_C) in terms of different stages of plant activities for cotton.

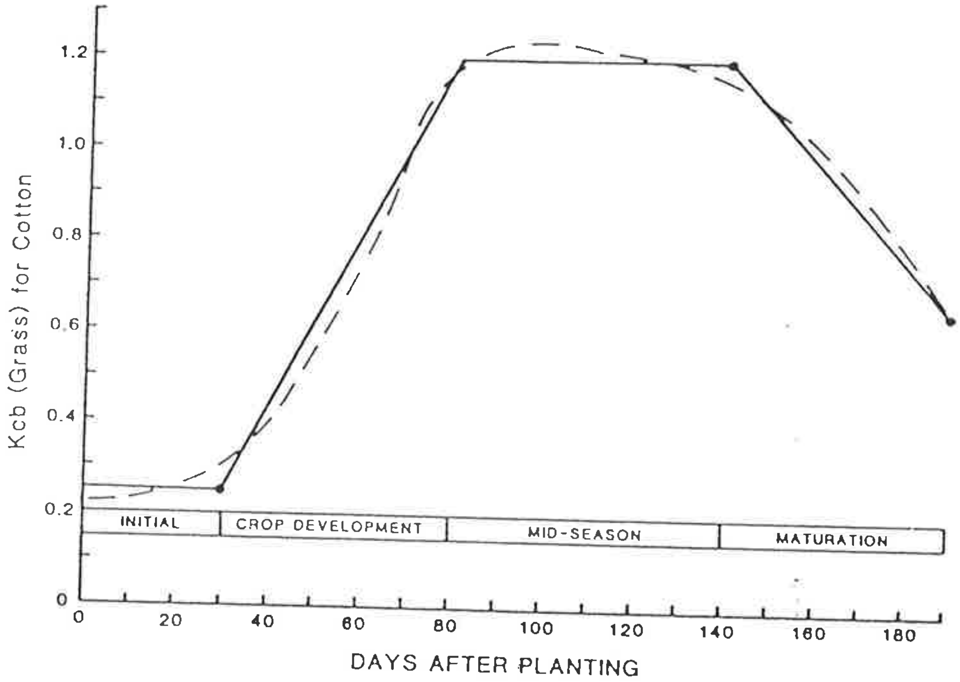


Fig. 5.9 Example of crop coefficient curve developed for cotton (Wu et al, 1986)

It should be noted that the actual irrigation frequency to be used depends on the management policy that can be adjusted in the schedule process. The duration of irrigation per shift (T, hr) can then be determined using Equation 5.29. Having selected N_{sh} and hence T, the emitter discharge (Q_E, L/hr) can be determined using Equation 5.30. The model allows the number of shifts (N_{sh}) to be chosen as a decision variable. The emitter discharge used should satisfy the following constraints:

(i) the percentage wetted area of the field should lie within a defined range in order to ensure that there is a reasonable volume of moisture stored in the soil (Keller and Bliesner, 1990);

(ii) the rate of application should not exceed the infiltration capacity of the soil.

The wetted area associated with a single emitter depends on the emitter discharge and soil properties. Karmeli et al (1985) give the following empirical relationship relating wetted diameter (W_d, m) to the emitter discharge rate:

$$W_d = \alpha + \beta\, Q_E \tag{5.64}$$

where values of α and β for different soils are given in Table 5.1.

TABLE 5.1 Parameters of dripper wetting diameter relating to emitter discharge for various soil types

Soil type	α	β
Fine soil	1.2	0.1
Medium soil	0.7	0.11
Coarse soil	0.3	0.12

For non overlapping wetted areas, the percentage wetted area is given by:

$$P_w = 100 \frac{\pi\,(W_d)^2}{4 d_x\, d_y} \tag{5.65}$$

in order to satisfy the minimum and maximum acceptable levels of P_w, W_d must lie within the following range.

$$\left(\frac{4P_w^{\min} d_x d_y}{100\pi}\right)^{1/2} \le W_d \le \left(\frac{4P_w^{\max} d_x d_y}{100\pi}\right)^{1/2} \tag{5.66}$$

where $P_w^{\min}$ and $P_w^{\max}$ = minimum and maximum acceptable values of the percentage wetted area (respectively). Constraints 5.66 effectively constrain Q_E through Equation 5.64.

Where overlap occurs, a more complicated equation for P_w needs to be developed. In this case Karmeli et al (1985) recommend the following constraint for efficiency reasons:

$$W_d \leq 1.6\ d_x \tag{5.67}$$

The rate of application from the emitters should not exceed the infiltration capacity of the soil.

$$\text{i.e.} \quad Q_E \leq I_{soil}\left(\frac{d_x d_y P_w}{100}\right) \tag{5.68}$$

where I_{soil} = infiltration capacity of the soil (mm/hr).

Constraints (5.66), (5.67) and (5.68) effectively limit the number of shifts which can be used. Having selected N_{sh}, the model allows for various shift patterns to be considered. A shift pattern is defined by *II* and *JJ* which are the number of subunits being irrigated simultaneously in the X and Y directions respectively (see Fig. 5.3). The process for estimating the irrigation requirements is presented in a flow chart given in Appendix G.

5.7.2 Uniform Distribution of Discharge

The hydraulic constraints are important to ensure discharge uniformity. In order to achieve an uniform water distribution along the manifold and laterals within the subunits, the pressure head variation along those pipes is limited. The head loss of multiple outlet pipes may be determined using the Hazen-Williams equation by considering a discharge factor as shown in Equation 3.15 or the equations developed in Section 4.7. In similar way to the model discussed in Chapter 4, the Hazen-Williams coefficient is usually taken to be 150 for polyethylene and PVC pipes (Oron and Walker, 1981). However, considering the additional roughness due to the emitters on the laterals and laterals on the manifolds, values of 130 for laterals, 140 for manifolds and 150 for the other pipes are assumed in this model.

Since, in drip irrigation systems, water is applied as discrete or continuous drops through the emitters, uniformity of emitter flow is very important. It depends on two factors: the emitter characteristics, and the water pressure variation along the lateral lines and manifolds. In general, the flow rate through the emitters is controlled by the hydraulic pressure and the flow path dimensions of the emitters. Three major groups of emitter types are: orifice or nozzle emitters, long flow path emitters and special types emitters such as pressure compensated, vortex and porous-pipe. The orifice and nozzle types usually have fixed geometry so the flow area is constant. According to Solomon and Keller (1978); Jensen (1983) and Wu et al (1986) the flow in orifice type emitters is largely dependent on the hydraulic pressure on emitters given in Equation 4.3.

The degree of emitter flow variation is important because it is one of the major components of the irrigation efficiency. It may be expressed by the emission uniformity (EU) or uniformity coefficient (UC) as defined by Christiansen (1942) for sprinkler irrigation systems. Uniformity coefficient (UC) was discussed and computed for emitters on lateral lines under various loading cases in Chapter 4 (Section 4.12). In a well design drip irrigation system, the emission uniformity for the emitters should be above a specified level. Karmeli and Keller (1975) define the emission uniformity for a dripper system as:

$$EU = 100\left(1 - 1.27\frac{\mu}{\sqrt{N_p}}\right)\frac{Q_n}{\overline{Q_E}} \tag{5.69}$$

where μ=emitter coefficient of manufacturing variation in emitters due to manufacture; N_p = number of emitters from which each plant receives water; Q_n= minimum emission rate from emitters (L/hr); $\overline{Q_E}$= average emission rate from emitters (L/hr). Keller and Bliesner (1990) recommend that EU should lie in the range 85% to 90% for drippers on flat terrain with fewer than 3 drippers per plant. An acceptable value of EU can be obtained by limiting the variation of pressure (Keller and Karmeli, 1974) of the emitters within a subunit to 20% (see Appendix E). For a working pressure of 10 m, this allows a total pressure loss within a subunit of 2 m in the manifold and laterals.

As explained previously the flow variation from emitters should not exceed a specific level. The relationship between the emitter flow variation and the uniformity coefficient is shown in Figure 5.10. Also the relationship between emitter flow variation and the pressure variation for different α values (x) is shown in Figure 5.11.

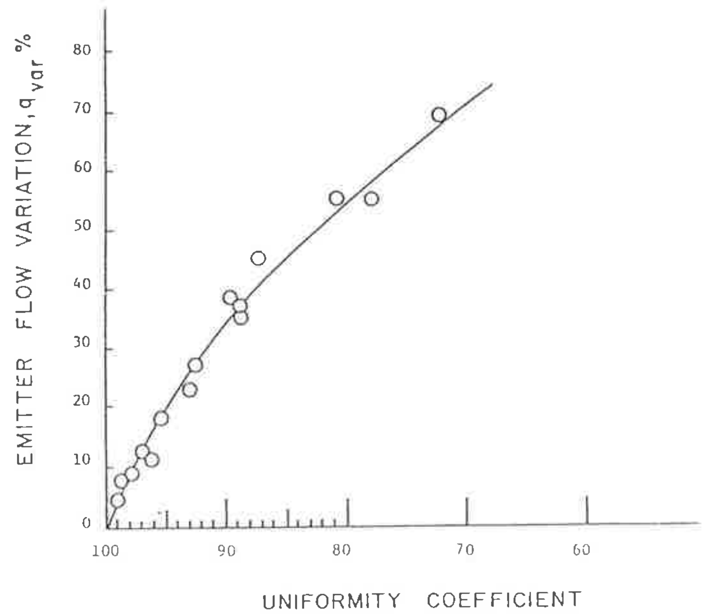


Fig. 5.10 Relationship between emitter flow variation and uniformity coefficient. (Howell et al, 1986)

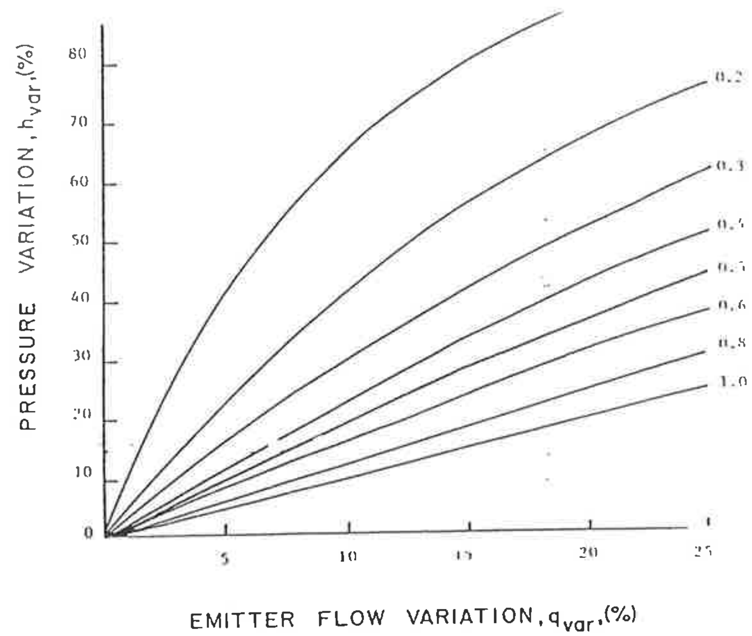


Fig. 5.11 Relationship between emitter flow variation and the pressure variation for different x-values (Wu et al, 1986)

The variation of the emitter flow rates is directly proportional to the maximum and minimum pressure head at the emitters as shown in Equation 4.42. The pressure and the emitter flow variation are related by the α-value given in the emitter flow equation (Equation 4.43). To compute the pressure head variation within the pipes computing the head loss is essential. The head loss in the two segments of laterals considering Q_E may be computed using procedure shown in Appendix F (Equation F.6).

Total head loss from the most distant dripper to the water source to be includes: the head loss of the most distant lateral, manifold, supply, submain and mainline pipes in series, the head loss of accessories including: valves, filter, chemical tank, pump shaft are considered in total hydraulic pumping head. Total system pressure or total dynamic pumping head is expressed as:

$$TH = HL_p + HL_{acc} + H_w + H_{wt} \qquad (5.70)$$

where TH = the maximum pressure head at the water source to be provided by the pump (m); HL_p= the head loss in pipes (m); HL_{acc} = the head loss of accessories including the head

loss of filter, (HL_{fil}, m), fertiliser (HL_{fer}, m), pump shaft (HL_{sh}, m) and head loss of valves (HL_{val}, m); H_w = working pressure at the most distant emitter (m); H_{wt}= groundwater depth (m).

The head loss of pipes in series may be presented as:

$$HL_p = HL_l + HL_m + HL_s + HL_{sm} + HL_{ml} \qquad (5.71)$$

where HL_{sm}, HL_{ml} = the head loss in the submain and the mainline pipes respectively (m).

The head loss of the supply, submain, and mainline pipes based on the Hazen-Williams equation are as following:

$$HL_s = 10.68 \frac{\dfrac{P_{sux}}{2} \cdot \left(2.78 \times 10^{-7} Q_E \dfrac{P_{sux}}{d_x} \cdot \dfrac{P_{suy}}{d_y}\right)^{1.852}}{CH_s^{1.852} D_s^{4.87}} \tag{5.72}$$

The term in brackets represents the flow in the supply pipe which is the product of the number of emitters in the subunits multiplied by the discharge of emitters.

$$HL_{sm} = 10.68 \frac{\left(\dfrac{F_y}{2} - \dfrac{P_{suy}}{2}\right)\left(Q_E \dfrac{P_{sux}}{d_x} \cdot \dfrac{P_{suy}}{d_y} \cdot N_{suy}\right)^{1.852}}{CH_{sm}^{1.852} D_{sm}^{4.87}} \tag{5.73}$$

where CH_{sm}= Hazen-Williams roughness coefficient of submain pipes; D_{sm}= internal diameter of submain pipes (m); N_{suy}= the number of subunits in the Y-direction or the maximum subunits which may be supplied by each submain line.

$$HL_{ml} = 10.68 \frac{\left(\dfrac{F_x}{2} - P_{sux}\right)\left(2.78 \times 10^{-7} Q_E \dfrac{P_{sux}}{d_x} \cdot \dfrac{P_{suy}}{d_y} \cdot n \cdot N_{suy}\right)^{1.852}}{CH_{ml}^{1.852} D_{ml}^{4.87}} \tag{5.74}$$

where HL_{ml}= the head loss of mainlines (m); CH_{ml}= Hazen-Williams roughness coefficient of mainlines; D_{ml}=internal diameter of mainline pipes(m); n=0 if $N_x \leq 2$, $n=2$ if $2 \leq N_x \leq 6$ and $n=4$ if $6 < N_x \leq 10$.

5.7.3 Size of Emitters

In the past the emitter flow rates of drip irrigation systems were selected just large enough to meet plant water requirements on a continuous basis. These small flow rates required small orifices or emitters that caused clogging problems. To minimise clogging, the emitter diameters were increased which led to increases in the emitter flow rates and changed the irrigation duration from a continuous to an intermittent system. In the design of a drip irrigation system it is necessary to make sure that an appropriate size is chosen for the emitters. Very small sizes may cause clogging problems and very large emitter sizes may cause run off and subsequently, lead to soil erosion. Single-outlet emitters can be used to irrigate small spots, or can be arranged around larger plants to serve the same function as dual or multiple outlet emitters or spray. Multiple-outlet emitters are used in orchards where large trees may each require several emission points. In Figure 5.12 various emission point layouts for a widely spaced tree crop are shown.

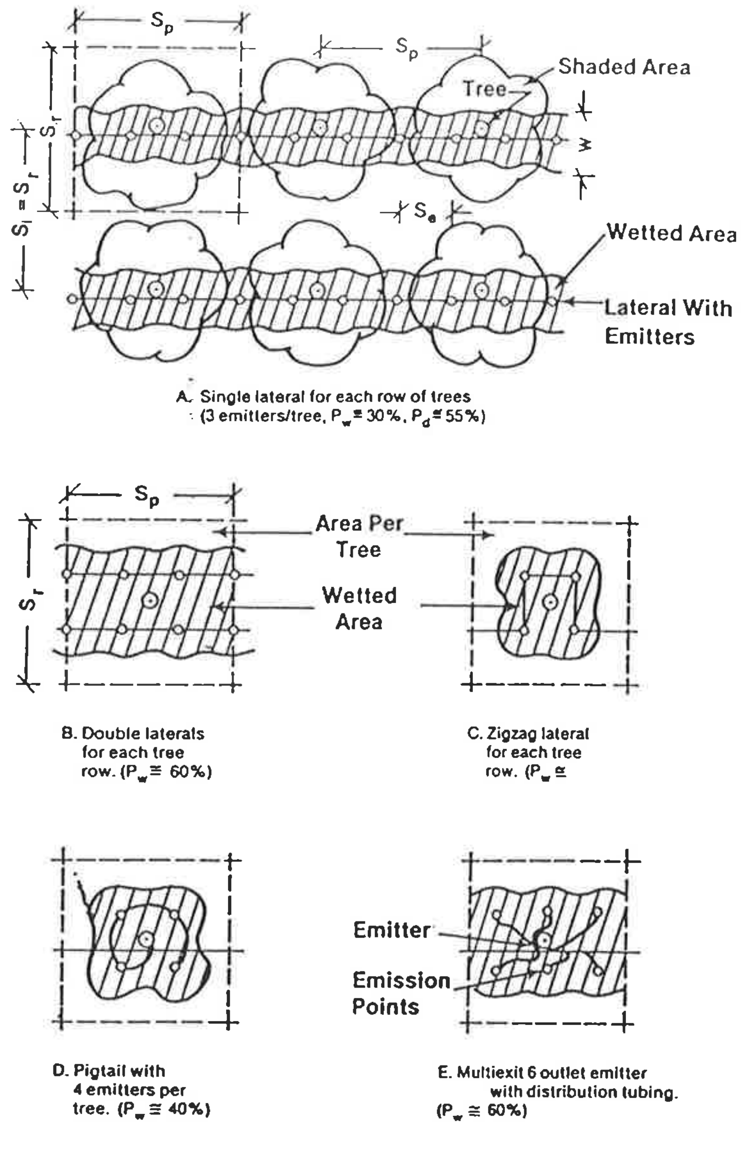


Fig. 5.12 Different layout of multi exit emitters (James, 1988)

5.8 OPTIMIZATION PROCEDURE

The model evaluates all combinations of subunit sizes, pipe sizes, shift numbers and shift patterns. The system cost is evaluated for various sizes of created subunits under three different numbers of irrigation shifts (1, 2 and 4). Optimisation is carried out by complete enumeration of all alternatives.

The following values are assumed to be known:

(1) The dimensions of the field, F_x (m) and F_y (m);

(2) The depth of the water table H_{wt} (m);

(3) The potential evapotranspiration, ET_0 (mm/day), the crop coefficient, K_c;

(4) The minimum and maximum percentage of wetted area, P_w, (%);

(5) The application efficiency of drip irrigation, E_a;

(6) The annual irrigation requirement for the crop, A_{ir}, (mm);

(7) The field capacity, FC and the permanent wilting point, PWP, of soil (% by weight);

(8) The depth of root zone, R, (mm), soil infiltration rate, I_{soil} (mm/hr) and specific gravity of soil, γ_s;

(9) The allowable fraction of the available moisture depletion, f;

(10) The spacing between emitters, d_x, and laterals, d_y, respectively (m);

(11) The pipe cost coefficients K_1, K_2, K_3 and ϕ; the Pump cost parameters K, a and b;

(12) Efficiencies for the electric motor, η_m, and pump, η_p, respectively;

(13) The discount rate, i , and expected project life, n, (years);

(14) A list of available diameters for all pipes;

(15) Two diameters for laterals and their cost per unit length;

(16) Hazen-Williams coefficients for all pipes;

(17) Cost information for all components;

(18) The head loss through the filter, fertiliser unit, valves and pump.

The acceptable range for emitter discharges, can be determined using the acceptable range for the percentage wetted area, P_w, and Equations 5.68 and 5.69.

The optimisation processes for the subroutine which optimises the subunits and the main program are illustrated in two flow charts shown in Figures 5.13 and 5.14 respectively. The Model is written in FORTRAN. It takes approximately 26 seconds CPU time on a mainframe machine, DEC server 5000/240 to run under Unix system.

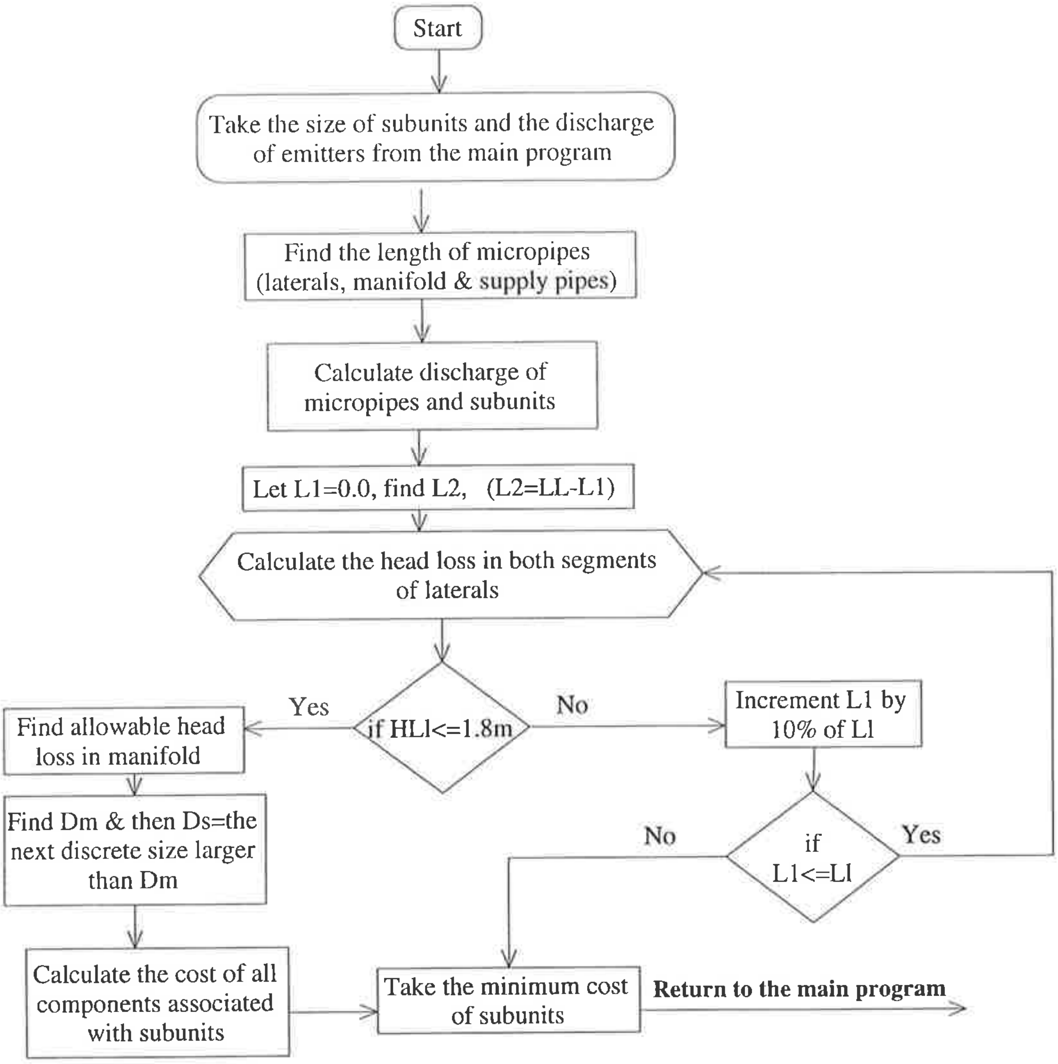


Fig. 5.13 Flow chart of subroutine optimizing the subunits

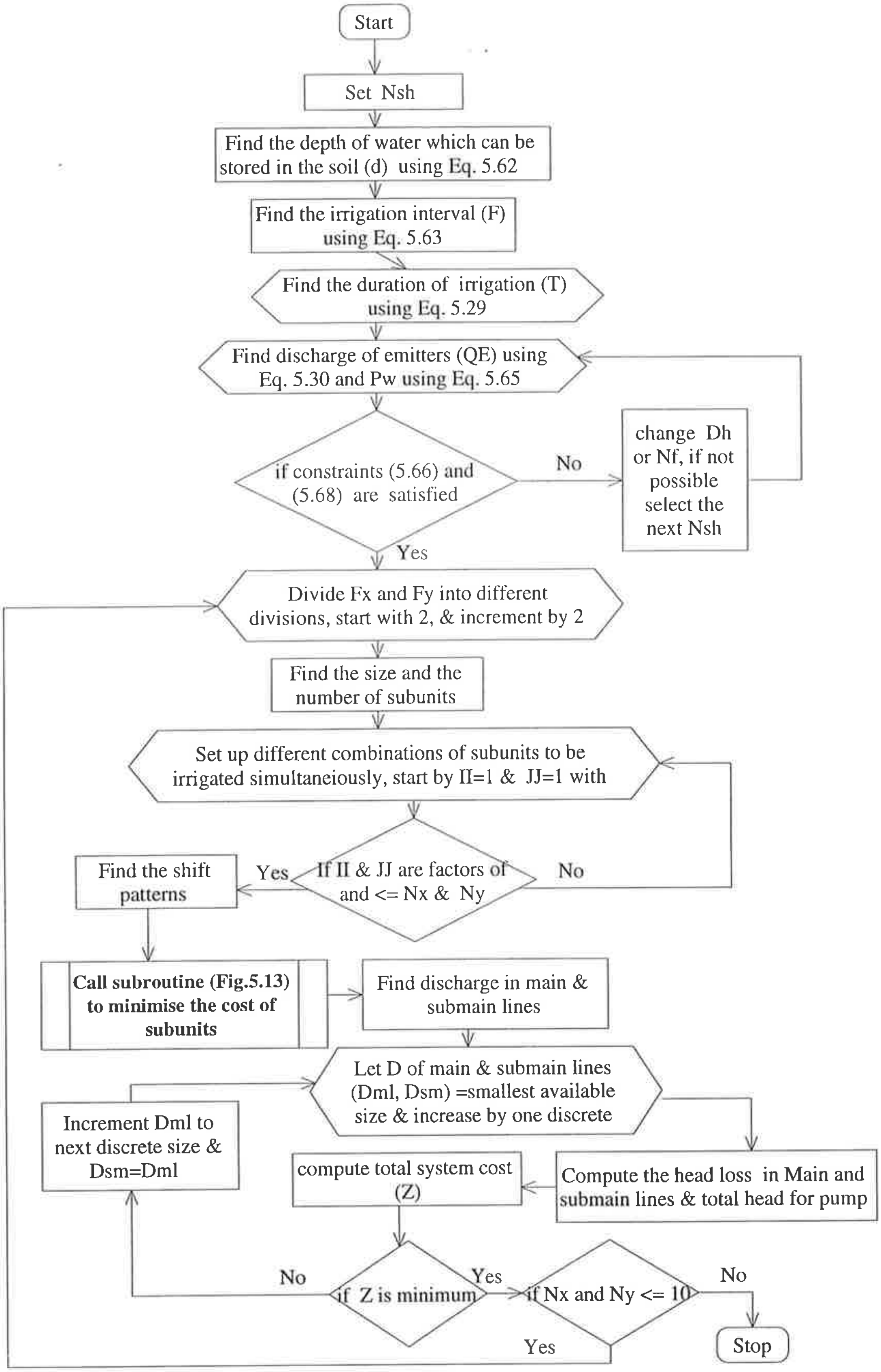


Fig. 5.14 Flow chart of main program for optimisation of a multiple subunit drip irrigation system for different operating conditions

5.9 MODEL ASSUMPTIONS AND DATA INPUT

In the present optimisation model the general configuration of pipes within the field (main and submain lines) and within the subunits (lateral, manifold and supply lines) is fixed. However, since the area and the dimensions of subunits in both X and Y directions change in each iteration of the field division, the lengths and the sizes of all pipes change as well. The model is developed for a field with given area and known dimensions for which the groundwater source is located at the centre of field. The model can be easily applied to any size and dimensions of field.

5.9.1 Case Study

The model is applied to a case study with the data given in Table 5.2. This represents irrigation of a crop with a maximum water requirement of 4.6 mm per day. Only 1, 2 and 4 shift operation are considered. In this table the coefficients for pipe cost (K_1, K_2 and K_3) were obtained by regression analysis of cost data for PVC pipes. The parameters for pump cost (K, a, and b) were found by non-linear regression analysis of the costs of various submersible pumps. The main purpose of this study is to identify an optimum design for drip irrigation based on multiple subunit systems. The model enables an examination of the influence of various subunit sizes, and shift patterns on the system cost and will find the global optimum solution among various local optima.

5.10 RESULTS AND DISCUSSION

As explained previously, the main purpose of the study outlined in this chapter is to develop an optimisation model for drip irrigations based on multiple subunit system with the following characteristics:

- Each given field can be easily divided into various subunits with different area and dimensions.

- The model enables an examination of the influence of various subunit sizes, and shift patterns on the system cost and will find the global optimum solution among various local optima under a known operating program.

TABLE 5.2 Input data for a case study

Variables (1)	Value (2)	Units (3)	Variables (4)	Value (4)	Units (5)
A_{ir}	1000	mm	$K1$	0.96×10^{-3}	--
Fx	800	m	$K2$	6×10^{-3}	--
Fy	600	m	$K3$	0.18	--
dx	2	m	ϕ	2	$/m
dy	3	m	K	1000	--
Db	19	mm	a	0.2305	--
Ds	13	mm	b	0.9038	--
Hw	10	m	η_p	72	%
Hwt	20	m	η_m	95	%
HLf	2	m	Ea	85	%
$HLfer$	1.5	m	Kc	1	--
$HLval$	0.7	m	FC	27	%
$HLsh$	3	m	PWP	10	%
CHL	130	--	γ_s	1.35	g/cm^3
CHm	140	--	R	1300	mm
CHs	150	--	I_{soil}	8	mm/hr
$CL1$	0.85	$/m	$ET0$	4.6	mm/day
$CL2$	0.42	$/m	f	0.45	-
Cf	360	$	N_f	1	day
$Cfer$	450	$	Dh	20	hr
$Ccon$	225	$	n	12	years
$Cval$	80	$	i	10	%
$Cvalsm$	150	$	α	1.20	m
$Cvalml$	180	$	β	0.10	m hr/L
$CUem$	0.90	$	P_w^{min}	25	%
Cen	0.09	$/kWh	P_w^{max}	65	%

- The optimum solution for each iteration of field division can be achieved by finding decision variables including: the optimum lengths of lateral segments, the optimum size of the manifold and supply pipes for an accepted discharge uniformity, the optimum size of submain and mainline pipes by caring out a trade-off between the cost of pipes and the cost of corresponding energy, the number of shifts and patterns.

A number of effects were evaluated. These are discussed in the following sections.

5.10.1 Effect of Subunit Area

Determining the optimum area of subunits is one of the aims of the model developed in this study. The effect of subunit areas and subunit dimensions on the system cost were examined by increasing the number of subunits in the model from 4 to 100. The results obtained under three selected number of shifts are given in Tables 5.3, 5.4 and 5.5. The minimum cost solution involves one shift operation and corresponds to a subunit area of 3 ha with dimensions of 400 m×75 m (Table 5.3). It has a system cost of $262782 or 5475 $/ha. Table 5.3 includes some other cases with the same subunit area, but higher system costs, due to the differences in subunit dimensions. For example, the subunits with an area of 3 ha but with dimensions, 200 m×150 m and 100 m×300 m, lead to system costs of $270130.2 (5627.7 $/ha) and $272486 (5676.8 $/ha) respectively. Although the optimum ratio of the X to the Y dimensions for fields with one subunit (one control head) is somewhere between 1.0 and 1.5 (discussed in Chapter 3), the optimum ratio for a field with multiple subunits under 1-shift is not consistent with that. This could be the effect of other parameters such as the cost of the submain and main line pipes and the pump. However, the dimensional ratios of subunits for the minimum system cost under 2-shift and 4-shift operation are more consistent with the results discussed in Chapter 3 (being 1.3 for 2-shift and 0.8 for 4-shift operation). The effect of subunit area on the system cost for one shift operation is shown in Figure 5.15.

The same analysis was carried out for 2-shift and 4-shift operations. The details are given in Tables 5.4 and 5.5 and shown in Figures 5.16 and 5.17. The results show that the system cost variation in terms of subunit area is different for the various number of shifts. As the number of shifts increases the system cost increases due to increase in flow rate of pipes.

TABLE 5.3 Total minimum cost, capital and operation costs for various subunit sizes under 1-shift operation

Subunit area (m²)	Psux (m)	Psuy (m)	Total head (m)	Pump Power (kw)	Capital cost ($)	Operating cost ($)	Total cost ($)
120000	400	300	44.7	27.01	216381	47331	263712
60000	400	150	44.6	26.93	216576	47189	263765
40000	400	100	48.9	29.28	213125	51293	264418
30000	**400**	**75**	**46.3**	**27.96**	**213789**	**48993**	**262782**
24000	400	60	51.8	31.32	217109	54871	271980
60000	200	300	45.4	27.47	219866	48123	267989
30000	200	150	45.9	27.73	221541	48589	270130
20000	200	100	53.8	32.19	220125	56406	276530
15000	200	75	49.8	30.1	223041	52730	275771
12000	200	60	58.1	35.14	222299	61566	283865
40000	133.3	300	45.5	27.25	224542	47737	272278
20000	133.3	150	46.8	28.03	220956	49106	270062
13333.3	133.3	100	47.5	28.16	220558	49339	269897
10000	133.3	75	50	29.94	223861	52466	276327
8000	133.3	60	48.1	28.8	225586	50468	276055
30000	100	300	45.6	27.59	224154	48332	272486
15000	100	150	47.3	28.59	226134	50094	276228
10000	100	100	48.4	28.93	226956	50689	277645
7500	100	75	46.7	28.23	231522	49467	280990
6000	100	60	50.2	30.33	233614	53138	286752
24000	80	300	44.3	26.79	229580	46932	276512
12000	80	150	49.8	30.12	224958	52769	277728
8000	80	100	50	29.93	224892	52443	277335
6000	80	75	52.6	31.81	229011	55727	284738
4800	80	60	51	30.8	231413	53961	285375

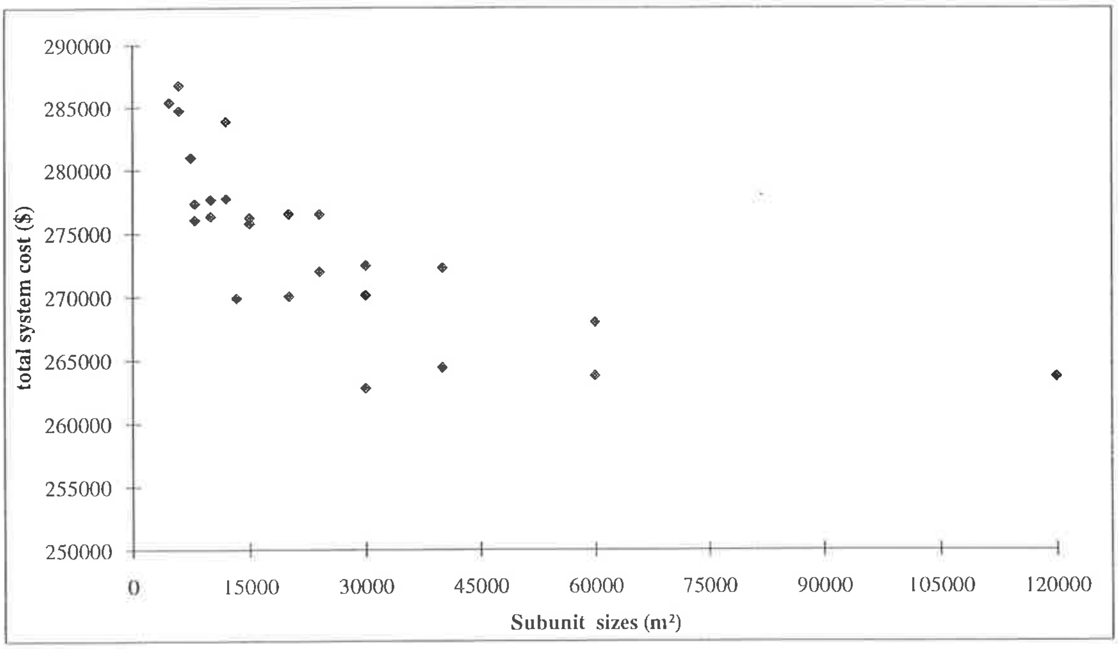


Fig. 5.15 Total minimum system costs for various subunit areas under 1-shift operation

On the other hand, as the subunit area decreases the system cost also decreases to reach the optimum cost and then increases. The increase in system cost from the optimum level for the smaller subunit area is due to the increase in the number of submain lines, valves, length of submain and main pipes and also the increase of corresponding head losses. The higher cost for very large subunit areas could be due to using pipes with larger diameters. As shown in Figure 5.17 the maximum feasible subunit size is 6 ha, for 4-shift operation. This is due to the fact that for larger sizes of subunits under high shift operation, the head loss in the laterals exceeds the allowable head loss and violates the pressure constraint for the laterals because of the greater length and higher flow rate. The optimum system cost for 2-shift occurs at a subunit area of 1.33 ha with dimensions of 133.3 m by 100 m, and for 4-shift occurs at subunit area of 2 ha with dimensions of 133.3 m by 150 m. This difference in the optimum size of subunits for different shifts may be due to the use of discrete pipe sizes.

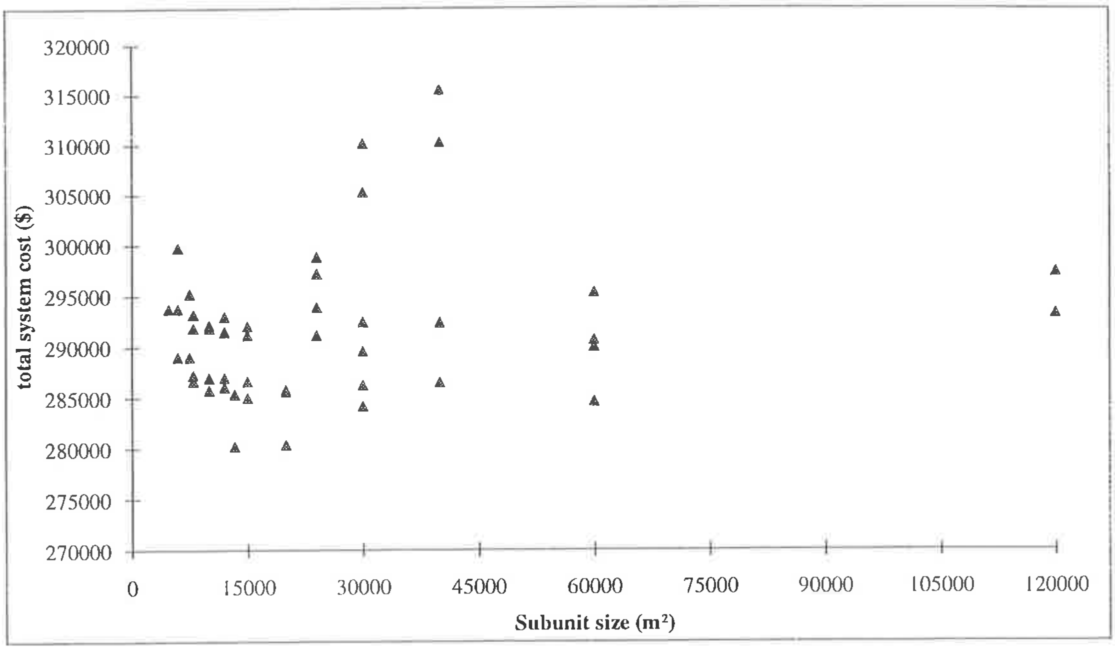


Fig. 5.16 Total minimum system costs for various subunit areas under 2-shift operation

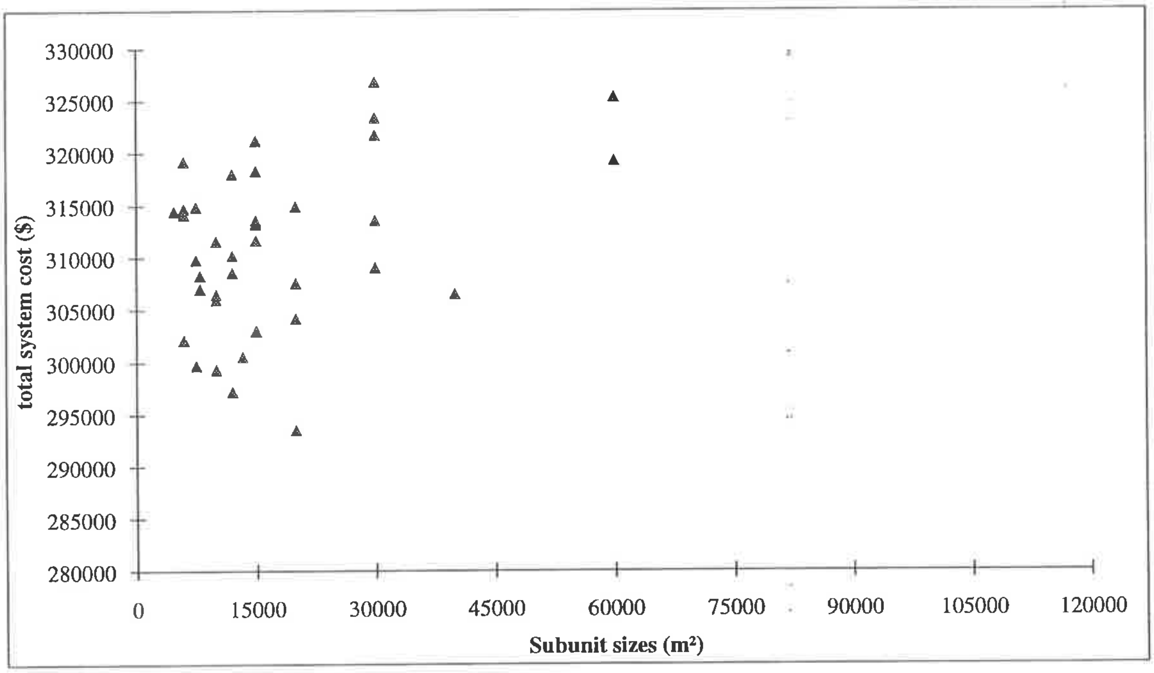


Fig. 5.17 Total minimum system costs for various subunit areas under 4-shift operation

TABLE 5.4 Total minimum cost, capital and operation costs for various subunit sizes under 2-shift operation

Subunit area (m^2)	Psux (m)	Psuy (m)	Total head (m)	Pump Power (kw)	Capital cost ($)	Operating cost ($)	Total cost ($)
120000	400	300	45	27.21	245527	47672	293199
120000	400	300	45.3	27.37	249317	47954	297271
60000	400	150	45.3	27.39	242707	47993	290699
60000	400	150	46.9	28.36	245663	49687	295350
40000	400	100	45.8	27.43	238398	48063	286461
40000	400	100	48.2	28.82	241883	50488	292370
30000	400	75	56.6	34.19	245386	59899	305285
30000	400	75	58.1	35.08	248678	61469	310147
24000	400	60	51.7	31.26	239112	54775	293888
24000	400	60	53.2	32.15	242482	56331	298813
60000	200	300	46.4	28.06	240858	49157	290015
60000	200	300	46.1	27.84	235805	48780	284586
30000	200	150	45.5	27.5	241393	48190	289583
30000	200	150	45.2	27.29	236339	47814	284154
20000	200	100	51.5	30.79	231831	53946	285776
20000	200	100	51.1	30.58	226782	53580	280362
15000	200	75	55.4	33.47	233392	58640	292032
15000	200	75	55	33.25	228342	58264	286605
12000	200	60	52.9	31.97	235474	56006	291480
12000	200	60	52.5	31.75	230422	55630	286052
40000	133.3	300	48	28.72	265133	50314	315447
40000	133.3	300	46.3	27.7	261708	48536	310244
20000	133.3	150	49.3	29.47	233955	51636	285592
20000	133.3	150	47.6	28.46	230532	49859	280391
13333.3	133.3	100	50.5	29.91	232924	52411	285334
13333	**133.3**	**100**	**48.8**	**28.93**	**229515**	**50683**	**280198**
10000	133.3	75	53.5	32.04	236008	56130	292139
10000	133.3	75	51.8	31.02	232590	54352	286943
8000	133.3	60	51.3	30.67	238104	53728	291832
8000	133.3	60	49.6	29.65	234683	51950	286634
30000	100	300	47.9	28.94	241722	50707	292428
30000	100	300	45.8	27.65	237781	48448	286229
15000	100	150	53.2	32.18	234805	56374	291178
15000	100	150	51.1	30.89	230872	54115	284987
10000	100	100	54.3	32.47	234985	56884	291869
10000	100	100	52.2	31.21	231071	54689	285760
7500	100	75	52.3	31.62	239801	55404	295205
7500	100	75	50.2	30.33	235867	53145	289012

6000	100	75	50.2	30.33	235867	53145	289012
24000	80	300	48.7	29.41	245630	51528	297158
24000	80	300	46.8	28.29	241588	49563	291151
12000	80	150	49.4	29.88	240610	52357	292967
12000	80	150	47.6	28.76	236568	50392	286960
8000	80	100	49.9	29.87	240835	52327	293162
8000	80	100	48.1	28.78	236809	50418	287227
6000	80	75	52.1	31.49	244538	55174	299712
6000	80	75	50.2	30.37	240500	53209	293709

TABLE 5.5 Total minimum cost, capital and operation costs for various subunit sizes under 4-shift operation condition

Subunit area (m²)	**Psux** (m)	**Psuy** (m)	**Total head** (m)	**Pump Power** (kw)	**Capital cost** ($)	**Operating cost** ($)	**Total cost** ($)
60000	200	300	46.6	28.15	269905	49327	319232
60000	200	300	48.6	29.4	273816	51508	325324
30000	200	150	47.9	28.96	262786	50733	313519
30000	200	150	50.7	30.66	269638	53726	323364
30000	200	150	45.6	27.57	260694	48306	309000
20000	200	100	49.9	29.87	255220	52334	307553
20000	200	100	50.6	30.25	261882	52993	314876
15000	200	75	55.3	33.43	254995	58568	313563
15000	200	75	55.9	33.81	261977	59233	321210
15000	200	75	52.7	31.84	255841	55789	311630
12000	200	60	52.8	31.94	254225	55959	310184
12000	200	60	53.5	32.31	261398	56618	318016
40000	133.3	300	49.9	29.83	254232	52269	306501
20000	133.3	150	48.5	29.05	253239	50891	304130
20000	**133.3**	**150**	**49.1**	**29.4**	**242004**	**51508**	**293512**
13333.3	133.3	100	57.2	33.9	241149	59391	300540
10000	133.3	75	55.8	33.38	247485	58491	305975
10000	133.3	75	53.1	31.75	243684	55632	299316
8000	133.3	60	58.8	35.19	245343	61649	306992
30000	100	300	46.7	28.23	277308	49456	326764
30000	100	300	53.7	32.45	264853	56847	321700
15000	100	150	50.2	30.31	265170	53108	318277
15000	100	150	51.8	31.29	258400	54814	313214
15000	100	150	46.8	28.3	253403	49585	302988
10000	100	100	50.8	30.39	258329	53245	311574

10000	100	100	52.4	31.34	251546	54903	306449
7500	100	75	52.8	31.9	258965	55891	314856
7500	100	75	54.4	32.87	252192	57598	309790
7500	100	75	54.6	32.99	241919	57809	299728
6000	100	60	56.3	34.03	259565	59620	319185
6000	100	60	57.9	35	252788	61327	314116
24000	80	300	49.9	30.13	302285	52795	355080
12000	80	150	50.5	30.51	255073	53455	308528
12000	80	150	50.3	30.4	243933	53262	297194
8000	80	100	51.4	30.77	254377	53905	308282
6000	80	75	60.2	36.36	250964	63702	314666
6000	80	75	53.7	32.47	245229	56895	302123
4800	80	60	57.3	34.6	253806	60626	314433

TABLE 5.6 Minimum cost design and some associated decision variables for 1-shift operation

Items	Tot.pipe length (m)	Diameter (mm)	Discharge (L/s)	Head loss (m)	Minimum cost ($)	Percent of total cost (%)
Field	--	--	42.0	15.57	**262782**	100.0
Laterals	158400	13	0.0527	1.66	95040	36.2
Manifolds	1152	43.4	2.63	0.34	2590	1
Supply lines	3200	56.6	2.63	3.95	11504	4.4
Submains	525	130	21.07	1.72	10125	3.9
Main lines	0.00	0.00	0.00	0.00	0.00	0.00
Pump	--	--	42.1	3.00	19462	7.4
Operating	--	--	--	--	48993	18.6
Emitters	--	--	1.9 L/hr	--	72000	27.4
Accessories	--	--	--	4.9	3066	1.2

5.10.2 Effect of Irrigation Shifts

Multiple subunit irrigation systems allow the application of a number of different shifts in the operating program. Irrigating a set of subunits instead of irrigating the whole system simultaneously increases the flexibility and reliability of system. A high number of shifts requires high emitter flow, which may overcome emitter clogging problems. It is also more flexible in relation to sharing irrigation water for a specified set of subunits when the available water is either provided from different sources or the field belongs to different owners. However, as the number of irrigation shifts increases, the irrigation time for a set of subunits irrigated simultaneously decreases, and as a result pipe flows and system costs increase. The value of irrigation interval, irrigation duration, emitter discharge and the percentage of wetted area associated with each selected number of shifts (1, 2 and 4) are given in Table 5.7.

TABLE 5.7 The value of F, T, Q_E and P_w for selected number of shifts

N_{sh}	F day	T hr	Q_E L/hr	P_w %
1	7	120	1.9	25.2
2	7	60	3.75	32.6
4	7	30	7.5	50.0

The global minimum system cost was obtained using one shift. The details of the design are given in Table 5.6. The optimum configuration does not require mainline pipes and only the minimum size of laterals is required. Tables 5.8 and 5.9 contain, similar information but for the optimum design under 2 and 4 shift operation. Once again only the smallest sized lateral is used in each design. Each of these systems is a local optimum which corresponds to a higher cost than the global optimum cost of the system under 1-shift operation. (Table 5.6). In fact, the optimum design with one shift operation represents a 6.2% cost saving compared to the optimum for two shift operation and 10.5% compared to the optimum for four shift operation.

5.10.3 Effect of Shift Pattern

For any particular number of shifts (greater than one) there are various possible combinations of subunits in the X and Y directions which can be irrigated simultaneously. Figure 5.3 shows an example of 5 possible shift patterns under 2-shift and 4-shift operation for an drip irrigation system with 16 subunits. As the number of irrigation shifts increases, the possibility of using more shift patterns increases as well. As shown in Figure 5.3 under 2-shift operation, two different system costs and under 4-shift operation 3 system costs exist which correspond to different shift patterns. In two shift operation, that pattern No. 5 will lead to a lower system cost than pattern No. 4 as the former involves lower flow in the mainlines than the latter. Similarly pattern No. 2 involves the lowest cost for four shift operation as each submain and each mainline is supplying only two subunits at a time. The details of system cost for the feasible shift patterns associated with 3 different shift numbers for subunit area of 3ha with the same dimension ratios are shown in Table 5.10. The following numerical examples illustrate the influence of shift pattern on the system cost. As mentioned above the optimum design for 2-shift operation given in Table 5.8 uses shift pattern No. 5 (Fig. 5.3, II=4, JJ=2). If the same subunit size is used (133.3 m by 100 m) but the shift pattern is changed to No. 4 (Fig. 5.3, II=2, JJ=4) the system cost increases from $280,198 to $285,334 (1.83% increase in system cost).

Similarly the optimum design for 4-shift operation (Table 5.9) uses shift pattern No. 2 (Fig. 5.3, II=1, JJ=4). For the same size of subunits and shift pattern No. 1 (Fig. 5.3, II=1, JJ=4) the system cost increases from $293,512 to $304,130 (3.6% increase in the system cost). In this study only contiguous shift patterns were considered. Some further cost saving can be achieved by using non-contiguous patterns. For example, irrigating the four subunits on the left and at the same time as the four subunits on the right in Figure 5.3 under two shift operation. The influences of shift patterns on the system cost for each subunit under 2-shift and 4-shift are given in Tables 5.4 and 5.5. The variation of system cost for different shift patterns but for the same subunit size and the same shift number is due to an increase in the size of pipes and in the energy requirements.

TABLE 5.8 Minimum cost design and some associated decision variables for 2-shift operation

Items	Tot. pipe length (m)	Diameter (mm)	Discharge (L/s)	Head loss (m)	Minimum cost ($)	Percent of total cost (%)
Field	--	--	41.3	18.83	**280198**	100.00
Laterals	153648	13	0.0348	0.24	92189	32.9
Manifolds	3492	34.6	2.29	1.76	5367	1.9
Supply lines	2400	43.4	2.29	3.71	5397	1.9
Submains	1500	102	13.77	2.59	19291	6.9
Mainlines	533	130	13.77	1.93	10286	3.6
Pump	--	--	41.3	3.00	20344	7.3
Operating	--	--	--	--	50683	18.1
Emitters	--	--	3.8 L/hr	--	70567	25.2
Accessories	--	--	--	5.6	6075	2.2

TABLE 5.9 Minimum cost design and some associated decision variables under 4-shift operation

Items	Tot. pipe length (m)	Diameter (mm)	Discharge (L/s)	Head loss (m)	Minimum cost ($)	Percent of total cost (%)
Field	--	--	41.8	19.16	**293512**	100.0
Laterals	155200	13	0.0695	0.87	93120	31.7
Manifolds	3528	56.6	6.95	1.13	12683	4.3
Supply lines	1600	83.8	6.95	1.17	11879	4.1
Submains	1350	102	13.91	5.39	17362	5.9
Mainlines	533	130	13.91	1.97	10286	3.5
Pump	--	--	41.8	3.00	20503	7
Operating	--	--	--	--	51508	17.6
Emitters	--	--	7.6 L/hr		71280	24.3
Accessories	--	--	--	5.6	4890	1.8

TABLE 5.10 System cost, capital and operating costs for shift patterns associated with 3 different shift numbers (1, 2 and 4)

Subunit area (m^2)	Psux (m)	Psuy (m)	Total head (m)	Nsh --	II --	JJ --	Capital cost ($)	Operation cost ($)	Total cost ($)
30000	400	75	46.3	1	4	4	213789	48993	**262782**
30000	200	150	45.5	2	2	4	241393	48190	289583
30000	200	150	45.2	2	4	2	236339	47814	284154
30000	200	150	47.9	4	1	4	262786	50733	313519
30000	200	150	50.7	4	2	2	269638	53726	323364
30000	200	150	45.6	4	4	1	260694	48306	309000

5.10.4 Optimum Solutions for Various Irrigation Intervals

On the basis of the management policy, the real irrigation interval in practice may be selected as a desirable cycle (as in the previous section where it was fixed to be 7 days). In the model it is determined using Equation 5.63 with a lower integer value being selected. The net irrigation depth (water readily available to the crop) which can be stored in the root zone is estimated using Equation 5.62. In Equation 5.62 an initial estimate for P_W was selected to be 50% and the irrigation interval then was calculated considering the amount of water which can be stored in the soil and the average evapotranspiration based on the peak-use period. The irrigation interval was then rounded to a lower integer number. The irrigation time (duration) then was computed on the basis of the selected number of shifts and daily available hours (Equation 5.29). The irrigation time and irrigation interval were considered to be the basic parameters needed to calculate the discharge rate of emitters for each selected number of shift using (Equation 5.30).

The relationship between the final irrigation interval and irrigation times considering the day(s) free of irrigation in each irrigation cycle for 3 different shift numbers is illustrated in Figure 5.18. When a low number of shifts is used in which a larger number of subunits and consequently, a larger number of emitters work simultaneously, the irrigation duration is increased for a given depth of irrigation. This affects the flow rate of the emitters and also the percentage of wetted area. As indicated in Equation 5.62 for a lower P_W the depth of

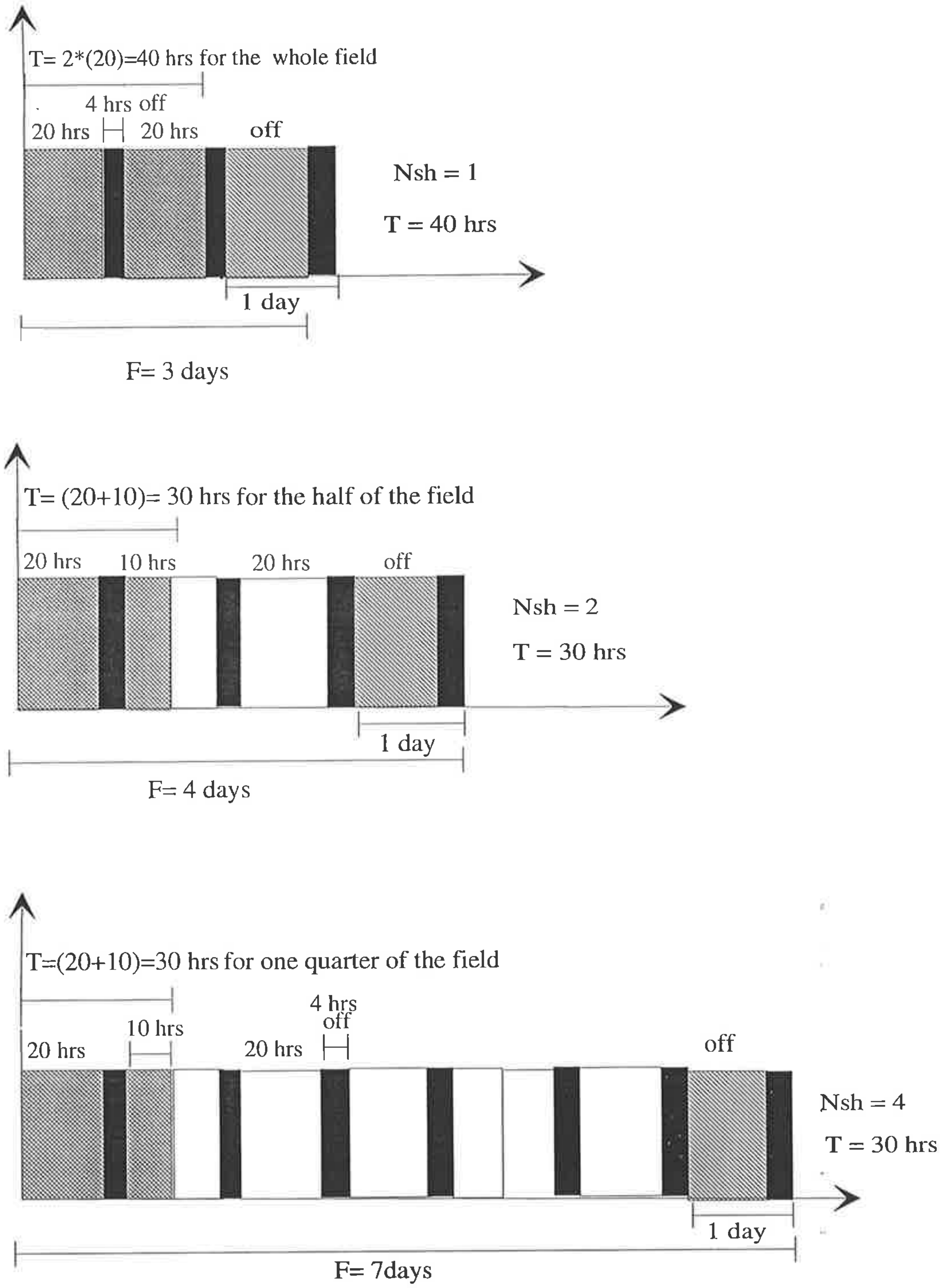


Fig. 5.18 The graphical demonstration of irrigation interval, and time for different shift numbers

irrigation water which can be stored in the root zone is reduced and consequently it affects the irrigation interval. In this part of the work for an initial value of P_w the required depth of irrigation for three different numbers of shifts was estimated. Then it was used to estimate the other irrigation parameters such as F, T, Q_E, and the new P_w. The new percentage wetted area, P_w, was again used in Equation 5.62 and a new d, was recalculated. This process was repeated until a negligible difference of the new and old P_w was obtained as below:

$$\Delta\left|(new\ P_w - old\ P_w)\right| \leq 0.02 \tag{5.75}$$

A simple algorithm was developed to reach a final P_w and the other associated parameters as illustrated in Figure 5.19. The model was examined by an example similar to the case study given in Table 5.2 with the following changes in some input data:

TABLE 5. 11 The modified input data used for the second example

FC	22	%	R	1000	mm
PWP	11	%	initial P_w	50	%
f	0.45	-			

Based on the data given in Tables 5.1 and 5.2 and 5.11 for each irrigation shift the main irrigation parameters are given in Table 5.12. In a similar manner to the previous sections the total system costs and the other cost components corresponding to the optimum design for various alternatives were obtained. As it is clear for a larger wetted area a higher net depth of water per irrigation is needed (Equation 5.62) which affects the irrigation interval. Once a greater number of shifts is selected the larger flow rate for emitters should be considered to provide the required irrigation water. On the other hand, as the irrigation interval increases because of more accumulated evapotranspiration more water should be provided in each irrigation. As discussed previously, a large flow rate in pipes will require either larger pipe sizes or will cause a higher head loss. In ether case the system cost will increase.

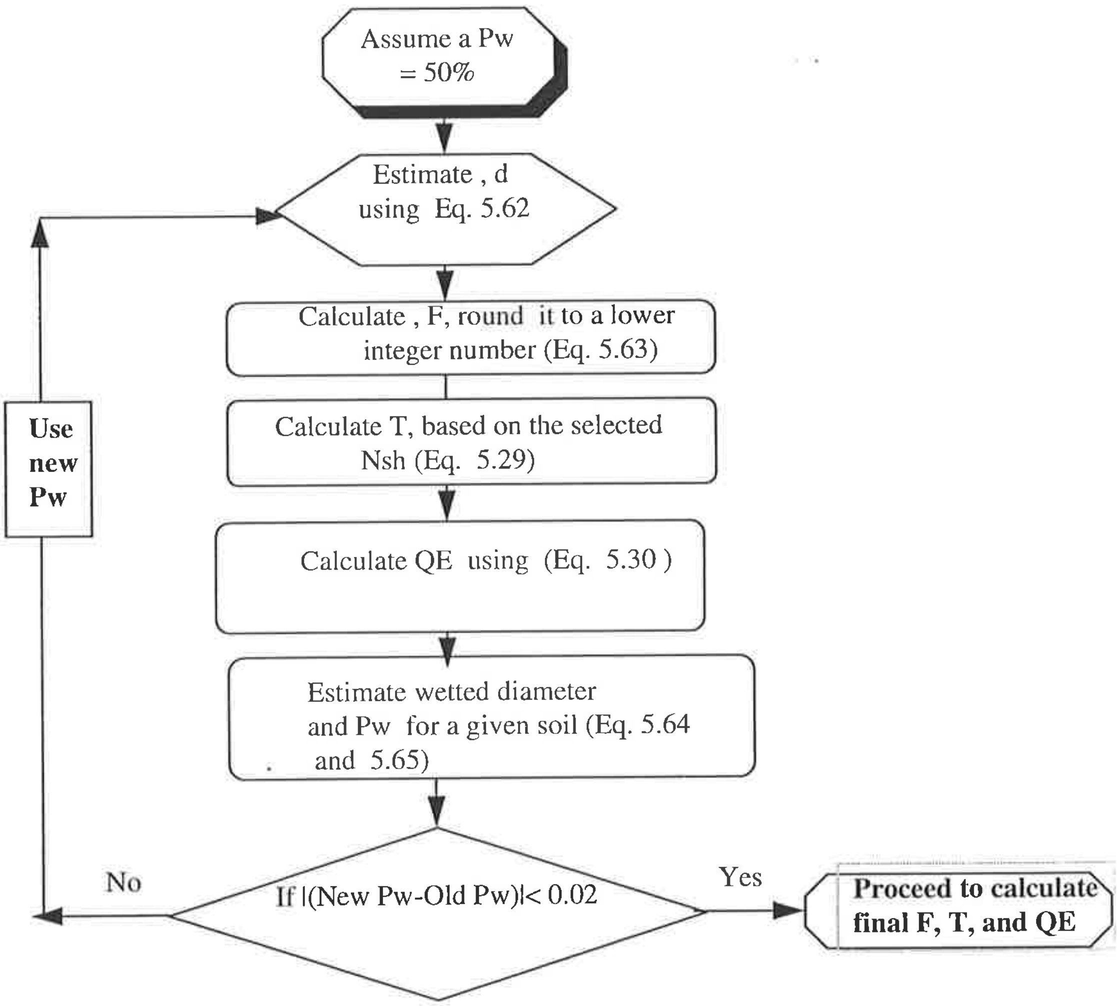


Fig. 5.19 The process for estimating an appropriate P_w

TABLE 5. 12 The irrigation parameters based on different irrigation intervals for three shift numbers

shift numbers	F (day)	T (hr)	Q_E (L/hr)	P_w (%)
Nsh=1	3	40	2.43	27
Nsh=2	4	30	4.33	35
Nsh=4	7	30	7.57	50

To select an optimum design a trade off between the size of pipes and the corresponding head loss was carried out by the model to select an optimum design. The minimum cost solution for each irrigation shift considering the effect of subunit dimensions and shift patterns with some other irrigation parameters are given in Table 5.13. The influence of shift patterns corresponding to various shift numbers is also shown in Figure 5.20.

In each shift operation a set of local optima and also the minimum cost solution are computed. As discussed previously, the total system cost is not the same for various subunit sizes. For each shift number a minimum cost solution exists which is shown in Table 5.13. It is clear that the global optimum cost mainly occurs when a lower number of shifts is selected for operation. This concept is presented graphically in Fig. 5.21.

TABLE 5.13 The global optimum cost for three different selected numbers of shifts and some other irrigation parameters

Psux (m)	Psuy (m)	Hpu (m)	Pm (Kw)	Nsh	II	JJ	F (day)	Pw (%)	QE L/hr	operating cost($)	Total cost($)
200	150	49	38	1	4	4	3	0.27	2.4	51861	275516
133.3	150	48.5	31.1	2	6	2	4	0.35	4.1	51322	282241
133.3	150	49.1	29.4	4	6	1	7	0.5	7.6	51982	293688

Furthermore, the constraints related to the percentage wetted area, soil infiltration capacity and the maximum application depth of irrigation in each irrigation cycle were verified. To avoid using extra water the applied water which is directly affected by the emitter flow rates and the irrigation duration should be less than the gross depth which can be stored in the root zone. Table 5.13 shows that the dimension ratios of subunits for minimum cost solutions is consistent with results were found in Chapter 3 and also the configuration of irrigating shift patterns (combination of II & JJ) are consistent with the results discussed in section 5.10.3.

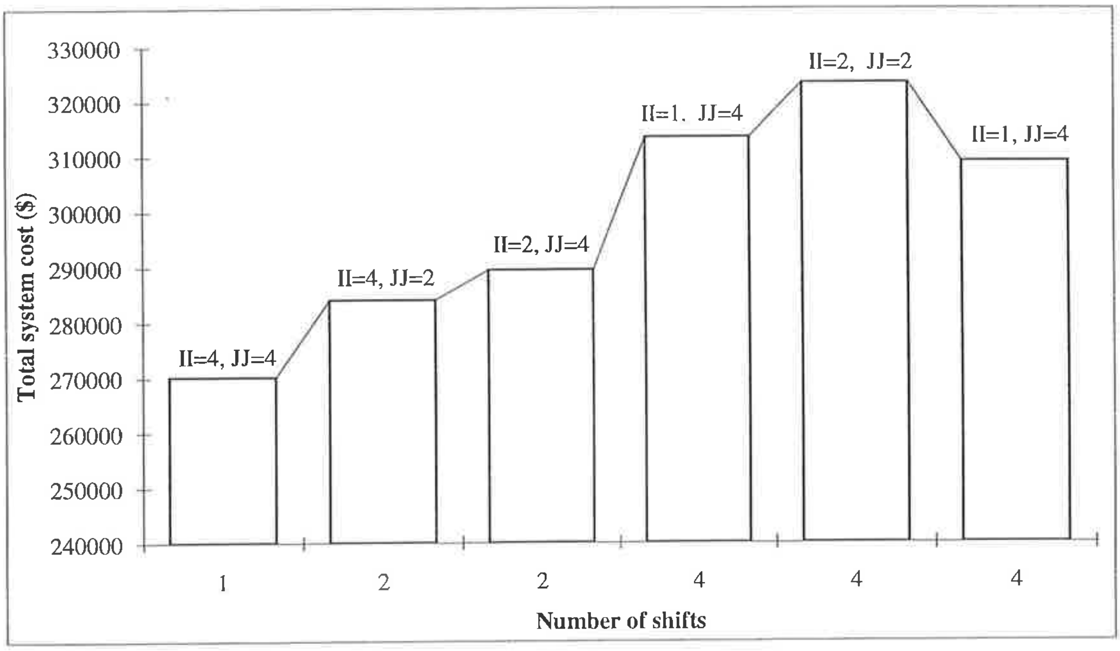


Fig. 5.20 The effect of shift patterns on the system cost for each shift number

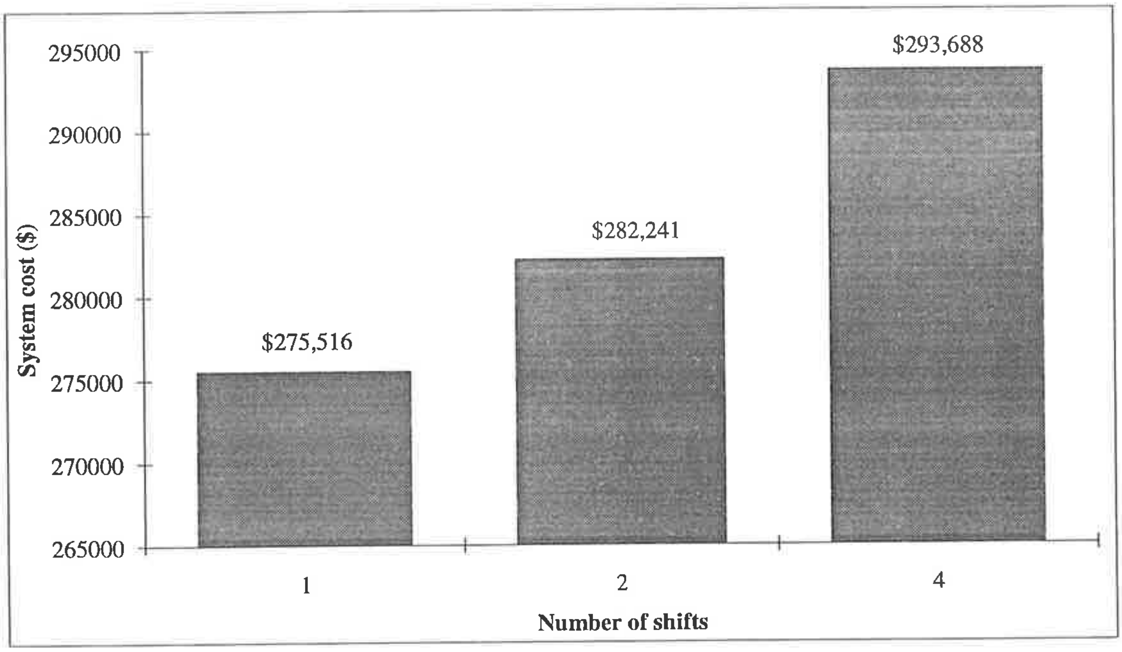


Fig. 5.21 Global optimum cost for each selected number of shifts

5.10.5 Cost of Different Pipes in A Multiple Subunit System

The cost of five different pipes that deliver and distribute the irrigation water throughout the system is displayed in Figure 5.22. As is clear, the lateral pipes constitute a large portion of the system cost. Furthermore, the findings indicate that the effect of subunit sizes on the lateral costs is not large, as the laterals are limited to two small sizes and usually the smaller size is used. The size of subunits therefore has virtually no effect on the total length of laterals. However, the costs of manifold and supply pipes are affected by subunit sizes since as the size of subunits decrease the rate of flow which is delivered by these two pipes is decreased, therefore, a smaller pipe size is selected. But the cost of main and submain lines increase as the size of subunits decreases. This is due to the configuration of pipes in the distribution system. As the size of subunits decreases the number of subunits increases, as a result, the length and the numbers of submain and also the length of mainlines increase. For subunits with a length of 400m, the cost of mainlines is zero, since they are not needed.

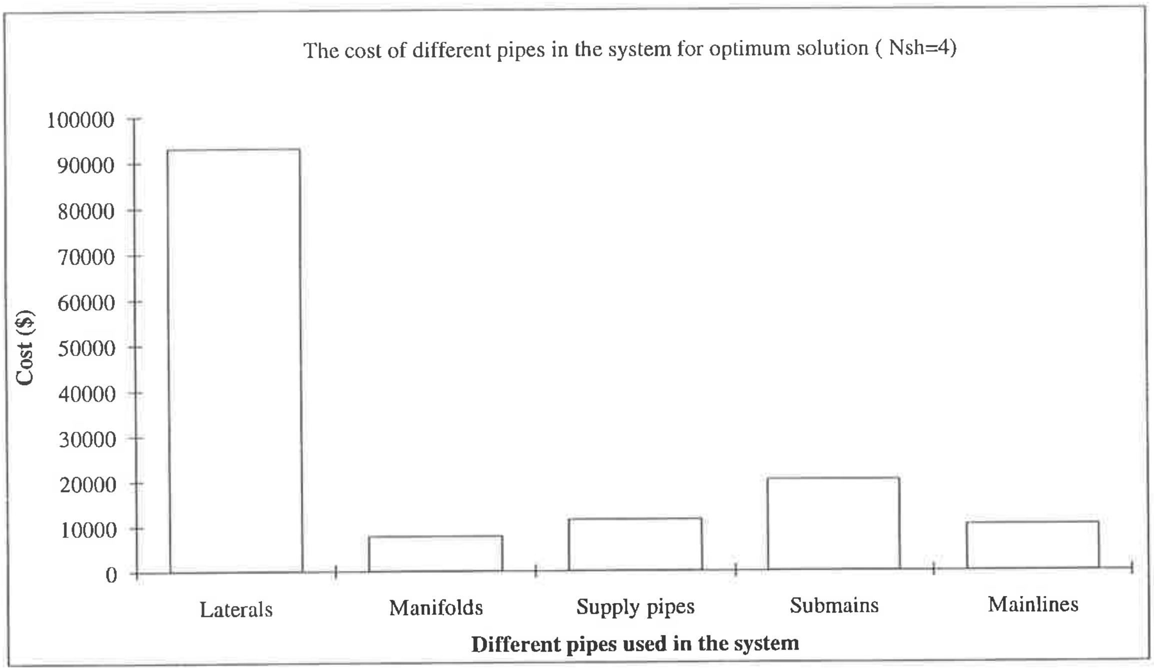


Fig. 5.22 Cost of different pipes within the multiple subunit system for optimum solution

5.11 SUMMARY AND CONCLUSIONS

An optimisation model for a drip irrigation on a field with multiple subunits has been developed. The model partitions a field into subunits with an assumed layout and configuration of the piping system. The model evaluates various shift patterns and determines the minimum cost design for each. The design variables are the length of each of two diameters of laterals, the diameters of the manifold, supply line, submain and mainline pipes as well as the power of pump required. In addition, the model identifies the optimum sizes of subunits as well as the optimum shift pattern.

The cost includes the capital and installation cost of all pipes, the pump, emitters, valves and accessories and the present values of annual operating costs of the system. The model can be applied to a level rectangular field with a groundwater source at the centre. It can be applied to various field sizes and crops in different regions. This can be achieved by specifying the input data such as: dimensions of the field, emitter and lateral spacing, potential evapotranspiration and crop coefficient, and the annual crop irrigation requirement.

When applied to a particular case study, the model showed that one shift operation was more efficient than the multiple shift operation. In general, it would appear that the minimum number of shifts should be used, consistent with achieving a reasonable flow rate through the emitters. The model evaluates various dimensions of subunits for one , two and four shift operation. The effect of subunit areas of various divisions of the field in the X and Y directions for three applied irrigation shifts are shown in Figures 5.15 to 5.17. The least cost solution in each case has obtained when the size of subunits was between 3 and 1.3 ha (the field was allowed to be divided into subunits with areas between 12 and 0.5 ha). In each case, these corresponded to using the smaller size of laterals. This is reasonable given that the laterals constitute 30 to 36% of the system cost in this case (Tables 5.6, 5.8 and 5.9). In general, it is considered that efficient designs will use the smallest possible size of laterals. The optimum design for one shift operation does not have any mainline pipes and only two submains. Again this gives some general guidance to try to reduce the number of feeder pipes where possible. In this case the optimum ratio of the X to Y dimension of the subunits is 5.33 to 1 (X parallel to the laterals). This differs from the general experience with optimising single subunits where this ratio lies in the range 1 to 1.5 but is reasonable when the other costs are considered. It should be noted that, except for 1-shift operation, the minimum cost solutions under 2- and 4-shift operations are fairly consistent with the result obtained in Chapter 3 and given by Oron and Walker, 1981.

Division of large field into subunits facilitates the use of various combinations of subunits to be irrigated simultaneously (i.e. Fig. 5.3). A number of possible cases in which different combinations of subunits in the X and Y directions may be selected to be irrigated in one time are discussed in Section 5.10.3. The findings indicate that a significant saving can be achieved in this regard. Table 5.10 shows a number of examples with different shift patterns resulted from the model.

However, in a version of model in which the irrigation interval is not limited to 7 days, the results show that the subunit dimension ratios for the global minimum solution is consistent with the results obtained in Chapter 3. The results show that under high number of shift operations the size of subunits is limited comparing to the low number of shifts. For example, there is no feasible solution under 4-shift operation when the size of subunits is 12 ha (2 divisions in the X and Y directions). This is due to the head loss violation in micro pipes unless larger diameter is considered for those pipes or less distribution uniformity is expected. Table 5.13 summarises the results corresponding to the global minimum cost for three irrigation intervals.

Chapter 6

Genetic Algorithms Methodology

6.1 Introduction

This part of the thesis is concerned with the use of genetic algorithms (GAs) as a natural way to search for the best. Firstly, the history and theory of genetic algorithms and a review of their application to pipe network optimisation problems are examined in the present chapter. The application of GAs to pressure irrigation systems for the optimisation of layout, pipe sizes and pump selection are then discussed in Chapters 7, 8 and 9.

In nature, natural populations evolve over many generations according to the principles of natural selection and survival of the fittest (Beasley et al, 1993). By mimicking this process, GAs are able to evolve efficient solutions to real world problems. GAs are adaptive and structured search methods which may be used to search and solve optimisation problems. They are a set of search techniques based on natural selection and the mechanisms of population genetics (Holland, 1975; Goldberg, 1989). GAs use a direct analogy of natural behaviour based on the genetic processes of biological organisms. They work with a population of individuals, each representing a possible solution to a given problem. GAs have been successfully applied to many different problems including:

engineering, music generation, genetic synthesis, industrial planning, etc.. This indicates the capability and the flexibility of the method.

6.2 OVERVIEW AND LITERATURE SURVEY

6.2.1 The Fundamental Principles of Genetic Algorithms

Genetic algorithms as search methods are rooted in the mechanisms of evolution and natural genetics. They are based on the principles of natural selection and survival of the fittest. Professor John Holland proposed genetic algorithms in the early 1970's as computer based methods that mimic the evolutionary process in nature. Holland (1975) in his book *adaptation in natural and artificial systems*, sets the framework for this special approach to search and adaptation. Holland's book is a projection of separate ideas and realisations on which he worked for many years proceeding the book's publication. Starting with a broad outline of adaptive systems during the early 1960s. They have been used over two decades to solve a wide range of search, optimisation and engineering problems. Goldberg (1989) presents a good survey of the nature and use of genetic algorithms.

When Holland (1975) first proposed genetic algorithms, this interest guided Kirkpatrick et al (1983) to introduce simulated annealing as another random search method for solving optimisation problems. Simulated annealing is based on thermodynamic considerations with annealing interpreted as an optimisation procedure. It probabilistically generates a sequence of statistics based on a cooling schedule to ultimately converge to the global optimum (Srinivas and Patnaik, 1994). Ribeiro Filho et al (1994) classified search techniques into three broad classes. In their classification the random search techniques (stochastic search methods) are divided into two categories; Genetic algorithms and evolutionary strategies (Figure 6.1). GAs are divided into two main classes, sequential and parallel. The sequential GAs are, in turn, divided into three subclasses: standard (simple) genetic algorithms (SGA), Genitor and Genesis. In this review only the SGAs are discussed in more detail. The GAs and their subclasses of search techniques as well as the evolutionary strategy techniques are illustrated in Figure. 6.1. Goldberg (1989) describes the GA as a

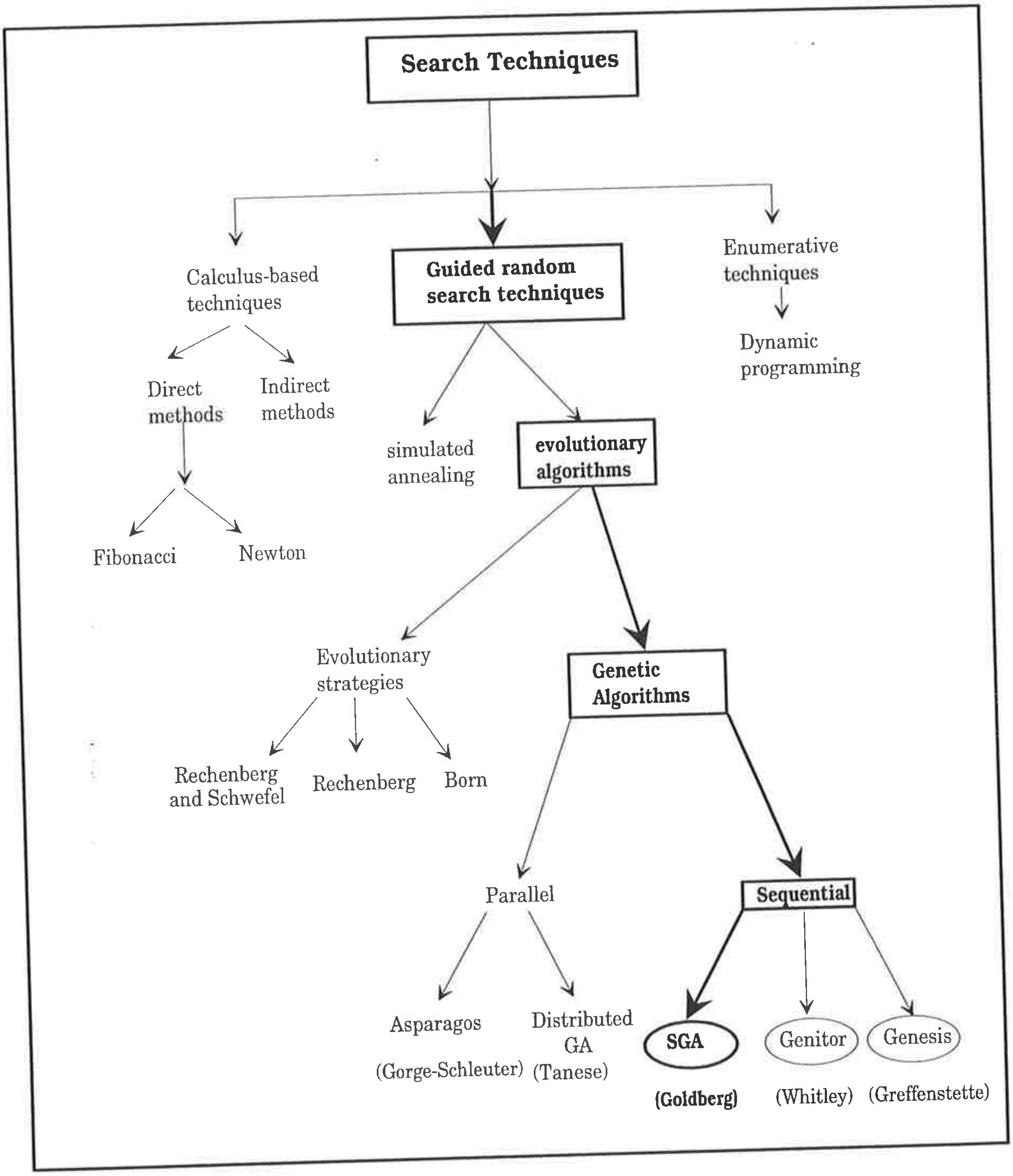


Fig. 6.1 The classification of the main search techniques (Ribeiro Filho et al, 1994)

stochastic optimisation technique which is based on the genetic process and natural selection. GAs attempt to simulate the near-optimal process of the evolution of living things. They are modelled on nature's very effective optimisation techniques of evolution, whereby a species adapts itself to a particular environment over a large number of generations" (Walters and Lohbeck, 1993). GAs combine artificial survival of the fittest with genetic operators abstracted from nature to form an efficient and strong search mechanism that is suitable for a variety of search problems (Goldberg, 1989). The genetic algorithms represent an important subclass of the evolutionary programming approaches. They are random search algorithms that start with a population of randomly selected feasible solutions. Each member of the subsequent generation is selected from the current one using an appropriate selection technique (proportionate or tournament selection). Essentially, a genetic algorithm simulates the evolution of a population in an environment which is characterised by the function to be optimised (Bethke, 1980). The principle of the genetic algorithm process is demonstrated in Figure 6.2.

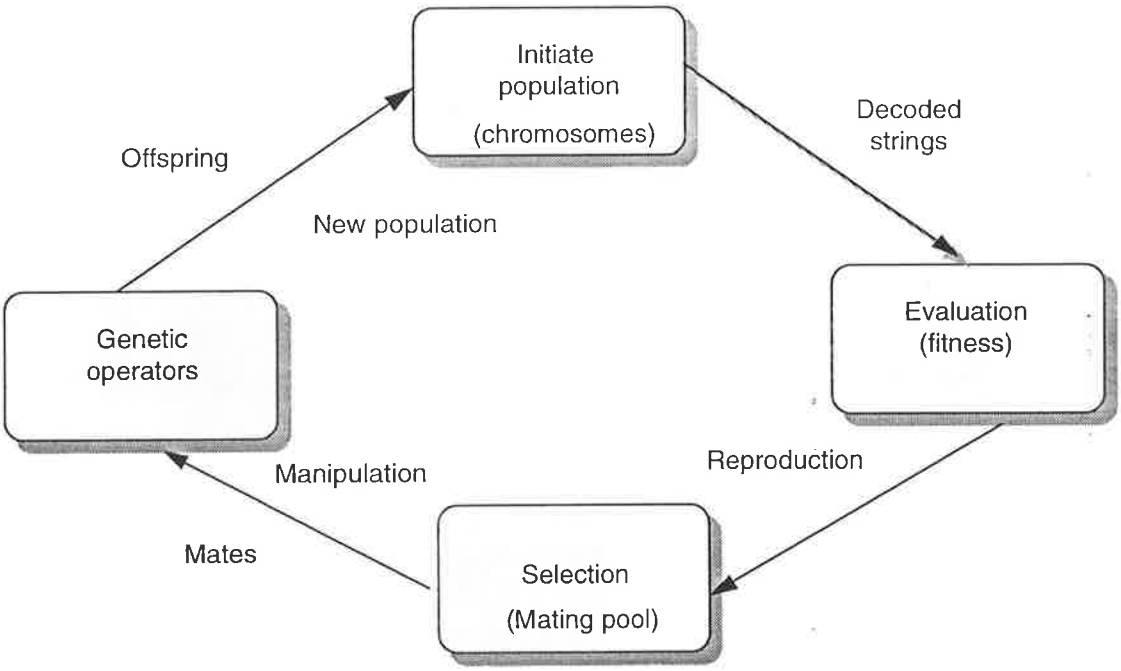


Fig. 6.2 The principle of the genetic algorithm process (Ribeiro Filho et al, 1994)

In recent years, genetic algorithms have emerged as practical robust optimisation and search method. Diverse areas such as music generation, genetic synthesis, VLSI technology, strategy planning, and machine learning have profited from these methods (Srinivas and Patnaik, 1994). From the early 1980s the GA community has experienced a lot of GA applications which spread across a large range of disciplines. Each and every additional application gave a new perspective to the theory. Golberg's work (1983) on optimisation of a gas pipeline for the steady-state case using GAs is a classic example.

There are other notable applications in engineering and pipe network optimisation (Murphy and Simpson, 1992; Davidson and Goulter, 1992; Murphy et al, 1993; Simpson et al, 1994; 1995; Dandy et al, 1993; Murphy et al, 1994; Walters and Lohbeck, 1993; Simpson and Goldberg, 1994; Hassanli and Dandy, 1994, 1995b). GAs are different from the normal methods of optimisation in a number of ways.

As Goldberg (1988) explains:

(1) GAs are blind; many other search methods use much knowledge about their intended problem class to obtain a solution. By contrast, GAs are blind and treat the problem as a black box.

(2) GAs use coding of decision variables not the parameters themselves; GAs achieve their relative efficiency by adapting coding of the decision parameters using artificial chromosomes.

(3) GAs search a population of points; many search methods work from point to point, they move gradually from a single point in the decision space to the next, using local information to decide which point to explore next. GAs however, work from a number of points simultaneously and climb many peaks in parallel.

(4) GAs use probabilistic not deterministic rules (randomised operators). As Goldberg (1986, 1988) emphasises, using randomised operators instead of deterministic transition rules should not cause confusion with a simple random walk.

A genetic algorithm technique simulates mechanisms of natural population genetics in an artificial evolutionary strategy (Murphy et al, 1993). It comprises a set of individual elements each representing a potential solution. According to evolutionary theories only the more suited elements in a population are likely to survive and generate offspring. In fact, GAs work with a set of strings, each string representing a potential solution (Ribeiro Filho et al, 1994) which is associated with a fitness value that reflects how good it is compared with other solutions. The principle steps of genetic algorithms which operate through a simple cycle of stages are given as below:

1) Generate an initial population of strings (chromosomes);
2) Evaluate each string of the population by its fitness;

3) Use the fitness to select the pairs of strings for the next population;
4) Examine the selected string to see whether crossover will occur using the value of Pc. If crossover occurs do crossover;
5) Examine each bit of each string with a probability of Pm for mutation, if mutation is true change the bit value;
6) After crossover and mutation, insert the new strings in the next generation;
7) Repeat steps 3 to 6 until all strings are examined and a new generation has received the same number of population as the current generation;
8) Repeat steps 3 to 7 to create a new generation until a specified number of generations has been reached.

The above process is demonstrated by the flow chart shown in Figure 6.3. In the first stage of the process the GA creates a number of strings in a random manner. The creation of strings is based on the coding format and the number of decision parameters. These parameters (known as genes) are joined together to form a string of values (Beasley et al, 1993). The GA uses the current population of strings to create a new population (Spillman, 1993) such that the new strings in the new population inherit some characteristics of the old population. Each individual string represents one of many possible solutions within the large solution space and is evaluated on the basis of a fitness function. The GA successively evaluates the members of a population which are in fact a set of trial solutions (Simpson et al, 1994). The idea is to use the better elements from the current population via a selection trial considering the fitness or worth function of each individual string. If this is carried out properly, then the new population of strings will , on average, be better than the old population. For a given chromosome the fitness function returns a single numerical figure of merit, which is related to the ability of the solution which that chromosome represents. As Beasley et al (1993) explain the power of GAs comes from the fact that the technique is robust, and can deal successfully with a wide range of problem areas. GAs are not guaranteed to find the global optimum solution to a problem, but they are generally good at finding good solutions close to global optimum. The GA in a simple form uses three operators to make the transition from the population of one generation to the next generation: reproduction, crossover and mutation. These are described in the following sections:

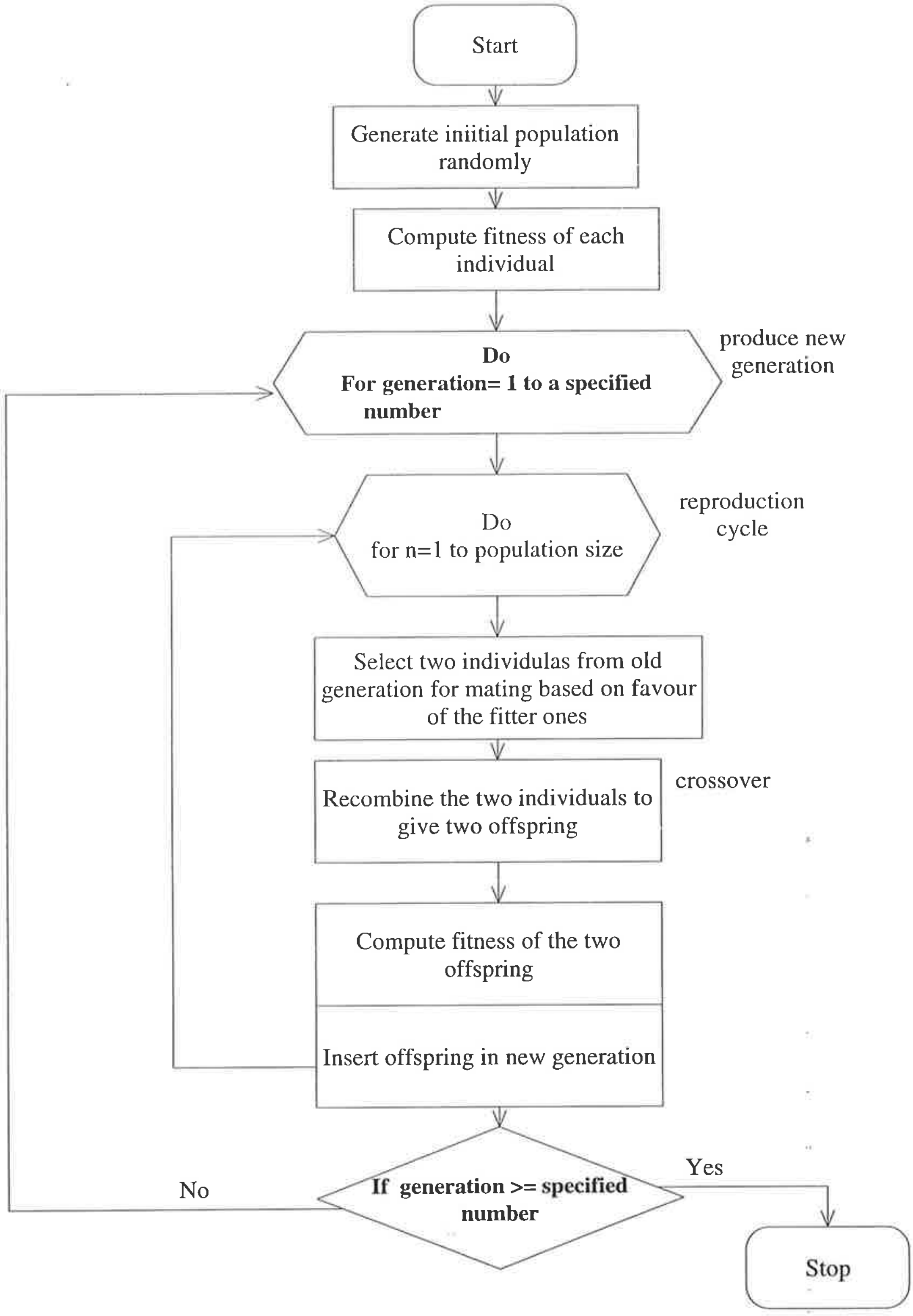


Fig. 6.3 Flow chart showing the principle of genetic algorithm process

6.2.2 Reproduction Scheme

The first operator of a simple GA is called selection or reproduction. The aim is to determine which strings in the current generation will be used to create the strings within the next generation. This is carried out by implementing a biased random selection methodology. Parents are selected randomly from the population using a selection scheme which favours the more fit individuals. Good individuals which posses high fitness will probably be selected several times in a generation, while poor ones with the low fitness may not be selected at all (Beasley et al, 1993). In the SGA a fitter string receives a higher number of offspring and thus has higher chance of surviving in the subsequent generation (Srinivas and Patnaik, 1994).

Reproduction is an application of Darwin's survival-of-the fittest philosophy (Murphy et al, 1992). The fittest and strongest chromosomes survive to continue to be a part of the search. In this way less fit strings will die off while some weak strings will survive by chance. Goldberg (1989) believes that the success of the GA is due to using higher fitness strings to cerate the new generation. Accordingly, if the selection scheme could guarantee that the best strings are represented in the parent set then this should provide a better GA process (Connarty, 1995). A number of selection schemes for GAs were examined by Goldberg and Deb (1991). The examined selection schemes were proportionate selection, tournament selection, linear ranking selection and Genitor or steady state selection.

In the simple genetic algorithm which was examined by Goldberg (1989), proportionate selection is used. The fitness of chromosomes may be used to make a weighted roulette wheel as illustrated in Figure 6.4. A chromosome is associated with a segment of the weighted roulette wheel. The size of the segment is a function of the fitness of the chromosome. The chromosomes with higher fitness values have larger segments of the wheel and therefore a greater probability of selection. For example, the segments in Figure 6.4 each represents a particular chromosome. The chromosome assigned segment 6 has more chance to be selected to produce new offspring for the next generation.

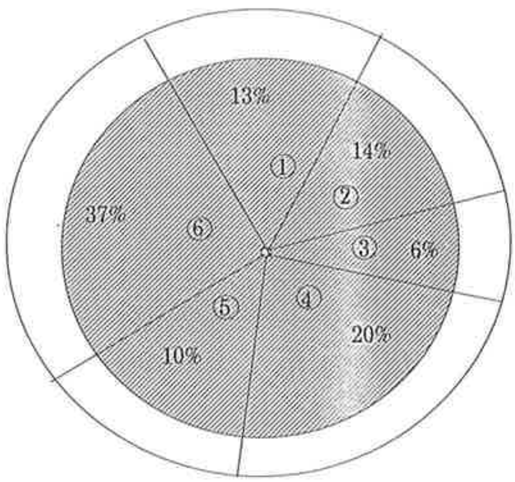


Fig. 6.4 Roulette wheel with slots sized according to fitness used (proportionate selection)

In proportionate selection, the members of the next generation are randomly selected in proportion to their fitness relative to the fitness of all the other strings. Therefore, the probability of selection is:

$$P_i = \frac{f_i}{\sum_{i=1}^{n} f_i} \tag{6.1}$$

where P_i = probability of selection of member i ; f_i = fitness of member i of the current generation; n = population size.

Simpson and Goldberg (1994) explain that a disadvantage of the proportionate selection is that later on in a GA run all members usually have very similar fitness values and thus there is a lack of selection pressure to cause the better strings to dominate. This leads to loss of good solutions in subsequent generations. This shortcoming of proportionate selection has caused researchers to turn to scaling techniques and ranking methods. The use of an exponential scaling function to control the degree of competition is recommended by Goldberg and Deb (1991). Murphy et al (1993) outline satisfactory results of implementing a power scaling function in proportionate selection in their pipe optimisation problem.

The next selection scheme examined by Goldberg and Deb (1991) is tournament selection. In this scheme, some number of strings are chosen from a population at random (with or without replacement). The best string from the chosen group is then picked up to go into the next generation. The process is repeated until enough strings are selected to fill the new population. In this way, several copies of the string with the largest fitness will go onto the next generation and no copies of the least fit string will go on. As outlined by Simpson and Goldberg (1994), on average one copy of the median fittest string will go onto the next generation. Goldberg and Deb (1991) examined the effect of tournament size on convergence time. They reported that as the tournament size increases, the convergence time decreases by the ratio of the logarithm of the tournament sizes. The tournament size was also investigated by Simpson and Goldberg (1994) for the pipe network optimisation problem. They report that tournament selection performs better than proportionate selection in terms of a significant reduction in the number of evaluations needed to achieve the global optimum. A comparison showed a reduction in the number of evaluations for tournament sizes of 2, 5 and 20 compared with proportionate selection. Their investigation shows that the number of evaluations can be further reduced by increasing the tournament size and consequently the selection pressure.

The ranking selection scheme was introduced by Baker (1985). In this scheme, the population is sort from best to worst. The fitness of each individual is assigned according to a non-increasing assignment function of its rank order and then proportionate selection is performed according to that assignment. The performance of ranking selection was analysed quantitatively by Goldberg and Deb (1991).

The Genitor selection or steady state method was developed by Whitley (1990). As Goldberg and Deb (1991) explain Genitor works individual by individual. An offspring is chosen by linear ranking for the next generation and then currently the worst individual is chosen for replacement. In other words, not only two individuals by their ranked fitness score are selected to be parents but also two unlucky individuals from the population should be selected to be killed off (Beasley et al, 1993). Goldberg and Deb (1991) analysed and simulated the Genitor method and compared the results with other selection schemes. The selection schemes were compared on the basis of their difference equations, growth ratio estimations and take over time computations. Proportionate selection was found to be slower than the other three types to find the optimum solution. This was verified for two reservoir pipe network problem by Simpson and Goldberg (1994). They report that tournament selection outperforms proportionate selection and a 5 size tournament produces the optimal solution in the least number of evaluations over ten runs. As Goldberg and Deb (1991) observe a binary tournament is preferred over linear ranking while larger tournament sizes or non-linear ranking can give growth ratios similar to Genitor.

After the performance of the selection scheme, crossover (as the second operator) is the main operator in making diversity in the population. The crossover is explained in more detail in the following section.

6.2.3 Crossover Operator

The mechanism of crossover is the breakage, partial exchange and reunion of pairs of chromosomes. In this way each pair of selected chromosomes are crossed over to generate new chromosomes on the base of a particular probability of crossover. This operation is analogous to sexual reproduction in nature (Ribeiro Filho et al, 1994). In the 1-point crossover process, a random location along the two chromosomes is chosen. The chromosomes are then separated at this point and recombined to form two new chromosomes. In this process the left hand part of one chromosome combines with the right hand part of the other and vic versa. Crossover occurs with a specified probability. It is not usually applied to all individuals selected for mating. A random choice is made where the probability of crossover being applied is typically between 0.6 and 1.0 (Beasley et al, 1993). If crossover is not applied, offspring are produced simply by duplicating the parents. A traditional GA uses 1-point crossover. Figure 6.5 demonstrates a 1-point crossover between two parents.

However, a number of different crossover algorithms have been proposed. For example, Dejong (1975) investigated the effectiveness of multiple-point crossover. As reported by Goldberg (1989) two-point crossover gives an improvement in performance, but adding further crossover points above 2, reduces the performances of the GA. Two-point crossover has two randomly chosen cut points instead of one, the chromosome segments are swapped between the two cut points between the two strings. Simpson and Goldberg (1994) report that there are some schemata that two-point crossover can not combine.

However, researchers agree that two-point crossover is generally better that one-point crossover (Beasley et al, 1993). Uniform crossover is the third method which is radically different from one-point crossover. Each gene in the offspring is created by copping the corresponding gene from one or the other parent, chosen according to a randomly generated crossover mask. The offspring produced by the uniform crossover contain a mixture of genes from each parent and the number of effective crossing points is not fixed (Beasley et al, 1993).

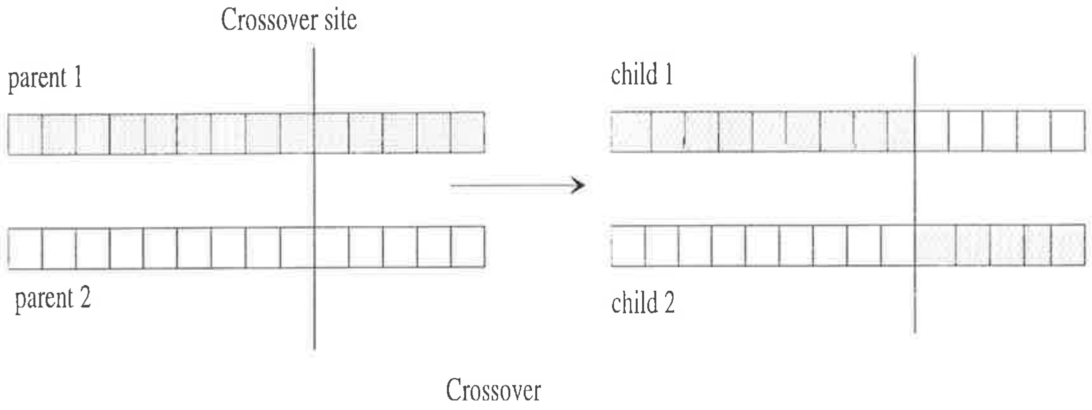


Fig. 6.5 One-point crossover between mating chromosomes

Goldberg (1985, 1989) describes a rather different crossover operator called, partially mated crossover (PMX). In PMX it is not the values of the genes which are crossed, but rather the order in which they appear. Offspring have genes which inherit ordering information from each parent (Beasley et al, 1993). Simpson and Goldberg (1994) examine the first three methods of crossover operators for a pipe network optimisation problem. They report that there seems to be little difference in various crossover methods for that problem.

6.2.4 Mutation

Mutation is the occasional random change of coded bits of chromosomes. It is applied to each chromosome individually after crossover. Mutation randomly alters each gene with a small probability. This operator of GA is an insurance against the loss of potentially useful genetic material. As Goldberg (1989) states mutation is a simple random walk through the string space. It is an insurance policy against the loss of important genetic material at a particular position. Simpson and Goldberg (1994) explain that mutation can be applied in two ways: bit-wise mutation in which a bit is randomly chosen in a string and changes to the opposite value and; adjacency mutation. In this method, the selected complete decision variable sub-string is altered to an adjacent decision variable sub-string up or down the pipe choice list (in a pipe optimisation problem). They compared the results of a pipe network problem by using a random mutation with a value of zero ($P_m = 0$) and adjacency mutation with a probability of 0.02. They obtained an improved result for adjacency mutation. Davis and Coombs (1987) use an operator similar to adjacency mutation (called creeping) in their study of the design of packet switching communication networks. Murphy et al (1993) report the use of adjacency mutation in their pipe network optimisation problem. They observe an improvement in their model efficiency as a result of using adjacency mutation. Figure 6.6 shows the effect of a traditional mutation on a 15 bit chromosome at genes 9.

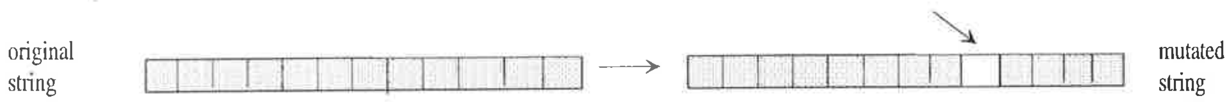


Fig. 6.6 Mutation operator at gene 9

To increase the effectiveness of the GAs for different problems it is sometimes necessary to change the characteristics of the simple GA. A number of researchers have attempted to examined different methods of crossover and mutation. Also some researchers examine the effect of different coding schemes on the GA process. In the following section, various coding schemes are discussed in detail.

6.2.5 Coding Scheme

As discussed previously, one of the main features of GAs is their potential adaptation to coding the decision parameters. A successful coding scheme is one which encourages the formation of building blocks by ensuring that: "related genes are close together on the chromosome, which there is little interaction between genes (Beasley et al, 1993). In the simple genetic algorithm, Goldberg (1989) uses binary codes consisting of 1s and 0s. Any combination of 1s and 0s can be representative of a decision variable. 2-bit, 3-bit or 4-bit binary numbers could be implemented to problems with 4, 8 and 16 choices of each decision variable respectively. Murphy and Simpson (1992) use a 3-bit binary numbers for a pipe network to represent each of eight choices of a decision variable. In their work all eight 3-bit sub strings were joined together to form a 24-bit binary string.

The use of Gray coding has been used as an alternative to binary coding (Caruana and Schaffer, 1988). Simpson and Goldberg (1994) state that the Gray coding has been suggested as a better alternative coding to binary. Gray coding, in fact, uses the same binary variables as binary coding. Nevertheless, in Gray coding there is only a one bit change between adjacent strings. For example, a using binary coding the strings "101" and "110" represent adjacent strings and using Gray coding the same adjacent strings are represented by "111" and "101". In Gray coding only a change in one bit is needed to move to an adjacent string whereas in binary coding may be up to three changes needed for a 3-bit sub string to move to an adjacent string. For example, a problem with eight options (i.e. a pipe network with 8 discrete pipe sizes) may be represented as a 3-bit string in binary or Gray coding scheme as shown in Table 6.1.

Table 6.1 Binary, Gray and integer coding schemes with 3-bit for eight options

Binary coding	Gray coding	Integer coding
000	000	0
001	001	1
010	011	2
011	010	3
100	110	4
101	111	5
110	101	6
111	100	7

Murphy et al (1993) use a Gray coding scheme to improve the genetic algorithm formulation in a pipe network optimisation problem. They argue that the use of Gray coding in their model for the New York city tunnels problem with 4 bit binary strings for each decision variable reduces the distance between adjacent decision variables. This is because to move to one pipe size to an adjacent size only involves altering one bit rather than 4 bits of the sub string.

Real coding is another type of coding scheme which is used in GAs. It may be used as floating point or other higher cardinality integer coding. The success of the GA coding scheme is dependent on the cardinality of the coding used. The cardinality of a coding scheme is defined as the possible range of values which can occur in a gene. In the case of integer numbers it is 9 for the numbers between 0 and 8 and 2 for a binary or Gray coding. More recent empirical work has indicated that real-coded GAs have given satisfactory results in a number of practical problems (Simpson and Goldberg, 1994). Goldberg (1990) examined the role of real-coded genetic algorithms which use floating-point or other high cardinality coding. He shows that the use of higher cardinality alphabets cause some solutions to converge more quickly than those coded over a smaller alphabet. Much research using different coding schemes has been carried out in relation to pipe network and water resource optimisation problems (Dandy et al, 1993; Simpson and Goldberg, 1994; Hassanli and Dandy, 1994; Connarty, 1995) and; also in relation to job scheduling and communication network problems (Davis, 1985; Coombs and Davis, 1987; Davis and Coombs, 1987).

6.2.6 Genetic Algorithm Parameters

In this section, the effects of three genetic algorithm parameters including the size of population, the probability of crossover and the probability of mutation are discussed. One of the major decision for every user of GAs is the selection of the appropriate size of population. If too small a size is selected, the GAs will converge too quickly, with insufficient processing of low order schemata (Goldberg, 1985). Selection of a very high size of population will increase the run time required for a significant improvement. Some researchers have attempted to examine different sizes of population to find the optimal size by a set of empirical studies. Dejong (1975) and Grefenstette (1984) demonstrate that simple GAs show a good performance at a moderate size (35-200) of population. The existing theory shows that larger populations give better results Goldberg (1985). Grefenstette (1984) used population sizes between 10 and 160 in increments of 10. As

Goldberg (1985) notes the optimal average population size for empirical examples were 90 for a string length of 30. On the other hand, Goldberg (1985) predicted an optimal size of 106 with theoretical studies. He concluded that although the results of experimental and theoretical work are close, a further study is needed. Goldberg (1985) examined the existing theories of schemata processing as related to population size. He develop a new theory for the calculation of optimal population size as a function of string length.

A typical population size ranging between 35 and 200 was suggested by Goldberg and Kuo (1987). Simpson and Goldberg (1994) performed a number of simulations to determine the performance of the genetic algorithms for various population sizes. They present a formula for population size on the basis of the string length, cardinality of the coding, standard normal deviate and probability of error on a string trial. They test their formula for a water pipe network with string length of 24. The predicted population size obtained (with a range from 6206 to 477107) were quite large which were dependent on the estimated parameters. They use a formula suggested by Goldberg, Deb and Clark (1992) for lower bound population size for uniformly scaled problems. The population size obtained for a pipe network problem with string length of 24 was 106. They conclude that since the pipe network problem would not be expected to be linear and uniformly scaled, the required population size may be expected to be larger than 106. They state that for a two reservoir pipe network optimisation problem a population size larger than 200 is required to ensure convergence.

Simpson and Goldberg (1994) evaluated the effect of different population sizes of 100, 200 and 1000. They suggest a population size in the range of 8(L-1) to 45(L-1) is required to optimise the two reservoir pipe network problem (where L = string length). Simpson et al (1994) suggest a population size between 30 to 200 for GAs. Walters and Lohbeck (1993) use a constant size of 20 for all trial runs of their branched pipe network problem. They report that a population size between 10 and 50 is typically used. Murphy et al (1993) investigated different population sizes for a pipe network with 84 bit string. In their work, population sizes of 100, 200 and 500 were used. Davidson and Goulter (1992) used a population size of 15 for a small pipe network example. They concluded that experiments on small pipe network problems with six or seven nodes indicate that a population size of 15 is adequate and preferable to a large size population. Dandy et al (1993) use a population size of 100 to optimise the Loubser and Gessler problem.

Probabilities of crossover and mutation are also two genetic algorithm parameters which can be varied. If crossover does not occur the two selected chromosomes are simply left in the new generation unchanged. Goldberg (1987) indicates that a GA achieves the most success with high probabilities of crossover in the range 0.7 to 0.9. In a similar way, Dejong (1975) suggests that a good results can be achieved with GAs by using a low mutation probability and high crossover probability. According to Goldberg and Kuo (1987), GAs are not highly sensitive to these parameters. They also suggest the use of high crossover probability (0.5 to 1.0) and low mutation probability. They suggested mutation probability may be inversely proportional to the population size (P_m = 1/N to 5/N). Murphy and Simpson (1992) use 0.9 and 0.02 for the probabilities of crossover and mutation respectively. Dandy et al (1993) use 1.0 and 0.0 for the probabilities of crossover and mutation. Beasley et al (1993) suggest a value between 0.6 to 1 for crossover probability and 0.001 for mutation probability as typical GA parameters. Walters and Lohbeck (1993) report that for their example (a branched pipe network) a crossover probability of 1.0 and mutation probabilities between 0.02 and 0.002 lead to the most successful solutions.

Simpson and Goldberg (1994) use a probability of 0.5 for the crossover and 0.0 for bit-wise mutation. However, instead of bit-wise mutation they use creeping mutation with probability of 0.02. They suggest that mutation probability is usually selected within the range of the 1/N to 1/L).

6.3 APPLICATION OF GAs TO WATER PIPE NETWORK PROBLEMS

The applications of GAs to water resources to date are mainly limited to pipe network optimisation problems (Connarty, 1995). As this study is concerned with the optimisation of pressure irrigation systems the application of GAs to pipe network optimisation problems will be discussed. Application of the GAs to pipe network optimisation was initiated by Goldberg (1983). He used this new probabilistic method to optimise a gas pipeline for the steady state flow case. In the optimisation of a simple pipe network, the decision variables are the sizes of pipes for delivering a specified volume of water under an allowable pressure level at nodes.

In more complicated problems, the optimum layout of links between nodes also could be unknown. Based on link connectivity, pipe network systems are divided into either looped or branched systems. The hydraulic analysis of looped systems is more complicated than branched ones and; a hydraulic solver would be necessary for the analysis of these systems. Murphy and Simpson (1992) use genetic algorithms to optimise the Gessler problem (1985) consisting of two reservoirs with 14 pipes (8 pipes to be sized). They use a 3-bit binary coding to represent eight decision variables. The length of strings used was 24 bits. Each 3-bit binary number could represent either a pipe size from a given range or cleaning the pipe or duplicating a particular link. A Newton-Raphson network solver is utilised for hydraulic analysis and determining the pressure at the nodes. They report that GAs could find optimal and near-optimal solutions after searching only a small portion of the search space. To eliminate the infeasible solutions they use a penalty function. Penalty costs are computed if the pipe network design does not satisfy the system performance constraints such as minimum pressure at nodes for a given demand pattern.

Dandy et al (1993) compare different optimisation methods including linear programming, non-linear programming and partial enumeration with the genetic algorithms. They use a water distribution network described by Loubser and Gessler (1990) and apply the above mentioned optimisation techniques. For the GA they use an integer coding for decision variables from 0 to 8 to indicate sizing of the new pipes and duplication of 8 existing pipes. The decision variables were 8 available pipe sizes and the option to leave existing pipes. They chose GA parameters which are consistent with the suggestions of Dejong (1975) and Goldberg and Kuo (1987). Dandy et al, also use a penalty factor of $175000/m for the head below the minimum allowable pressure at nodes. They add that increasing the penalty factor will exclude infeasible solutions while reduction of the penalty factor makes the marginally infeasible solutions more prominent in the search space. They conclude that genetic algorithms identify the best solution for the case study. The use of a penalty cost to eliminate the infeasible solutions is discussed in Chapter 9 of this thesis.

Walters and Lohbeck (1993) applied the GAs to a branched pipe network. They used GAs to identify the optimum layout and then dynamic programming approach was used to find the optimum pipe sizes. In their study they developed two alternative GA search models. In both models the base graph has been used to reduce the size of the search space. The first model deals with low degree of connectivity (only two alternative links supply each node). The second model deals with networks with a high degree of connectivity. They applied the binary coding scheme to the first and a novel integer coding to the second model. Their results show that, for low connectivity networks the binary coding works efficiently. However, for base graphs of high connectivity, an integer representation was found more effective. They argue the results show that the dynamic programming is limited to small networks with a small degree of connectivity. The result obtained shows that the GA is not very sensitive to population size and crossover probability. However, mutation probability has a significant influence on the result. Walters and Lohbeck indicate that although there is not a definite rule to determine the optimal mutation rate, it was found that having one change of bit for each string could be a reasonable size for this operator. They use values between 0.05 and 0.1 for the probability of mutation depending on the network size. They concluded that for small networks, the GAs were comparable in performance to a dynamic programming formulation. Although the GAs can't guarantee to reach the global optimal solution they can greatly reduce the computer memory requirements. For large networks they gave a rapid convergence to near-optimal solutions, whereas no algorithm will guarantee to find the global optimum.

Optimisation of pipeline and pumping operation is another problem investigated using GAs by Goldberg and Kuo (1987). A simple GA was utilised to solve the problem. In this model, the aim was to determine which pumps should be operated to provide a specified flow rate while the total required power was minimised. As the authors report, the GA found a near optimal solution which was only slightly worse than the optimum results obtained by an integer programming model. They present a useful comparison to determine the efficiency of the GA process. Davidson and Goulter (1992) applied GAs to the design the rectilinear branched pipe network. They investigate the problem of finding the optimal layout geometry for a network with a single source and multiple nodes. The fitness in their model was the total length of pipes in system. Therefore the least cost solution is associated with the layouts with the smallest length. This is not always correct because pipe networks usually consist of pipes with variety of diameters with different costs per unit length. Davidson and Goulter developed a method for removing redundant links to improve the efficiency of both the initial random search and the genetic algorithm.

Murphy et al (1994) applied GA search to the complex solution space for a pumped pipe network optimisation problem. They attempt to identify the optimum design and operation of the Anytown network considered in the Battle of the Network Models study (Walski et al, 1987). In this work they identified a number of alternative solutions. The sizes of new pipes, pump operation schedule and location of new tanks are chosen simultaneously by GAs.

Simpson et al (1994) compare the performance of GAs with both complete enumeration and non-linear optimisation techniques by using a simple GA with three operators. In their work the available pipe sizes were coded for selection as binary strings. They argue that genetic algorithms are very effective in finding near-optimal or optimal solutions for the Gessler network. They suggest that non-linear and enumeration optimisation are effective technique for relatively small networks. The other shortcoming of these methods is rounding up and down of the continuous solutions to discrete pipe sizes. Whereas GAs generate a whole class of alternative solutions close to optimum with discrete pipe sizes. A further research would provide improvement in these search methods for practical problems. Simpson et al (1995) apply GAs to a case study to optimise the selection of pipe sizes for the expansion of an existing water supply system. They concluded that the genetic algorithm technique has consistently achieved cost savings in the order of 5%-15% in their present and previous work.

6.4 SUMMARY

The history and theory of genetic algorithm methodology (GAs) and a review of their applications to pipe network optimisation problems were examined. Genetic algorithms as random search techniques (stochastic search methods) are rooted in the mechanisms of evolution and natural genetics. They attempt to simulate the near-optimal process of the evolution of living organisms. According to Ribeiro Filho et al (1994) genetic algorithms are divided into sequential and parallel. The sequential GAs are, in turn, divided into three subclasses. Standard (simple) genetic algorithms (SGAs), Genitor and Genesis. In this thesis SGAs are utilised. SGAs consist of three main operators: selection (reproduction), crossover, and mutation. The principle of genetic algorithms in which the above three operators are shown in a cyclic process is demonstrated in a flow chart shown in Figure 6.3. A number of selection schemes including proportionate, tournament, linear ranking and Genitor or steady state selections were described. Literature shows that GA with tournament selection leads to a least cost solution with a smaller number of evaluations (needs smaller run time) comparing to the proportionate selection.

The concept of crossover and mutation with different values of probability were examined. The use of a high probability ranging from 0.5 to 1.0 for crossover and a low probability ranging within 1/N to 1/L for mutation are recommended. In this chapter, different coding schemes including binary, Gray, integer and floating point were also discussed. The size of population is an important decision in any GA model. If too a small is selected, the GAs will converge too quickly with insufficient processing of low order schemata. Selection of a very high size of population will increase the run time required for a significant improvement. Finding an optimal size or an efficient range of population sizes is examined by a number of researchers. Degong (1975) and Grefenstette (1984) reported that a moderate size (35-200) of population shows a good performance.

The applications of SGAs to water distribution pipe systems for looped and branched networks were reviewed. The conclusion shows that GAs are effective in pipe network and have consistently achieved a good cost saving for different problems. However, the further research is needed to improve these search techniques for practical problems.

Chapter 7

Model for the Optimum Layout of Branched Pipe Networks

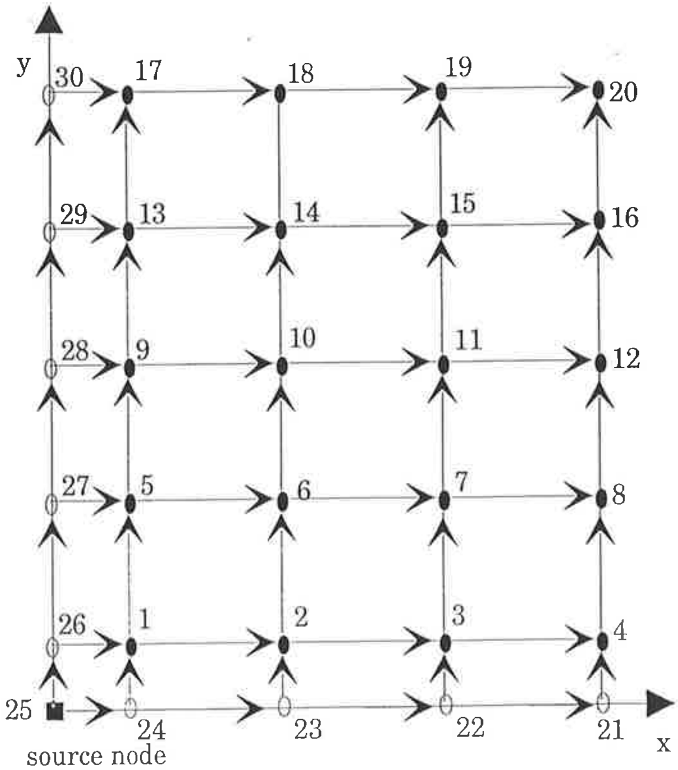
7.1 INTRODUCTION

An essential first step in the design of a water distribution pipe network is the determination of the location of pipes or layout of the links (Rowell et al, 1982). In designing such networks the selection of layout is important because it will serve as the foundation of the design. A water distribution network is an essential component for urban development and irrigation projects. A branched (tree) network of a water distribution system consists of one or more source node(s), a number of demand nodes and a number of pipes which link the demand nodes to the source node(s). The links are pipes of specific diameters delivering flow to satisfy demands which are assumed to be concentrated at the nodes. Flow in pipes may be delivered either by gravity or pressure provided by pumps which are usually located at the source node. When the system operation is based on gravity, the total head at the source will equal the minimum required pressure at nodes plus the total head loss of the connecting pipes.

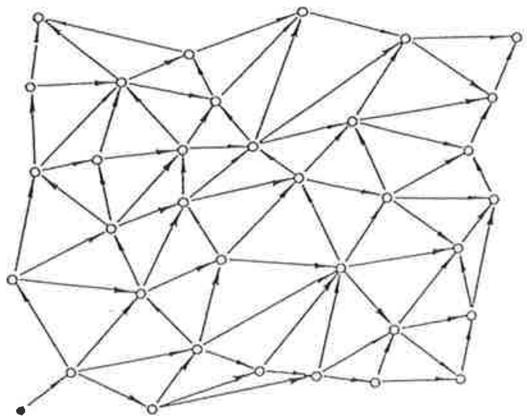
Depending on the position of demand nodes, the network could be classified as regular or irregular with different levels of connectivity as shown in Figure 7.1. The achievement of minimum cost is an important part in the design of a water supply distribution network. Since the capital cost of such networks is high, it is important to ensure that the design of a new system or modification to an existing system are as efficient and economical as possible. Many studies have been carried out on the design and analysis of tree networks and much research has been devoted to the development of optimisation models. Nevertheless, the development of methods to achieve the best possible design for any branched network still needs further investigation.

As noted by Walters (1987), many standard algorithms exist which enable optimal decisions to be found for simplified models of fairly complex systems. Many of these procedures are based on linear programming (LP). Dynamic programming (DP) is another approach which has been implemented for the optimal layout of pipe networks by many researchers. Dynamic programming methods always guarantee to achieve global optimal for the simplified problem (Walters and Lohbeckl, 1993). However, DP is limited to small and medium size networks because it needs a large amount of computer memory. Computation time for the analysis of branched layout problems using dynamic programming depends on the connectivity of nodes. In a highly connected network the number of states increases very quickly and as a result, dynamic programming could be inefficient for pipe layout problems with high connectivity.

Thus for large networks with a high degree of connectivity, an alternative technique is required. The genetic algorithm is a random search method which is efficient and powerful for pipe networks. Genetic algorithms have been used for the optimal layout of branched networks by a few researchers (Davidson and Goulter 1992; Walters and Lohbeck, 1993). Generally, in the design of branched pipe networks, two main aspects are taken into account. The selection of the layout and connectivity of the network, and the selection of pipe diameters. In the problems where layout is known, the flow in each link can be determined when the demands at the nodes are known. In such problems, the diameters of pipes are considered to be the decision variables for the optimum search. There are a number of methods which can be used to identify least cost solutions.



(a) A regular pipe network



(b) An irregular pipe network

Fig. 7.1 Typical examples of regular and irregular pipe networks

Dynamic programming, linear programming, non-linear programming and full enumeration techniques are the well known approaches for this type of problems. The problem of searching for the least cost pipe layout and connectivity is more complex. This is because the flows in links are not known in advance and there is no a simple relationship between flow and pipe cost.

The purpose of the part of the research reported in this chapter is to develop an optimisation model based on genetic algorithms to find an optimum layout for a branched network. This model can be modified and extended to full design of multiple subunit pressure irrigation (drip and sprinkle) systems. The model presented here deals with searching for the optimum links connecting demand nodes and the optimum size of pumping system in a regular branched piping system.

7.2 BASE GRAPH AND DIRECTED BASE GRAPH

Generally, water in a distribution network should be delivered to a number of demand nodes from one or more source node(s). The demand nodes are connected to the source node by a number of direct or indirect links. For example, as shown in Figure 7.2a in order to connect two demand nodes, Y and Z to the source node X, there are several different possibilities. One possibility is connection from X to Z, then from Z to Y (Figure 7.2c). Another is direct connections from X to both demand nodes, Z and Y (Figure 7.2d). The third alternative is connection from X to Y, then from Y to Z (Figure 7.2e). In addition to those 3 alternatives, which are carried out by direct links from the source node to the demand nodes, it is possible to use an intermediate node or junction which enables one to connect the demand nodes to the sources node indirectly, as shown in Figure 7.2f. Connection of the source node to the demand nodes considering all direct possibilities without specifying any direction as shown in Figure 7.2b, is called a base graph. This base graph contains all figures from 7.2c to 7.2e but not 7.2f. However, in reality, the direction of flow in branched networks is often known in advance. In such cases, the term directed base graph is used instead of base graph.

Implementing the directed base graph is helpful in order to reduce the size of search space or the number of possible solutions. In Figure 7.2a using the base graph for connecting the demand nodes to the source node (apart from introducing a junction) three different layouts could exist (Figures 7.2c, 7.2d and 7.2e). However, using a directed base graph (Figure 7.3a) the number of solutions will be reduced to two as shown in Figures 7.3b and 7.3c.

Another example of a pipe network with four demand nodes and one source node is presented in Figure 7.4. Since there are four demand nodes and each may receive water from two different links, the number of feasible solutions will equal 16. While the same example using the base graph considering the maximum possible links for each node may generate 72 solutions ($72 = 2^1 \times 3^2 \times 4^1$, one node with 2 possibilities, two nodes with three and one node with 4 possibilities). For a constant demand and given distance between nodes (flow rate at each node = 0.0027 m^3/s; distance between nodes in the X direction = 200 m; distance between nodes in the Y direction = 150 m) the optimum layouts resulting from the base graph is circled (Figures 7. 4f and 7.4j).

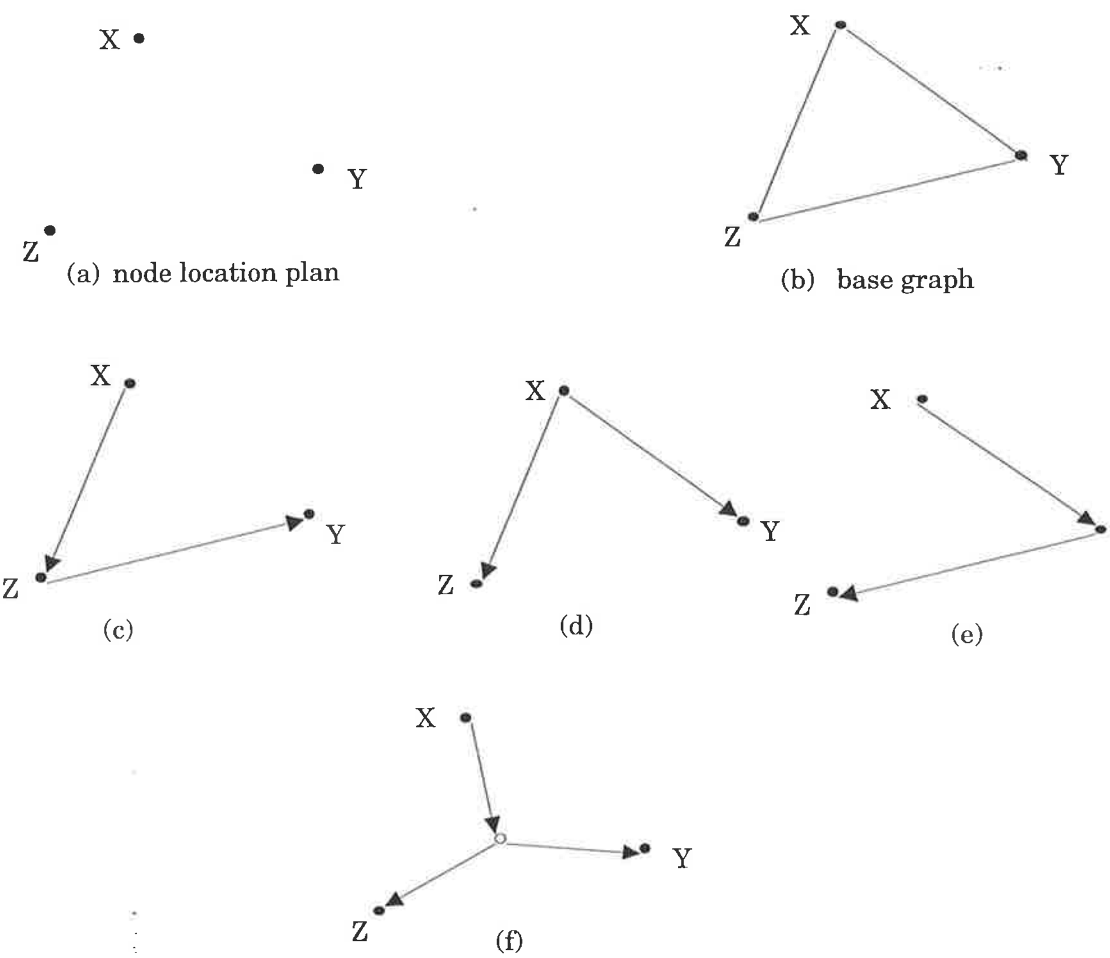


Fig. 7.2 A base graph with 2 demand nodes (y and z) and associated possible solutions (Walters and Lohbeck, 1993)

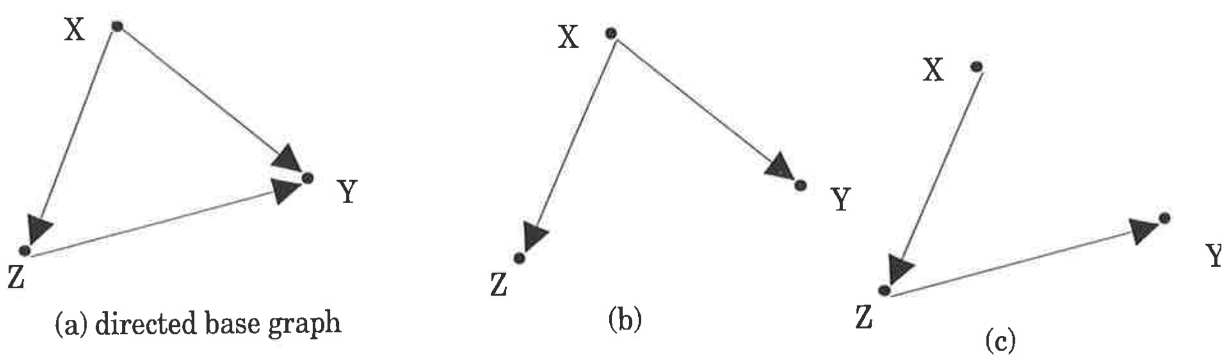


Fig. 7.3 A directed base graph with two demand nodes (y and z) and associated possible solutions

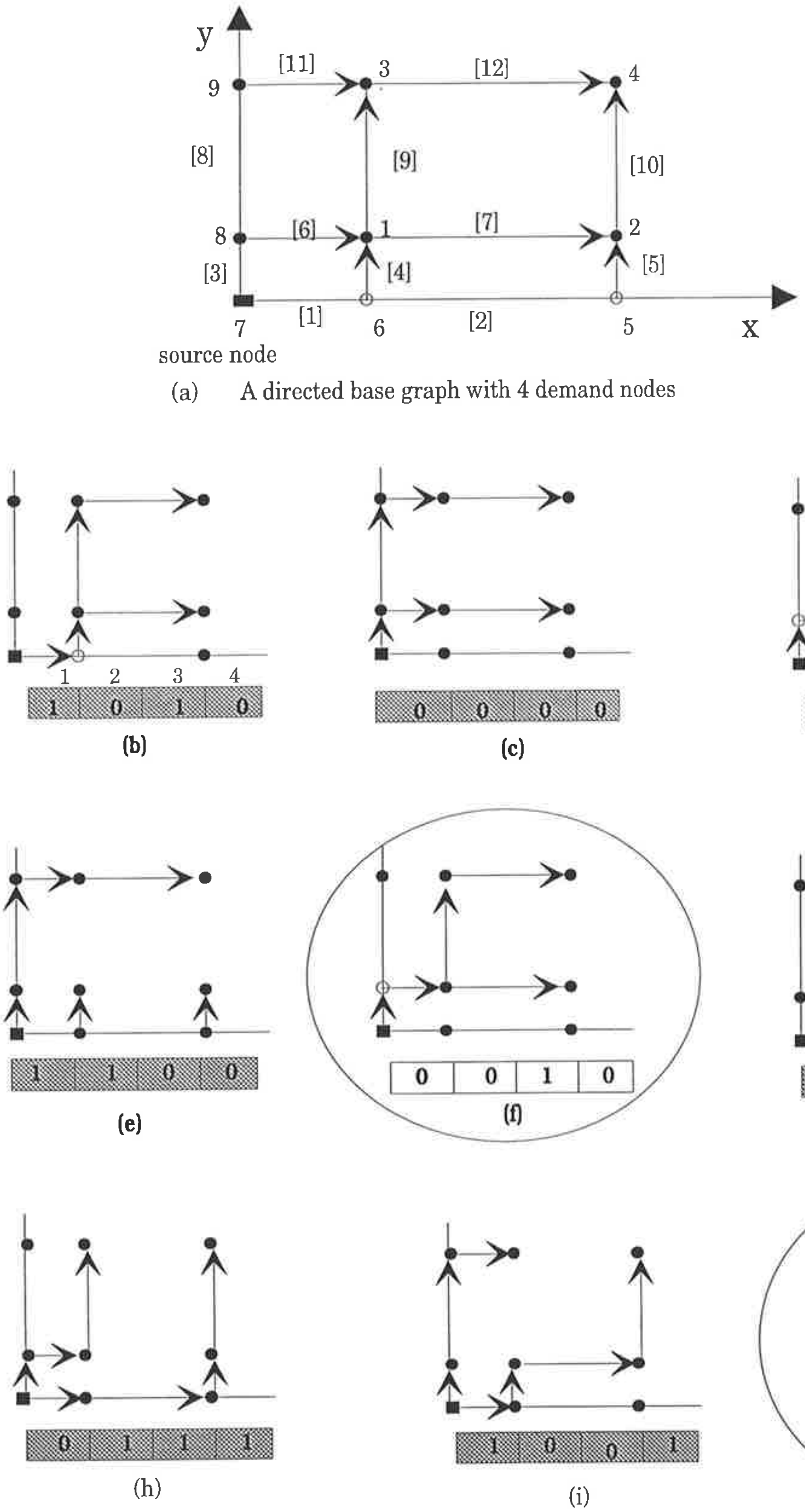


Fig. 7.4 A directed base graph and a number of associated solutions. (Optimum solutions are circled f and j)

7.3 MODEL AIMS

The main purpose was to develop a model to identify the least cost layout for branched water distribution networks. It was then applied to multiple subunit pressure irrigation systems with known demand at nodes. In a multiple subunit pressure irrigation system, the cost of main and submain pipes is one of the major components of the system cost. Initially, an optimum layout model was developed for a branched pipe network consisting of number of demand nodes (n) and a source node with a configuration shown in Figure 7.10. It then was extended to a network 4 times larger than the original network ($4n$) using properties of symmetry as shown in Figure 7.5. This causes less computer memory and time, since only the original network is considered in the GA process.

7.4 PIPING CONFIGURATION AND NETWORK COMPONENTS

It is assumed that the network consists of a number of demand nodes located in a rectangular pattern as shown in Figure 7.6. This represents the top right hand quadrant of the field shown in Figure 7.5 which is considered in the GA process. Each node potentially can be served by a maximum of two links which are at right angles to each other. The source node of the network is located at the bottom left hand corner of the grid. The distance between nodes and the position of the source node are known in advance. In addition to the demand nodes with a known demand a number of non-demand nodes (dummy nodes) are assumed in order to connect the main lines to the submain lines and also to ensure feasibility of the network. The non-demand nodes are located on the main lines connecting the source node to the other demand nodes. The main pipes are connected to the source node directly and deliver the flow to the submain pipes.

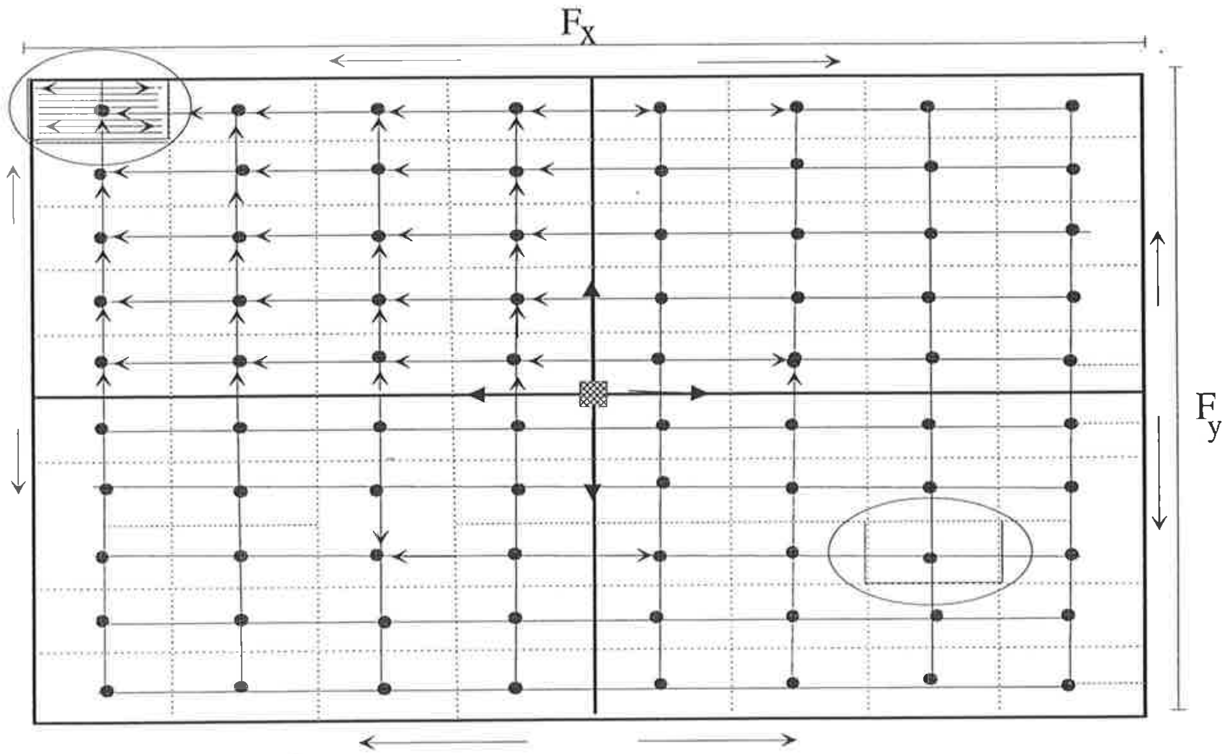


Fig. 7.5 A typical multiple subunit pressure irrigation system supplied from the centre (a subunit with micropipes is shown at top left)

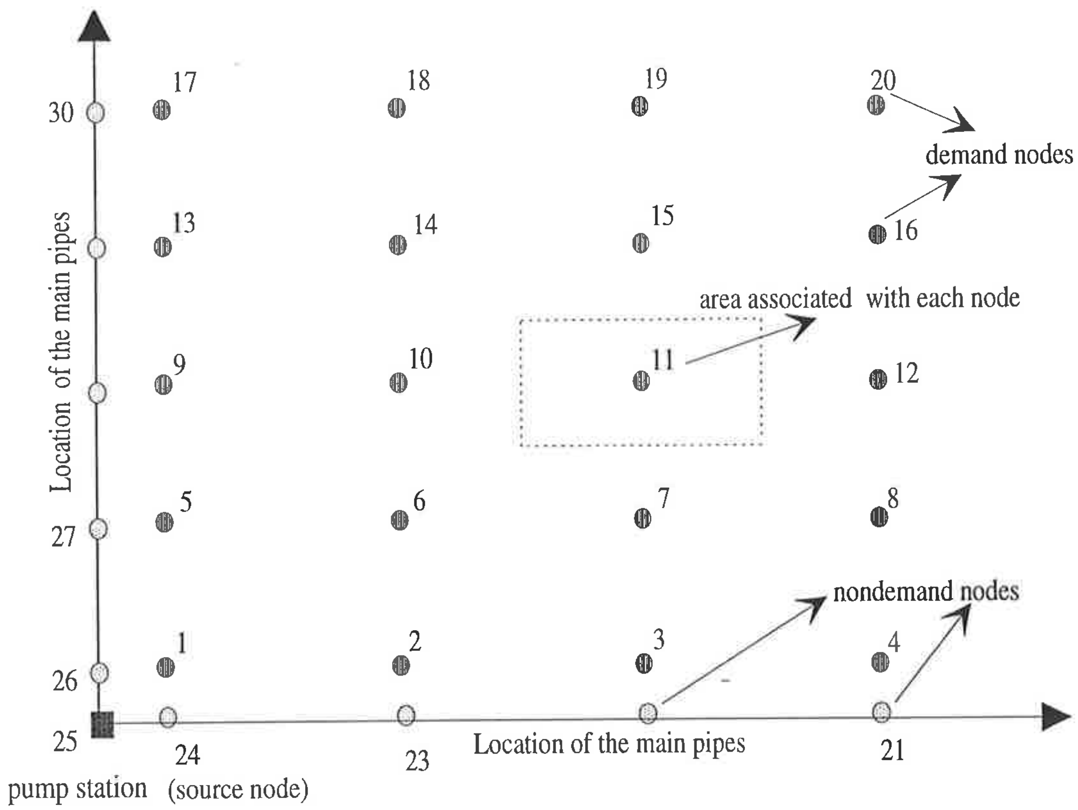


Fig. 7.6 A typical example of demand nodes, non-demand nodes and source node

7.5 GENETIC ALGORITHMS

As explained in more detail in Chapter 6, genetic algorithms (GAs) are search procedures based on the mechanics of natural genetics and natural selection (Goldberg, 1988). GAs attempt to simulate the near-optimal process of the evolution of living things. They are random search algorithms that start with a population of randomly selected feasible solutions. Each member of the subsequent generation is selected from the current one with a probability proportional to its function value or fitness. The highly fit individuals are given opportunities to reproduce by "cross breeding" with other individuals in the population. This produces new individuals as "offspring", which share some of the features of each parent (Beasley et al, 1993). Genetic algorithms achieve their relative efficiency from coding the decision parameters by adapting artificial chromosomes, rather than by adapting the parameters themselves (Goldberg, 1988).

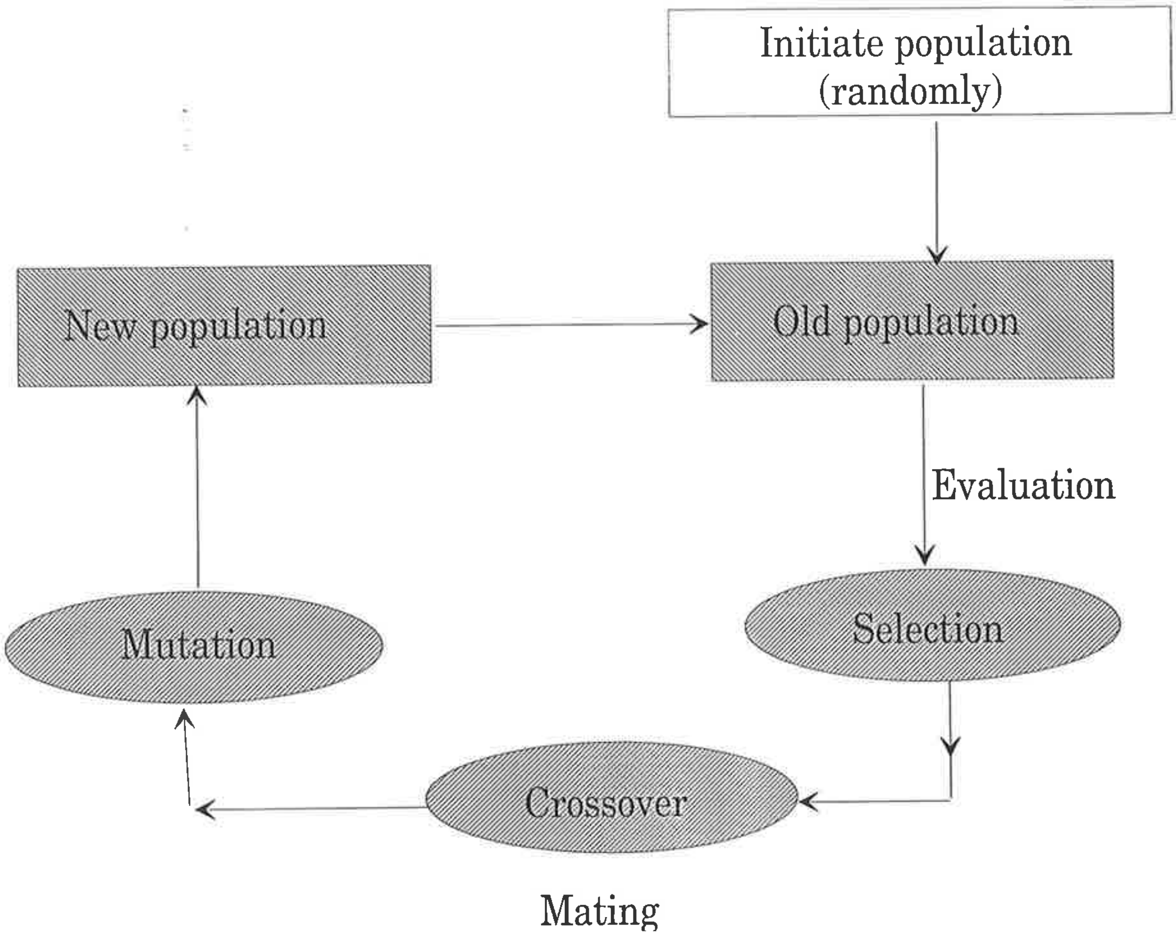


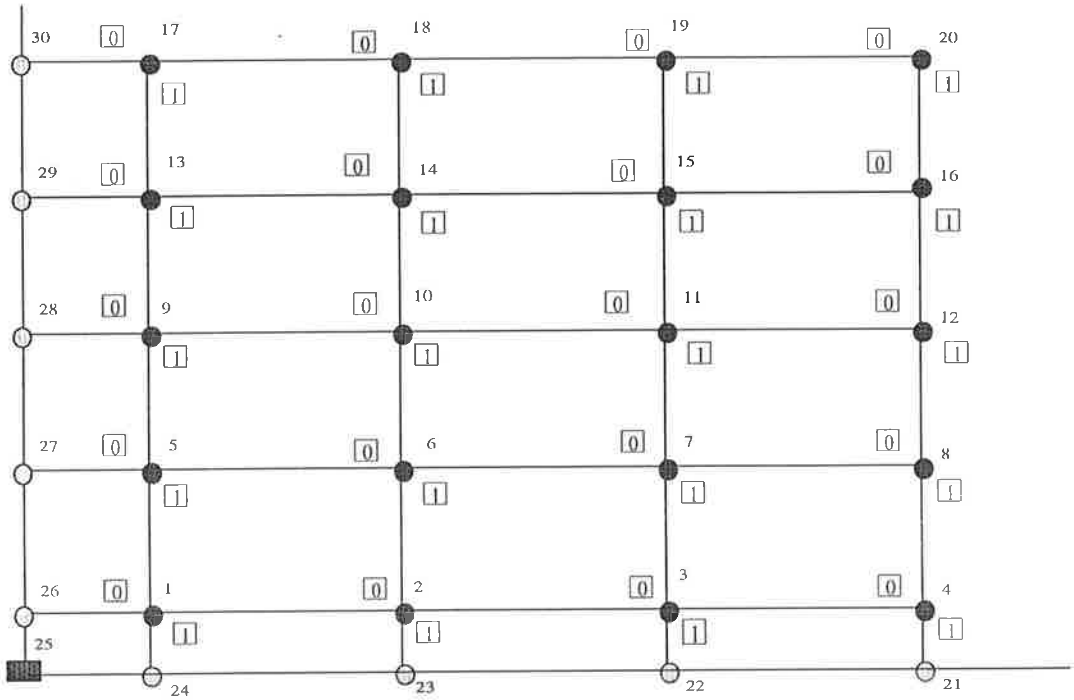
Fig. 7.7 The principle of the genetic algorithm process

In practice the possible solutions to the problems are coded as a finite-length string (often a bit string), and the genetic algorithm processes successive generations. GAs in a simple form have three basic operators including reproduction, crossover and mutation and perform as shown in Figure 7.7. These three basic GA operators were described in Chapter 6 comprehensively.

7.6 CODING FORMAT

In the problems formulated using genetic algorithms the decision variables describing trial solutions should be represented in a unique finite length coded string. The coded string representing a possible solution is similar to the chromosome in genetics. In the present model the links between demand nodes are considered as decision variables. Initially all nodes may be connected to adjacent nodes and the system looks like a fully looped system as shown in Figure 7.8. On the basis of their position, some nodes have two, some three and some have four adjacent nodes. Obviously, the number of potential links for each node should be equal to the number of adjacent nodes as indicated in Figure 7.9. The assumption that all nodes are connected to the adjacent nodes may create a looped network with a high degree of connectivity, as indicated in Figure 7.8.

However, it seems that the minimum cost design for a single loading case would be a branched network. In this model, the looped system is converted to a branched system. This conversion should be carried out in a manner such that the redundant links are removed and the feasibility of solutions is ensured. The development of a branched pipe network from an initial looped and high connectivity pipe network is performed by using a directed base graph. As shown in Figure 7.10, each demand node potentially can receive water from only two links. In order to provide a simple, efficient coding format for genetic algorithms to create initial solutions and also to represent the subsequent solutions in different generations, a binary format consisting "1's' and "0's" was defined. The length of each string is equal to the number of demand nodes. According to the format illustrated in Figure 7.11, "1's" indicate the existence of pipes in the y direction and "0's" indicate the existence of pipes in the x direction for each associated demand node. Each generated chromosome corresponds to one of the possible solutions.



Source node

Fig. 7.8 A pipe network with the possible links and associated binary formats

Thus a string consisting of a number of bits equal to the number of demand nodes is considered for each solution as shown in Figure 7.11. Immediately after the generating the initial population in the genetic algorithm process, a set of coded chromosomes each representing a particular solution of branched network is obtained. Each randomly created chromosome has a set of 1's and 0's as illustrated in Figure 7.11. In this figure each bit corresponds to a demand node and is specified by a binary number. In the created branched pipe network the links to nodes can be simply found from each generated chromosome. For example, if bit number 10 has been assigned a "0", it means that a pipe in the x direction delivers water to the demand node number 10. In the same way, if it is assigned a "1" it means that a pipe in the y direction delivers water to that node. The binary format corresponding to links for the all demand nodes is shown in Figure 7.8.

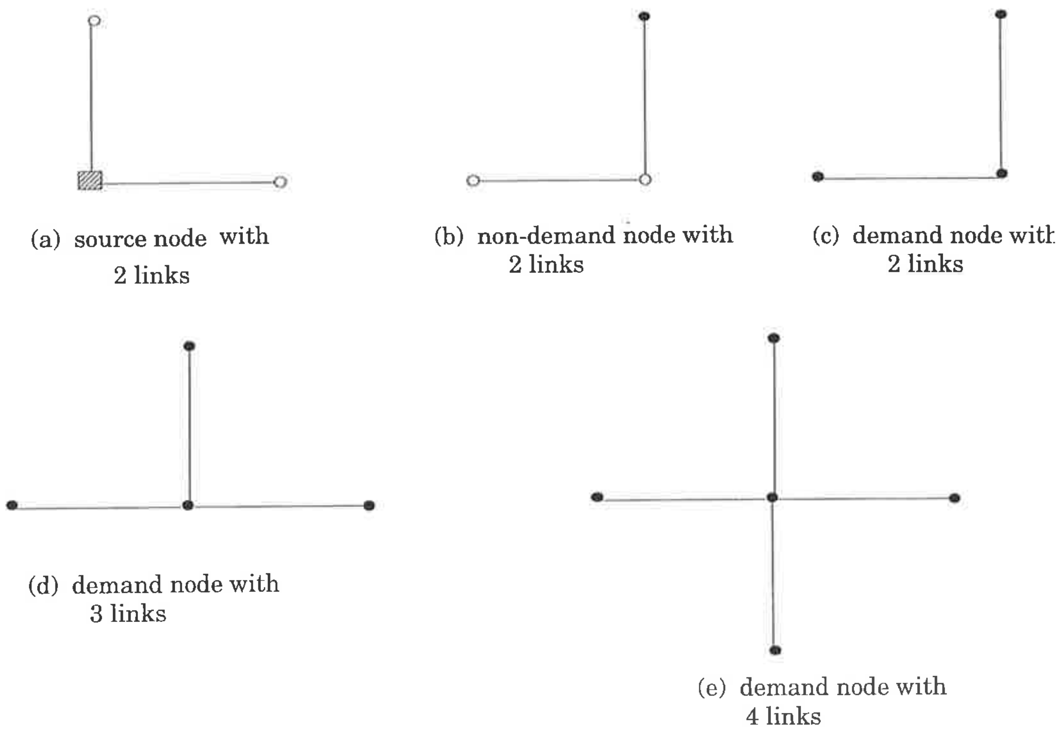


Fig. 7.9 A number of examples for connectivity of nodes before removing the redundant links

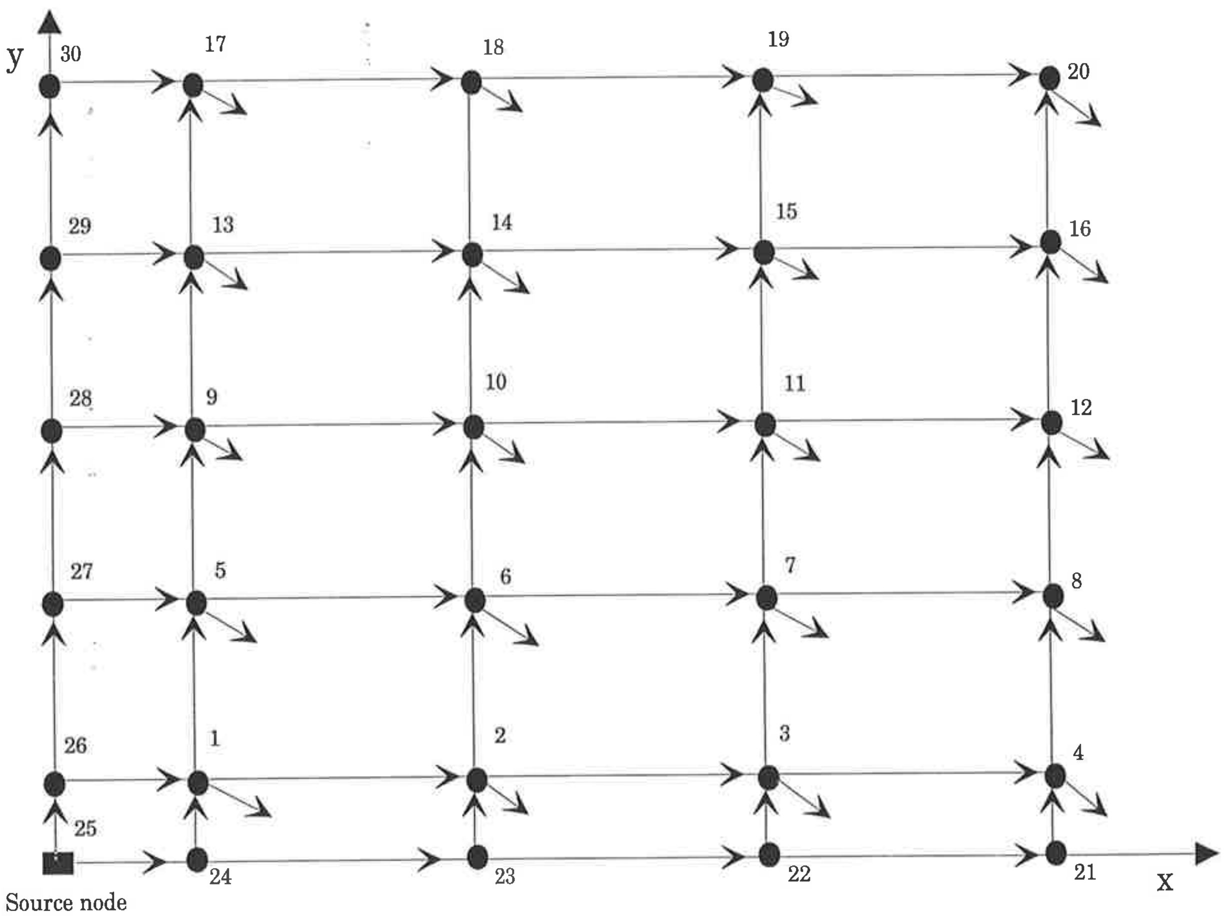


Fig. 7.10 A directed base graph with 20 demand nodes

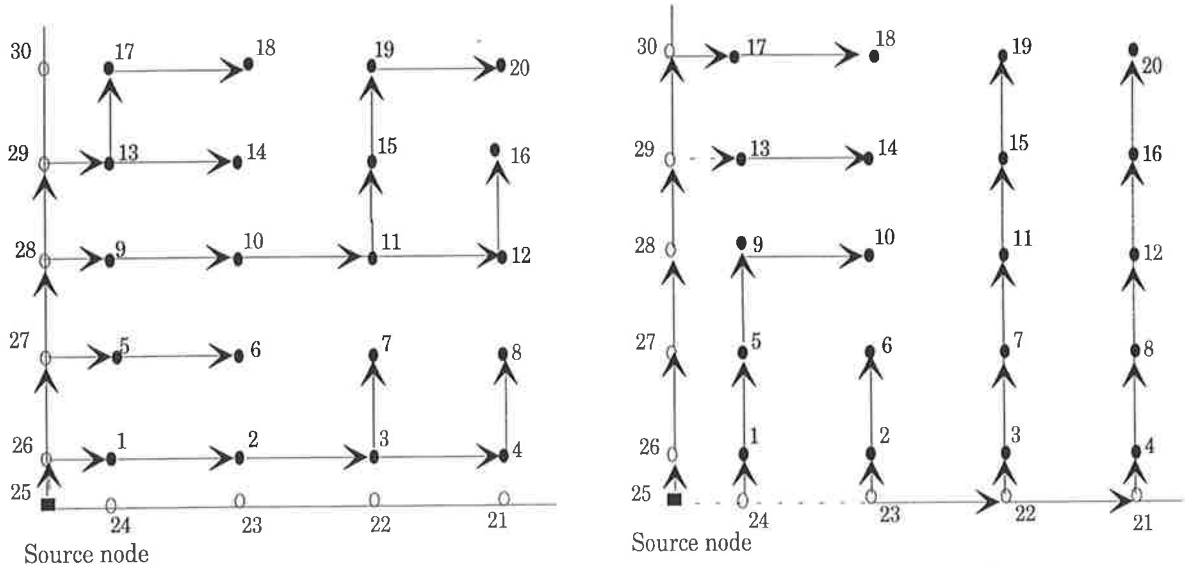
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	0	0	1	0	1	1	1	0	0	0	1	1	1	1	0	0	1	0	0

Fig. 7.11 A trial solution (chromosome) with binary coded

7.7 DETERMINING THE FEASIBILITY OF SYSTEMS

Since genetic algorithms use a random search to create the starting population of solutions and the genetic algorithm operators are essentially blind search techniques (Davidson and Goulter, 1992), some of the solutions created by GAs will be infeasible. The infeasible solutions should be specified and removed from the search space. In the optimisation of looped networks normally a penalty cost is defined. If a network does not satisfy the minimum pressure requirement, the maximum pressure deficit is multiplied by a penalty factor (Simpson et al, 1994). However, in this study the pumping system at the source is designed to ensure that the pressure at all nodes will not drop below a specified pressure. The major concern regarding feasibility in this model is to ensure that there is a continuous path from any node in the system to the source node. Otherwise some nodes will not receive water.

Typical examples of feasible and infeasible solutions in a branched network are presented in Figure 7.12. In the current model a simple method is considered for eliminating the infeasible solutions. As stated previously, in addition to the demand nodes some non demand nodes are also considered in the system. The non demand nodes are located on the paths where main lines may exist and their function is to connect the main pipes to the submain pipes.



(a) Feasible solution

(b) Infeasible solution

Fig. 7.12 Typical examples of feasible (a) and infeasible solutions (b) in a branched pipe network

The cost of each individual pipe depends on the associated flow in that pipe. If there is no flow in a pipe it means that sub-branch or pipe should not exist and no cost will be assigned to such pipes. In order to ensure feasibility, all pipes connecting the non demand nodes are assumed to exist. Those pipes with zero flow are then not included in the cost evaluations. In this way only feasible solutions will be obtained.

7.8 FLOW IN PIPES

In a branched network the flow in a link must equal the sum of the demands downstream. However, the connectivity in branched networks is not initially known, thus the flow of each link is unknown at the beginning the optimisation process (Walters and Lohbeck, 1993). Developing an efficient method for determining the flow in each link considering the demands at downstream nodes is essential in the least cost layout problems. In the current model a method has been developed based on backtracking movement to branch nodes. The details are discussed in Section 7.8.3. In this method each demand node has a known constant demand, and the non demand nodes (dummy nodes) have zero demand.

7.8.1 Connectivity

The connectivity of a network may be determined on the basis of generated chromosomes, each one representing a possible solution. In fact, the connectivity is defined as the existence of a link between two adjacent nodes and therefore is specified on the basis of the feasibility of the links. For instance, in one of the examples used in this study, connectivity of node 11 with two adjacent nodes 10 and 7, as shown in Figure 7.13, may be formulated as follows:

If node [11]=1 then

Connectivity [11,7] = link [26]

Connectivity [7,11] = link [26]

else (If node [11] = 0 then)

Connectivity [10,11] = link [30]

Connectivity [11,10] = link [30]

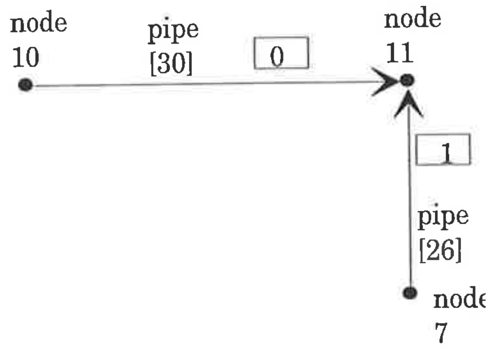


Fig. 7.13 A demand node and its possible associated directed links

The same procedure is applied to all demand nodes to determine the existence of links associated with each node. The flow chart determining the existence of links based on the "1" or "0" associated with node 11 as a typical node is presented in Figure 7.14. Clearly, the non demand nodes may only receive water from one node. Thus the link directed to a non-demand node would be the only connection between that node and the adjacent upstream node. In the process of determining the connectivity it is assumed that all connections between non-demand nodes exist.

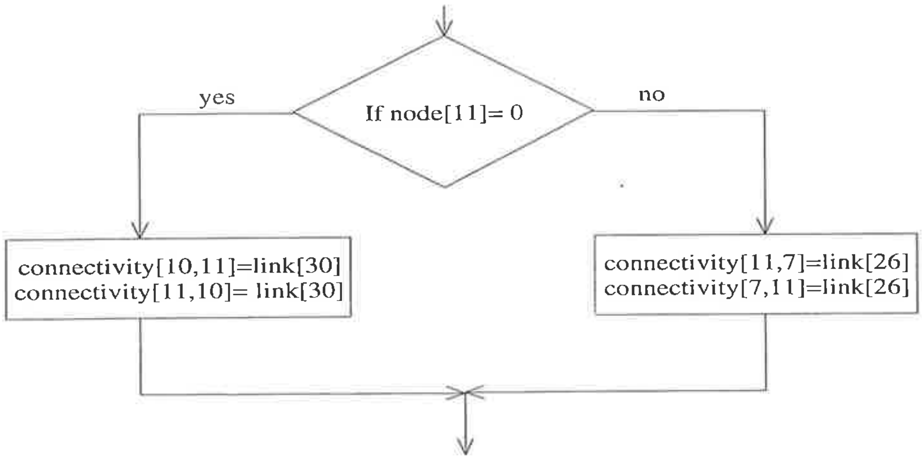


Fig. 7.14 The flow chart determining the connectivity of a demand node

Some of these pipes may have zero flow and not contributed to the total system cost. An example of the connectivity matrix for the network shown in Figure 7.4f is illustrated in Figure 7.16. To determine the connection between nodes it is essential to identify the degree of each node. This is discussed in the following section.

7.8.2 Determining the Degree of each Node

In the current model, the degree of a node is defined as the number of links directed to or directed away from that node as shown in Figure 7.17. A method based on the connectivity matrix has been developed for determining degree associated with each node. Considering the directed base graph shown in Figure 7.10, each node may be connected to 2, 3 or 4 adjacent nodes. After the performance of the connectivity procedure and before removing the redundant links, the system has maximum possible connectivity between nodes. Before the execution of the process leading to removal of the redundant links, the degree of all nodes is initialised to zero.

A connectivity matrix based on the number of nodes is then determined. The number of columns and rows of that matrix is equal to the number of nodes including the source node. However, after generating a chromosome which identifies a typical solution for a branched network the degree of nodes changes. This concept for the network shown in Figure 4a is demonstrated in two matrices illustrated in Figures 7.15 and 7.16. The number in the matrix corresponding to two nodes represents the pipe number that connects those two nodes (see Figure 7.4a). In a regular branched water distribution system only one link may be directed to a node and a maximum of 2 links directed away from that node. Thus in a regular branched system the degree of each node could be 1, 2 or 3. Some nodes corresponding to infeasible solutions may have no links to adjacent nodes. The algorithm setting up the connectivity matrix which determines the degree of each node is shown by a flow chart in Figure 7.18.

b \ a	1	2	3	4	5	6	7	8	9
1	0	7	9	0	0	4	0	6	0
2	7	0	0	10	5	0	0	0	0
3	9	0	0	12	0	0	0	0	11
4	0	10	12	0	0	0	0	0	0
5	0	5	0	0	0	2	0	0	0
6	4	0	0	0	2	0	1	0	0
7	0	0	0	0	0	1	0	3	0
8	6	0	0	0	0	0	3	0	8
9	0	0	11	0	0	0	0	8	0

Fig. 7.15 Connectivity matrix for a system with 4 demand nodes before the GA process, showing the connection between nodes (Fig. 7.4a)

b \ a	1	2	3	4	5	6	7	8	9
1	0	7	9	0	0	0	0	6	0
2	7	0	0	0	0	0	0	0	0
3	9	0	0	0	12	0	0	0	0
4	0	0	12	0	0	0	0	0	0
5	0	0	0	0	0	2	0	0	0
6	0	0	0	0	2	0	1	0	0
7	0	0	0	0	0	1	0	3	0
8	6	0	0	0	0	0	3	0	8
9	0	0	0	0	0	0	0	8	0

Fig. 7.16 Connectivity matrix for a system with 4 demand nodes after the GA process, showing the connection between nodes (Fig. 7.4f)

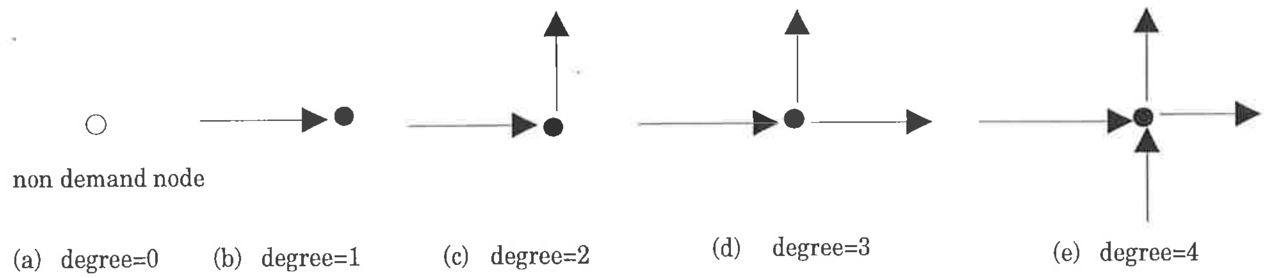


Fig. 7.17 Different nodes with different number of degrees

This algorithm checks the links connecting a node to all other adjacent nodes. If the link (connectivity) between that node and any adjacent node exists it then adds one degree to that node for each adjacent node. This process is repeated for all nodes within the connectivity matrix. The degree of connection of each node for the network shown in Figures 7.4a, 7.4f and matrices 7.15 and 7.16 is given in Table 7.1. The number of degrees for each node varies from 1 to 4. Table 7.1 represents the degree of connectivity of a looped and a branched network. Column three of Table 7.1 represents a regular branched network which could have a maximum of three at any one node. In a regular branched network a terminal node has only one adjacent link while all other nodes on a single path will have two adjacent links.

TABLE 7.1 Degree of connectivity for nodes in two networks before and after the GAs process

Node	degree Before the GAs process (Fig.7.4a)	degree After the GAs process (Fig.7.4f)
1	4	3
2	3	1
3	3	2
4	2	1
5	2	1
6	3	2
7	2	2
8	3	3
9	2	1

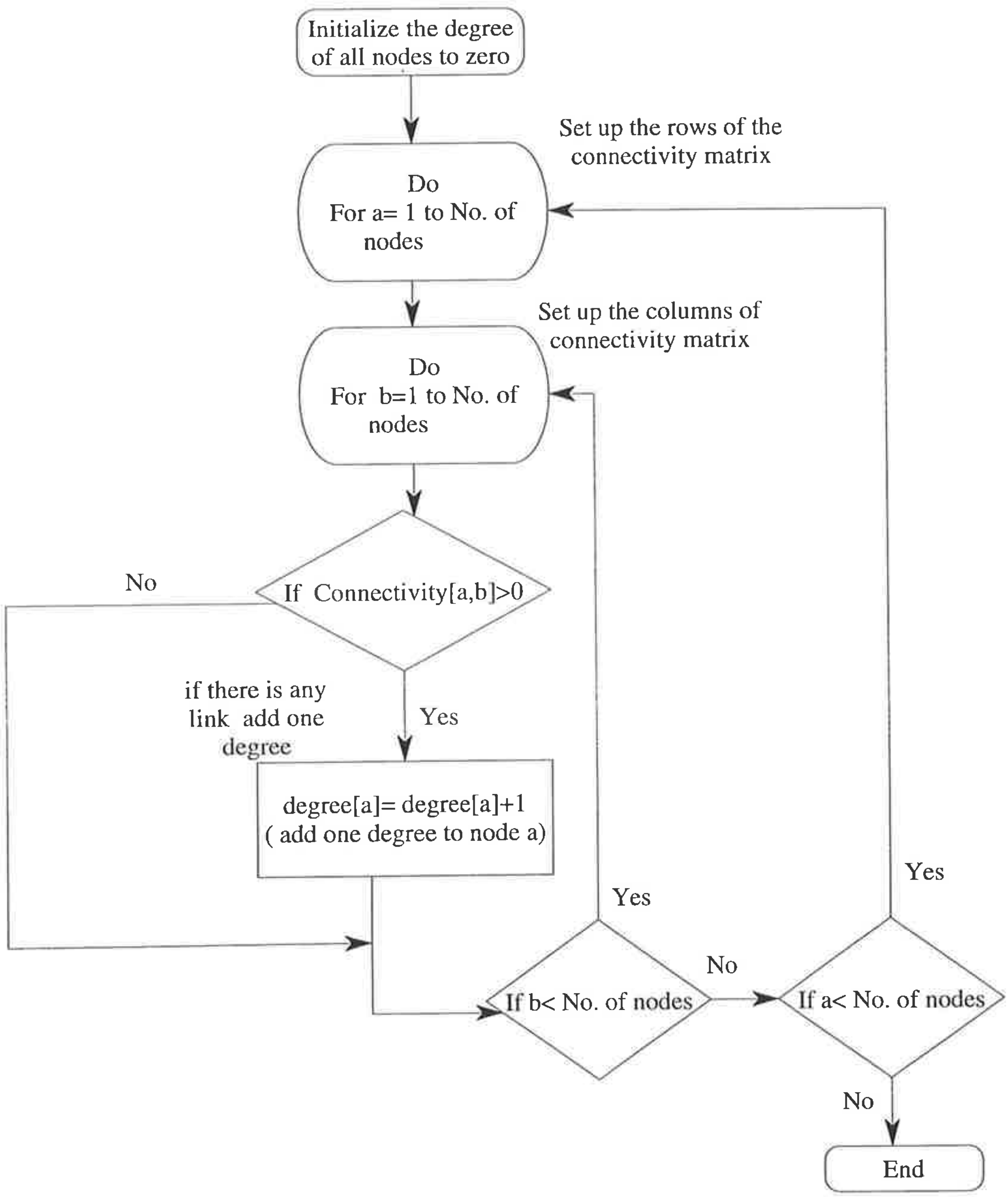


Fig. 7.18 Flow chart of determining the degree of nodes for connectivity matrix

7.8.3 Determining the Flow in Pipes

A method based on backtracking has been developed for identifying the flow in each pipe. The flow in all pipes is initially set equal to zero. As stated previously, a terminal node locating at the end of each single path has only one link or one degree. It should be noted that in a branched system only the terminal nodes have a degree of one. Determining the terminal nodes having a degree of one is the main aim of the algorithm developed for flow determination. The algorithm uses the connectivity matrix and the degree of all nodes and searches for terminal nodes. The search will be continued by checking all nodes until a node with a degree of one is found. At this stage the demand of the current node (with a degree of one) will be assigned to the corresponding link. The algorithm continues to find all other nodes with the degree of one to calculate the flow rate in the links associated to those nodes. It then assigns the demand of each terminal node to the associated links and then removes the link from that terminal node. In other words, only link between the terminal nodes and adjacent nodes is removed and the degree of the terminal nodes changes from one to zero. Then the demand at each terminal node is added to the demand at the adjacent node. In addition, the degree of this adjacent node is decreased by one. In this way the new terminal nodes with a degree of one are generated, while the previous terminal nodes with a degree of one are removed. In fact, all nodes with a degree of one will now be the target of the algorithm. The process described above is repeated to add the demand of the current terminal node to the adjacent nodes, reducing their degree to zero and also taking one degree from the adjacent nodes. At the end of the above process, the accumulated flow, which is sum of all the demands at nodes is assigned to the source node. At this stage the flow in all existing pipes as well as the flow at the source node has been found. The flow chart concerning this process is illustrated in Figure 7.19.

7.9 HEAD LOSS IN PIPES

Although in the current optimum layout model, the cost of pipes is formulated in terms of the flow in the pipes, the cost of the pumping system is affected by the total head loss in the system. The head loss in a pipe can be computed using a number of empirical equations. Two commonly used equations are the Hazen-Williams and the Darcy-Wiesbach equations. The diameter of the pipes is one of the essential parameters in calculating the pipe head loss. However, in the current study the diameters of pipes are not known. Although in most least cost layout problems the diameter of pipes could be assumed as known parameters, in this part of research outlined here the system cost is modelled independent of the pipe diameters. As a result, the head loss cannot be calculated from the commonly used equations. Davidson and Goulter (1992) and Walters and Lohbeck (1993) use a different approach for their optimisation branched layout models. They don't consider the head loss in pipes. More details regarding their approach are given in Section 7.11.

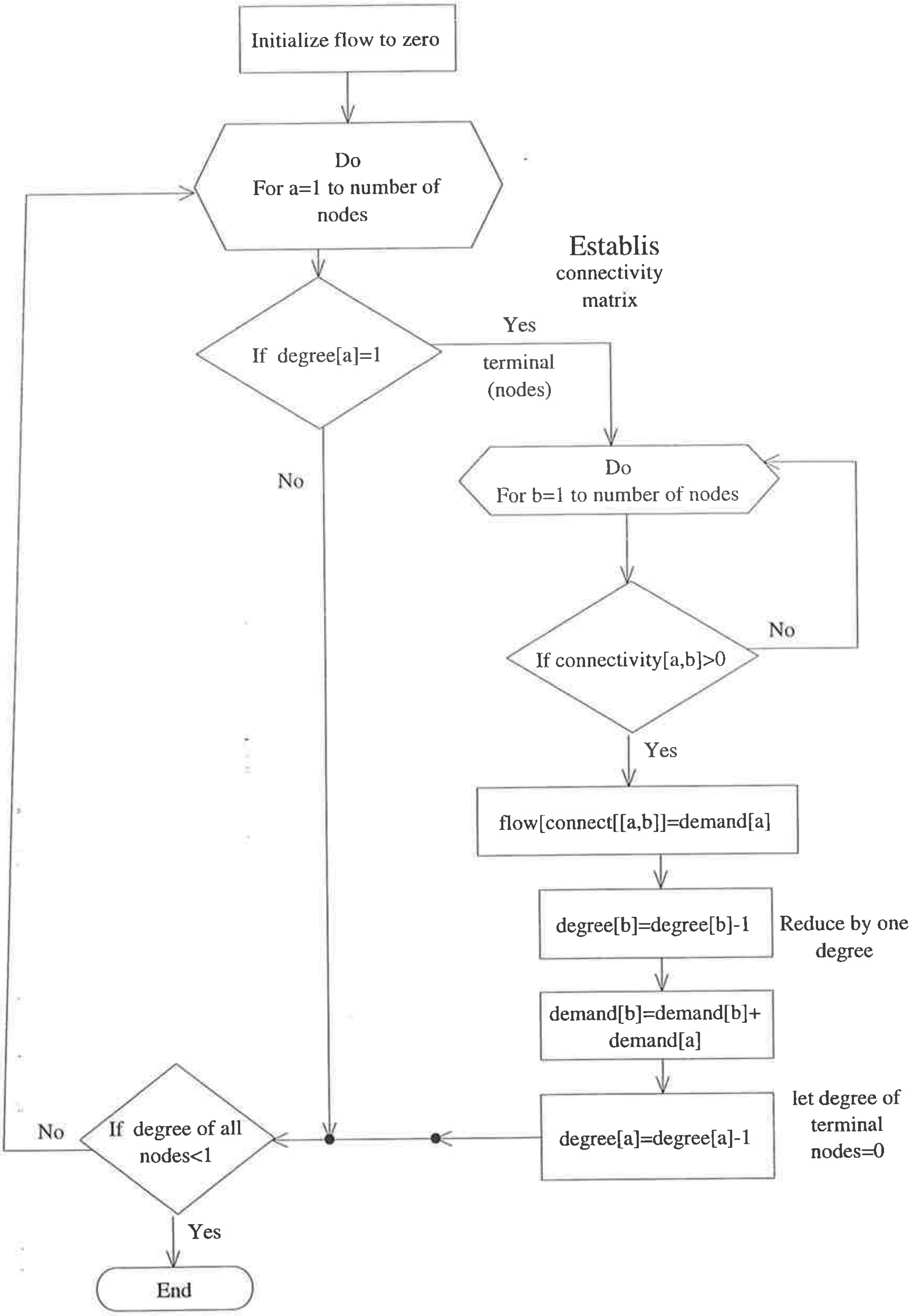
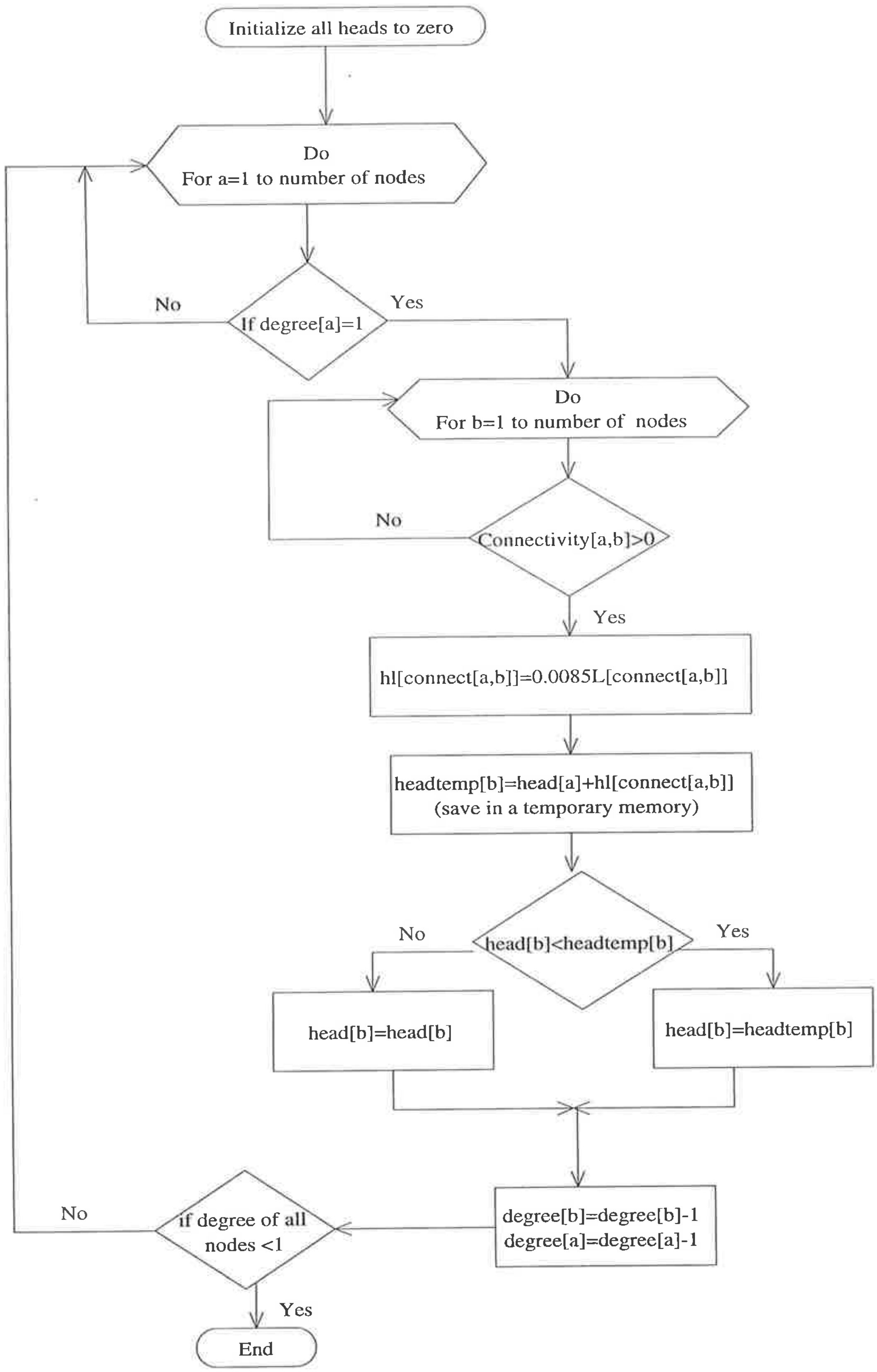


Fig. 7.19 Flow chart of backtracking method for computing the flow in pipes for a branched pipe network

In the current model the pumping system has also been considered. Consequently, the head loss affected by the flow in each pipe is needed to determine the pump size and pump cost. In order to overcome this problem a reasonably constant head loss in pipes is assumed. This assumption is based on the optimisation model developed for a multiple subunit pressure irrigation system in Chapter 5. In fact, the head loss resulted from one of the optimum solutions with the full enumeration technique discussed in Chapter 5 was considered as a constant head loss per unit length of pipes for this model.

7.10 HEAD AT NODES AND PUMPING SYSTEM

Examining the head at nodes in any pipe network is one of the essential stages in the analysis of the piping systems. The pressure on nodes should not be allowed to drop below the minimum required pressure and also should not exceed a specified level. The design pressure head at each node in a branched network consists of two components: the minimum required pressure at each node to allow the system to operate properly, and the pressure to compensate for the head loss in pipes downstream. Determining the head at each node is also carried out using the backtracking method in a similar fashion to the method discussed in Section 7.8 for flow determination. The head at a node, in contrast to the flow, which is sum of the demands in all nodes downstream, is determined considering the path or sub-branch with the highest accumulated head loss. This may result from pipes downstream where two or more sub branches connect to a single node. In the case of a single downstream path the head at a node is the accumulated head loss of pipes downstream. Obviously, design head at the source node should be the sum of the head loss of pipes connecting all nodes along the longest path plus the minimum required pressure at the extreme node for the system operation (assuming the minimum required pressure at all nods is the same). The flow chart of the algorithm for determining the design head at nodes based on the connectivity matrix discussed earlier is shown in Figure 7.20. The coefficient value of 0.0085 in the head loss computation is in fact the head loss per unit length of pipes which is taken from the model discussed in Chapter 5 for an optimum solution.



$L[connect[a,b]]=$ length of pipe between nodes a and b

Fig. 7.20 Flow chart for determining the head at nodes

7.11 FORMULATION OF COST EQUATIONS

The selection of the least cost tree layout from the directed base graph is a complex problem (Walters and Lohbeck, 1993). Two reasons for this complexity have been stated. One reason is due to the unknown connectivity of the network initially, which leads to unknown capacities of the links at the start of the optimisation process. The second reason for this complexity may be due to the fact that there is not a simple relationship between capacity and cost for a link. Generally, in a pressurised pipe network, the cost of links depends on the pipe diameter, the pipe length and sometimes on the internal pressure. Walters and Lohbeck (1993) assumed a constant head loss per unit length of pipe and used the following equation for their branched layout optimisation model.

$$C = KLQ^{\alpha} \qquad 0<\alpha<1 \qquad (7.1)$$

where C = total cost of pipe (\$); K = a constant; L = length of pipe (m); Q = discharge in pipe (m^3 /s).

They believe that this cost equation models the real situation in which the cost per unit capacity (discharge) of a pipe decreases as the capacity increases. Davidson and Goulter (1992) assumed that the cost of pipes depends only on the length of pipes. This is not a good assumption since the cost of pipes is also greatly affected by the pipe diameter particularly for the larger sizes. However, a new approach was developed for the formulation of the cost equation in the current optimisation layout model which is described in the next section.

7.11.1 Pipe Cost

In addition to a constant head loss per unit length, the Hazen-Williams pipe roughness (CH) is also assumed to be constant for all pipes. In the given example the head loss per unit length (0.0085m per m length) and the pipe roughness, ($CH =$ 150) are given. Considering these two parameters, the diameter of each pipe may be obtained using the Hazen-Williams equation as follows:

$$hl = \frac{10.68LQ^{1.852}}{CH^{1.852}D^{4.87}} \tag{7.2}$$

where hl l= head loss in pipe (m); D= diameter of pipe (m); CH= Hazen-Williams pipe roughness coefficient.

From Equation 7.2 pipe diameter (D) may be formulated as:

$$D = \left(\frac{10.68LQ^{1.852}}{CH^{1.852}hl}\right)^{0.2053} \tag{7.3}$$

Substituting the given values for the pipe head loss and the pipe roughness coefficient in Equation 7.3, the following equation representing the diameter of pipe in terms of discharge is obtained:

$$D = 0.644Q^{0.38} \tag{7.4}$$

An equation was developed for the cost of PVC pipes in terms of diameter using a non linear concave expression suggested by Oron and Karmeli (1979). The details are given in Chapters 3, Section 3.5. Considering a cost of installation per unit length (ϕ), the final equation showing the cost per unit length of pipe may be expressed as follows:

$$C = 960D^2 + 6D + (\phi + 0.18) \tag{7.5}$$

where C = the cost of a unit length of pipe with diameter of D, (\$/m); ϕ = the cost of installation per unit length of pipe (\$/m).

The diameter expressed in terms of flow (Equation 7.4) may be substituted in Equation 7.5. As a result, the final cost equation in terms of discharge in pipes may be formulated as follows:

$$C_p = L(400Q^{0.76} + 3.87Q^{0.38} + \phi + 0.18 \qquad (7.6)$$

where C_p is the total cost of pipe with the length of L (\$).

The above modified cost equation suggests that the pipe cost is a concave function of its capacity. In other words as the capacity in a delivery pipe increases the cost of delivery per unit flow decreases.

7.11.2 Pump Cost

In the current optimal branched layout model a pumping system has also been considered in the optimisation process. As stated earlier the head loss in the pipes is assumed to be constant 0.85 m per 100 m of length and the cost of the pumping system is a function of the power required to operate the system (Holzapfel et al, 1990). On the other hand, the power of a pump is a function of the discharge and the head which is in turn, affected by the head loss. A cost equation for a submersible pump with an electric motor was developed on the basis of the general pump equation suggested by Holzapfel et al (1990). Regression analysis was used to fit a function using a list of pump cost data issued by Southern Cross. More explanation is given in Chapter 4, Section 4.8.5. The final equation was formulated as below:

$$C_{pu} = 1000Q_{pu}^{0.2305}H_{pu}^{0.9038} \qquad (7.7)$$

Obviously, the total flow is the sum of the demands on all demand nodes within the system. While the total head is the sum of the head loss of pipes along the longest path of the branched system plus a minimum required pressure at the extreme nodes. In the backtracking method the source node is the last node considered. Its head is sum of the head losses in pipes along the longest path plus the operating head on the extreme node. As Equation 7.7 indicates the pump cost in this model is greatly affected by the pump head. As the head decreases the pump cost will decrease as well. On the other hand, the pump head is a function of the length of pipes located along the longest path. Although the pipe cost is a function of flow rate and length, it is most likely that the optimum solution might be the solution with the shortest length of pipes. Total system cost is the sum of the total pipe cost and the pumping cost which is used to calculate the fitness of each solution created by the genetic algorithm procedure.

7.12 EXTENSION OF THE MODEL

As stated earlier, the model deals with the optimum layout of branched pipe networks where the source node is located on one corner of the demand nodes as shown in Figures 7.4 and 7.10. It may be extended to other types of networks where the source node is located at the centre of the block of land. This should enable it to be applied to many different networks that may exist in reality. In the current model, the source node may be located at any corner of a rectangular shaped field. As a result, the model applied to examples shown in Figures 7.4 and 7.10 may be applied to four identical networks supplied from one corner of the field. If all four networks share a common source node, the system then will be the same as the another type of multiple subunit system as shown in Figure 7.5.

In the irrigation systems the demand at each node depends on the area allocated to each subunit, which is greatly affected by the irrigation requirements. The irrigation requirement in turn depend on the plant consumptive use, soil type and the irrigation method. In this way, the demands at the nodes represent the irrigation requirements of subunits associated with those nodes. The details of irrigation requirements on the basis of the agronomical and agro-technical aspects are given in Chapter 5 and Chapter 8. In this model, a reasonable demand for each node has been considered and only the cost of main and submain delivery pipes are taken into account. In the model outlined in Chapter 8, the cost of subunits considering micropipes will be discussed.

The modified model with four identical sets of subunits will not have any new constraints compared to the original one. As indicated in Figure 7.5, the modified model deals with networks with more demand nodes. Strictly speaking, it covers an area four times larger than the original one. It may also be applied to networks with a number of demand nodes four times larger than the original model. The interesting point regarding the modified model is that the evaluation process will not be more complicated than the original model. Since only the original nodes are evaluated by the GA. As a results, the required computer time and computer memory will be almost the same as for the original model. In the modified model, if irrigation is to be carried out simultaneously for all nodes, the main lines then may be common in two symmetrical set of subunits located either side of those main lines. This will mean that some changes to the original model are necessary.

7.12.1 Required Changes for the Modified Model

Changes required for the modified model relate to computing the flow rate in pipes and nodes. These changes should enable the model to compute the new flow rate and head loss in the main line pipes which deliver water to the symmetrical set of subunits. However, the head loss was assumed to be constant per unit length of pipe in this study, therefore only the flow plays an important role. To determine the total flow delivered by each main line pipe, it is essential to identify which submain pipe is connected to the mainline pipe in the both X and Y directions in the original model. When the model identified all submain pipes connected to the mainline pipes (two mainlines in original model) the flow then associated with each submain line is doubled. This is due to the fact that each mainline pipe feeds the same number of nodes from either side. In this way not only the cost of main line pipes which share a common flow, but also the total required flow from the pump is computed. As mentioned previously, the required changes may be necessary when the whole field is to be irrigated simultaneously. Nevertheless, in practice, only one or a number of subunits may be irrigated at each time. In this case, if each set of subunits which were covered by the original model are irrigated simultaneously, no changes will be needed. Only the cost of whole system should be considered

7.13 RESULTS AND DISCUSSION

In order to verify the model, it was applied to two examples consisting of 4 and 20 demand nodes in a grid network located in a rectangular pattern as shown in Figures 7.4 and 7.10. A constant demand at each demand node was determined on the basis of irrigation requirements from a previous study. As explained previously, the model was developed for a case where the nodes are located in a rectangular pattern. Any rectangular pattern branched system with a known demand at nodes may be solved with the current model.

Although the main purpose was to find out the optimum layout of a branched pipe network and it could be solved without considering the pumping system (as considered by Davidson and Goulter, 1992 and Walter and Lohbeck, 1993) in this case a pumping system was considered at the source node. Selecting a reasonable head loss per unit length is very important in such optimisation problems. It not only affects the size of pump but also is directly proportional to the flow and inversely proportional to the size of pipes. In the current model the head loss was selected on the basis of the previous study described in Chapter 5. This will ensure that the assumption of head loss might be more realistic and more consistent with the flow rates and the size of pipes.

7.13.1 The Use of Genetic Algorithms in the Model

The model was developed using the genetic algorithm approach. All links directed to each demand node were chosen as decision variables, and each decision variable was represented by a binary number. In the final design each demand node should have one link from two possible links to be supplied. Consequently, the length of each string was equal to the number of demand nodes in the original network. In this way, each possible solution or complete design was represented by a binary string as shown in Figure 7.11.

The optimisation process in GAs starts by generating a number of initial random solutions. The number of initial solutions and the number of subsequent solutions in each generation is defined as the size of the population. The cost of each solution was computed using Equations 7.6 and 7.7. Two different selection methods: proportionate and tournament were used to select the solutions with a higher fitness. The fitness of each solution was computed using an inverse function of total system cost as formulated in Equation 7.8.

$$Fitness = \frac{1}{Cost} \tag{7.8}$$

Generally, in the optimisation of pipe networks the aim is to achieve the minimum cost design while satisfying a number of constraints. Therefore, the solutions with the highest fitness, representing the lowest cost, are the main target. According to Darwin's survival-of-the-fittest philosophy, the fittest and strongest chromosomes reproduce in greater numbers in the next generation. In the tournament selection method (with a tournament of size two) two chromosomes of the current generation are picked at randomly, and the chromosome with the larger fitness is selected to be copied for the next generation. Another two chromosomes are then picked randomly and the one with the larger fitness is selected. The same process is repeated to select $\frac{n}{2}$ chromosomes of good fitness (n = size of population). This process is then repeated starting with the full population of the current generation. Thus a new population of n chromosomes is developed. In proportionate selection, members of the next generation are randomly selected in proportion to their fitness relative to the fitness of all the other members of the current generation (Simpson and Goldberg, 1994). After implementing the selection operator, the next operator of the GA is applied. In this stage the chromosomes are paired randomly. Each pair of selected chromosomes are then crossed over to generate new chromosomes on the basis of a particular probability of crossover as shown in Figure 6.5. It is expected that the children produced from parents after crossover will have larger fitness on average for the whole population. A Genetic Algorithm achieves the most success with high a probability of crossover such as 0.7 to 0.9 (Goldberg, 1987). Walters and Lohbeck (1993) applied a probability of 1.0 for their optimum branched pipe network model. In this model a probability of 0.8 was used.

Mutation is the last operator of the GA. It is applied with a low probability to every bit of each chromosome. If mutation occurs a "0" is changed to a "1" or a "1" is changed to a "0" in binary format. Too high a probability of mutation may cause the loss of good solutions in the process of random search. On the other hand using too low probability may provide a premature convergence on a local optimum. The children produced in this process become members of a new generation of solutions, and the complete process including reproduction, crossover and mutation is repeated for all members of population until a new generation has been created. The creation of a new generation from the old generation by applying the three GA operators is continued until the improvement in the fittest solution of each generation is negligible.

7.13.2 Case Study

The model was tested on two examples with 16 and 80 demand nodes. The first example is a simple rectangular pattern network with 16 demand nodes as shown in Figure 7.21. The genetic algorithm parameters for this example were chosen as follows:

probability of crossover	Pc = 0.8
probability of mutation	Pm = 0.02
population size	n =100

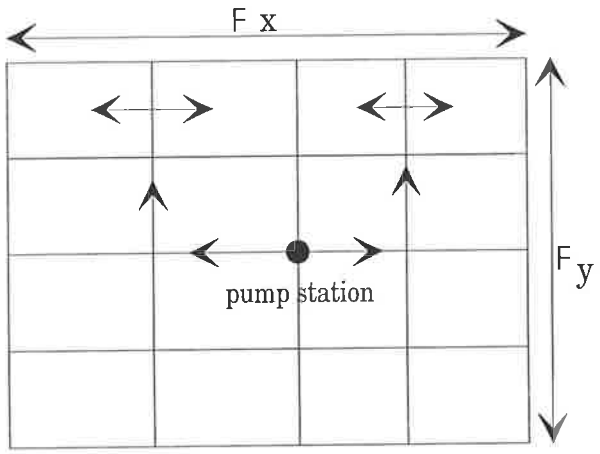


Fig. 7.21 A typical network with 16 demand nodes (resulted from extension of Fig. 7.4a)

The network parameters were given as follows:

Discharge at demand nodes	0.0027 m^3/s
Distance between demand nodes in the X direction	200 m
Distance between demand nodes in the Y direction	150 m
Distance between demand and non demand nodes in the X direction	100 m
Distance between demand and non demand nodes in the Y direction	75 m

Although the model deals with a network of 16 demand nodes only one quarter of demand nodes were participated in the GAs process. Consequently, the number of possible solutions considering 4 demand nodes each with two possible links was small ($2^4 = 16$). Therefore, in the first generation the minimum cost was obtained. The GA was allowed to run with proportionate and tournament selections. In both selection methods the best solution (least cost) was obtained in the first generation and the minimum cost remained constant in all tested generations using two different selection methods. Figures 7.22 and 7.23 show the variation of maximum, average and least cost of solutions in each generation. The optimum multiple subunit pressure irrigation system of 16 demand nodes is shown in Figures 7.24. This was obtained at a cost of $38,360. Finding the minimum cost solution in the first generation shows that using a small population size for the small problems with a narrow search space as tested in this example is adequate.

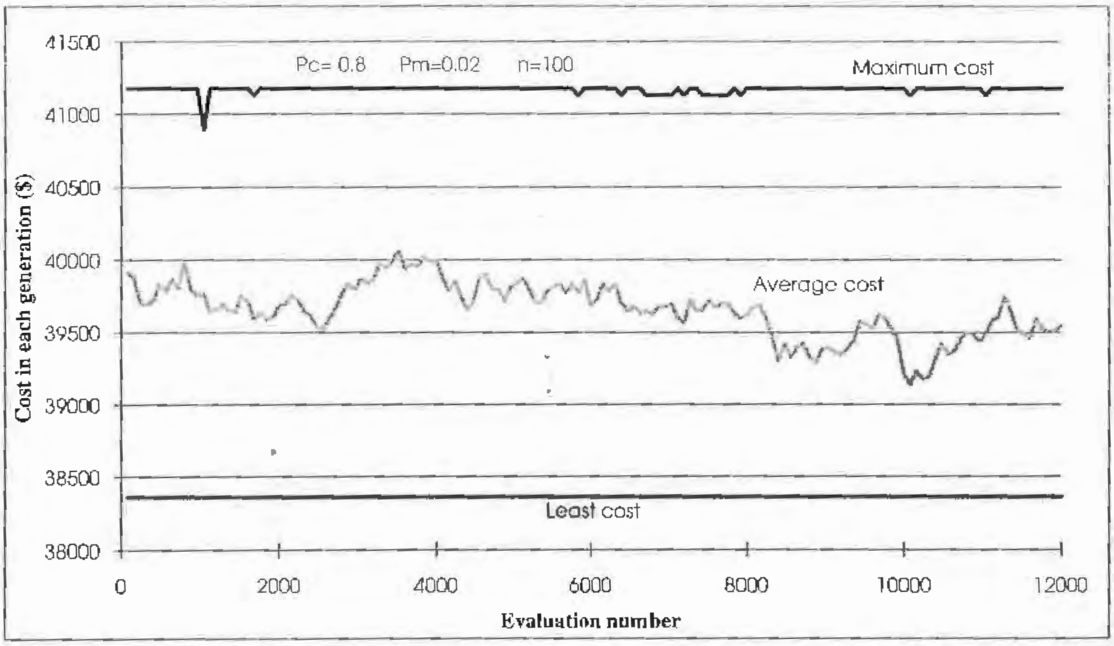


Fig. 7.22 The least, average and maximum cost in each generation for a multiple subunit system with 16 demand nodes (proportionate selection)

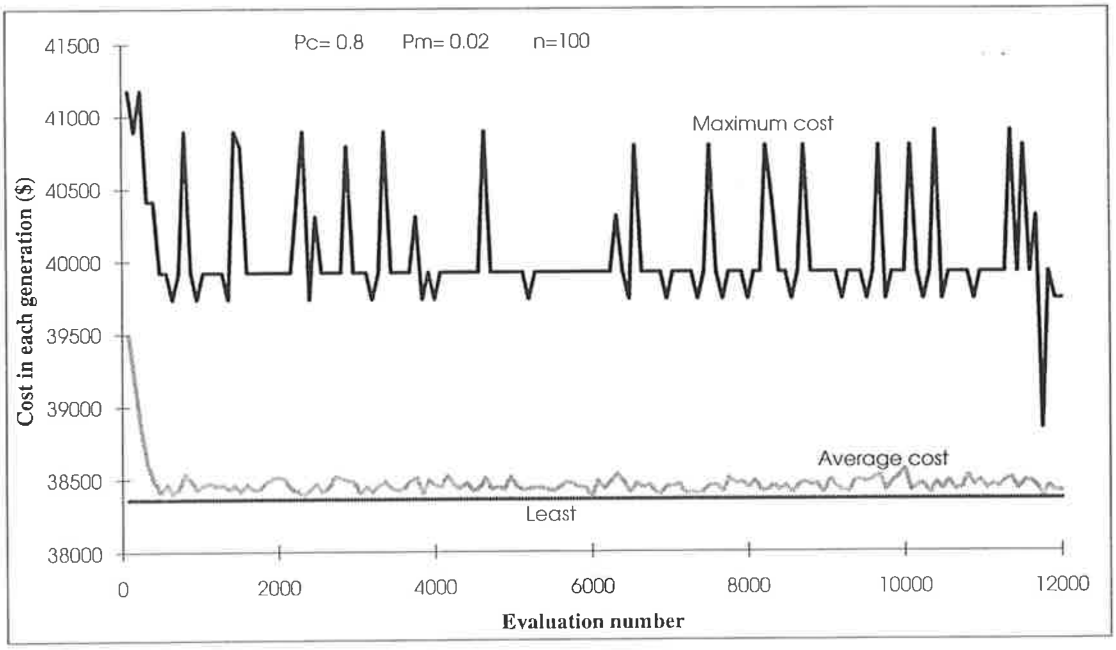


Fig. 7.23 The least , average and maximum cost in each generation for a multiple subunit system with 16 demand nodes (Tournament selection)

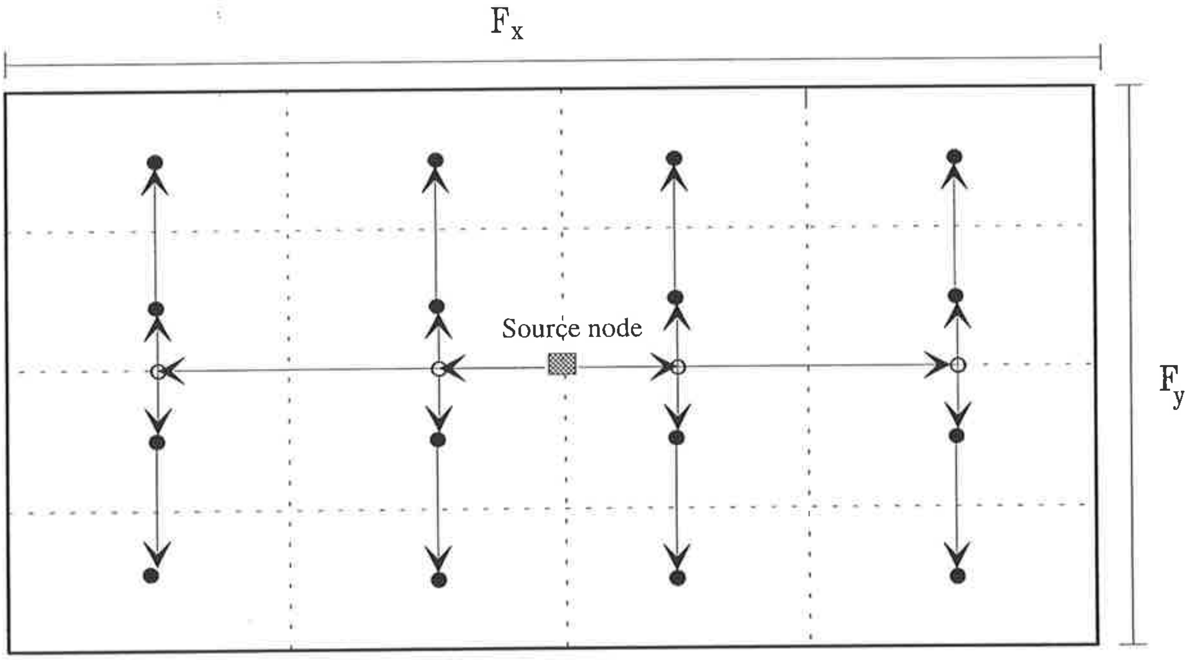


Fig. 7.24 The optimum solution (layout) of a multiple subunit with 16 subunits

The second example is a network with 80 demand nodes shown in Figure 7.5. As stated previously, when the modified model is tested on a network with n demand nodes, in fact, it is tested on a network of $4n$ demand nodes with a symmetrical configuration. In this case, the original network had 20 demand nodes (Figure 7.10) while the final network which was analysed had 80 demand nodes (Figure 7.5). The genetic algorithm parameters for this example were the same as the previous example except that the population size was 250. The pipe network parameters were given as follows:

Discharge at demand nodes	0.00053 m^3/s
Distance between demand nodes in the X direction	100 m
Distance between demand nodes in the Y direction	60 m
Distance between demand and non demand nodes in the X direction	50m
Distance between demand and non demand nodes in the Y direction	30 m

As the number of demand nodes increases, the number of possible alternative solutions increases dramatically. For the network with 80 demand nodes although only 1/4 of the demand nodes participate in the GAs process the size of search space equals 1,048,576. The model was allowed to run up to 300 generations with 250 population.

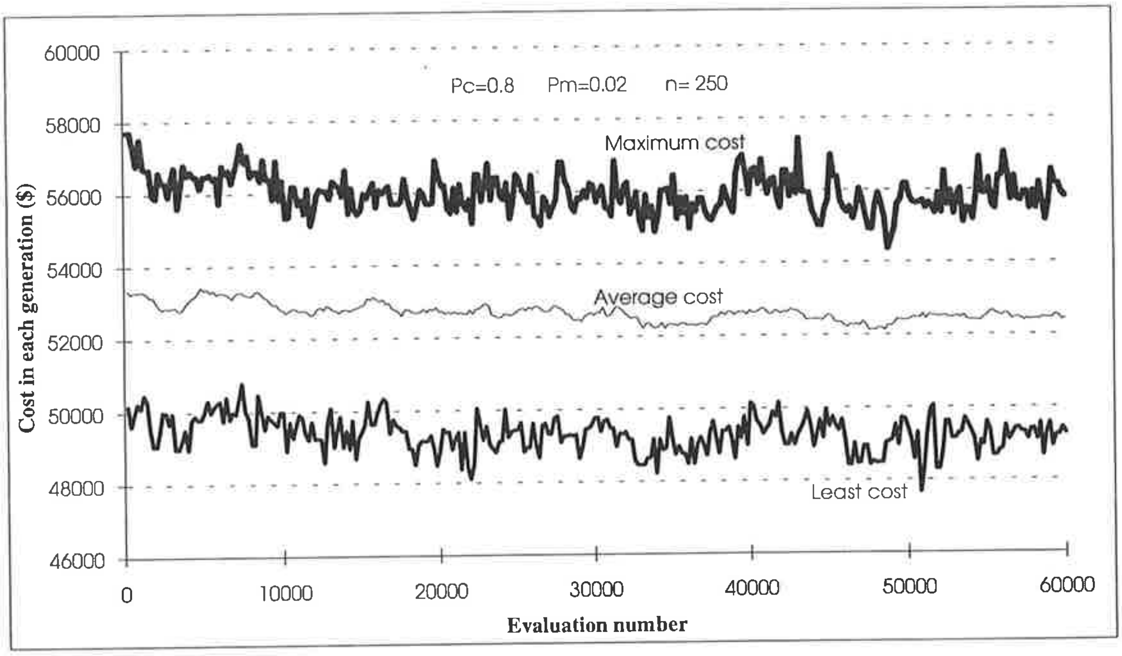


Fig. 7.25 The least , average and maximum cost in each generation for a multiple subunit system with 80 demand nodes (proportionate selection)

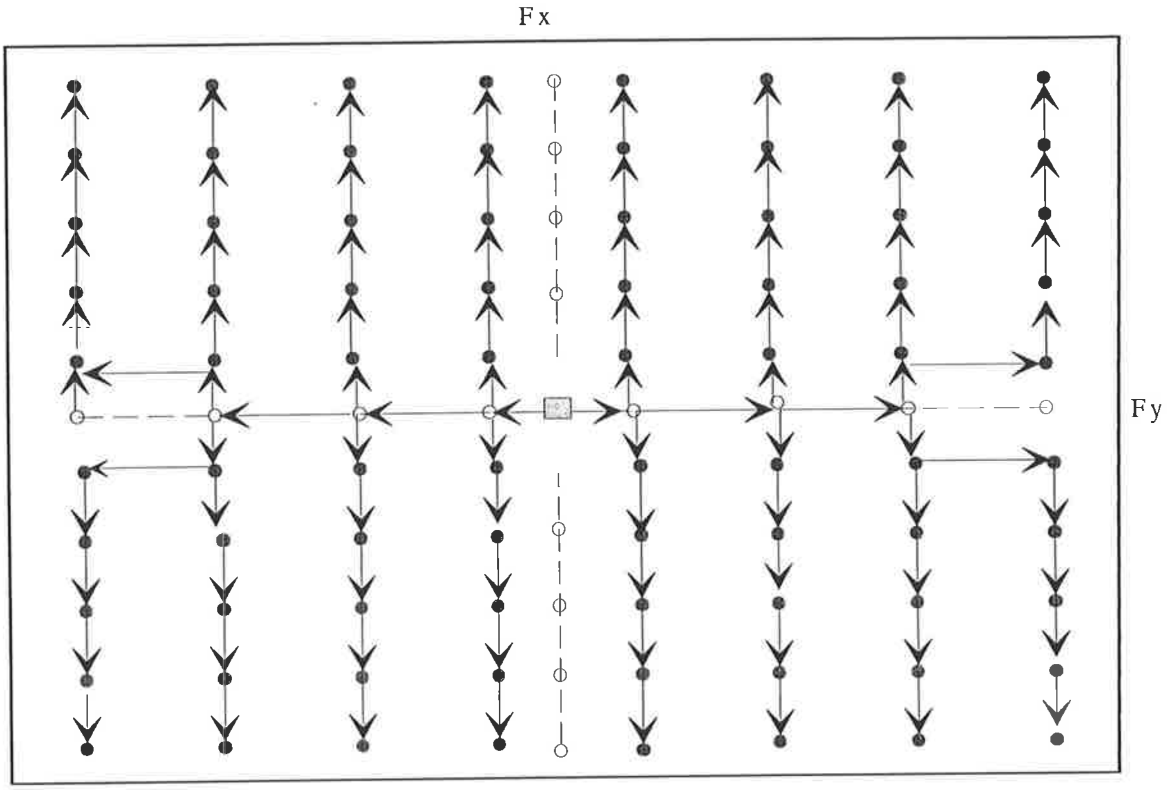


Fig. 7.26 Optimum layout of a multiple subunit system with 80 subunits applying GA with proportionate selection

In this process nearly 60,000 solutions were evaluated. The least cost solution was found after 50,800 evaluations in the 254th generation. Evaluation of 60,000 network designs took 37 minutes to run on a 80486 PC with Turbo Pascal compiler. In a similar way to the previous example, the modified model was tested considering proportionate and tournament selections. The results obtained for the least, average and maximum cost in each generation by applying GAs with proportionate selection are shown in Figure 7.25. The value of the least cost solution was $47,666 which obtained at generation 254 after 50,800 evaluations. For these number of evaluation (to reach the optimum) it took 30 minutes to run. The configuration of the least cost (optimum solution) obtained from the GA with proportionate selection is shown in Figure 7.26.

As shown in Figure 7.25, the curve for the least cost solution resulted using proportionate selection shows a very slow improvement towards an optimum solution. But it changes in an erratic fashion. In this way some solutions appear which are nearly close to the optimum solution. For example, a good solution before the best solution was found in generation 110 after 22,000 evaluations with a cost of $ 48,105. A rapid increase in the least cost in some generations is probably the effect of the crossover and mutation operators. The average and maximum costs in each generation change in an erratic fashion as well. The results indicate that the average generation cost curve shows very slow improvement in the fitness of the population. Some new solutions are created by crossover and mutation which may be substantially worse than the previous best solution. Although, in the long run this may lead to even better solutions. Nevertheless the best cost curve for the number of evaluations carried out in this run shown in Figure 7.25 dose not converge and still it changes with an unsteady manner. It is not clear how many evaluations are needed to reach the least cost solution. To ensure that the minimum cost obtained at generation 254 is the least cost, a sensitivity analysis using various GA parameters and also applying different seed numbers to generate different sequences of random numbers is necessary.

The same example was examined using tournament selection. Although the parameters of the genetic algorithm and network were not changed, the results obtained were different from the proportionate selection method. In this case not only the optimum solution had a smaller cost than the previous case but also it was found after a smaller number of evaluations. The minimum cost solution was appeared in generation 9 after 1,800 evaluations with a cost of $47,359. In this study it was found that tournament selection gave better results than proportionate selection. As shown in Figure 7.27, at the start the curves for the best, average and also maximum cost are very steep. At this stage it is not very difficult to achieve improvement in almost every generation as there are a lot of lower cost solutions. The results observed from testing the modified model on the example with 80 demand nodes show that in the first 1800 evaluations the genetic algorithm with tournament selection is very effective in reducing the best and the average cost of each generation. The best cost exhibits a rapid reduction from the start to generation 9 and after 1800 evaluations, remains constant at $47,359. (Figure 7.27). For all other subsequent generations no further improvement is observed and the curve is steady. The average cost of generations has also a rapid reduction, then shows small variations but with a very slow rate of decline. However, the maximum cost curve after a rapid reduction does not show a smooth variation. As the number of evaluation increases it changes in an erratic fashion without significant improvement. The configuration of layout corresponding to the minimum cost solution using tournament selection is presented in Figure 7.28.

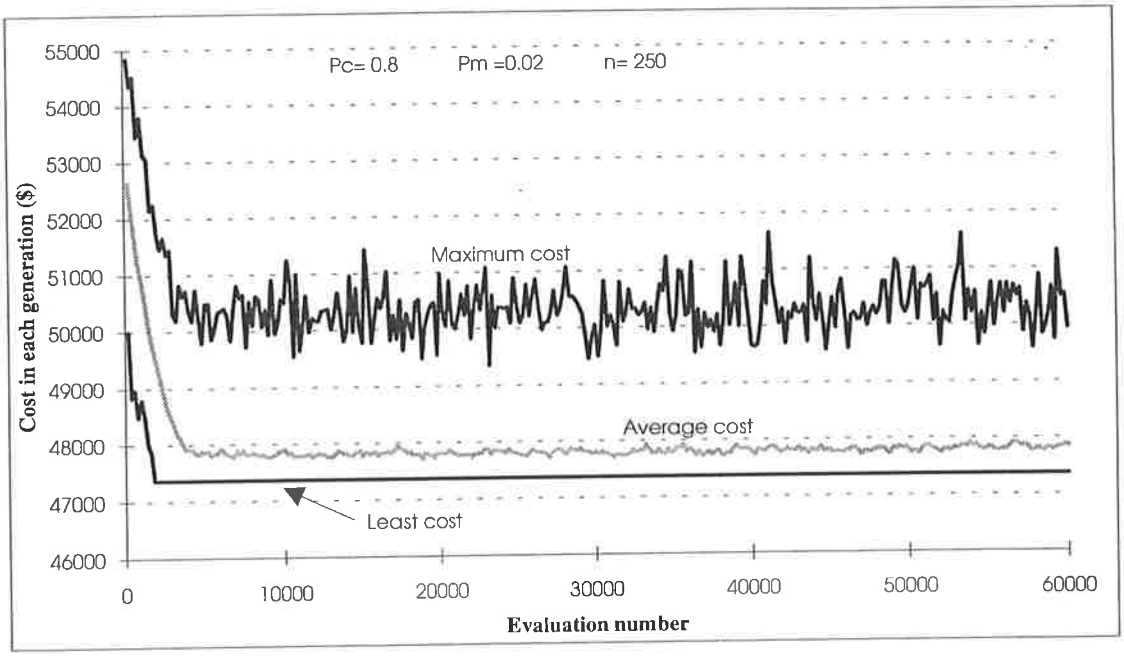


Fig. 7.27 The least , average and maximum cost for a multiple subunit system with 80 demand nodes (Tournament selection)

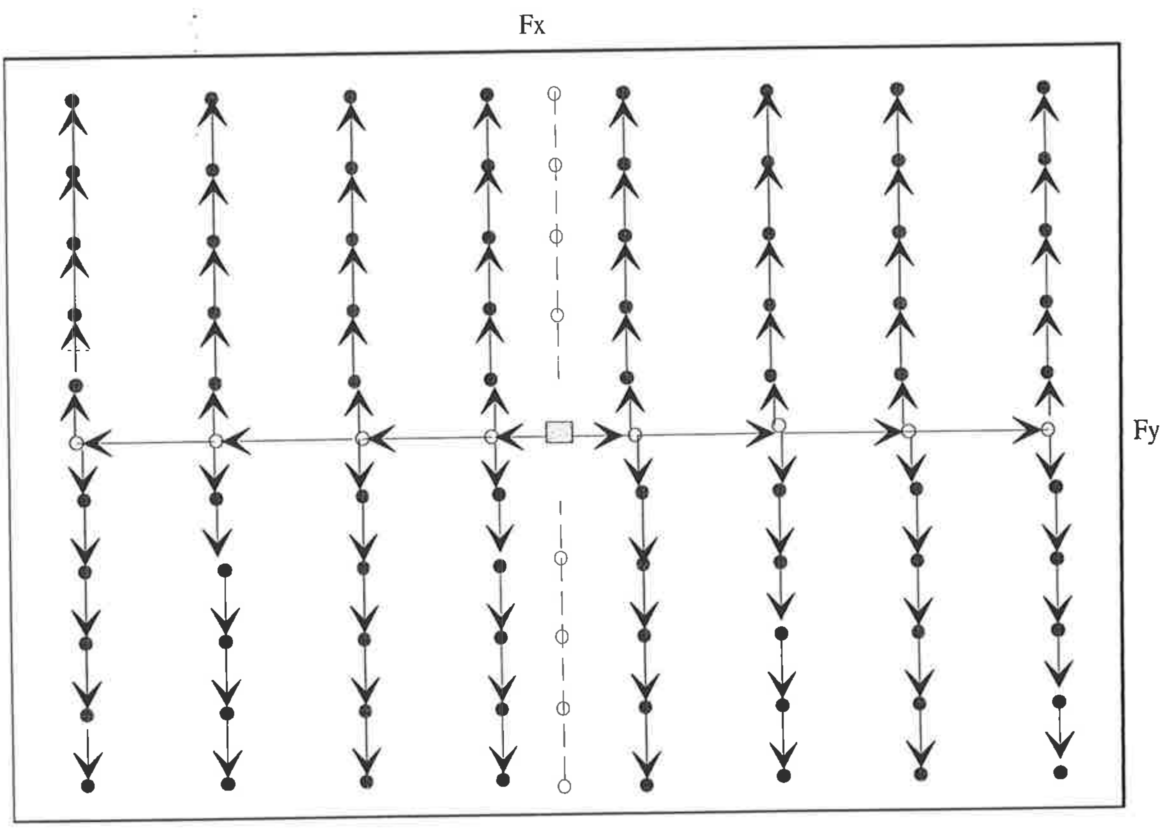


Fig. 7.28 The optimum solution (layout) of a multiple subunit system with 80 subunits (Tournament selection)

For 60,000 evaluations the model took 34 minutes to run on a 80486 PC with Turbo Pascal compiler. To identify the real time that the model needs to find the optimum solution (tournament selection) the model was run again with various population sizes. It was found that the minimum cost solution can be obtained with much less numbers of evaluations comparing to the proportionate selection. In the new run with the same seed number but with a population size of 25 the minimum cost solution of $47,359 was obtained after just 320 evaluations (12 generations). By this way the time to reach the optimum solution was decreased dramatically. It took only 15 seconds to reach the least cost solution. A number of chromosomes with binary presentation each showing a particular solution with associated costs are illustrated in Appendix G. The findings indicate that for the branched networks examined here using GA with tournament selection finds a better solution with a very few number of evaluations comparing to proportionate selection.

7.14 SUMMARY

An optimum branched layout model was developed using the genetic algorithm technique. Genetic algorithms as a random search method were found to be very efficient for identifying the minimum cost layout of pipe networks. The formulation of the model was based on the directed base graph. Using a directed base graph was useful to reduce the size of the search space. Any pipe network with a rectangular pattern of nodes which can be connected by a branched network may be solved by the current model. A technique based on the backtracking method for determining the flow rate in links and consequently, the pressure head at nodes was developed.

Each possible solution was coded using a binary format. In this way each demand node as a decision variable was assigned to one "1" or zero "0". As a result, the length of strings or chromosomes, each representing one of the trial solutions was equal to the number of demand nodes in the network. The "1" or "0" in each bit of a string corresponds to a particular node, and represents which of two possible links should exist.

The system cost consists of pipe and pump costs. As the diameter of pipes is unknown the cost of pipes is modelled as a function of the flow in the pipes. As indicated in Equation 7.6, the cost of a pipe is a mild concave function of its capacity, in such a way as the capacity of a pipe increases the delivery cost per unit flow decreases. The pump cost as the second component of the system cost was formulated on the basis of total head and total flow rate. A simple fitness function as an inverse function of total system cost was used as the objective function for the GA to select the better solutions.

The model was tested on two examples, one a network with 16 demand nodes and the other with 80 demand nodes. However, only a quarter of demand nodes in each network were used in the GA process. This was very useful to reduce the required computer time and memory. The GA was applied considering both tournament and the proportionate selection techniques. The results obtained using the tournament selection were much better than for proportionate selection. For a population size of 10 and generation number of 20 the model with tournament selection took about 1 second to run for the first example. However with proportionate selection much more numbers of evaluations need to be carried out. For example, for a population size of 250 and the generation number of 254 the model (with proportionate selection) took nearly 30 minutes to find the minimum cost solution for the second example on a 80486 PC with Borland Turbo Pascal compiler. The same model, but with tournament selection for the second example (80 nodes) took only 15 seconds with 320 evaluations to find the least cost solution on the same machine. This time is quite small comparing with proportionate selection which shows the better performance of tournament selection.

The total size of the search space for the example with 80 subunits equals 1048576 . The GA model with tournament selection found the best solution after 320 evaluations which is a very small fraction of the search space. The results of model using both selection methods for the least cost layout are shown in Figures 7.26 and 7.28. It is clear that installing the submain pipes in the vertical (Y) direction and the mainline pipes in the horizontal (X) direction is more economical than any other way.

Chapter 8

Optimum Layout and Design of Drip Irrigation Systems

8.1 INTRODUCTION

Optimum design of pipe networks has been the subject of research over the last two decades. Some optimisation methods apply to a looped network and some apply to a branched pipe system. Some researchers have considered a fixed layout and unknown pipe sizes (Karmeli et al, 1968; Kally, 1972, for branched network; Alperovits and Shamir, 1977; Quindry et al, 1981; Simpson et al, 1994 for looped networks) others have considered the optimum layout problem only (Walters, 1985a; Davidson and Goulter, 1992; Hassanli and Dandy, 1994). However, some authors have studied the problem of simultaneously optimising layout and pipe diameters (Walters, 1985b; Goulter and Morgan, 1985; Awumah et al, 1989; Walters and Lohbeck, 1993).

Usually, pipe networks designed for irrigation purposes are branched, consisting of one or more source node(s) and a number of demand nodes. The principles dealing with the hydraulic analysis and optimum design of branched networks for irrigation or urban systems are the same. The difference between them relates to the loading cases and the operating conditions.

A number of different optimisation techniques have been employed by researchers for the optimum solution of pipe networks. The most commonly used methods are linear programming (LP), dynamic programming (DP), non-linear programming (NLP) and genetic algorithms (GAs). The last of these have received special attention in the recent years. During the past few years, the GA as an optimisation method based on systematic search has received much attention. GAs represent an important part of the evolutionary programming approaches. They are based on random search algorithms that start with a population of randomly selected solutions.

In this chapter, genetic algorithms (GAs) have been utilised to identify a least cost branched layout and sizes of the corresponding components (pipes and pump). The model has then been extended to the optimal design of a multiple subunit drip irrigation system with a regular pattern and considers the minimum cost of the micro pipes inside the subunits by using a full enumeration approach. The discharge at demand nodes representing the irrigation requirements of subunits is calculated from the soil and crop characteristics using equations suggested by Karmeli et al (1985). The details concerning the least cost layout of branched networks using a directed base graph are discussed in more detail in Chapter 7.

8.2 CONFIGURATION AND COMPONENTS

In general, most irrigation, drainage and sewer systems are designed as branched (tree) networks having a root node (source node), trunk and branches (Walters et al, 1993). For irrigation networks, in addition to the major pipes which connect the demand nodes to the source node, there are a number of minor pipes (micro pipes) inside subunits which are supplied from a valve located at the centre of each subunit. Figure 8.1 illustrates all options of main and submain pipes for a type of regular irrigation pipe network which was analysed in this study. The model identifies an optimal tree network from this figure. It is composed of a single water source located at the centre of the field with a set of major pipes and a number of subunits containing lateral and manifold pipes. Each manifold pipe is fed by submain pipes from the centre of a subunit. The centre of each subunit is assumed to be a demand node for the submain lines. The laterals in each subunit have a regular spacing (d_y) on either side of the manifold and are fed by the manifold. The emitters are located on the laterals with regular spacing (d_x) and act as a point source of flow rates operating at low pressure heads.

Since the model is designed for a level field it is assumed that water allocated to each subunit is divided equally into the two halves of the manifold. The same assumption is considered for the laterals located on either side of the manifold. The submain pipes supply the manifolds from one end (at demand nodes) and are connected to the main pipes at the other end. The connection of the submain pipes to the main pipes is identified on the basis of the least cost layout search. A typical configuration of micro pipes within the subunits is shown in Figure 8.2. The main pipes which transport water from the source node are located along the X and Y axes as illustrated in Figure 8.1.

A pumping system is assumed to be located at the source node providing the total irrigation requirements from a bore hole. A filter unit, a fertiliser unit, and a number of controllers (one for each 8 subunits) are considered for the whole field. Additionally, a number of on-off valves are also considered for each subunit and submain pipes where connected to the main pipes and for main pipes where connected to the source node. The hydraulic properties and costs of the pumping system, pipes and the other equipment are similar to those discussed in detail in the previous chapters.

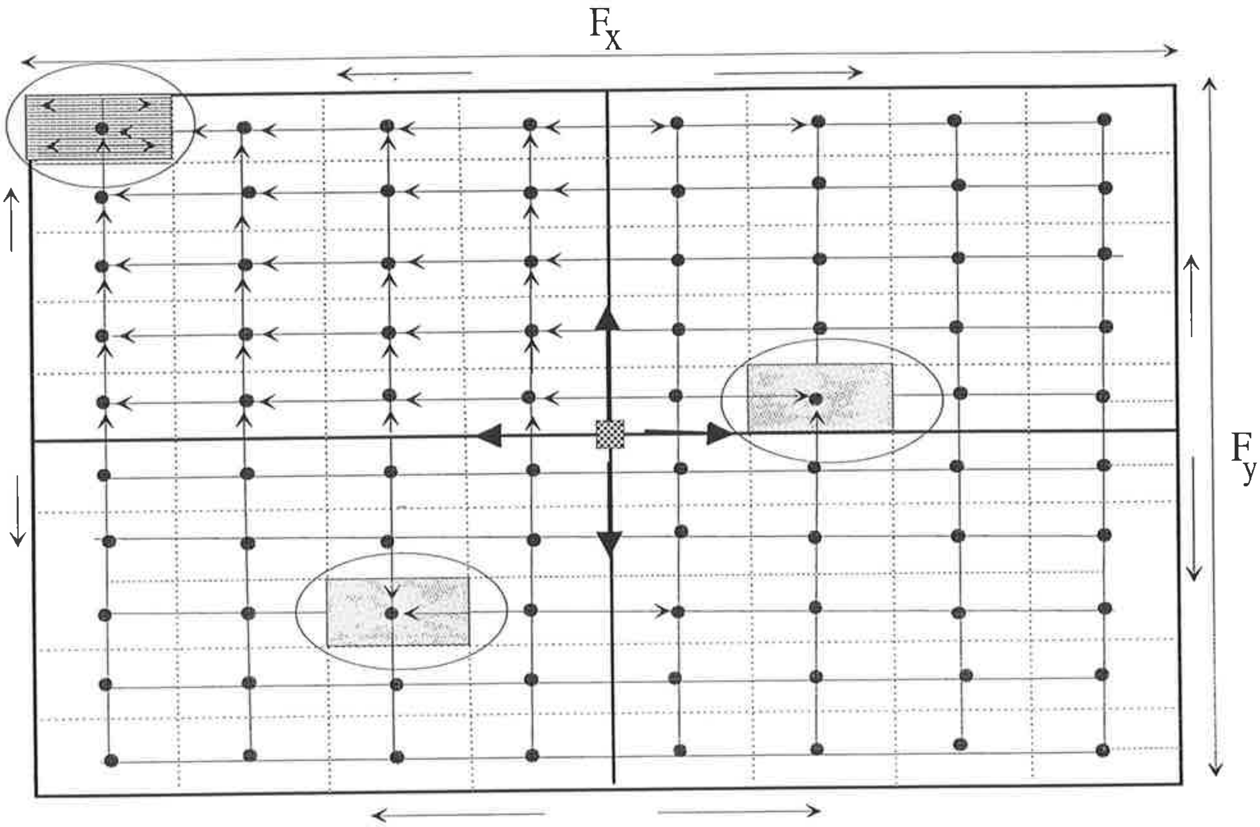


Fig. 8.1 A typical multiple subunit pressure irrigation systems with 80 subunits

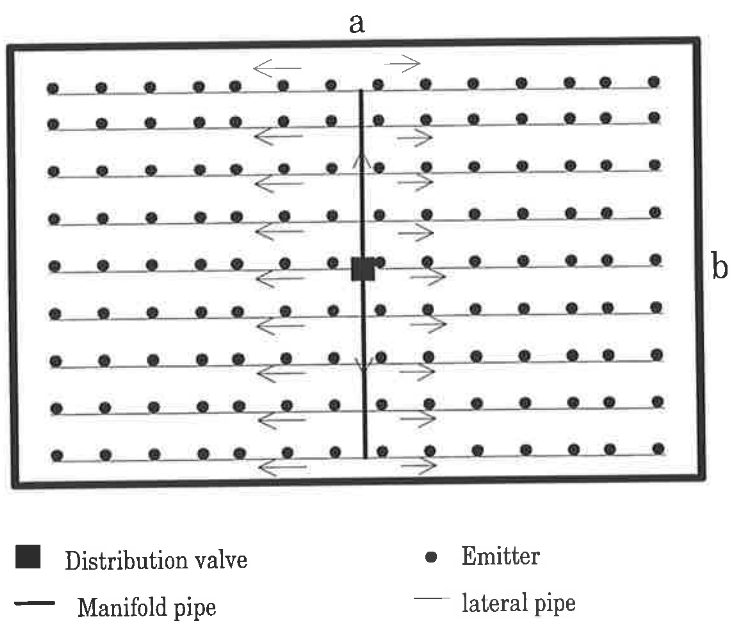


Fig. 8.2 A typical example of a subunit with micro pipes analysed in the model

8.3 OPTIMISATION METHODS USED

In general, there are two linked aspects to the design of a pipe network, i.e. selection of layout or connectivity of the network and the selection of pipe diameters. In a branched network, with a fixed layout and connectivity, the flow along each link is known. Thus the optimum size of links may be selected using the various methods mentioned in Section 8.1. When both the layout and component sizes are unknown, the problem is far more complex (Walters and Lohbeck, 1993; Hassanli and Dandy, 1995b). In this study, the optimum design of a multiple subunit drip irrigation system considering both the layout and component (pipe and pump) sizes including operating cost was investigated. In Chapter 7, GAs were employed as an efficient search technique to determine the optimum layout. The least cost layout solution was investigated on the basis of the flow rate in the pipes. This was undertaken by developing a modified cost equation considering cost per unit capacity (Walters and Lohbeck, 1993) and cost as a non linear concave expression of pipe diameters (Oron and Karmeli 1979). Additionally, a constant head loss per unit length of pipes (as considered by Davidson and Goulter, 1992 and Walters and Lohbeck, 1993) was assumed.

In the current model, GAs were applied to determine both the layout and component sizes of a network. The maximum accumulated head loss associated with the selected pipes was considered in the design of the pump. The optimum layout and the major pipe sizes were investigated by applying GAs, whereas the minimum cost of subunits including manifold and laterals were investigated using a full enumeration approach. In order to achieve an optimum cost associated with each subunit, two small sizes were considered for laterals. The optimum length of each size of laterals, as well as the optimum size of manifold, were considered as the decision variables. The irrigation requirement of subunits (flow rate at demand nodes) were obtained from agronomical and agro-technical factors using equations suggested by Karmeli et al (1985). Further details are given in Section 8.6.1.

8.4 TRIAL SOLUTION (CODED STRINGS) IN THE GA PROCESS

The coded strings representing a trial solution are similar to the structure of a chromosome of genetic code. The finite length string is called a *chromosome* and the bit positions are called *genes* (Goldberg and Kuo, 1987). A variety of coding schemes can and have been used successively in GA formulation (Goldberg, 1986). The strings may be expressed in a binary, integer or real formats. In the current model, two different coding formats are used. These are firstly, the binary coded containing the characters "0" and "1" representing the existence of horizontal (X direction) or vertical (Y direction) links directed to each node, and secondly, integer numbers between 1 and 9 each representing a particular discrete size for pipes. A full string with a binary and integer format showing a trial solution for a network with 20 demand nodes is shown in Figure 8.3. The coded string of 49 numbers consisting of binary and integer format may, for example, represent a branched pipe network design of 20 demand nodes as shown in Figure 8.4 (it should be noted that the original looped pipe network shown in Figure 8.4 will be converted to a branched pipe network after removal of redundant links by the GA). In this process only one of two possible links directed to each demand node is allowed to remain to supply the corresponding node. Details of the developed algorithm are given in Chapter 7.

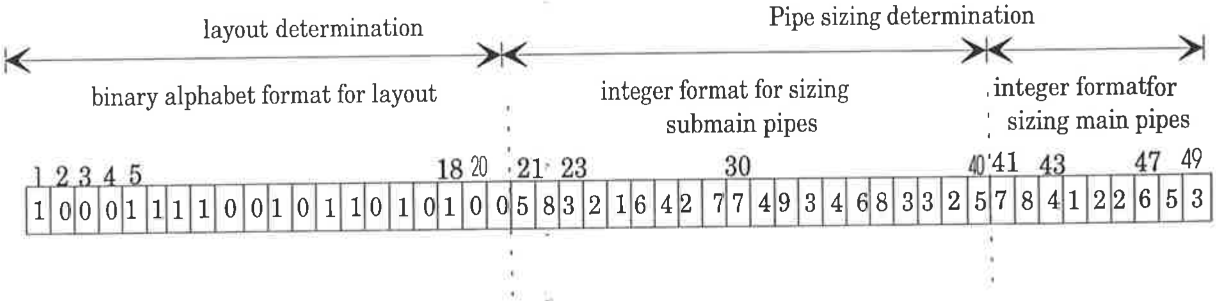


Fig. 8.3 A full length string (chromosome) for layout and component sizes of a network with 20 demand nodes.

In Figure 8.3, the numbers above the binary alphabets (1 to 20) represent the number of the demand node and above the integer numbers (21 to 40) represent the corresponding links directed to those demand nodes. On the other hand each binary number identifies which link should exist for the corresponding node and each integer number shows the size of the

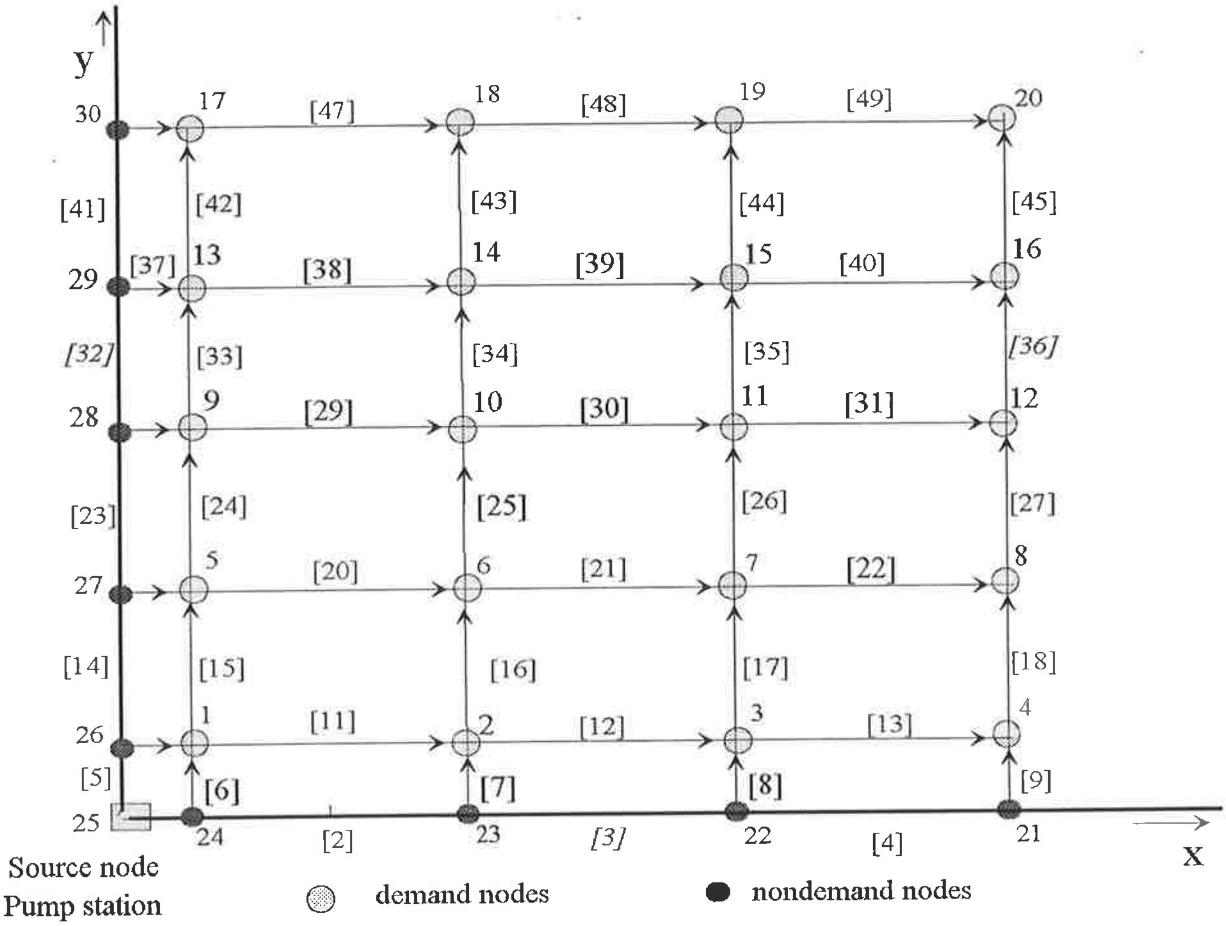


Fig. 8.4 A network with 20 demand nodes (the node numbers, pipe numbers and binary code of links are shown)

corresponding pipe. Obviously, in a branched pipe network with 20 demand nodes only 20 links (pipes), each one directed to one node would exist. However, there are 49 bits in the constructed string as shown in Figure 8.3. The last nine bits ranging from 41 to 49 denote the size of 9 possible existing pipes (main pipe segments) which may connect the source node to the submain pipes (connections between nodes 21 to 30, in Figure 8.4 including pipes numbered from [1] to[5] and [14], [23], [32], [41]). Although the model assumes that the main pipe segments exist initially, the final segments considered in the system cost, depend on the least cost solution and feasibility requirement. The feasibility requirement will be satisfied if there is a connection between all demand nodes and the source node. The decoded trial solution is evaluated and the coded string is then accompanied by a value corresponding to that string showing its fitness. The fitness of a coded string in fact, shows the ability of the artificial chromosome to survive. In other words, it reflects how good it is, compared with other solutions in the population. In nature, the fitness of a chromosome may reflect a living organism's compatibility with the surrounding conditions and eventually regulates its survival (Murphy et al, 1993). In the coded strings scheduled for the present model, for each demand node only one bit containing either 0 or 1 will exist. This is due to the fact that after removal of one of the links only one link will be directed to each node to build a branched pipe network. The second part of the string containing the integer numbers, consists of a string of numbers equal in length to the number of demand nodes plus the number of possible main pipe segments as already explained. In this manner all demand nodes and possible associated existing pipes will be assigned either 1's and 0's or a set of integer numbers.

8.5 FORMULATION OF MODEL

As pointed out previously, the present model optimises both the layout and the component sizes of the branched pipe networks with a known number of demand nodes located at a regular pattern. Although the main purpose was to develop an optimal model for multiple subunit pressure irrigation systems, the model may also be applied to any rectangular grid branched networks with known demands. The first process of a genetic algorithm is the generation of a number of strings or chromosomes as initial trial solutions with an appropriate coding format. In successive generations the GA generates a set of new strings using the fitness of old strings by implementing the selection, crossover and mutation operators. The decision variables which were coded in a finite-length string format may then be decoded to the actual pipe design. The formulation of coded strings utilised in the current model was discussed in Section 8.4. In the following sections the algorithms concerning the flow and head loss in pipes, pressure at nodes and the fitness of strings associated with the system cost will be discussed in detail.

8.5.1 Converting the Looped Network to a Branched Network

In the process of the least cost layout solution, the network was initially structured as a looped system. Two links were assumed to be directed to each node with a rectangular form as illustrated in Figure 8.4. Eventually, the least cost will be determined for a network with a branched configuration. This is achieved by removing one of the links directed to each node as redundant, in a manner such that each nodal demand is supplied by only one pipe. Although two alternatives exist for each node, the number of possible alternative solutions increases dramatically as the number of demand nodes increases. For example, in a network with 20 demand nodes used in case study, 1048576 (2^{20}) possible alternative solutions may exist just for the layout design. The final selected link to each node is based on the least cost solution. The principle of removing or selecting of links is initially based on the generation of 0's or 1's by the genetic algorithm. Generally, '0' represents the existence of link in the X direction for a demand node while '1' represents the existence of a link in the Y direction for the same demand node. Accordingly, the length of that part of string representing the layout will be equal to the number of demand nodes as illustrated in Figure 8.5.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
0	1	1	0	0	0	1	1	1	1	1	1	0	0	0	0	0	1	1	0

Fig. 8.5 First part of string with 20 bits corresponds to a network with 20 demand nodes for the layout problem (a trial solution)

Each bit will finally contain either 1 or 0 which depicts the final corresponding link associated with each node. This will represent a branched network with one link directed to each demand node. The existence of at least one link connecting the source node to any other links directed to demand nodes is essential to satisfy the feasibility requirement.

As discussed in Chapter 7, the choice of optimum layout was carried out considering a directed base graph. In branched networks, the required capacity of a link will be equal to the sum of the demands downstream. But when the layout is not fixed, the connectivity of the network is not known initially. In layout optimisation models in which the pipe diameters are not known but assumed to be available in continuous range, the head loss per unit length may be assumed to be fixed. In this case the relationship between cost and capacity of each link may be expressed as shown in Equation 7.1 in Chapter 7. This equation models the real situation in which cost per unit capacity for a pipe line decreases as the capacity increases. Further details are given in Chapter 7. However, the current model deals with both layout and pipe sizes in which the pipes are assumed to be available in discrete sizes. Hence the least cost solution, considering the optimum layout and component sizes, is undertaken by a trade-off between the pipe cost and the energy cost associated with the head loss in pipes.

8.5.2 Application of GA to Optimum Components

As pointed out previously, genetic algorithms work with strings in which the parameters of the optimisation problems are to be coded. In the first stage, a looped system is constructed in such a manner that each node can receive water from two directions. The final existing links are then specified by the GA process, considering the removal of redundant links randomly. In the second stage, a diameter from a set of discrete given sizes is also randomly selected for each remaining link. In this way, a set of integer numbers between 1 and 9, each corresponding to a particular size of available pipes is generated as illustrated in Figure 8.6.

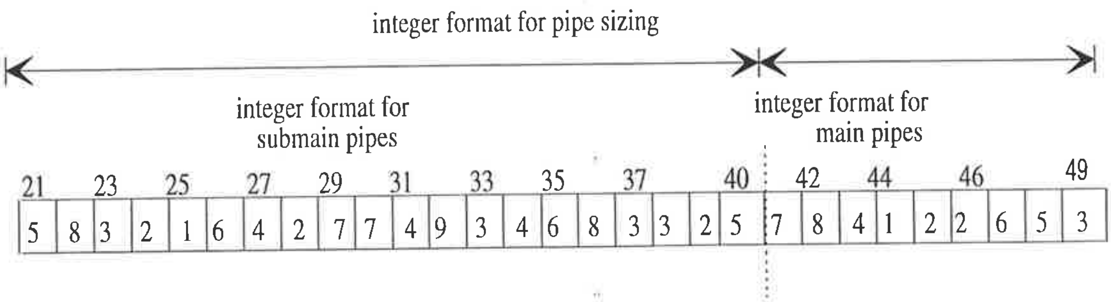


Fig. 8.6 Second part of the coded string with 29 bits

Each generated number then is assigned an associated actual pipe size (eg. Table 8.1). Any integer number generated for the bits 21 to 40 associated with demand nodes ranging from 1 to 20 represents a particular size for the corresponding links. Each number then is decoded to the actual pipe size using the look up table such as that shown in Table 8.1. Similarly, the integer numbers generated for the bits 41 to 49 associated with the non demand nodes each represents a particular size for the corresponding links. As shown in Figure 8.4, in contrast to demand nodes, only one pipe is considered to deliver water to each non demand node. In order to ensure feasibility, the links corresponding to non demand nodes (pipes [1] to [5] and also [14], [23], [32], [41] in Figure 8.4) will always be assumed to exist. However, only those links delivering water from the source node to the submain pipes will be considered in the network cost computation as the final existing pipes.

Table 8.1 Available discrete pipe sizes and associated integer numbers

Integer codes corresponding to pipe diam.	Pipe diameters (mm)	Integer codes corresponding to pipe size	Pipe diameters (mm)
1	16	6	102
2	21	7	130
3	35	8	149
4	57	9	187
5	84	-	-

This process is identified by the back tracking method developed in this research which basically depends on the GA search considering the least cost solutions. Including all links between non demand nodes ensures that infeasible solutions will not exist at any stage of the GA process. The results demonstrated in the later sections confirm this issue. A full length string for both the optimum layout and pipe sizing for the example used in the case study is illustrated in Figure 8.3.

8.5.3 Flow and Head loss in Pipes

The flow in pipes depends on the value of demands at nodes. The value of demands in turn depends on the irrigation requirements. Although the demand at nodes is known, determining the flow in each link is not a simple issue since the position of links is not known initially. To overcome this problem an algorithm based on back tracking movement was developed (detail is given in Chapter 7). The head loss of pipes in branched networks which affects the pressure at nodes and the selection of pipes and the pump is extremely important. This is particularly so in the current study in which a trade off is carried out to achieve an appropriate combination of pipes and pump sizes. The Hazen-Williams equation was used to calculate head loss. In each trial the length of each link as well as the Hazen-Williams roughness coefficient are known. The flow corresponding to each segment of path (pipes) considering the nodes downstream is also obtained as explained above. In addition, the pipe diameters from a set of given discrete sizes are selected by the genetic algorithm process. As a result, the head loss in each connection may be computed as follows:

$$hl[connect[a,b]] = \frac{10.68 \times flow[connect[a,b]]^{1.852} \times Leng[connect[a,b]]}{C[connect[a,b]]^{1.852} \times D[connect[a,b]]^{4.87}} \tag{8.1}$$

where

$hl[connct[a,b]]$	head loss in connection between nodes a and b; (m);
$flow[connect[a,b]]$	flow in connection between nodes a and b; (m^3/s);
$Leng[connect[a,b]]$	length of connection between nodes a and b; (m);
$C[connect[a,b]]$	Hazen-Williams roughness coefficient for connection between nodes a and b;
$D[connect[a,b]]$	diameter of pipe between nodes a and b (m).

In this process different pipe sizes are examined. If a smaller pipe size is selected (although the pipe cost is decreased) the head loss is increased and as a result, the size of the required pump and also the value of the operating cost are increased. In some solutions the accumulated head loss within different segments of a branch and consequently the total head loss is quite large. Although no particular constraints have been considered to control the head loss in major pipes, the trial solutions with a high accumulated head loss will not be selected because they will have a high cost for the pumping system and operation.

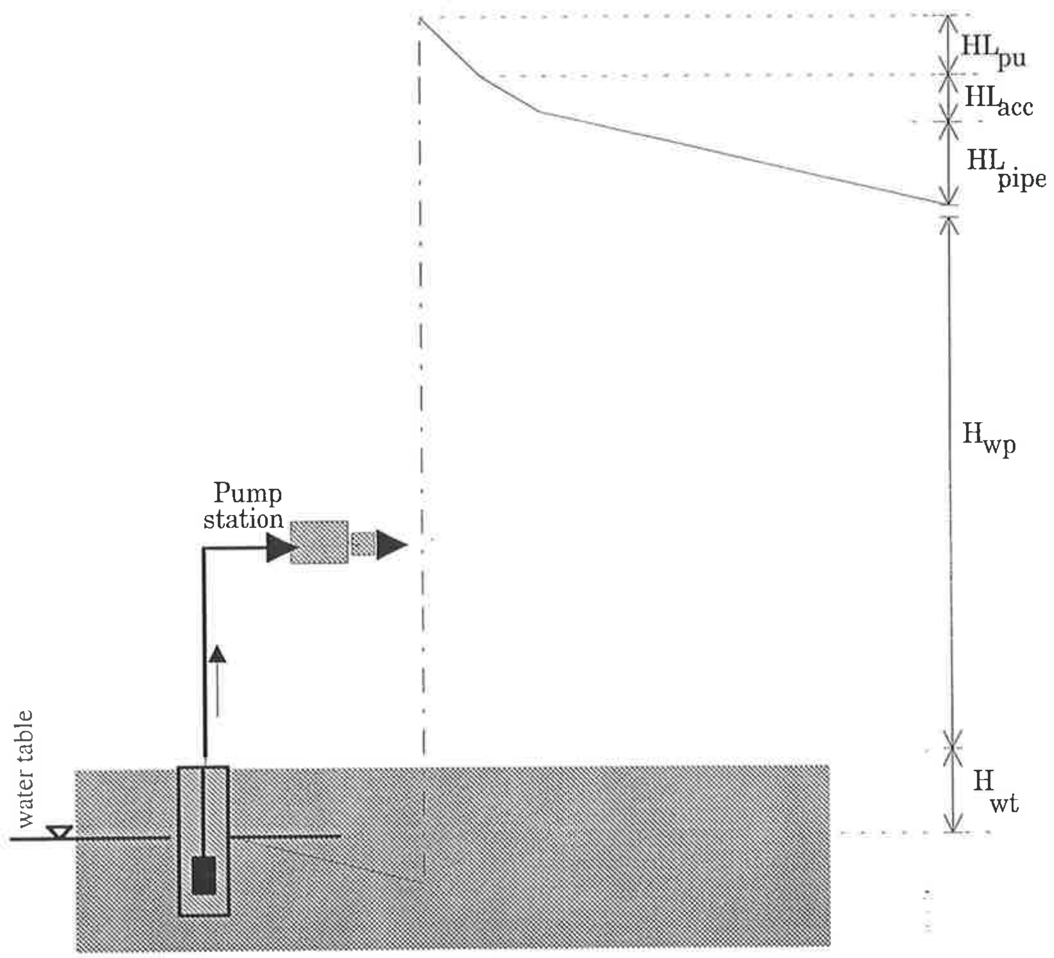
8.5.4 Head at Nodes

A constant head equals to the minimum required working pressure for emitters or sprinklers was assumed at each node. This working pressure or the minimum allowable hydraulic head may vary for different pipe networks. This variation depends on the characteristics and performance of system. For example, in the case of drip irrigation systems a pressure of 3.5 to 30 m (35 to 300 kpa) is recommended (Finkel, 1982; James, 1988), while for sprinkler irrigation systems a higher pressure is needed which varies for different type of sprinklers as well.

For example, for low pressure sprinklers, 10 to 21 m (99-207 kpa) and for high pressure sprinklers 28 to 90 m (276-896 kpa) is recommended (James, 1988). In addition to the minimum allowable hydraulic head at nodes the accumulated head loss in pipes downstream also is considered. In branched pipe networks more than one branch may exist and possibly some sub-branches also may be directed away from each branch. As a result, a different accumulated head loss may exist corresponding to each branch. The minimum required pressure provided by the pump will be the sum of the minimum allowable hydraulic head (working pressure), the depth of the water table (ground water level), the highest accumulated head loss in one of the branches, and the total minor head losses in accessories. A schematic diagram for the total pressure provided by the pump is illustrated in Figure 8.7.

8.6 EXTENSION OF THE MODEL TO DRIP IRRIGATION SYSTEMS

Although the model may work simply by assuming a certain demand at nodes in this study, both agronomic and economic considerations were taken into account to extend the model to a drip irrigation system. It is worth mentioning that the model may be easily applied to sprinkler irrigation systems as well. In such cases only the value of the demand rate at nodes may be increased. Additionally, for sprinkler irrigation system designs two procedures (sub programs in the model) dealing with micro pipe design and irrigation requirements need to be modified. Subunits containing laterals and manifold are optimised locally utilising the enumeration approach. Hence the evaluation of subunits was not included in the GA process.



$$H_{pu} = H_{wt} + HL_{pipe} + H_{wp} + HL_{acc} + HL_{pu}$$

Fig. 8.7 The components of total head in a typical pressure irrigation system

In other words, the optimum design of subunits is the same for all solutions resulting from the GAs performance. This remains constant unless the dimensions of subunits or the agronomical or agro-technical parameters change. Furthermore, the genetic algorithm parameters such as the probability of crossover and mutation and also the seed number (for generating the random numbers), were assumed to be constant. In the following sections the design of subunits considering the agronomical parameters, the plant water requirements and also the flow rate at nodes are discussed.

8.6.1 Design of Subunits

In contrast to the optimal model OPSHIR discussed in Chapter 5 in which the dimensions of subunits were considered as decision variables, in the current model, the system evaluation is carried out on the basis of the constant dimensions of subunits. The subunit dimensions depend on the dimensions of the field and the number of subunits in each network. The length of the laterals and the manifold are a function of the dimensions of subunits. In the optimisation process the optimum length of two given pipe sizes for laterals and also the optimum size of manifold were considered as decision variables.

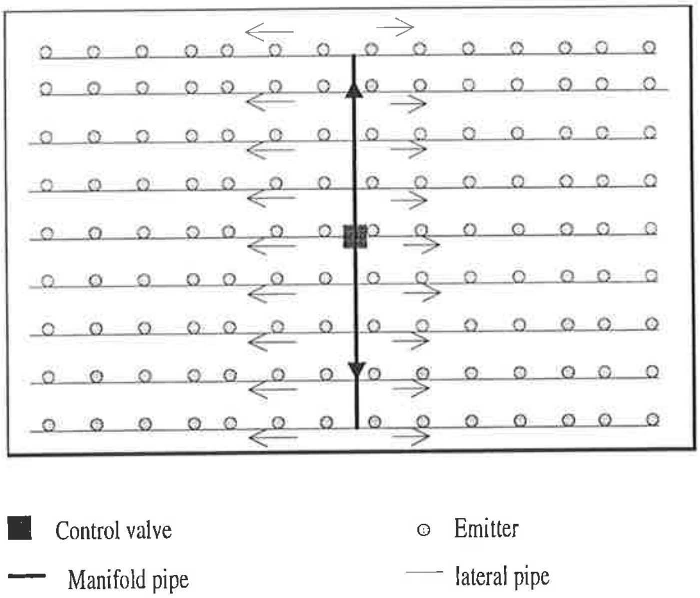


Fig. 8.8 A subunit with associated piping configuration

Decision variables in subunits were identified on the basis of a maximum 20% total pressure drop in the manifold and laterals from the control valve. The irrigation requirements were computed for a known agronomical and agro-technical information. Details related to this concept including the corresponding equations are given in Section 5.7. A typical subunit with associated piping configuration is shown in Figure 8.8.

8.6.2 Irrigation Requirement

The Irrigation requirement is one of the most important pieces of information for the design of irrigation systems (Karmeli et al, 1985). Reliable information requires extensive research and is often subject to changes. In general, use of data of maximum water requirement, in order to design a system to be able to provide maximum possible demand is recommended. The irrigation requirement of each subunit and eventually the total field discharge (pump discharge) are computed based on the agro-technical information. First the depth of water to be stored in the soil by each irrigation is estimated using field capacity, permanent wilting point, depth of root zone and the bulk density of the soil. It may be expressed using an equation suggested by Keller and Karmeli, 1979; Keller and Bliesner, 1990 as shown in Equation 5.72. Details related to the estimation of irrigation depth, irrigation interval, application rate, design discharge and the discharge of emitters are given in Section 5.7 and the process for determining the irrigation requirement used in this study (Chapter 5 and Chapter 8) is shown in a flow chart given in Appendix G.

8.7 SYSTEM COST

Computing the objective function to evaluate each trial solution on the basis of its fitness in GAs is essential. The fitness of each solution is inversely proportional to the total system cost. The objective function to be optimised, provides the mechanism for evaluating each string as a trial solution. In this study, the main focus is upon finding the minimum cost solution. The total system cost consists of pipe, pump, accessories and operating costs. In the design of irrigation pipe networks the cost of micro pipes, including drippers or sprinklers, significantly affects the total system cost. Otherwise the cost of main pipes supplying the demand nodes, pumping system and operating cost will be the main elements of the system cost. In the following sections different cost components considered in the total system cost are discussed in detail.

8.7.1 Fitness Function of Coded Strings

Along with the coding scheme used, the fitness function is one of the main aspect of any GA. Given a particular chromosome the fitness function returns a single numerical fitness which is supposed to be proportional to the utility or ability of that chromosome (Beasley et al, 1993). The fitness of a coded string associated with a branched pipe network may be determined by considering the cost of pipes, pump, accessories and the system operation. The fitness function represents a relationship between the fitness of the coded string and the objective function value. The general rule in constructing a fitness function is that it should reflect the value of the chromosome in a realistic way (Beasley et al, 1993). In the optimisation of pipe networks the optimum solution with minimum cost is investigated. The objective function therefore will be the total cost of the trial solutions as given by Equation 8.2. The objective function may also be expressed as a fitness function which is inversely proportional to the total cost as shown in Equation 8.3. In this way, as the system cost decreases the fitness will increase. Consequently, maximising the fitness function will be investigated through GAs process.

$$(C_{total})_j = (C_p + C_{pu} + C_{op} + C_{acc})_j \tag{8.2}$$

$$objfunc_j = fitness_j = \frac{1}{(C_{total})_j} \tag{8.3}$$

where $(C_{total})_j$ = total cost of string j (\$) and $objfunc_j$ = objective function of string j.

In the GA process the fitness value of strings is the main criteria for the strings to be selected. Highly fit strings receive a higher number of offspring thus having a higher chance of survival in subsequent generations. In tournament selection from some randomly chosen number of strings the fittest (the best) one is selected for further genetic processing. Moreover, the most fit string will be copied twice, while the least fit string in the population has no chance of being selected. Similarly, in a proportionate selection the strings are selected according to their fitness with respect to the fitness of all other strings in the population. This concept which represents the probability of selection may be formulated as shown by Equation 6.1.

Needless to say, the fitness assigned to strings must provide a criterion for the GA to differentiate the stronger strings (high fitness) with more chance of being selected from the weaker strings (low fitness). After selection each pair of selected chromosomes are subjected to crossover and the new chromosomes are then subjected to mutation as discussed in Sections 6.2.4 and 6.2.5.

8.7.2 Pipe Cost

The pipes used in this model are assumed to be available in discrete sizes. Although the associated costs are also available in a discrete manner, an equation using a non linear concave expression suggested by Oron and Karmeli (1979) was developed for this study as given in Equation 3.4. It facilitates the use of discontinuous pipe sizes as well. The cost of each link connecting two adjacent nodes considering the installation cost (ϕ) and using Equation 3.4 which computes the cost of unit length of a pipe with diameter of D is shown below:

$$C[connect[a,b]] = L[connect[a,b]] \times (C + \phi) \tag{8.4}$$

where C = cost of pipes per unit length with diameter of D (\$/m); $C[connect[a,b]]$ = cost of connection between nodes a and b (\$).

The GAs generate a set of random integer numbers within a specified range, each representing a specific diameter for pipes. Similarly, in successive generations the new strings each accompany a set of integer numbers which represent a size for each pipe. The generated strings then are decoded to the corresponding pipes in each generation. Equation 8.4 is then used to determine the pipe material and the installation costs. In addition to the major pipes there are a set of micro pipes (laterals and manifold) which distribute the irrigation water within the subunits. The cost of subunits including micro pipes, emitters and valve is one of the major components in the objective function. In the current model, the optimum design of subunits was not incorporated into the GA process. However, the enumeration approach was employed to find the optimum solution of subunits. The total minimum cost of subunits in each trial solution then was included in the objective function for further investigation by GAs.

8.7.3 Pumping System Cost

A cost equation for a submersible pump with electric motor drive head with standard solid shaft and associated accessories was developed on the basis of general pump equation suggested by Holzapfel et al (1990). The process of developing the cost of pumping system including the corresponding equation (Equation 4.39) is discussed in Section 4.8.5.1.

8.7.4 Present Value of Operating Cost

The present value of operation cost was computed over an expected life of project. Only the energy cost was considered in computation of the annual operating cost. However, the energy cost required for chemical injectors and fertiliser was assumed to be small and therefore was ignored. The annual required energy is dependent on the annual irrigation requirement and the total head satisfying the system pressure. It is directly affected by the total hours that the pump operates in a season to meet the annual water requirement. The present value of operating cost considering the annual irrigation depth, may be formulated as follows:

$$V_{an} = d_{an} \times (Fx \times Fy) \tag{8.5}$$

$$T_{an} = \frac{V_{an}}{Q_{pu} \times 3600} \tag{8.6}$$

$$A_{en} = P_{pu} \times T_{an} \tag{8.7}$$

where V_{an} = volume of annual irrigation requirements (m^3); d_{an} = depth of annual irrigation requirements (m); T_{an} = annual irrigation hours (hr); A_{en} annual energy requirements (known).

The annual energy requirement (A_{en}) and the unit cost of energy (C_{en}) are used to compute the present value of annual operating cost as shown in Equation 4.41.

8.8 OPTIMISATION PROCESS

As explained previously, the model was structured to optimise a branched pipe network system with known demands at the nodes. Achieving a fully optimised pipe network design to supply a multiple subunit irrigation system however may require further processes. This would incorporate the optimum cost of subunits receiving water from demand nodes. In the current model, GAs were used to optimise the layout and the size of the major pipes, while the optimum cost of subunits covering the cost of laterals and manifold was obtained utilising the enumeration approach. The structure of the main program is represented by the flow chart shown in Figure 8.9. Some procedures and functions used in the main program are discussed in the following sections:

8.8.1 Initial Data (input data)

The GAs receive their required data from an input file, created by the user. This input file contains the genetic algorithm parameters such as: size of population, length of string, probability of crossover and mutation and seed number. The main network parameters include: number and position of demand nodes, position of source node, length, roughness, connectivity map and unit cost of available pipes. Agronomic and agro-technical parameters including: field capacity, permanent wilting point, soil bulk density, effective root depth, wetted portion, allowable moisture depletion, irrigation efficiency, evapotranspiration rate, annual irrigation requirement and also lateral and emitter spacing as well as the operating parameters.

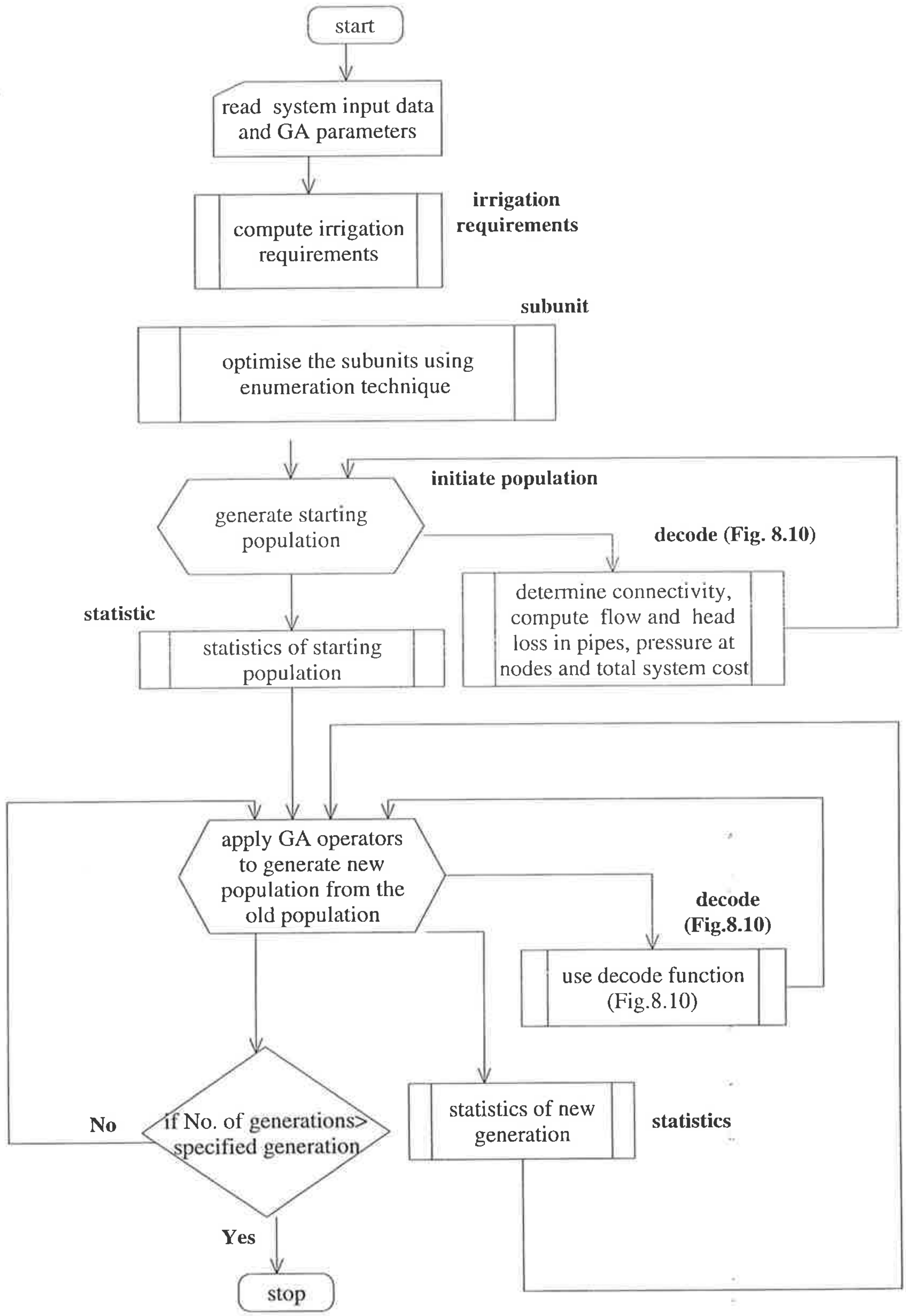


Fig. 8.9 Flow chart of main program

8.8.2 Generation of Initial Population

The GAs start their process with an initial population generated randomly. The sequence of generated random numbers depends on the seed number. The same seed number produces the same sequence of random numbers. Accordingly, with a given seed number the same starting population of chromosomes is generated for a genetic algorithm run. As noted earlier, the full length of each individual chromosome is composed of two different coded formats, binary and integer. Thus the procedure that generates the initial population is devised to create a set of binary numbers (0's and 1's) for the first segment of strings and a set of integer numbers (1 to 9) for the second segment in a random manner. The full procedure creating the initial population is demonstrated by the flow chart shown in Appendix I.

8.8.3 Decode Function

This important part of the model differentiates the application of GAs to different networks. In other words, for any particular problem a new decode procedure or function should be developed while the other parts of GAs may either need a slight modification or may remain unchanged. For example, the binary numbers 0's and 1's in this model, each represents the existence of a horizontal or a vertical link directed to each node. In the same way, the integer numbers (in the second segment of chromosomes) each represent a diameter for a particular existing link directed to a particular node. Once the existing links are specified, the flow rate, diameter, head loss and the pressure at nodes including the source node are determined. In the decode function decision variables are coded for each individual chromosome. The cost of pipes considering the length and the size and also the cost of pump considering the total required head and required flow are then computed. In addition, the optimum cost of subunits resulted from the procedure ***subunit,*** the present value of the annual operation cost resulted from the procedure *opcost* and the cost of accessories were computed to identify the total system cost. In fact, each decoded string is converted to the real parameters and then the total associated cost is computed. This is then used to compute the fitness or objective function of each trial solution. The process of the decode function devised for the model is represented by a flow chart shown in Figure 8.10.

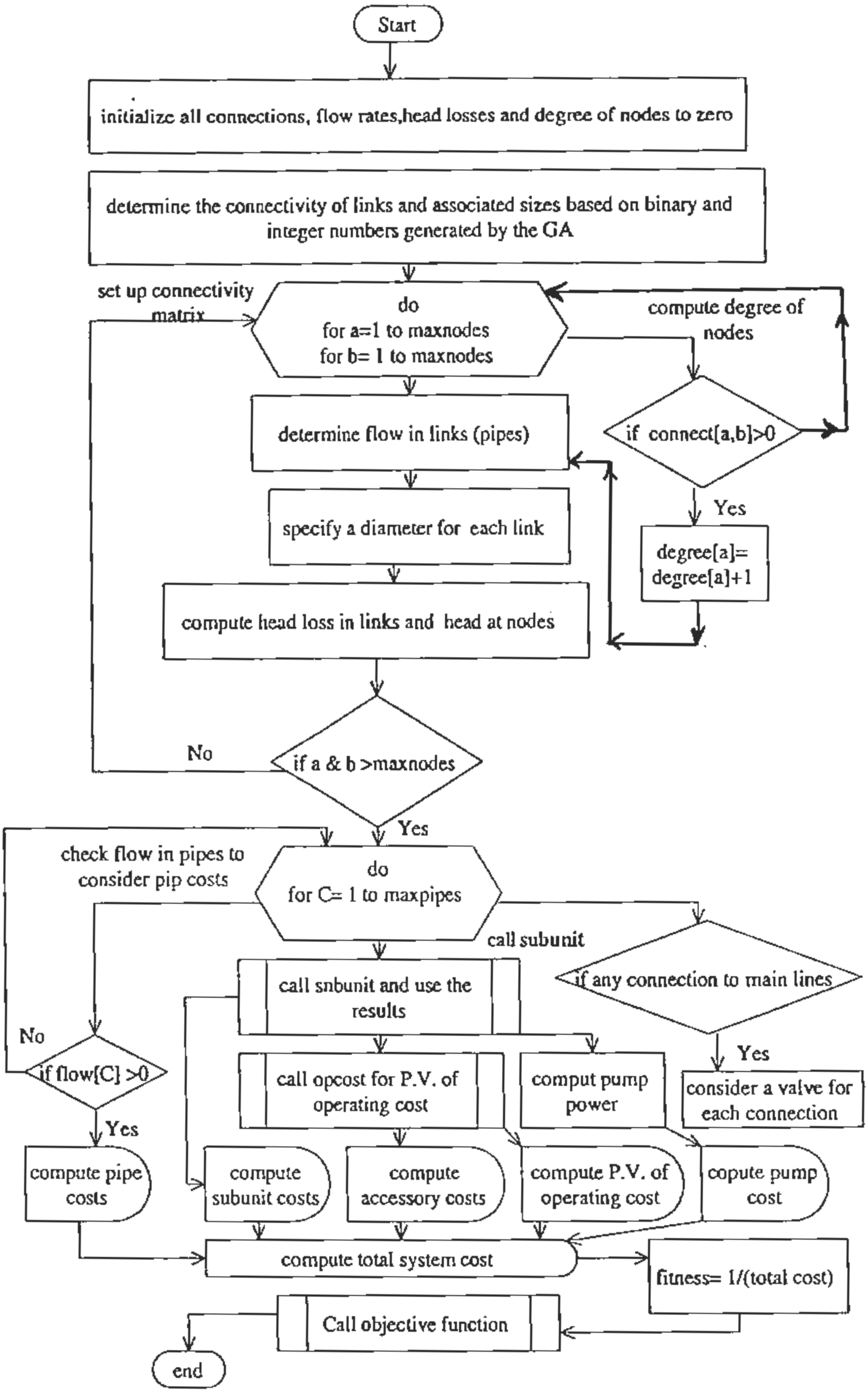


Fig. 8.10 Flow chart of the decode function for each chromosome

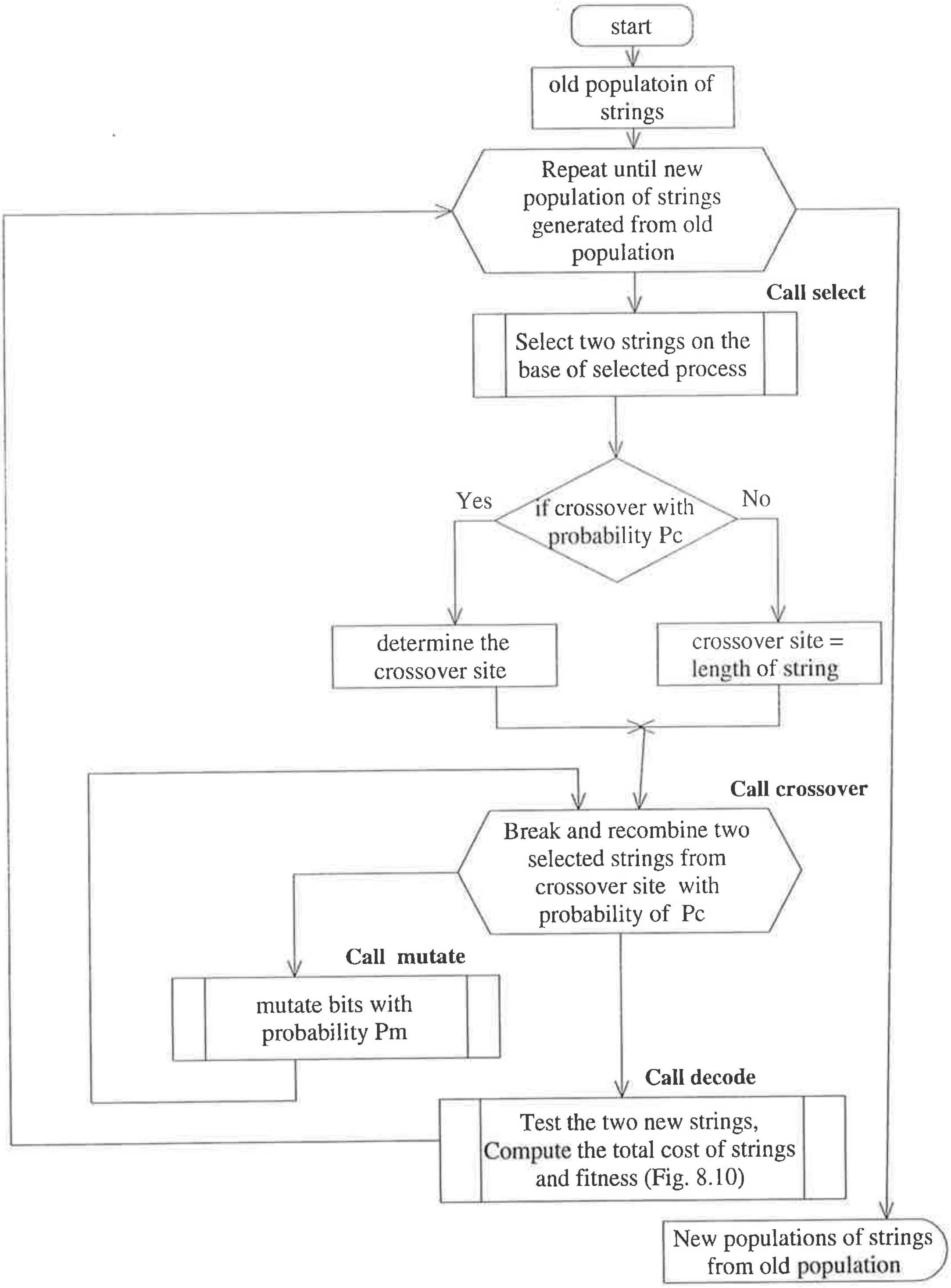


Fig. 8.11 Flow chart of generation procedure

8.8.4 The Generation Procedure

One of the most important features of the GAs is the evolution of their population over the successive generations. In each generation a new population is generated from the old population so that after many generations the population will have largely converged. The first generation begins with a population of chromosomes generated randomly. Two individuals are selected by the reproduction operator using function *select.* The crossover operator then takes two selected individuals and cuts their chromosome strings at some randomly chosen position to recombine and form two new individuals. Procedure *crossover* calls the function *mutate.* The function mutate will randomly alter each gene (bit) with a small probability (typically 0.01- 0.001, Beasley et al, 1993). The above process is applied to all individuals in the old population. The result of such a process is a new population of individuals with different properties belonging to a new generation. This process is repeated to form new generations until the fitness of the best individual increases towards the global optimum or no further improvement in the fitness occur. It may also be repeated until a preselected generation number is specified by the user. The generation procedure is represented by the flow chart shown in Figure 8.11.

8.9 CASE STUDY

The model OPDESGA was applied to two drip irrigation networks with 4 and 20 demand nodes. As pointed out previously, the model has the capability to assess networks four times larger than the original ones. Accordingly, two branched networks supplying multiple subunit drip irrigation systems with 16 and 80 subunits were considered. The original network with 20 demand nodes and its extended corresponding network with 80 demand nodes are shown in Figures 8.4 and 8.1 respectively. Also the original and final networks (extended) with 4 and 16 demand nodes are shown in Figures 8.12a and 8.12b. In multiple subunit pressure irrigation systems it is supposed that a field is divided into a number of subunits. The size and dimension of the subunits are a function of the size and dimension of the field and also the number of divisions as shown in Table 8.2 for a field with. 800m length and 600m width.

TABLE 8.2 Dimensions of field and subunits

Dimension	Field	subunits in a field with 16 demand nodes	subunits in a field with 80 demand nodes
Length (m)	800	200	100
Width (m)	600	150	60
Area (ha)	48	3	0.6

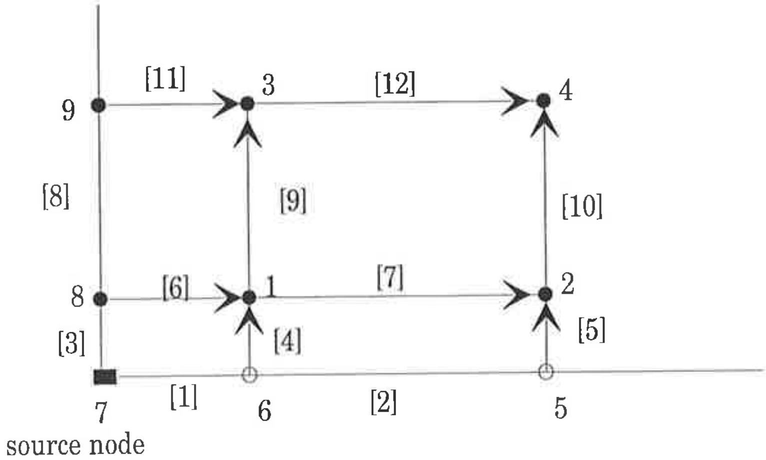


Fig. 8.12a A typical example of a network with 4 demand nodes (original network for 16 demand noes

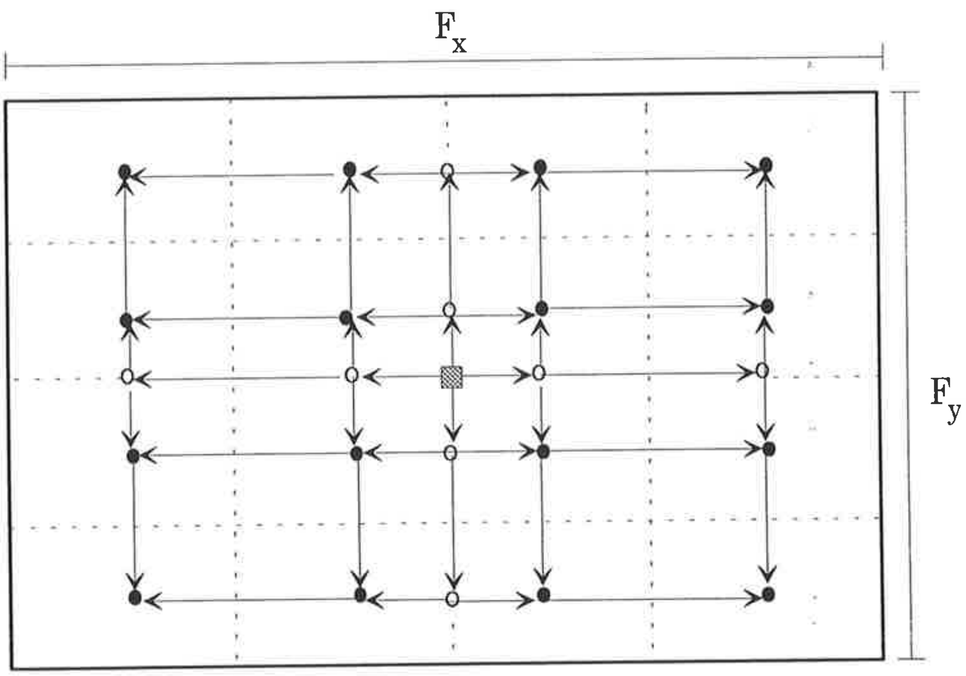


Fig. 8.12b A typical network with 16 demand nodes resulting from the extension of Fig. 8.12a

TABLE 8.3 Demand nodes and corresponding links in the network with 20 nodes

Node number	start node	end node	pipe number	length (m)	bit number (for sizing)
1	26	1	[10]	50	21
	24	1	[6]	30	
2	1	2	[11]	100	22
	23	2	[7]	30	
3	2	3	[12]	100	23
	22	3	[8]	30	
4	3	4	[13]	100	24
	21	4	[9]	30	
5	27	5	[19]	50	25
	1	5	[15]	60	
6	5	6	[20]	100	26
	2	6	[16]	60	
7	6	7	[21]	100	27
	3	7	[17]	60	
8	7	8	[22]	100	28
	4	8	[18]	60	
9	28	9	[28]	50	29
	5	9	[24]	60	
10	9	10	[29]	100	30
	6	10	[25]	60	
11	10	11	[30]	100	31
	7	11	[26]	60	
12	11	12	[31]	100	32
	8	12	[27]	60	
13	29	13	[37]	50	33
	9	13	[33]	60	
14	13	14	[38]	100	34
	10	14	[34]	60	
15	14	15	[39]	100	35
	11	15	[35]	60	
16	15	16	[40]	100	36
	12	16	[36]	30	
17	30	17	[46]	50	37
	13	17	[42]	60	
18	17	18	[47]	100	38
	14	18	[43]	60	
19	18	19	[48]	100	39
	15	19	[44]	60	
20	19	20	[49]	100	40
	16	20	[45]	60	

Note: The numbering of nodes and pipes is shown in Figure 8.4

For a fixed size (area) of field as the number of divisions or the number of demand nodes increases the size of subunits, the value of the required demand at nodes and also the length of laterals and manifold in each subunit will decrease. This may affect the total system cost due to the changes of flow rate in delivery pipes and also in micropipes. In the current model, the decode procedure differentiates between the application of the GA to branched networks with a different number of demand nodes with different configuration. The structure of the decode procedure for connectivity depends on how the nodes, links and the source node are formulated. For example, Table 8.3 tabulates the relationship between nodes, links and bit numbers on the basis of the original network shown in Figure 8.4.

The dimensions of the subunits in both examples used in the case study were obtained considering the number of divisions in the X and X directions as tabulated in Table 8.2. The value of demand at nodes was computed on the basis of agro-technical and agronomical considerations. This depends greatly on various parameters. For example, for a vineyard in a medium soil in South Australia some typical parameters used in the case study are summarised in Table 8.4.

TABLE 8.4 Parameters used in the case study

Soil parameters	Crop parameters	Emitter parameters	Operating parameters	Others
FC=25 %	R = 1 m	d_x = 1.5 m	N_{sh} = 4	ET_c= 6.5 mm/day
PWP=11%	P_w = 0.45	d_y = 2 m	T_d = 22 hrs	E_a = 0.95
I_{soil} = 6 mm/day	f = 0.50	ne = 1	T_{off} = 1 day	d_{an} =600 mm
γ_s= 1.3 g/cm^3	crop: vineyard	emitter: orifice		

The other parameters used in the case study are: depth of water table =20 m; expected life of project =12 years; interest rate =10%; pump efficiency =72%; electrical energy cost = \$0.09/Kwh; probability of crossover = 80% and probability of mutation=2%.

8.10 RESULTS AND DISCUSSIONS

The model OPDESGA was used to optimise two multiple subunit drip irrigation systems with 16 and 80 subunits (respectively). In both examples all agro-technical, agronomical and system parameters are given in Table 8.4. The main genetic algorithm parameters including the probability of crossover and mutation and the size of population were chosen by a limited sensitivity analysis. The results concerning both networks and the final optimum solution using the two mentioned selection methods (proportionate and tournament selection) are discussed in the following sections.

8.10.1 Network with 16 Subunits

As discussed previously, the formulation of the GAs for a network with 16 demand nodes was based on a network with 4 demand nodes. In such a network the first segment of the string dealing with layout consists of 4 bits each associated with one node. On the other hand, for each link an appropriate pipe size should be assigned. As a result, another 8 numbers are required, 4 members for the links supplying demand nodes and the remaining members for the sizes of the mainline pipes which satisfy the feasibility of network. Consequently, the basic length of each string is 12. The GAs generate a set of binary and integer numbers randomly for the first and second segments of each string in the initial population. In the subsequent process the crossover and mutation operations will be carried out on the selected strings as shown in Figure 8.13. In this way, a new set of strings (new population) which inherit some characteristics from parent strings are generated, each showing a trial solution for the pipe network with 16 demand nodes.

8.10.2 Application of Crossover and Mutation

Crossover and mutation as two operators of GAs were discussed comprehensively in the Chapter 6. In the current model, a single point crossover was utilised. Two individual chromosomes were cut at some randomly chosen position with probability of Pc. In this way, two "head" segments, and two "tail segments were produced.

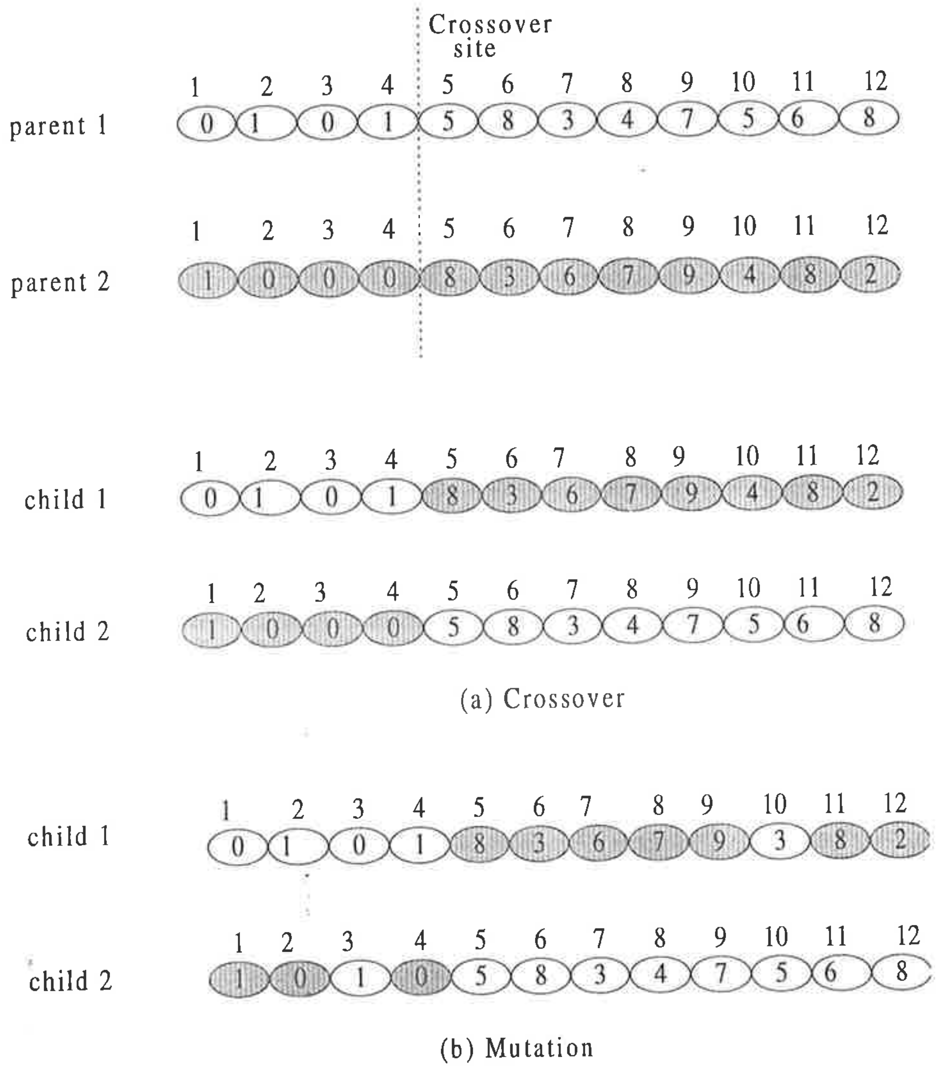


Fig. 8.13 Crossover and mutation applied to strings associated with the network of 16 demand nodes

The tail segments were then swapped over to produce two new full length chromosomes. Consequently, two offspring (new individuals), each inheriting some genes from each parent were produced. Mutation was applied to each offspring (child) after a random change of gene with a small probability. Figure 8.13(b) shows the 10th gene from child 1 and the 3rd gene from child 2 have been mutated.

To find the least cost solution within the search space a limited sensitivity analysis for GA parameters was carried out. This analysis increases the possibility that there will be no solution with lower cost for the examined parameters. Both models (with proportionate and

TABLE 8.5a Least cost solutions for values of population sizes (tournament selection, Pc=0.8, Pm= 0.02, Gen=200; seed= 1000, 16 subunits)

Population size	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
50	396,537	1,240	16.7 (Gen=400)
80	396,537	3,280	11.4
100	392,339	6,560	13.1
120	**392,089**	**4,992**	**15.3**
150	396,537	2,520	18.5

TABLE 8.5b Least cost solutions for values of Pc (tournament selection, pop=120, Pm= 0.02, Gen=200; seed= 1000, 16 subunits)

Probability of crossover	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
0.9	392,339	7,236	15.1
0.8	**392,089**	**4,992**	**15.3**
0.7	392,339	2,604	15.2
0.6	396,537	5,680	15.3
0.5	394,283	3,540	15.1

TABLE 8.5c Least cost solutions for values of Pm (tournament selection, pop=120, Pc= 0.8, Gen=200; seed=1000, 16 subunits)

probability of mutation	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
0	396,805	864	15.2
0.001	**392,089**	**6,624**	**15.3**
0.002	393,321	1,920	15.3
0.02	**392,089**	**4,992**	**15.3**

TABLE 8.5d Least cost solutions for different runs (tournament selection, pop=120, Pc= 0.8, Pm=0.001, Gen=200; 16 subunits)

seed number	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
100	392,339	2,592	15.3
500	397,892	2,208	15.2
10,000	**392,089**	**20,604**	**15.2**

tournament selection) were examined with a number of population sizes. In each case the probabilities of crossover and mutation were kept constant (Table 8.5a). For the population size which lead to the minimum cost, the probability of crossover was varied. This was carried out for a constant value of population size and mutation probability (Figure 8.5b). In the third stage of evaluation, for the selected values of population size and crossover probability which led to the minimum cost, the mutation operator was also varied (Table 8.5c). At the end, the models was again examined for the selected GA parameters but using different random seed numbers. Using various seed numbers changes the sequence of random numbers for the entire simulation. The findings show that the same minimum cost solution was obtained with the two random seeds of 1000 and 10,000 (Table 8.5d) The results summarised in Tables 8.5a to 8.5d show that the least cost solution using tournament selection is obtained at a value of $392,089 when a population size of 120, probability of crossover of 0.8 and probability of mutation of 0.02 or 0.001 are used. As the solution corresponding to a cost of $392,089 occurred several times, it would appear to be close to the global optimum.

The model was written in Pascal, running under Unix on a mainframe machine (DECstation 5000/240). It took 4.2 seconds (user time plus the system time) to find the minimum cost solution after 4992 evaluations. However, the runtime shown in Tables 8.5a to 8.5d is that for all generations being evaluated rather than for the time taken until the lowest cost solution is obtained. The given time is largely independent of the load on the system because it is the sum of the system time and user time not the elapsed time. The optimum solution which is most probably the global optimum or very close to it (with string and layout presentation) is shown in Figure 8.14.

The variation of least cost solutions in each generation using tournament selection is shown in Figure 8.15. This curve shows that there is a good improvement in the first generations. The best cost curve drops very quickly and converges towards its minimum level. It then remains constant without further improvement. Since in tournament selection the best cost curve reaches its minimum level after a number of evaluations and remains steady it allows the use of less number of evaluations to reach the optimum. When it approaches to its minimum level and remains constant there would be no point to have further evaluations.

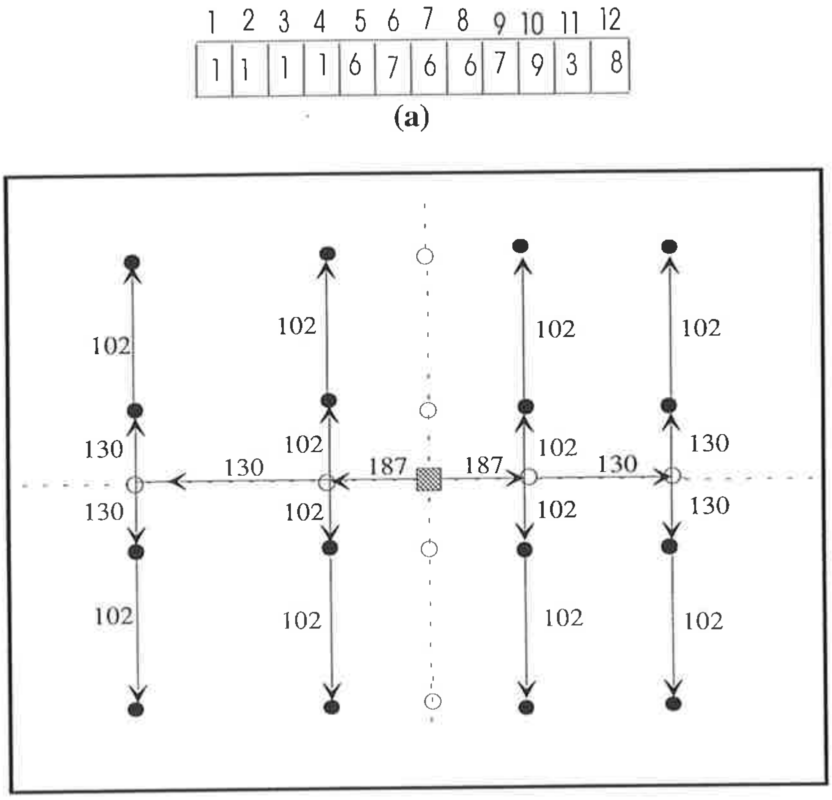


Fig. 8.14 The most fit coded string (a) and the corresponding optimum design (b) for a network with 16 demand nodes (tournament selection)

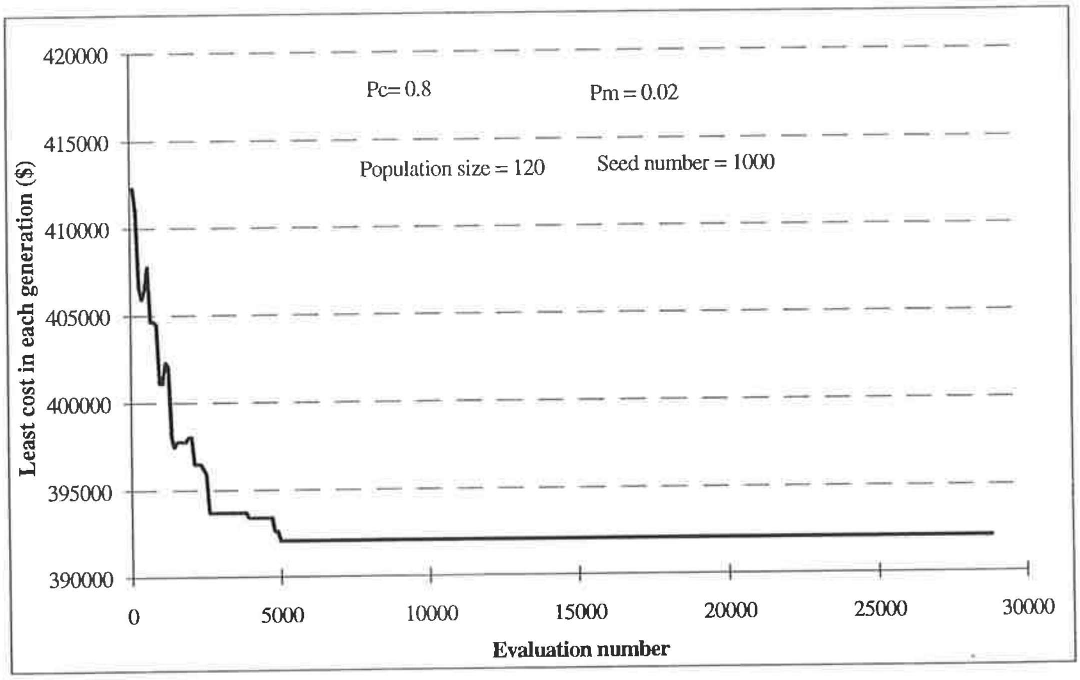


Fig. 8.15 The cost variation of the minimum cost solutions for the network with 16 subunits using tournament selection

For the subunit size shown in Table 8.2 (3 ha) and also the input data tabulated in Table 8.4, the results associated with the optimum solution shown in Figure 8.14 are summarised in Table 8.6. The model with proportionate selection was examined using the same example (a network with 16 demand nodes). In this case, a limited analysis to select the more appropriate GA parameters was also carried out.

All results corresponding to different values of population sizes, the probabilities of crossover and mutation are summarised in Tables 8.7a to 8.7d. The results indicate that the least cost solution is obtained at a value of $392,339 by using appropriate GA parameters as given in Tables 8.7a to 8.7d. This solution was achieved by using population size of 400 and probabilities of 0.9 and 0.002 for crossover and mutation respectively.

TABLE 8.6 Details associated with the optimum cost solution (tournament selection, 16 subunits)

cost component (items)	($)	subunit information		operating information	
pump	17,807	L_{ss}	78.8m	F	5 days
pipes	39,973	L_{bs}	19.7m	T	22 hrs
subunits	299,320	L_m D_m	148m 84mm	P_w	44 %
P.V. of annual operation	32,229	Q_E	4.9 L/hr	I_{app}	1.64 mm/hr
accessories	2,760	Q_{su}	13.6 L/s	N_{sh}	4
TOTAL	**392,089**	n_e	9900	$^*A_{sh}$	12 ha

*area under each shift irrigation (4 subunits)

pump head = 51m; pump power= 37.8 Kw; pump discharge = 54.6 L/s

TABLE 8.7a Leat cost solutions for values of population sizes (proportionate selection Pc=0.8, Pm= 0.02, Gen=200; seed= 1000, 16 subunits)

Population size	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
120	398,011	16,800	16.1
150	398,488	11,880	19.6
200	398,646	8,000	25.4
300	398,011	7,920	40.0
400	**394,010**	**10,560**	**53.7**

TABLE 8.7b Least cost solutions for values of Pc (proportionate selection pop=400, Pm= 0.02, Gen=200; seed= 1000, 16 subunits)

Probability of crossover	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
1.0	397,892	3,400	131.0
0.9	**393,901**	**4,680**	**132.2**
0.8	394,010	10,560	134.2
0.7	396,480	5,320	136.1
0.5	395,981	2,400	133.4

TABLE 8.7c Least cost solutions for values of Pm (proportionate selection pop =400, Pc= 0.9, Gen=200; seed=1000, 16 subunits)

probability of mutation	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
0	393,947	19,080	137.5
0.001	392,887	70,560	131.0
0.002	**392,339**	**4,320**	**131.4**
0.02	393,901	4,992	132.2

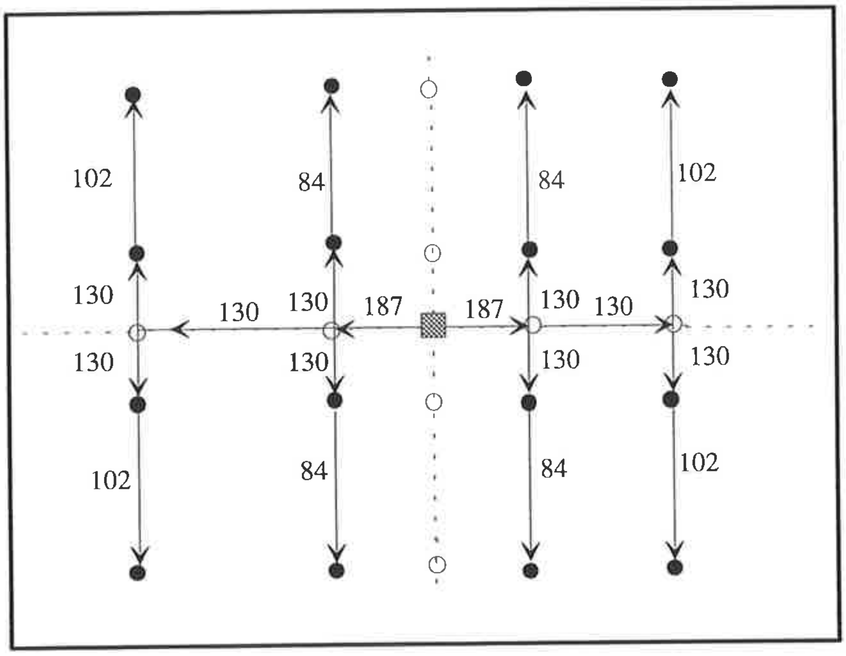
TABLE 8.7d Least cost solutions for values of runs (proportionate selection pop =400, Pc= 0.9, Pm=0.002, Gen=200; 16 subunits)

seed number	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
100	397,350	16,560	131.2
500	395,958	10,800	131.6
1000	**392,339**	**4,320**	**132.2**
10,000	393,321	15,480	131.2

The findings show that the GA parameters that lead to the minimum cost solution are different from those used when tournament selection was tested (Tables 8.5a to 8.5d). However the overall costs of both optimum solutions are within 0.1%. Comparison of Figures 8.14 and 8.16 showing the layout and pipe sizes of both solutions indicates that layout of pipes in both solutions is the same. The only differences are: the size of pipe number [4] which was found 102 mm by the first model and 130 mm by the second one, and pipe [9], 102 mm and 84 mm respectively. The optimum solution found using tournament selection is just $250 less cost than using proportionate selection. The minimum cost solution was obtained after 4320 evaluations. The runtime required to reach this optimum using proportionate selection was 3.6 seconds which is close to the time needed to reach the optimum by using tournament selection. The coded presentation and also the layout with pipe sizes corresponded to the minimum cost solution using proportionate selection are shown in Figure 8.16.

1	2	3	4	5	6	7	8	9	10	11	12
1	1	1	1	7	7	5	6	7	9	8	6

(a)



(b)

Fig. 8.16 The most fit coded string (a) and the corresponding optimum design (b) for a network with 16 demand nodes (proportionate selection)

The variation of least cost curve corresponding to proportionate selection is shown in Figure 8.17. At the start the decrease in cost for the best result is very sharp and very quickly it reaches its minimum level of $392339 after 4320 evaluations. It then increases considerably and varies in an unsteady manner with increasing evaluations. The model was allowed to run up to 500 generations (180,000 evaluations) but no improvement was observed even in four different runs. It appears that this low cost solution which is considerably less than the other minimum cost solutions could be due to the effect of the mutation operator. Details of optimum design solution are summarised in Table 8.8.

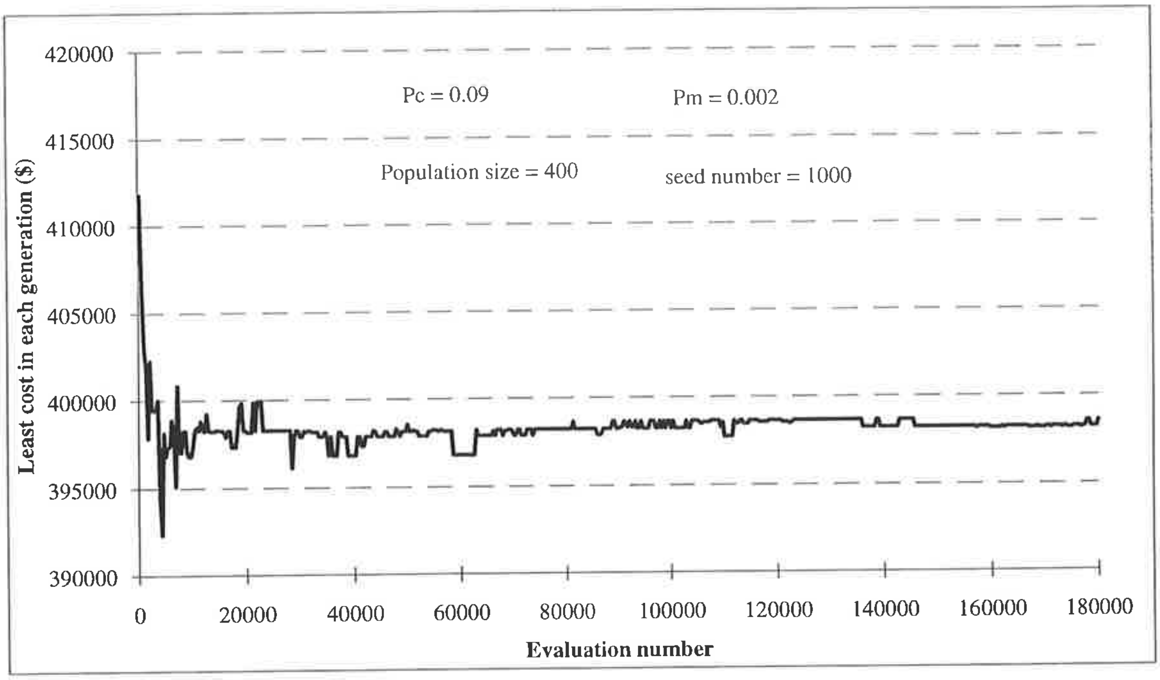


Fig. 8.17 The cost variation of the minimum cost solutions for network with 16 subunits using proportionate selection

TABLE 8.8 Details associated with the optimum cost solution (proportionate selection 16 subunits)

cost components (items)	($)	subunit information		operation information	
pump	17,915	L_{ss}	78.8 m	F	5 days
P.V of annual operation	32,443	L_{bs}	19.7 m	T	22 hr
pipes	39,901	L_m D_m	148 m 84 mm	P_w	44 %
subunits	299,320	Q_E	4.9 L/hr	I_{app}	1.64 mm/hr
accessories	2,760	Q_{su}	13.6 L/s	N_{sh}	4
TOTAL	**392,339**	n_e	9900	A_{sh}	12 ha

pump head = 51 m; pump power = 38 Kw; pump discharge = 54.6 L/s

8.10.3 Networks with 80 Subunits

In a similar way to the previous example, the model was applied to a network with 80 demand nodes as well. As stated earlier in the current model only one quarter of the demand nodes (subunits) are considered in the GAs process. The overall configuration of major pipes and micropipes within subunits are similar to the previous model. A similar principle with the same input data used in previous model is implemented in this model as well. Although the size of population and generation have a significant effect on the running time, the number of demand nodes in the original network also has a great effect. As the number of demand nodes increases the number of possible solutions increases dramatically. For example, for just the layout problem there are only 16 (2^4) possible solutions for the first example (with 16 demand nodes) and 1,048,576 (2^{20}) solutions for the current example (with 80 demand nodes). However, when the full design (layout and component sizes) are considered the number of possible alternative solutions increases to a huge number.

For instance, there are at least 9.45×10^5 ($2^4 \times 9^5$) possible alternative solutions for the first example with 9 available pipe sizes (one link directed to each node plus at least one link connected to the source node). The search space for the network with 20 demand nodes and the same number of available pipe sizes would be at least a size of 1.15×10^{26} ($2^{20} \times 9^{21}$). This size of search space needs much greater time to find the optimum solution compared with the first model. In a similar manner to the previous model, a limited sensitivity analysis for GA parameters was carried out to select the most appropriate parameter values for the genetic algorithm operators. The results concerning this analysis for both selection methods (tournament and proportionate) are given in Tables from 8.9a to 8.9d and 8.11a to 8.11d respectively.

It can be concluded from the results of analysis given in Tables 8.9a to 8.9d that the appropriate GA parameters leading to least cost solution (within the range of analysis) are 300, 0.5 and 0.002 for the population size, crossover and mutation probabilities (respectively). Using these GA parameters the best cost solution was obtained with a cost of $390,169. However, a further analysis was carried out to check that there is no solution with a lower cost than those already found. In this process a better result was achieved. An optimum solution which is probably very close to the global optimum was obtained at a cost of $387,769. This solution was found using a population size of 400 and probabilities of 0.9 and 0.01 for crossover and mutation respectively. It was achieved after 45,000 evaluations in generation 125. The result (corresponding to this solution) shows a good consistency with the results obtained for the layout problem discussed in Chapter 7 (Figure 7.28). To evaluate how the best cost solution in each generation varies, it was plotted against the evaluation number as shown in Figure 8.18. The best cost curve shows a significant improvement in the first few generations. As Figure 8.18 indicates the cost of the best solution in each generation drops sharply from around $760,000 in the first few generations to around $400,000. It then reaches a minimum of $387,769 after 45,000 evaluations and then remains almost constant in the subsequent generations. The details of this optimum solution are listed in Table 8.10. The coded presentation with layout and pipe sizes associated with this optimal solution are shown in Figure 8.19 (a and b).

TABLE 8.9a Least cost solutions for values of population size (tournament selection Pc=0.8, Pm= 0.02, Gen=500; seed= 1000, 80 subunit)

Population size	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
100	401,788	21,280	199.8
200	400,261	48,640	350.3
300	**399,657**	**76,080**	**523.0**
400	400,219	48,960	713.5
500	400,681	121,600	864.0
600	401,477	120,000	1032.0

TABLE 8.9b Least cost solutions for values of Pc (tournament selection pop=300, Pm= 0.02, Gen=500; seed= 1000, 80 subunits)

Probability of crossover	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
1.0	399,611	96,300	564.4
0.9	400,924	54,810	543.8
0.8	399,657	76,080	523.0
0.7	399,372	81,480	536.1
0.5	**397,592**	**48,750**	**538.0**

TABLE 8.9c Least cost solutions for values of Pm (tournament selection pop =300, Pc= 0.5, Gen=500; seed=1000, 80 subunits)

probability of mutation	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
0.00	409,398	54,810	520.8
0.001	395,221	27,750	522.4
0.002	**390,169**	**21,750**	**511.6**
0.02	397,592	48,750	536.1

TABLE 8.9d Least cost solutions for different runs (tournament selection pop =300, Pc= 0.5, Pm=0.002, Gen=500; 80 subunits)

seed number	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
100	391,221	21,750	534.0
1000	**390,169**	**27,900**	**523.0**
10,000	**390,169**	**21,750**	**541.3**

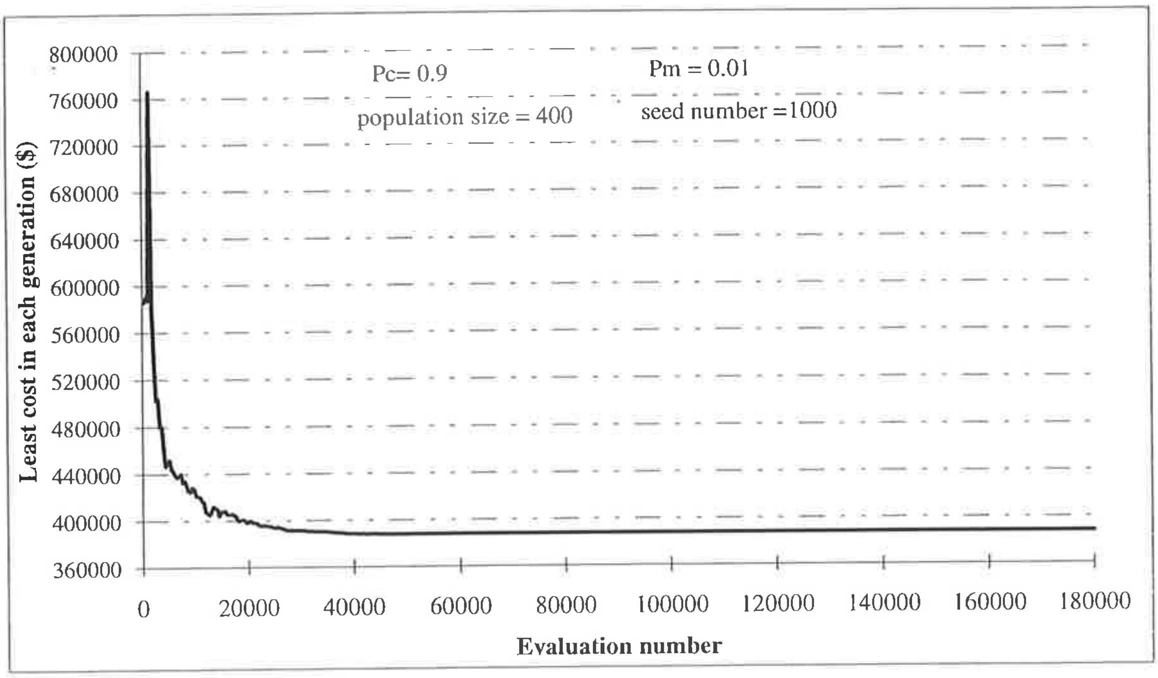
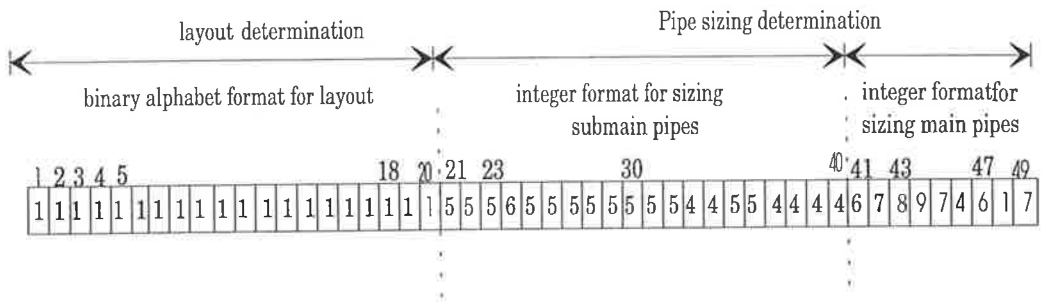


Fig. 8.18 The variation of least cost solution for a multiple subunit drip irrigation system with 80 subunits (tournament selection)

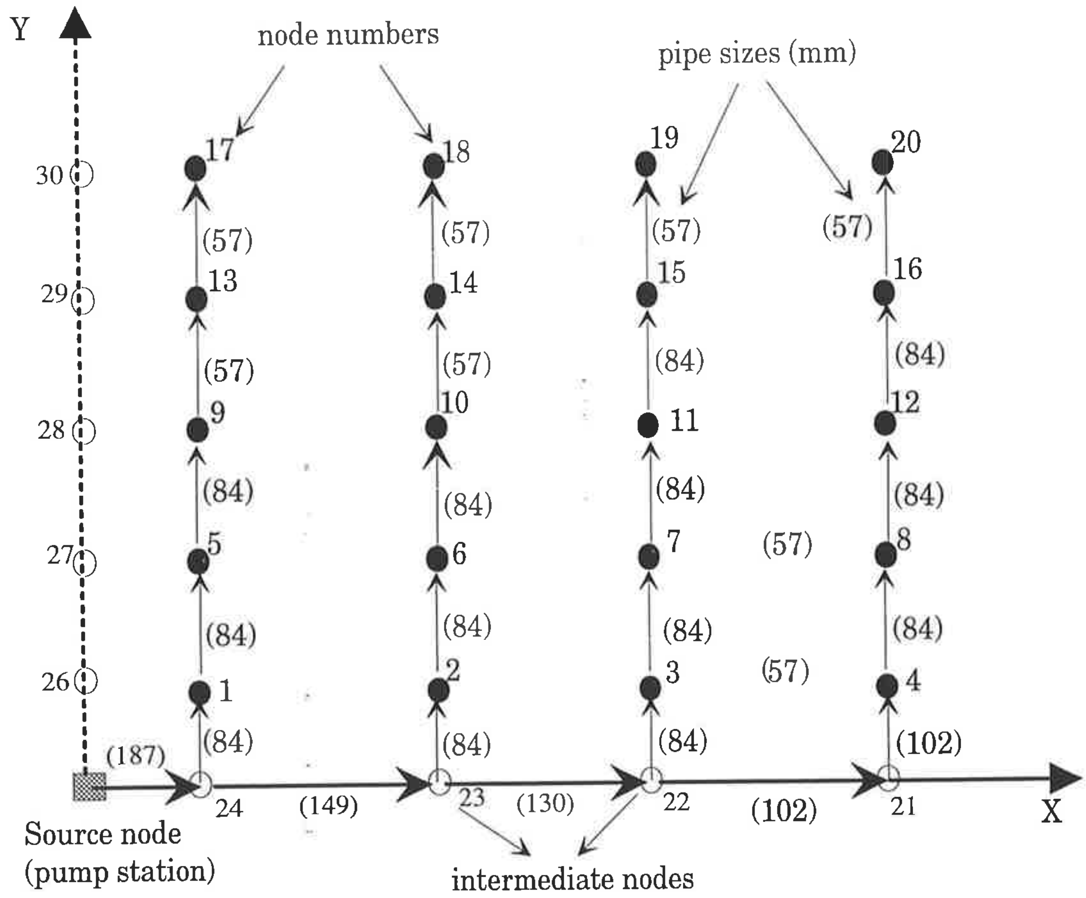
TABLE 8.10 Details associated with the minimum cost solution (tournament selection, 80 subunits)

cost components (items)	($)	subunit information		operation information	
pump	18654	L_{ss}	48.5 m	F	5 days
P.V of annual operation	32,572	L_{bs}	0.00 m	T	22 hr
pipes	50,717	L_m D_m	58 m 35 mm	P_w	44 %
subunits	280,066	Q_E	4.9 L/hr	I_{app}	1.64 mm/hr
accessories	5,760	Q_{su}	2.73 L/s	N_{sh}	4
TOTAL	**387,769**	n_e	2000	A_{sh}	12 ha

pump head = 53.43m; pump power = 38Kw; pump discharge = 54.6 L/s



(a)



(b)

Fig. 8.19 The most fit coded string (a) and the corresponding solution with layout and pipe sizes (b) for a network with 20 demand nodes (tournament selection)

The least cost was identified within a small fraction of the total search space. It took approximately 125 seconds (2.6 minutes) of the total CPU time (user time + system time) to run on a DEC server 5000/240 with MIPS R3000 processor. The number of alternative solutions evaluated by the GAs was only a small fraction of the whole search space. As an example, consider the solution space of 1.15×10^{26} network designs represented by the total land area of Iran which is 1.648×10^{6} square kilometres. The GAs search of 45,000 evaluations for the network with 80 demand nodes is equivalent of investigation of about 0.0000065 square centimetres of the total area of Iran. This represents how small is the size of the search space which has been evaluated by the GAs. The final design of the multiple subunit system which is the extension of Figure 8.19 is illustrated in Figure 8.20. This is the same layout that was obtained using the model discussed in Chapter 7.

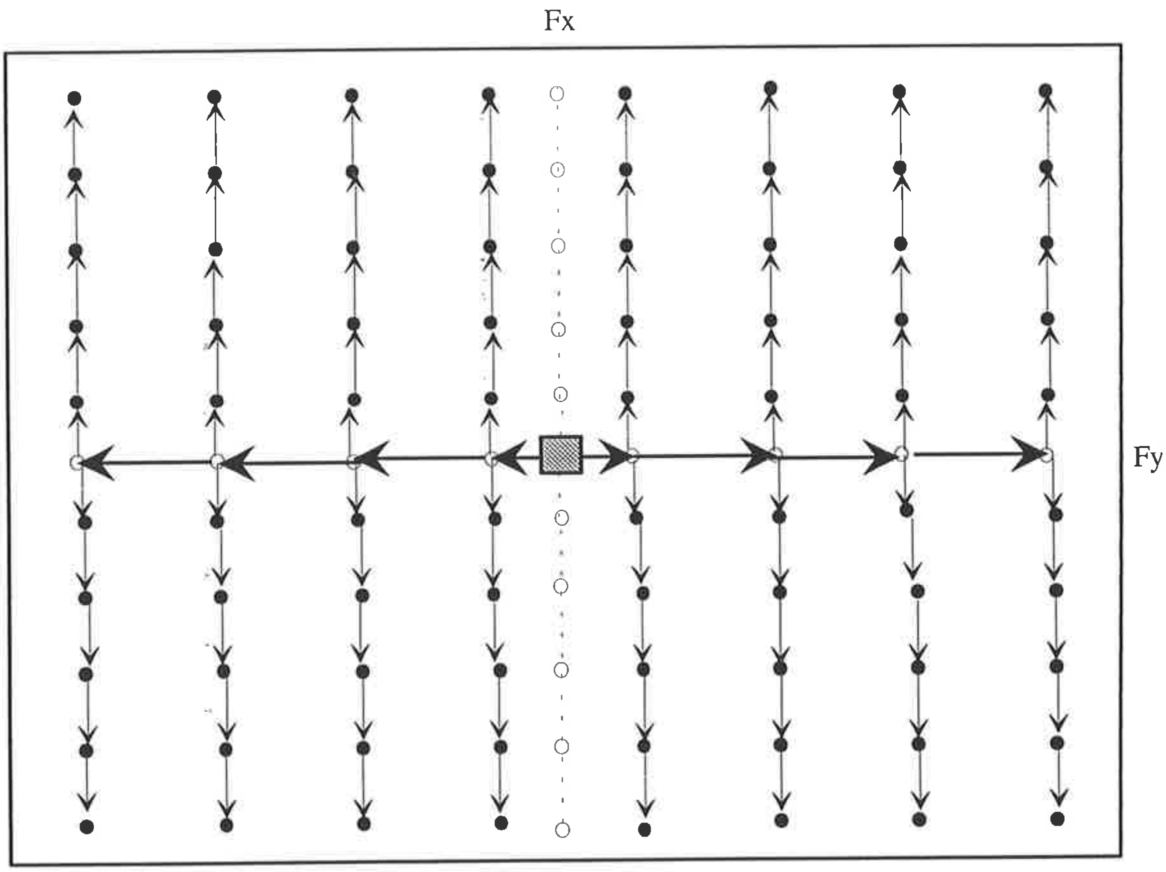


Fig. 8.20 The optimum layout of least cost solution associated with the network of 80 demand nodes using tournament selection

A similar process to find appropriate GA parameters was carried out for the model when the proportionate selection is used. The results are summarised in Tables 8.11a to 8.11d. As shown in Tables 8.11a to 8.11d the minimum cost solution using proportionate selection is achieved using a population size of 500 and probabilities of 0.8 and 0.01 for the crossover and mutation operators. The variation of the least cost solutions against the number of evaluations is illustrated in Figure 8.21. Although at the beginning rapid improvement is made, in the subsequent generations improvement is slow. In fact, the minimum cost changes in an erratic fashion with very slow improvement. Finally the least cost solution was identified after 106,000 evaluations at a cost of $406759 (in the 265th generation).

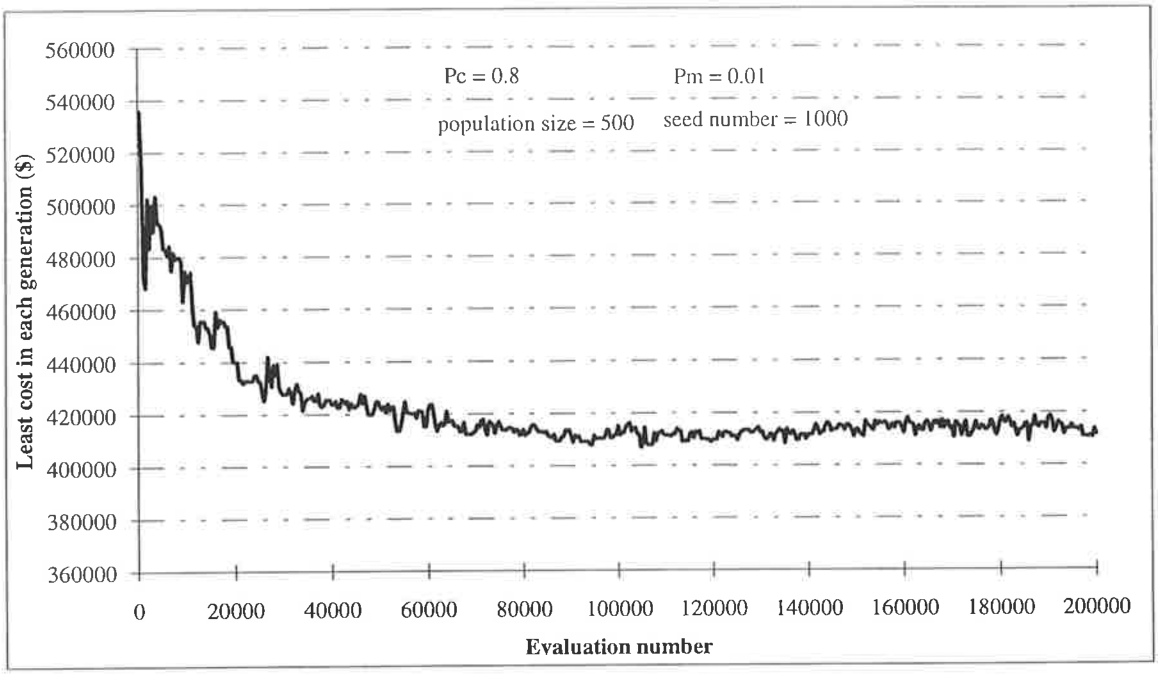


Fig. 8.21 The variation of least cost solution for a multiple subunit drip irrigation system with 80 subunits (proportionate selection)

The coded string and the layout with pipe sizes corresponding to the least cost solution are shown in Figure 8.22. A comparison between the layout configuration found by the model using two different selection methods indicates that installing all pipes parallel to the Y axis (Fig. 8.19b) or parallel to the X axis (Fig. 8.22b) may produce the minimum cost. The fact that some lines decrease and then increase in size in Figure 8.22 suggests that lower cost solutions are possible for the pipes aligned parallel to the X axis.

TABLE 8.11a Least cost solutions for values of population sizes (proportionate selection Pc=0.8, Pm= 0.02, Gen=500; seed= 1000, 80 subunit)

Population size	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
100	415,717	37,520	202.6
200	417,539	176,000	377.0
300	413,914	93,600	558.3
400	414,572	106,560	743.0
500	**410,066**	**178,800**	**945.3**
600	410,511	188,400	1130.8

TABLE 8.11b Least cost solutions for values of Pc (proportionate selection pop=500, Pm= 0.02, Gen=500; seed= 1000, 80 subunits)

Probability of crossover	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
1.0	411,732	110,000	948.8
0.9	413,631	151,200	939.8
0.8	**410,066**	**178,800**	**945.3**
0.7	413,626	64,000	929.3
0.5	413,871	215,600	920.0

TABLE 8.11c Least cost solutions for values of Pm (proportionate selection pop =500, Pc= 0.8, Gen=500; seed=1000, 80 subunits)

probability of mutation	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
0.00	465,351	83,200	945.3
0.001	409,182	196,000	905.0
0.002	414,470	182,400	898.0
0.01	**406,759**	**106,000**	**943.3**
0.02	410,066	178,800	921.8

TABLE 8.11d Least cost solutions for different runs (proportionate selection pop =500, Pc= 0.8, Pm=0.01, Gen=500; 80 subunits)

seed number	least cost ($)	number of evaluations to reach the least cost	time for evaluation of each run (seconds)
100	409,453	154,000	906.2
1000	**406,759**	**106,000**	**943.3**
10,000	410,008	125,200	925.1

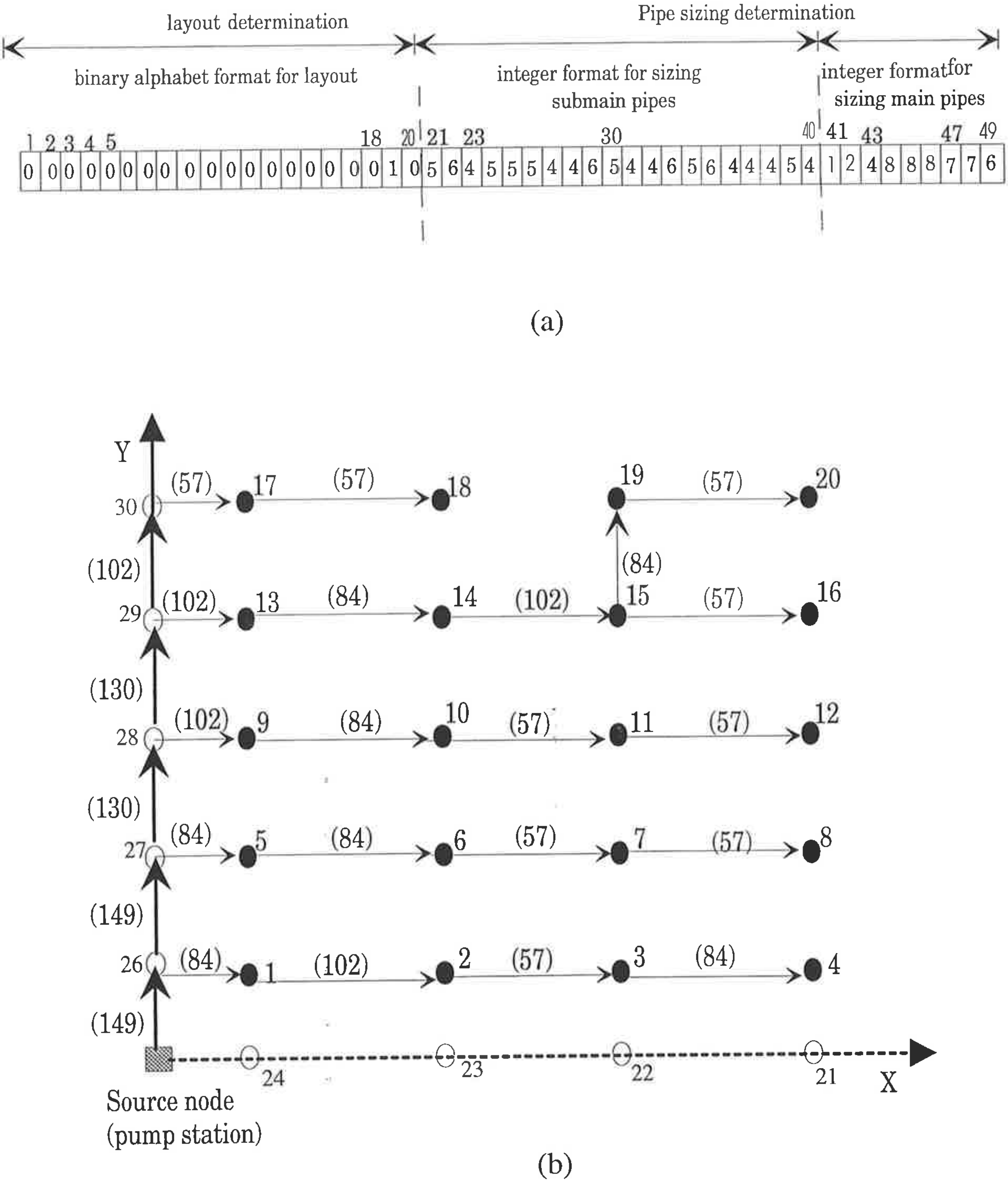


Fig. 8.22 The most fit coded string (a) and the corresponding solution with layout and pipe sizes (b) for a network with 20 demand nodes (proportionate selection)

The optimum solution found by tournament selection is $18,990 (4.67%) cheaper than the minimum cost solution found by proportionate selection. However, it appears that apart from having all pipes in the Y direction which corresponds to the optimal solution, the arrangement of pipes parallel to the X direction could also produce the low cost solutions. Detail related to the minimum cost solution found by proportionate selection (Fig. 8.22) is given in Table 8.12. The variation of the least and the average cost of solutions in each generation for both selection methods are represented in Figures 8.23 and 8.24. The average cost for both selection methods shows a very unstable variation. In the case of proportionate selection there is a significant difference between the least cost and the average cost curves, but there is more variation the average costs using tournament selection.

A typical set of examples resulted from the model showing a number of chromosomes with corresponding information including: number of generations, evaluations, least cost and average cost are given in Appendix J.

TABLE 8.12 Details associated with the optimum cost solution (proportionate selection, 80 subunits)

cost components (items)	$	subunit information		operation information	
pump	19,759	L_{ss}	48.5 m	F	5 days
P.V of annual operation	34,346	L_{bs}	0.00 m	T	22 hr
pipes	66,228	L_m D_m	58 m 35 mm	P_w	44 %
subunits	280,066	Q_E	4.9 L/hr	I_{app}	1.64 mm/hr
accessories	6,360	Q_{su}	2.72 L/s	N_{sh}	4
TOTAL	**406,759**	n_e	2000	A_{sh}	12 ha

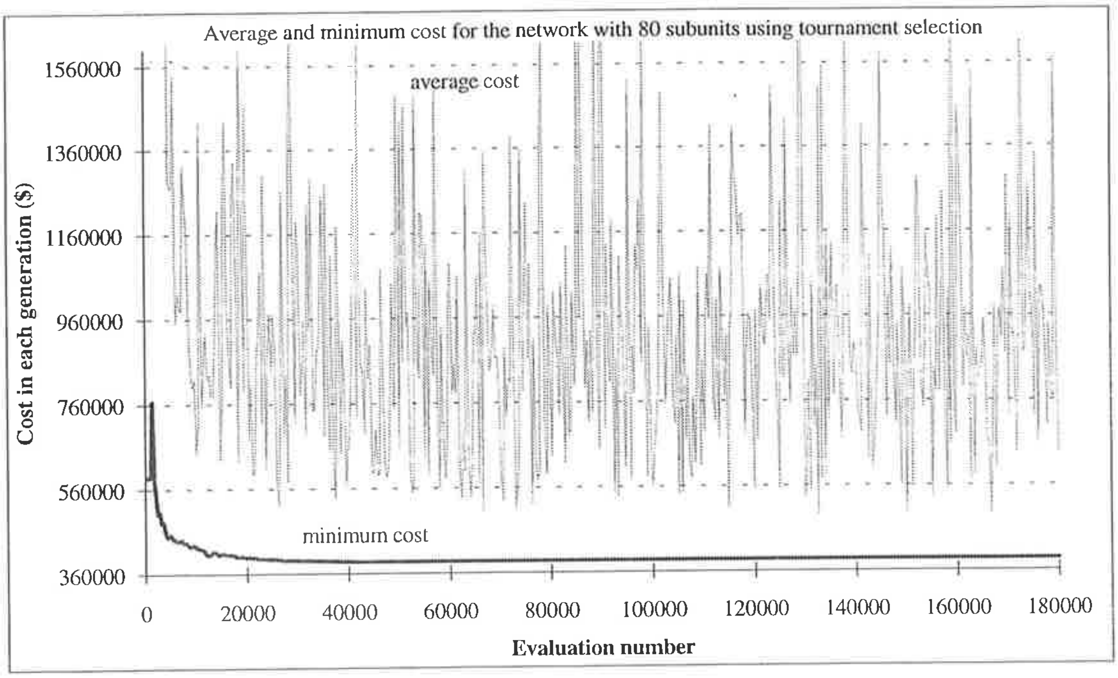


Fig. 8.23 The least and average cost in each generation of a multiple subunit drip irrigation system with 80 subunits (tournament selection)

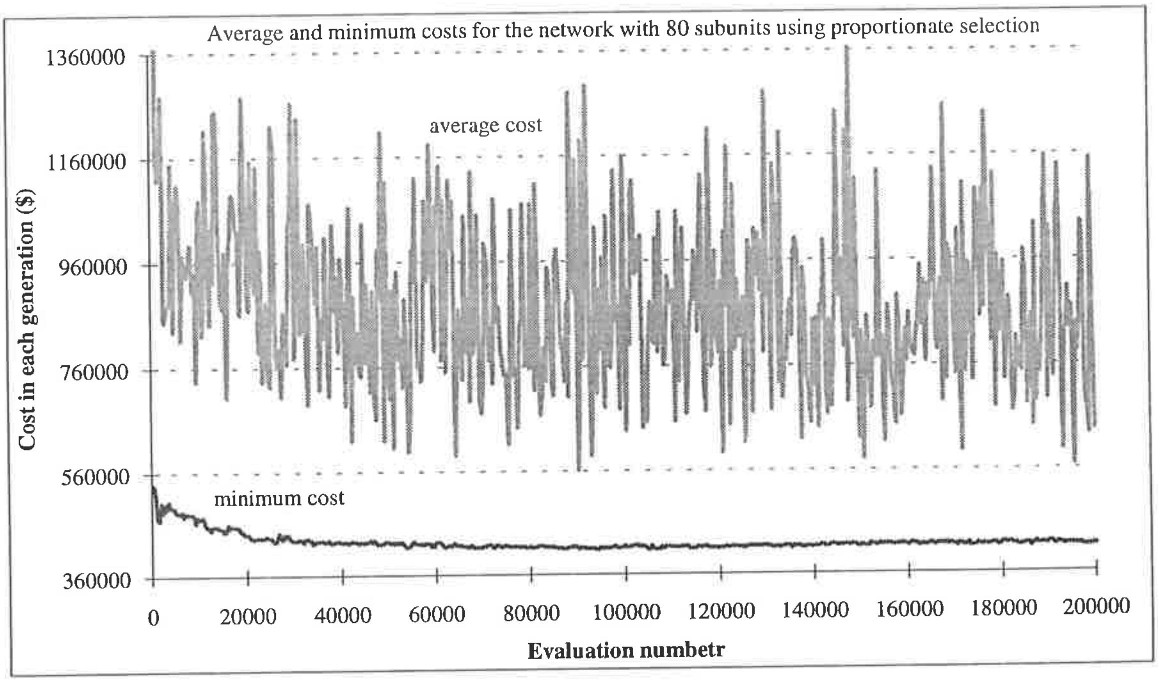


Fig. 8.24 The least and average cost of a multiple subunit drip irrigation system with 80 subunits (proportionate selection)

8.11 SUMMARY

A model ("OPDESGA") was developed for the full optimal design of multiple subunit pressure irrigation systems. Originally it was formulated for a branched pipe network in which demand nodes are located in a rectangular pattern (Fig. 8.4). It has then been extended to a multiple subunit pressure irrigation system in which the source node is located at the centre (Fig. 8.1). The layout and component sizes, including the major pipes and the micropipes (within the subunits) are considered as decision variables in this model. Selection of both the connectivity between nodes (layout) and also the component sizes leads to a complex procedure for the optimum design. Because the connectivity in pipe branched networks is not known initially, the flow in each pipe is unknown at the beginning of the optimisation process. To overcome this, a method based on backtracking movement which proceeds from the terminal nodes towards the source node was developed.

GAs were utilised as the main optimisation approach for the layout and major pipe sizes and complete enumeration was used to size the micropipes within each subunit. The results from enumeration were then passed to the GA to be included in the objective function for fitness computation. In the decoding function, the real parameters such as links between nodes and the size of the major pipes (main and submain) were coded as binary and integer numbers respectively. The length of strings (chromosomes) consists of two segments: a binary segment, representing the layout, and an integer segment, representing the size of links connecting the demand nodes to the source node (Figure 8.3). For each demand node two alternative links and for each link nine alternative discrete sizes were considered. Accordingly the size of the search space increases dramatically with an increase in the number of demand nodes. The model was applied to two branched pipe networks with 16 and 80 subunits respectively. Although the results corresponded to networks with 16 and 80 demand nodes, the original networks considered (using symmetry to reduce the sizes) consisted of 4 and 20 demand nodes respectively. This makes a significant reduction in the size of the search space and consequently, in the utilised computer memory and the run time. Indeed, GAs deal with a network of n demand nodes where the results obtained were extended to a multiple subunit system with $4n$ demand nodes. The size of solution space for a network of 20 demand nodes is 1.15×10^{26}, whereas only a very small fraction of search space was evaluated. A limited sensitivity analysis was carried out to select an appropriate value for each GA parameters. The process was applied to the model for each selection method (tournament and proportionate) separately for both cases studies.

Table 8.13 The final results of two networks with 16 and 80 subunits

Network	Selection method	Generation	Evaluation number	Least cost ($)	Time to reach optimum (s)
16 subunits	**Tournament**	**52**	**4992**	**392,089**	**4.2**
16 subunits	**Proportionate**	**12**	**4320**	**392,339**	**3.2**
80 subunits	**Tournament**	**125**	**45000**	**387,769**	**156.0**
80 subunits	**Proportionate**	**265**	**106000**	**406,759**	**475.4**

The least cost solutions associated with the four cases (two networks with two selection methods) are summarised in Table 8.12. The model was run on a mainframe machine DEC server 5000/240. The computer time required to find the optimum for each case is given in the same Table (Table 8.12). The findings provided evidence that tournament selection gives better results than proportionate selection. Although the model was allowed to run without changing any input data for both selection techniques the cost savings obtained using tournament selection was 4.66 %($18,990) for the 80 subunit system. However, the difference in cost for the first example (16 subunits) was not considerable ($250). The final solutions considering the optimum layout and pipe sizes for a network with 20 demand nodes(using the two GA selection methods) are shown in Figures 8.19 and 8.22. The overall finding related to the layout shows that when all pipes are laid in the Y direction the system would be more economical than when some are in the Y and some in the X directions. This shows a good consistency with the results obtained from the model discussed in Chapter 7. Also the result using proportionate selection shows that the second economical option would be the case that all pipes are laid in the X direction and lower cost solutions are possible..

Chapter 9

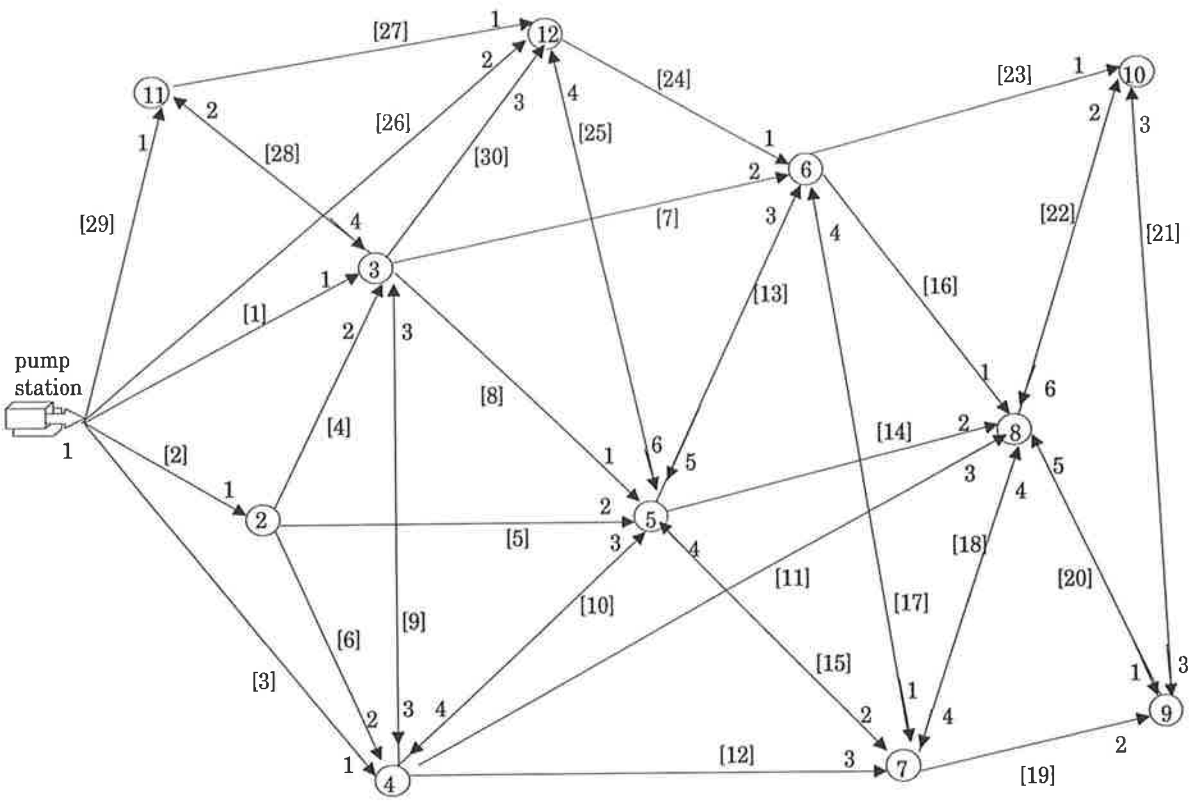
Optimal Layout, Pipe Sizing and Pump Selection for an Irregular Branched Network

9.1 INTRODUCTION

In this chapter, the model discussed in Chapter 8 is modified to apply to any irregular branched piping system. The new model "OPIRSYS" deals with a branched piping system with any configuration and different demands at nodes supplied from a single source node. In this part of the study, the model is formulated on the basis of the same principles applying to genetic algorithms (GAs) discussed in Chapters 7 and 8, but for irregular networks considering varying topography and various demands at nodes. In addition, the design of the pumping system is considered as a decision variables in the GA process. The model simultaneously searches for the optimum links between nodes, the optimum diameters for the pipes within a given range of discrete sizes and an appropriate pump speed from a given range for the feasible solutions to provide adequate pressure head at all demand nodes. To reach an optimum solution a trade off for the selected pump, pipe diameters and the connections between nodes is carried out. Two penalty costs are considered: one for connection feasibility and the other one for pressure violations in order to discourage infeasible solutions.

9.2 MAIN FEATURES OF THE MODEL

Since the research outlined in this chapter is an extension to the previous work discussed in Chapters 6, 7 and 8, it contains many of the features explained in those chapters plus the new characteristics which enable the model to be generalized to any branched piping system. It deals with a network with unknown layout consisting of a source node and a number of demand nodes supplied by a pumping system located at the source node. A typical example of such a network with possible alternative links between nodes is shown in Figure 9.1. The position of the source and demand nodes and also the elevations and demands at nodes are not limited as in the previous model. Hence the model is general and may be applied to any type of branched pipe system with one source node. A minimum design pressure head and a known demand must be specified at each node. On the basis of the total required head, a turbine pump with a number of different impeller speeds and known characteristic curves (which provides the required design flow rate) is considered for selection.



(1) = node number   [2] = pipe number   3 = coding of connection to each node

Fig. 9.1 A typical irregular pipe network used as a case study

The demand nodes shown in Figure 9.1 each may be a control head of a drip irrigation system supplying a subunit or farm, a sprinkler or a hydrant or any control valve for an urban water distribution system.

9.3 OPTIMIZATION TECHNIQUE

Genetic algorithms are utilized in this part of the study in similar way to the previous two chapters. A review of the principles and main features of this technique was carried out in Chapter 6 and its application to pipe networks was presented in Chapters 7 and 8. In Chapter 7, the GA strings represent just the layout of a branched pipe system with binary coded while in Chapter 8, each represents the layout and pipe sizes with binary and integer numbers. The strings constructed for the present model each consists of three segments showing the layout, pipe sizes and pump speed. A typical example of such strings for a network with 12 demand nodes is illustrated in Figure 9.2.

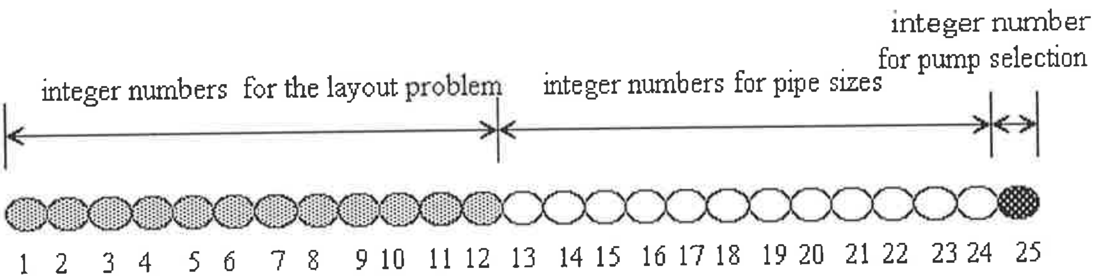


Fig. 9.2 A typical string for a network with 12 demand nodes for layout, pipe sizes and pump selection

Since the simple genetic algorithm is used the same procedures described in the previous chapters are applied to the genetic algorithm operators in this model. However, all bits of each string consist of integer numbers even for the layout segment which was represented by binary alphabet codes in Chapters 7 and 8. The range of integer numbers used in the first segment is limited to the maximum possible number of links that may potentially connect to each node. In the same way, the integer numbers allowed (to be generated) for the pipe diameters (second segment) depend on the number of available discrete pipe sizes. Each one corresponds to a pipe diameter as tabulated in Table 8.1. The third segment of strings consists of only one bit which contains an integer number and corresponds to a particular pump speed. An example is given in Table 9.1. In the GA process if a node does not satisfy the minimum required pressure the corresponding solution is discouraged by employing a penalty cost. The GA operators considered include: two selection schemes (tournament and proportionate), one-point crossover and a normal mutation with a low probability.

TABLE 9.1 Look up table showing pump impeller speeds and the corresponding integer numbers

integer number	pump speed rpm
1	2800
2	3000
3	3200
4	3400
5	3600

9.4 FORMULATION OF THE MODEL

On the basis of the position of nodes all reasonable connections between nodes are identified as shown in Figure 9.1. The model attempts to eliminate the redundant links in such a manner that each node is supplied by a single link. This is based on the fact that a treed network is the most efficient for a single demand loading case. In fact, in each iteration (cycle) of the genetic algorithm process a set of branched networks as potential solutions are identified to be selected for the further process. Converting a network with a high degree of connectivity to a branched network with only one link directed to each node is fulfilled by the generation of only one integer number for each bit. For example, in Figure 9.1, there are 4 possible pipes which may connect to node 3 (pipes [1], [4], [9], and [28]). These are given a coding of 1 to 4 in the GA. Only one of these pipes is allowed in the final solution. The allocation of a pipe size to a link is again based on the generation of a set of integer numbers each representing a specific pipe size as given in Table 8.1. Selection of an appropriate speed among the various given speeds for a known type pump is the third part of the process. This is carried out by generation of an integer number given in Table 9.1. Once the diameter of each existing link is identified the head loss in the pipe may be determined using a method similar to that discussed in Chapter 8. Obviously, the selected pipe sizes should satisfy the minimum pressure constraints. Selection of a very small pipe size (low cost pipe) for a link may cause a large head loss and therefore lead to an infeasible solution. These infeasible solutions are those in which one or more nodes do not satisfy the minimum required pressure head. A solution may also be infeasible if not all nodes are connected directly or indirectly to the source node. Once the links between nodes and the corresponding diameters are identified the hydraulic grade line (HGL) of nodes may then be computed. This process is based on the total head provided by the selected pump and the head loss within the pipes.

9.4.1 Total System Head (Pump Head)

Two alternative methods are used to determine the total required system head. The first method is similar to the process explained in Chapters 7 and 8. The second method which is based on the selection of an appropriate pump using the GA process is discussed in detail in the following section.

9.4.2 Total System Head through Selection of a Pump

In practice, pumps are available in discrete sizes with specific characteristic curves. Hence for any particular required head and discharge the most compatible pump should be selected for the system. The model discussed in this chapter is developed in such a manner to select the speed for a known type of pump. A pumping system will operate at the design discharge only if the exact total dynamic head is available. When more or less pressure head is available the system will discharge less or more water than the design discharge unless a pressure or a flow regulator is provided to regulate the flow rate. This can be seen from the sample pump characteristic curves shown in Figure 9.3.

In choosing a pump, the aim is select a unit that will operate at or near peak efficiency while supplying the design discharge at the total dynamic head of the irrigation system. Pump characteristic curves are a useful tool in this selection process. They show the range of head and discharge as well as the efficiencies at which the pump operates within that range (Keller and Bliesner, 1990). All pump characteristic curves are related to the

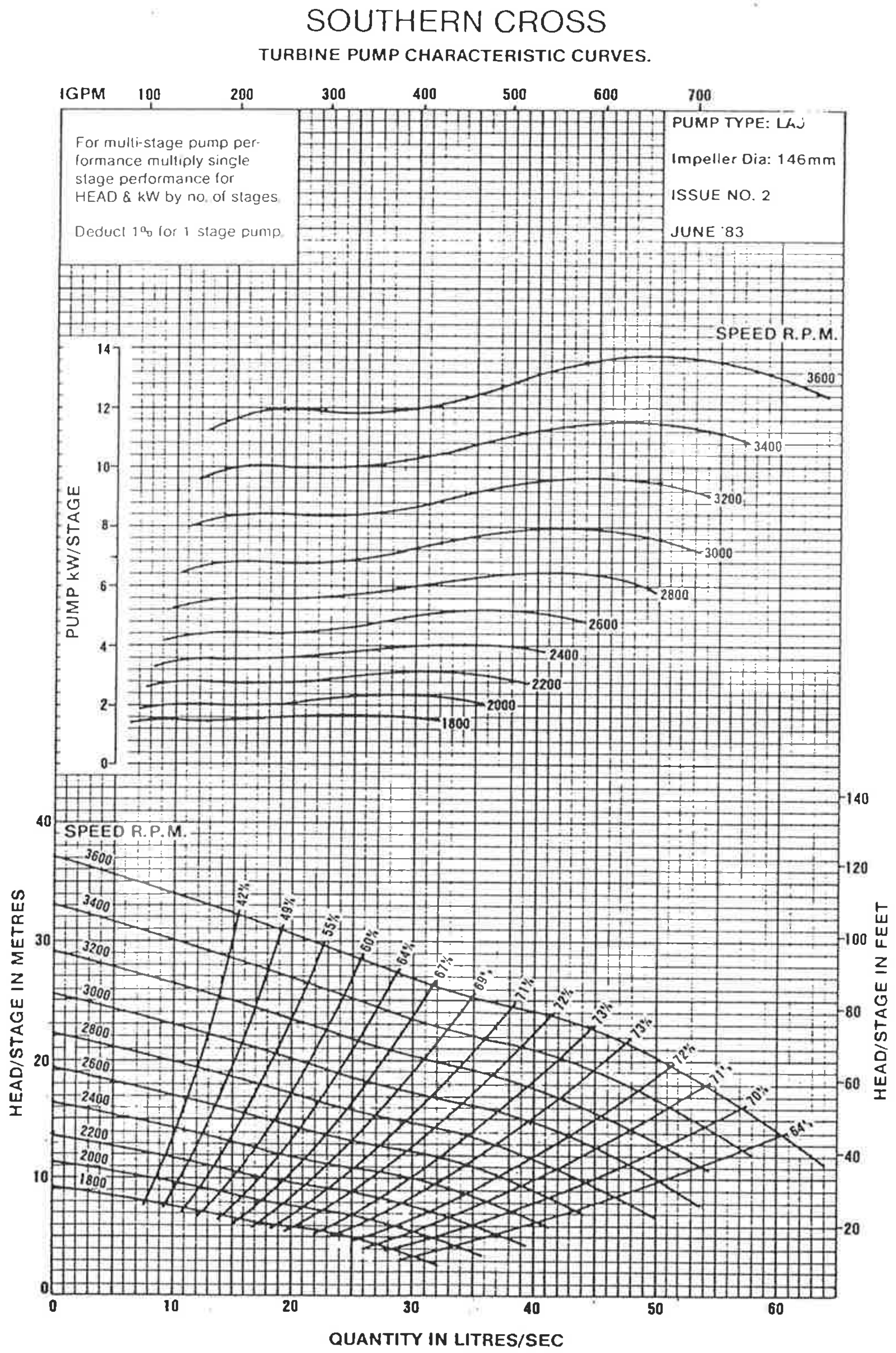


Fig. 9.3 Pump characteristic curves for different impeller speeds (Southern Cross, 1992)

discharge. The efficiency at any given discharge gives the fraction of the energy input needed to drive the pump which is converted to useful energy transferred from the pump to the water. The head, efficiency and power curves are interrelated in accordance with the following equation:

$$BP = \frac{KH_{pu}Q_{pu}}{\eta_p} \tag{9.1}$$

where BP= brake power input (kW or hp); K = conversion factor for units; η_p = pump efficiency.

Once the design discharge and total dynamic head are determined the next step in the irrigation system design process is to select a suitable pump for the operation. The selection process involves finding an economical pumping plant that will provide the required design discharge and head and while operating at high efficiency. In this section an approach is outlined while takes into account the discrete sizes of pumps. For a turbine pump with a number of different impeller speeds as a higher pump speed is selected for a known discharge a higher head may be provided by the pump as shown in Figure 9.3. Once a pump with a known impeller speed is selected the total head and also the efficiency (at which the pump may operate) are found by using the corresponding characteristic curve (see Figure 9.3). The hydraulic grade line of the pump is simply calculated by adding its elevation to the head provided.

Once the HGL of the pump is identified the hydraulic grade line of demand nodes is then computed using the following equation:

$$HGL[a_i] = HGL[b_j] - hl[connect[b_j, a_i]] \tag{9.2}$$

where $HGL[a_i]$ and $HGL[b_j]$= hydraulic grade line at nodes a_i and b_j (respectively). In this approach b_j and a_i are the adjacent upstream and downstream nodes in a system respectively (see Figure 9.4).

The hydraulic grade line can be determined at all demand nodes by working through the network in this way. The flow chart showing the process of HGL computation is given in Appendix K.

After identifying the hydraulic grade line the actual pressure head at each node is computed using the following expression:

$$H_{act}[a_i] = HGL[a_i] - elv[a_i] \tag{9.3}$$

in which $H_{act}[a_i]$ and $elv[a_i]$ are the actual pressure head and the elevation at node a_i (m).

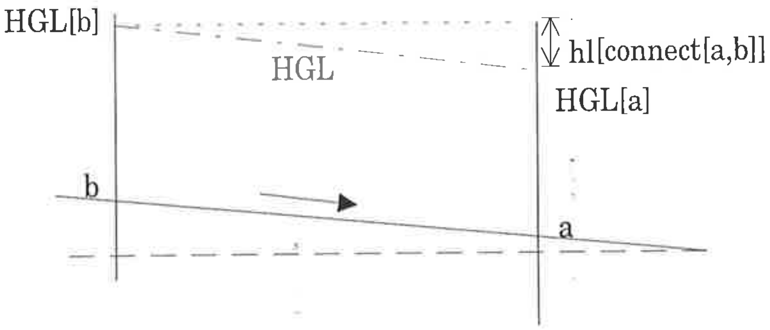


Fig. 9.4 Hydraulic grade line in a pipe between two nodes b and a

In irrigation systems various operating pressures may be needed depending on the type of outlets (sprinklers or drippers). The actual pressures obtained from Equation 9.3 is compared with the minimum required pressure. If the actual pressure is less than the minimum required operating pressure it means that the design is infeasible. In the GA process the infeasible solutions which fail to meet the system constraints are discouraged by applying a penalty cost.

9.4.3 Matching a Pump to the System

The impeller speed and impeller diameter of a pump can be adjusted to select an appropriate pump for a required design discharge and total dynamic head. The pump discharge is directly proportional to the impeller velocity and diameter assuming the crosssectional flow area remains constant (Keller and Bliesner, 1990). On the other hand, the pressure head is proportional to the square of the impeller speed and diameter. Therefore, the effect of impeller speed and /or diameter changes on the pump discharge and pump head may be expressed as below (Keller and Bliesner, 1990):

$$\frac{Q_1}{Q_2} = \frac{rpm_1 \times D_1}{rpm_2 \times D_2} \tag{9.4}$$

$$\frac{H_1}{H_2} = \left(\frac{rpm_1 \times D_1}{rpm_2 \times D_2}\right)^2 = \left(\frac{Q_1}{Q_2}\right)^2 \tag{9.5}$$

where rpm= speed of the impeller in revolutions per minute; D = diameter of impellers (mm); 1 and 2 are subscripts showing different sets of rpm, D, Q, and H.

Since the pump power is proportional to its H and Q, if the pump efficiency remains nearly constant, the effect of speed and the diameter of impeller on the pump power input may be expressed as follows:

$$\frac{BP_1}{BP_2} = \left(\frac{rpm_1 \times D_1}{rpm_2 \times D_2}\right)^3 = \left(\frac{Q_1}{Q_2}\right)^3 \tag{9.6}$$

Equations 9.4, 9.5 and 9.6 are known as pump affinity or pump similarity laws. They can be used to predict pump characteristics to extend the information obtained from hydraulic tests. Since in this model one loading case is applied (design discharge is constant) the impeller diameter for a particular pump is constant and only the impeller speed is considered to vary. Thus for a constant impeller diameter the above pump affinity equations may be expressed as follows:

$$\frac{Q_1}{Q_2} = \frac{rpm_1}{rpm_2} \tag{9.7}$$

$$\frac{H_1}{H_2} = \left(\frac{rpm_1}{rpm_2}\right)^2 = \left(\frac{Q_1}{Q_2}\right)^2 \tag{9.8}$$

$$\frac{BP_1}{BP_2} = \left(\frac{rpm_1}{rpm_2}\right)^3 = \left(\frac{Q_1}{Q_2}\right)^3 \qquad (9.9)$$

A family of characteristic curves for a LAJ type of turbine pump with a constant diameter of 146 mm which is used in the current model is shown in Figure 9.3. A deep-well turbine pump in fact is a specialized multistage application of the centrifugal pump. It operates on the same principle as a centrifugal pump except that the housing is designed to direct water from the discharge of one stage to the inlet of the next stage (Keller and Bliesner, 1990). A typical turbine pump and its characteristic curves are shown in Appendix C.

Characteristic curves of turbine pumps are similar to those of centrifugal pumps. Turbine pump selection is considerably more complicated than is the selection of a centrifugal pump. Determining how best to fit a pump to a given well and irrigation system requires judgment on: the proper size of pump column, size, type, and number of stages, impeller shaft diameter and spacing of bearing. However, in this study since the primary aim is to develop an optimization model for the design of pipe irrigation systems the selection of the pump is considered to involve finding an appropriate speed for a known pump type. It is not within the scope of this study to consider this selection process in any more detail. Values of the characteristic curves for impeller speeds from 2800 rpm to 3600 rpm with an increment of 200 rpm are given in Table 9.2. Xmaths from Microsoft Excel was used to develop the corresponding equations for head and efficiency to be used in the model. The values in Table 9.2 correspond to the equations developed for this purpose. There is a small difference in values between Table 9.2 and the values which may be obtained directly from the curves shown in Figure 9.3.

TABLE 9.2 The fitted characteristic curves for different pump impeller speeds (for one stage)

rpm=2800			rpm=3000			rpm=3200			rpm=3400			rpm=3600		
Q	H	Eff	**Q**	H	Eff	**Q**	H	Eff	**Q**	H	Eff	**Q**	H	Eff
L/s	m	%	**L/s**	m	%	**L/s**	m	%	**L/s**	m	%	**L/s**	m	%
15	21	48	**15**	24	44	**15**	28	43	**15**	32	41	**15**	36	39
20	19	59	**20**	23	56	**20**	26	54	**20**	30	51	**20**	34	50
25	18	67	**25**	21	65	**25**	24	62	**25**	28	60	**25**	32	58
30	17	71	**30**	20	69	**30**	23	68	**30**	26.5	66	**30**	30	64
35	16	73	**35**	18.5	73	**35**	22	71	**35**	25	70	**35**	28.5	69
40	14	74	**40**	17	74	**40**	20	73.5	**40**	24	72.5	**40**	27	72
45	12	71	**45**	15	73	**45**	18.5	74	**45**	22	74	**45**	26	73.4
50	9	64.5	**50**	13	70	**50**	16	72	**50**	20	73.2	**50**	24	74
55			**55**	10	64.5	**55**	14	68	**55**	17	71	**55**	21	71
60			**60**			**60**	10.5	60	**60**	14	66	**60**	18.5	70

9.5 PUMP SELECTION PROCESS

As explained previously five different pump speed curves for a turbine pump are assumed to be available (Figure 9.3). A pump with an appropriate impeller speed and efficiency is to be selected in terms of the design discharge and required head. Each specific pump speed is represented by an integer number in the GA string. In the first stage of the GA process an integer number (from 1 to 5) is generated randomly as shown in Figure 9.5. Each generated number corresponds to one of the pump impeller speeds. Details of the pump characteristic curves for each pump impeller speed and corresponding equations for pump head and pump efficiency are given in Appendix L. For a given design discharge and using a particular speed a specific head is determined. Based on the total head provided by the pump and also the head loss in each pipe the hydraulic grade line at each node is computed using the procedure outlined in Section 9.4.2. The feasibility of each trial solution is then verified. If the pressure head at any node(s) does not satisfy the minimum required pressure, the solution is infeasible. Infeasible solutions are discouraged from the evaluation process by applying a penalty cost. As a result, another pump speed or possibly other pipe sizes for the corresponding links will be selected to meet the feasibility requirements. This process is illustrated in the flowchart shown in Figure 9.5.

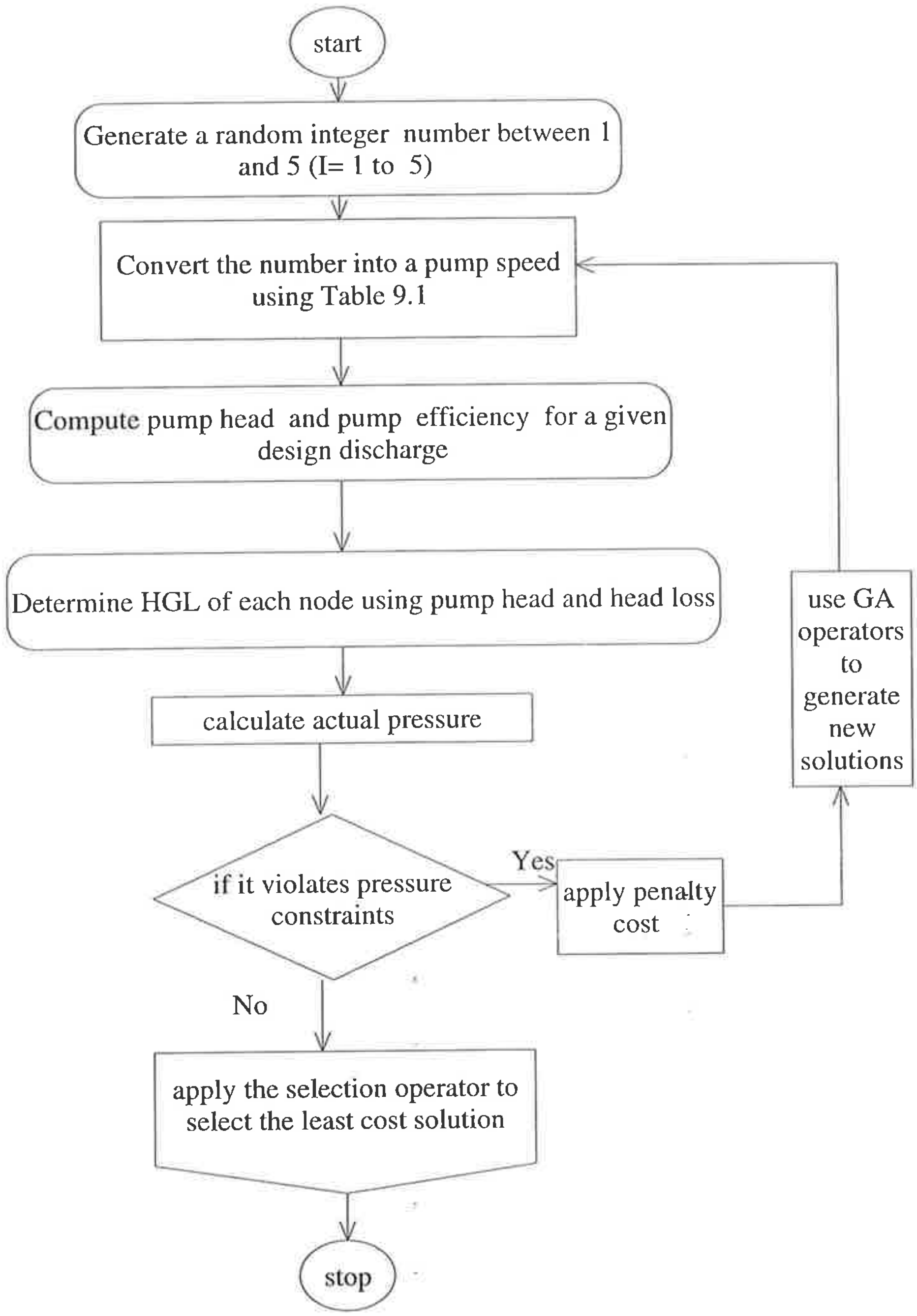


Fig. 9.5 Flowchart for pump selection by GA

9.6 TOTAL SYSTEM COST

The total system cost in this model consists of the pump, valve and pipe costs, the present value of operating cost and the penalty cost. The pipes are available in discrete sizes and the cost per unit length of each size is given in Table 9.3. Although the pump is also available in discrete speeds the cost is computed using Equation 4.39 which is discussed in detail in Section 4.8.5. In this equation the cost of the pump is dependent on its discharge and dynamic head. Equation 4.39 is developed for a turbine pump and electric motor with all accessories. However, a known pump type in reality has a constant cost and its cost may not be changed with speed, whereas in this analysis the cost of pump varies with the head which is affected by speed and discharge. In other words, the cost of pump varies for different speeds which seems to be unrealistic. To overcome this shortcoming the pump cost needs to be modified.

A number of valves (one for each node) with a constant cost are also considered for the system. The present value of the annual operating cost is computed in a similar manner to that used in the previous models. The annual cost of energy is calculated on the basis of the cost of a unit of energy and the total number of hours that the system will work per year. The present value of the annual operating cost is then calculated using Equation 4.41. The penalty costs applied to infeasible solutions are discussed in the following section.

9.6.1 Penalty Costs and Infeasible Solutions

As mentioned already the total system cost is taken as the sum of the network component costs, operating cost and the penalty costs. The penalty costs are used to ensure that there is no violation of the minimum pressure requirements and the connectivity between the demand nodes and the source node. The pressure violation penalty cost is a function of the degree by which the design violates the hydraulic head constraints. In this model, two types of penalty costs are considered: one for violation of the hydraulic head constraints and the other for possible disconnections in the system. The first one is calculated on the basis of the suggestion of Richardson et al (1989) which implies a penalty as a function of the distance from feasibility rather than a function of the number of violated constraints. The pressure violation penalty function used in this model investigates every node and finds the maximum positive difference between the minimum required pressure and the actual pressure. The pressure violation penalty cost is a product of the maximum difference of actual and required pressure and a penalty factor as shown below:

$$Penalty\ cost = f\left\{\max\left(H_{req}[a_i] - H_{act}[a_i]\right)\right\} \qquad (9.10)$$

where f = penalty factor ($/unit of pressure head); $H_{req}[a_i]$ = minimum required pressure at node a_i (m) and $H_{act}[a_i]$ = actual pressure at node a_i (m).

Equation 9.10 is used when the actual pressure is less than the minimum required pressure at the node. This process is demonstrated in a flowchart shown in Figure 9.7. The value of the penalty factor (f) must be selected. The penalty multiplier may be considered as the economic worth of a deficit of one unit of head. The pressure violation penalty function is selected so that the search may approach the optimum solution from both above and below the feasible region of the solution space. In this model, the penalty factor with a value of 20,000 $/m of pressure deficit was selected through a limited sensitivity analysis.

The second penalty cost is a large value of cost for any case of disconnected pipes leading to an infeasible solution. The algorithm developed in this model ensures that each demand node is supplied by a pipe, for the case in which a pipe is assigned two possible directions of flow (as shown in Figure 9.6) there is a likelihood of a disconnection between sets of nodes. If such an infeasibility exists then this penalty cost is added to the total system cost.

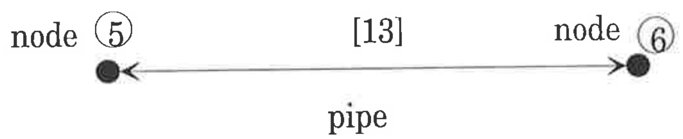


Fig. 9.6 An example of a pipe which is assigned two possible directions between nodes 5 and 6

9.7 MODEL ASSUMPTIONS AND DATA INPUT

In the current GA model in which the pipe layout, pipe sizes and the pump selection are investigated the following major assumptions are made. (i) The pipe network is a branched type and only one link should be directed to each node. (ii) The network may be represented by a base graph and all possible pipes that may deliver water to a node are specified. (iii) To generalize the model some pipes are examined with flow in either possible direction. (iv) Water is supplied from groundwater and a turbine pump with an electric motor is assumed. All other details are given in the next section.

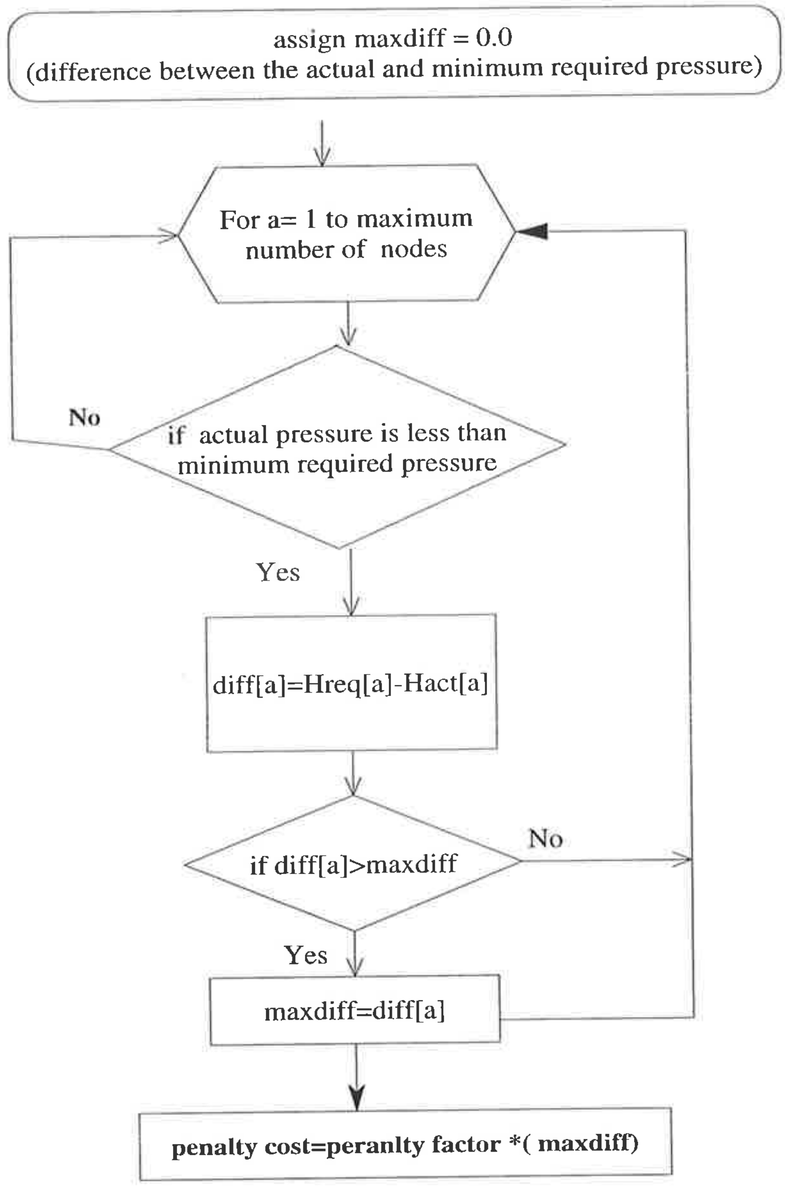


Fig. 9.7 Flow chart of penalty cost for pressure violation

9.7.1 Case Study

The OPIRSYS model is applied to a network consisting of 12 demand nodes and a source node with a pump station. The case study described in this section allows the methodology of Chapter 6 to be assessed for an irregular branched pipe network. The configuration of the piping system and general layout of network used in the case study is shown in Figure 9.1. In this model, eight different pipe sizes are considered for the pipes. Each size is selected on the basis of an integer number which is generated among eight integer numbers by the GA process. The coding is shown in Table 9.3. The pipe data and also the nodal information including the elevations, demands and also the minimum required pressures (working pressures) for the nodes are given in Tables 9.4 and 9.5 respectively.

As can be seen from Figure 9.1 the network used for the case study has a high degree of connectivity and the direction of flow of water is not limited. Some links therefore are specified as the reciprocal links (i.e. links No. [9], [10], [13], [15], [17], [18], [20], [21], [22], [25], [28]). It is expected that the model will identify which direction should finally be considered. In general, all links that may reasonably be connected to each node are taken into account. However, genetic algorithms attempt to select just one link as a supply pipe for each node and the remaining links are eliminated as redundant links.

TABLE 9.3 Pipe sizes with corresponding integer codes and price per unit length

integer codes	pipe diameter (mm)	pipe price ($/m)
1	35	2.26
2	57	4.99
3	84	15.50
4	102	23.80
5	130	32.00
6	149	43.00
7	187	70.00
8	205	80.00

TABLE 9.4 Pipe data for the case study

pipe No.	start node	end node	length (m)	pipe No.	start node	end node	length (m)
[1]	1	3	260	[16]	6	8	335
[2]	1	2	185	[17]*	6	7	500
[3]	1	4	435	[18]*	7	8	325
[4]	2	3	285	[19]	7	9	285
[5]	2	5	415	[20]*	8	9	260
[6]	2	4	275	[21]*	9	10	590
[7]	3	6	428	[22]*	8	10	375
[8]	3	5	375	[23]	6	10	400
[9]*	3	4	500	[24]	12	6	325
[10]*	4	5	375	[25]*	6	12	483
[11]	4	8	710	[26]	1	12	600
[12]	4	7	558	[27]	11	12	385
[13]*	5	6	335	[28]*	3	11	300
[14]	5	8	385	[29]	1	11	350
[15]*	4	7	350	[30]	3	12	250

* reciprocal pipes: for this pipes the start and end nodes are not known initially;

- all pipes assumed to be polyethylene;
- Hazen-Williams roughness coefficient =150
- annual operating hours =1600 hrs
- cost of a valve =$80
- project life =12 years
- interest rate =10% p.a.
- unit energy cost =$0.09/kWh

TABLE 9.5 Nodal data used in the case study

Node	elevation (m)	demand (L/s)	minimum required pressure head (m)
pump (water level)	*100.0	-41.0	-
1	107.0	0.0	-
2	103.0	6.0	30.0
3	104.5	5.0	30.0
4	104.0	4.0	30.0
5	107.5	2.0	40.0
6	108.0	6.0	40.0
7	108.2	3.0	40.0
8	110.0	2.0	35.0
9	110.0	3.0	35.0
10	112.0	2.0	35.0
11	102.0	4.0	30.0
12	106.0	4.0	30.0

* groundwater level (including draw down)

9.8 RESULTS AND DISCUSSION

The present GA model has the capability of being applied to any branched pipe network supplied from one source node. The current model is more elaborate than the previous GA models outlined in Chapters 7 and 8, because it deals with irregular networks and varying topography. In addition, the required pump is considered as a decision variable and the GA finds a discrete size for the pump. This model was tested on the network discussed in Section 9.7.1 using two different GA selection methods (tournament and proportionate). The results are discussed in detail in the following sections.

9.8.1 Optimum Solutions using Tournament Selection

In this analysis tournament selection with the size of 5 is used. Therefore, for each 5 individuals which are selected randomly the one with the highest fitness is chosen and goes to the mating pool to be processed by the GA operators. The genetic algorithm parameters were selected for the initial examination are given in Table 9.6.

TABLE 9.6 Genetic algorithm parameters used in the case study

probability of crossover Pc	probability of mutation Pm	population size N	Tournament size
0.8	0.02	1000	5

The seed number which specifies the sequence of generated random numbers was chosen to be 1000. The length of string is constant for this case study and equals 25 bits, of which 12 bits correspond to layout specification, 12 bits to pipe sizes and 1 bit to the pump speed.

Implementing a population size of 1000 and a probability of crossover of 80% causes an expected population of 800 (1000×0.8) new strings to be involved in the next generation. A new string is created in the next generation when two old strings are crossed over (Murphy et al, 1993). There are expected to be 200 strings that pass to the next generation without any change by crossover. Increasing the probability of crossover will decrease the number of strings which pass to the next generations without any change by crossover. Mutation alters the bits within strings that have been crossed over. Since this operator is carried out with a low probability its effect is not very important but it does maintain some diversity in the gene pool. Some researchers used a GA model with zero mutation (Simpson and Goldberg, 1994; Connarty, 1995). Using a probability of mutation of 2% implies a change of 2 bits in 100 bits (on average). Since the length of the strings is 25 it is expected therefore that each string that has been crossed over will be altered with a probability of 0.397 ($1-0.98^{25}$) by the mutation operator. Once a new string is created by crossover and/or mutation an evaluation of this string is required.

In each generation about 800 evaluations were carried out. The performance of a genetic algorithm can be outlined by observing the progression of cost values for successive generations during the evaluation process. Some of these will be infeasible. Figure 9.8 shows the cost of the best solution in each generation for a typical computer run. These values decline with increasing evaluations and the best result of the population converges to a near optimal solution. At the start, the decrease in cost for the best solutions is very steep. At this stage, it is not very difficult to achieve improvement in almost every generation as there are a lot of potential solutions with lower cost. As more evaluations are carried out, the search space becomes progressively narrower and the best solution can be improved only occasionally. It seems that the genetic algorithm is actually very effective within the range that the minimum cost solution curve is steep (Walters and Lohbech, 1993). As shown in Figure 9.8, the minimum cost solution curve in the first stage of evaluation drops from around \$190,000 to around \$110,000 within about 23000 evaluations. It then reaches its minimum value of \$107035 after 40,000 evaluations at generation 50. Although the model was allowed to run up to 300 generations no progress was observed after generation 50.

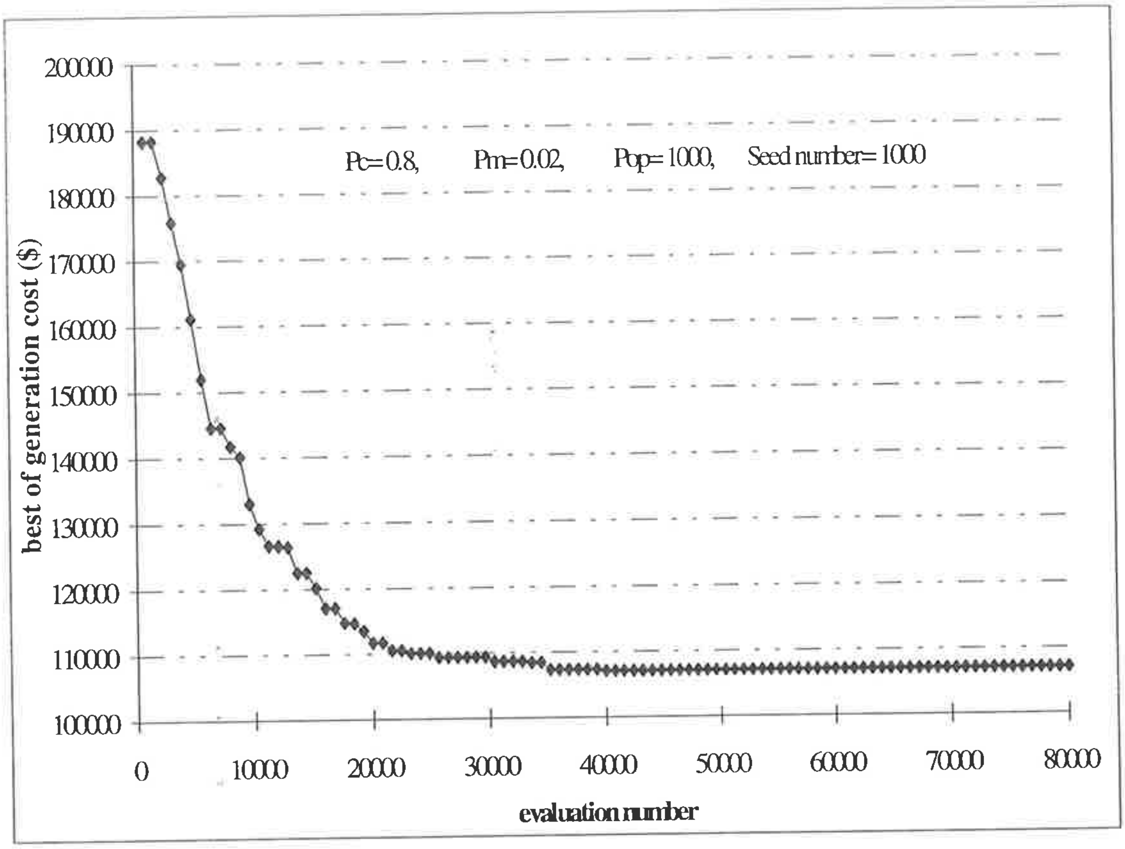


Fig. 9.8 The variation of least cost solution in each generation (tournament selection)

The model was written in Pascal to run on a DECstation 5000/240 running under Unix. To determine the real run time for the GA model to find the minimum solution, the model was run again and it was allowed to run up to 50 generations. The number of evaluations made to find the best design equals 40,000 ($50\times1000\times0.8$). The model took 788 seconds (13.13 minutes) to run. This time is the user plus the system time. It is largely independent of the load on the system. The real run time for finding the minimum cost solution depends largely on the population size. If we can use a smaller population size the run time will largely decrease. The integer presentation of the optimal solution (string) and also the layout of the design corresponding to the minimum cost solution are shown in Figures 9.9a and 9.9b. More details including nodal information, pipe information and component costs corresponding to the minimum cost solution (using tournament selection) are given in Tables 9.7, 9.8 and 9.9 respectively. The HGL at node 1 is estimated by subtracting the head loss in accessories from the HGL of pump.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
0	1	1	1	1	2	2	2	2	1	2	3	0	2	5	2	4	3	3	2	2	2	2	2	5

Fig. 9.9a Integer coded presentation of optimum solution (tournament selection)

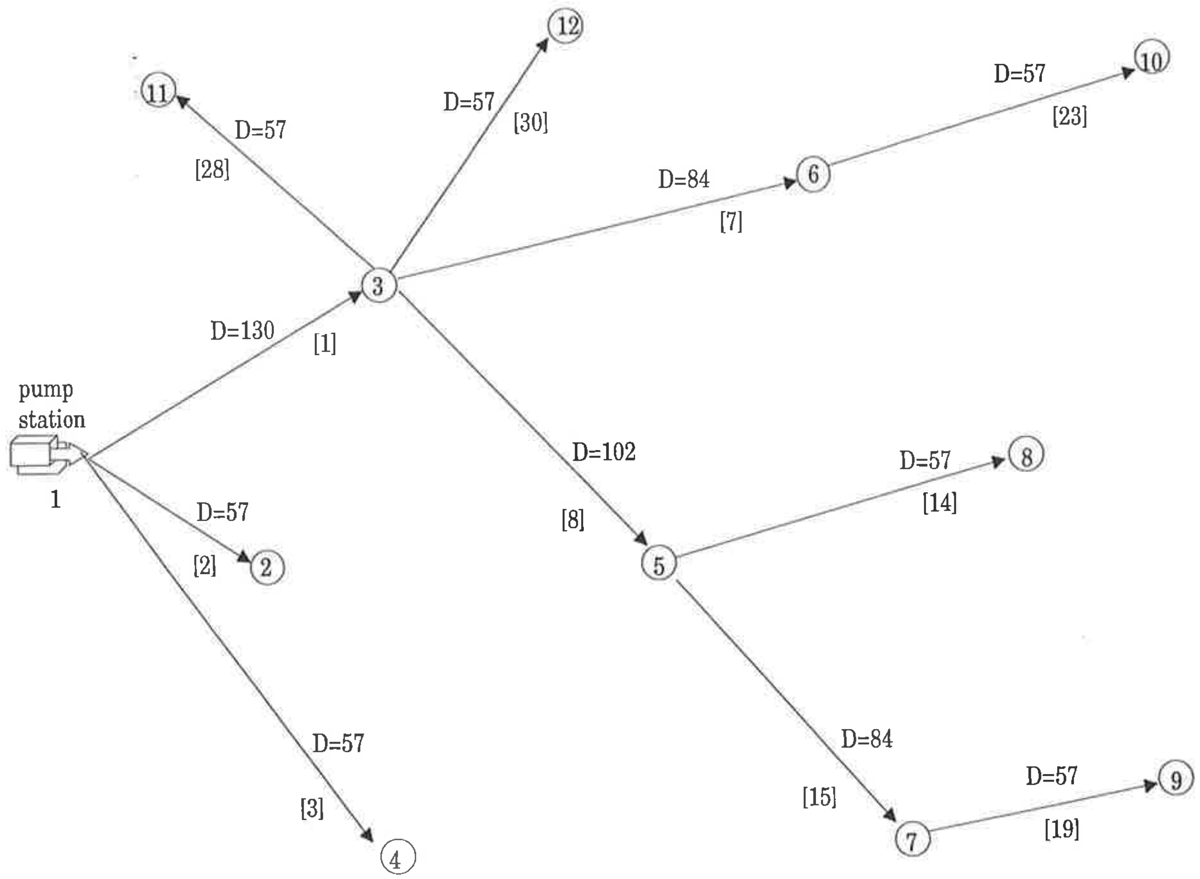


Fig. 9.9b The optimum design resulted from GA using tournament selection

TABLE 9.7 Nodal information for the optimum solution (tournament selection)

node	elevation (m)	HGL (m)	minimum req. pressure(m)	actual pressure (m)
1	107.0	*170.7	-	63.7
2	103.0	154.5	30.0	51.5
3	104.0	162.1	30.0	57.6
4	104.0	152.7	30.0	48.7
5	107.5	157.1	40.0	49.6
6	108.0	152.4	40.0	44.4
7	108.2	152.5	40.0	44.3
8	110.0	152.7	35.0	42.7
9	110.0	145.6	35.0	35.6
10	112.0	147.8	35.0	35.8
11	102.0	149.7	30.0	47.7
12	106.0	151.8	30.0	45.8

*HGL[1]= HGL[pump]- hl_{acc}

hl_{acc} = head loss in pump shaft and all accessories between pump and node 1

TABLE 9.8 Pipe information for the optimum solution (tournament selection)

pipe number	length (m)	diameter (mm)	head loss (m)	cost ($)
[1]	260	130	8.6	8320
[2]	185	57	16.2	923.0
[3]	435	57	18.0	2170.6
[7]	428	84	9.7	6634.0
[8]	375	102	5.0	8925.0
[14]	385	57	4.4	1921.0
[15]	350	84	4.6	5425.0
[19]	285	57	6.9	1422.0
[23]	400	57	4.6	1996.0
[28]	300	57	12.4	1497.0
[30]	250	57	10.3	1247.5

TABLE 9.9 Minimum system cost and the other cost components corresponding to the optimum solution (tournament selection)

Total pipe cost	$40482
Pump cost	$24790
P.V. annual operating cost	$40804
Accessory costs	$960
Penalty costs	$0
Total system cost	**$107036**

TABLE 9.10 Pump information for the optimum solution (turbine pump with impeller diameter of 146 mm and 3 stages; tournament selection)

pump head (m)	pump power (kW)	pump efficiency (%)	pump discharge (L/s)	pump speed (rpm)
78.8	**41.6**	**76.2**	**41**	**3600**

The pump information given in Table 9.10 are the results obtained from the model using equations developed on the basis of the pump characteristic curves shown in Figure 9.3 and Microsoft Excel. Thus the values of head, efficiency and power may not exactly matched with the values obtain from Figure 9.3 directly. For a pump with N stages the pump head and the pump power should be multiplied by N. (In this model, a pump with 3 stages produces the head and discharge within the required range).

9.8.2 Optimum Solution using Proportionate Selection

Proportionate selection is the traditional method which was used in standard (simple) genetic algorithms (SGA) by Goldberg (1989). The problem with this method is that there is no guarantee that the highest fitness string in one generation will be selected for the next generation nor is there a guarantee that the string with the lowest fitness will not be selected for the next generation. If a selection scheme could guarantee that the best strings are represented in the parent set in the new generation then this could provide a better GA process (Connarty, 1995).

The same network with the same genetic algorithm parameters used in the previous section (with tournament selection) was examined using proportionate selection. The model was allowed to proceed up to 800 generations. In this run $1000 \times 800 \times 0.8$ (i.e 640,000) evaluations were examined. Although all parameters and network data were exactly the same as the previous case the cost of the optimum solution obtained was higher than the cost of optimum solution obtained with tournament selection. Not only the cost of the optimum solution is higher but also it is obtained after a large number of evaluations. The optimum solution was obtained in generation 742 with a cost of $110840 after 593,600 evaluations. The variation of the best cost solution at each generation using this selection technique is demonstrated in Figure 9.10. The variation of the best cost solution within 640,000 evaluations shows that in the first stage of evaluation, there is a significant improvement in reducing the cost. The cost of the optimum solution drops sharply from the first generation up to generation 30 (i.e 24,000 evaluations), it then changes with an erratic manner and improves at a slow rate. The minimum cost solution finally occurs after 593,600 evaluations and then increases again. It is not quite clear whether there would be further improvement if the evaluation were allowed to proceed (details are discussed in the next section). The comparison between the two selection techniques indicates that tournament selection performs much better than the proportionate selection.

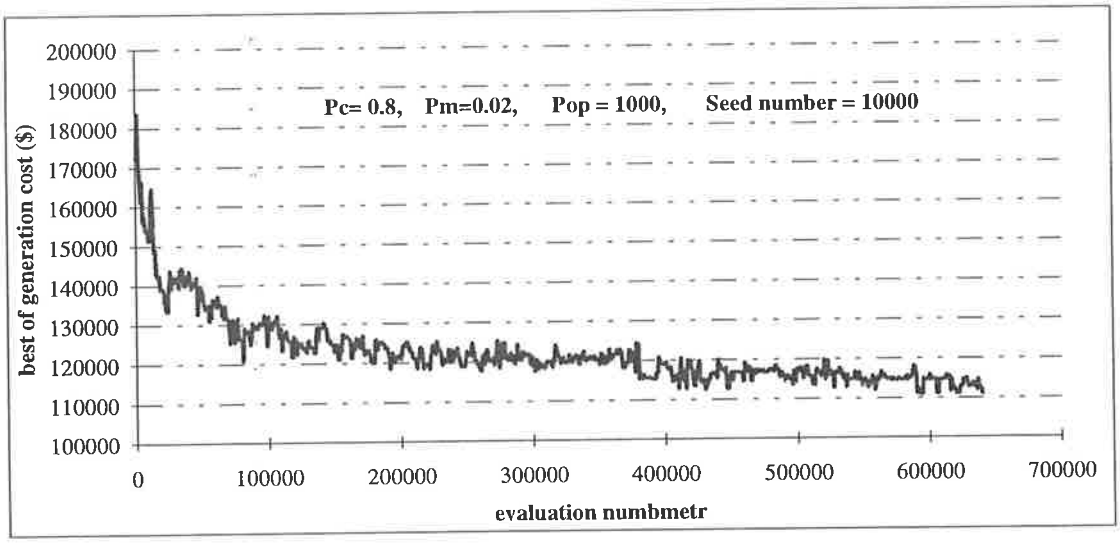


Fig. 9.10 The variation of the least cost solution in each generation (proportionate selection)

It reaches the optimum solution with a lower cost within many fewer evaluations compared with proportionate selection. Clearly, reaching a minimum cost solution within a larger number of evaluations increases the computer time required. This is a big disadvantage particularly for complicated problems in which the number of nodes and connectivities is large. However, the computer time for running the same example for the same number of evaluations with proportionate selection is much less than the time needed with tournament selection. But since proportionate selection requires many more evaluations to find the optimum solution, on average it takes longer than tournament selection to reach a near-optimal solution. The average computer time for running the model for 800 generations in which 640,000 evaluations were examined on the same mainframe computer (Unix with DECstation 5000/240) was 1136 seconds (≅ 19 minutes) which is longer than the time needed to reach a near-optimal solution with tournament selection (≅13 minutes). The model information for the solution using proportionate selection is summarized in Table 9.11. This indicates that the actual pressures at some nodes particularly those that are terminal nodes could be reduced down to the minimum required pressure.

TABLE 9.11 Nodal information for the optimum solution (proportionate selection)

node	elevation (m)	HGL (m)	minimum req. pressure(m)	actual pressure (m)
1	107.0	170.7	-	63.7
2	103.0	167.3	30.0	64.3
3	104.5	154.4	30.0	49.9
4	104.0	156.0	30.0	52.0
5	107.5	160.1	40.0	52.6
6	108.0	152.5	40.0	44.4
7	108.2	151.6	40.0	43.4
8	110.0	156.4	35.0	46.4
9	110.0	150.0	35.0	40.0
10	112.0	147.9	35.0	35.9
11	102.0	156.2	30.0	54.2
12	106.0	140.2	30.0	34.2

In this way the use of smaller pipe sizes and hence a cheaper design may be possible. The coded presentation (strings) and the layout of the optimum solution found by proportionate selection are shown in Figures 9.11a and 9.11b. More details including nodal information, pipe information and component costs corresponding to the minimum solution (using proportionate selection) are given in Tables 9.11, 9.12 and 9.13 respectively.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
0	1	1	2	2	3	2	2	1	1	1	4	0	6	2	2	5	3	2	3	2	2	2	2	5

Fig. 9.11a Integer coded presentation of optimum solution using proportionate selection

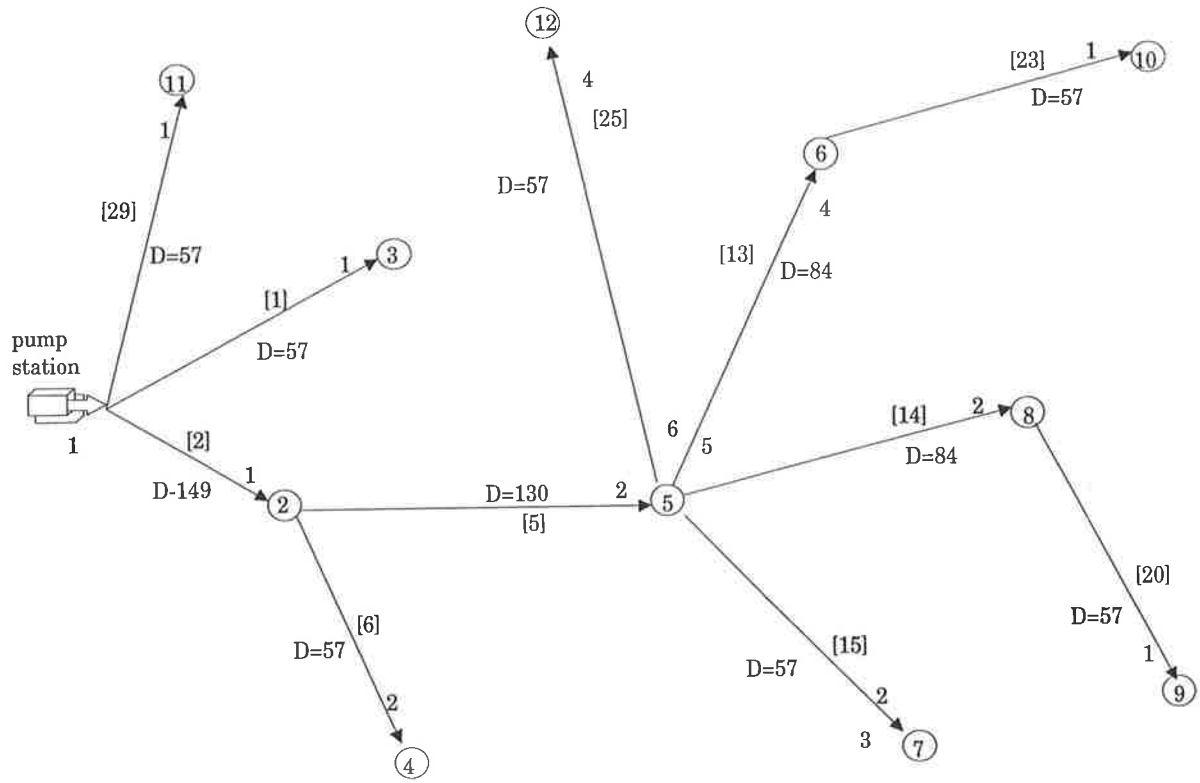


Fig 9.11b The optimum design resulted from proportionate selection

TABLE 9.12 Pipe information for the optimum solution (proportionate selection)

pipe number	length (m)	diameter (mm)	head loss (m)	cost ($)
[1]	260	57	16.2	1297.4
[2]	185	149	3.3	7955
[5]	415	130	7.3	13280
[6]	275	57	11.4	1372.2
[13]	335	84	7.6	5192.5
[14]	385	84	3.6	5967.5
[15]	350	57	8.5	1746.5
[20]	285	57	6.4	1322.0
[23]	400	57	4.6	1966.0
[25]	483	57	19.9	2410.2
[29]	350	57	14.5	1746.5

TABLE 9.13 Minimum system cost and the other cost components corresponding to the optimum solution (proportionate selection)

Total pipe cost	$44286
Pump cost	$24790
P.V. annual cost	$40804
Accessory costs	$960
Penalty costs	$0
Total system cost	**$110840**

TABLE 9.14 Pump information for the optimum solution (turbine pump with impeller diameter of 146 mm and 3 stages, proportionate selection)

pump head (m)	pump power (kW)	pump efficiency (%)	pump discharge (L/s)	pump speed (rpm)
78.8	**41.6**	**76.2**	**41**	**3600**

9.9 SENSITIVITY ANALYSIS OF GENETIC ALGORITHM PARAMETERS

The values of the GA parameters utilized in the GA model in this chapter, were a result of recommended values in the literature and limited sensitivity analysis. The purpose of this part of study is to perform a more detailed analysis of GA methods utilized in this thesis and also to identify the parameter values which provide the best and most effective GA models. The literature contains some recommended values for the GA parameters in pipe networks dealing with the optimisation of component sizes only (Walter, 1993; Murphy et al, 1993; Dandy et al, 1993; Simpson and Goldberg, 1994). However, there are no guidelines for models dealing with layout, component sizes and also pump selection in the same GA model. Thus it is expected that a different range of parameter values could exist that may be more efficient for this sort of problem and the models outlined in this thesis.

The GA parameters which are investigated are

- Population size;
- Probability of crossover (P_c);
- Probability of mutation (P_m).

In addition, the seed number is also examined for a number of runs to evaluate the effect of different sequence of random numbers on the system cost and performance of the GA. The size of the tournament in tournament selection is also another factor that will be examined.

A higher population size is more likely to contain the necessary sequence of bits which will produce the optimal string particularly in complex problems. The current model, which deals with layout, component sizes and pump selection has a large search space particularly when a large number of demand nodes with a high degree of connectivity is considered. Thus using a high population size will increase the possibility of achieving the better solution even though only a small fraction of the total solution space is searched. Simpson and Goldberg (1994) found that for larger population sizes higher fitness results were obtained and convergence was more likely. In this part of the study, the population size will start at 20 and will be increased until it is considered that there will be no further significant improvements in the results obtained. To evaluate the effect of population size on the optimum solution, the other parameters such as P_c, P_m, seed number and tournament size are kept constant. As the population size increases the number of evaluations also increases. In order to assess the effect of population size within the same number of evaluations, as the population size was increased the number of generations was decreased. Thus in each run the value of $Pc \times Pop \times Gen$ was kept constant. The results given in Table 9.15 show that the minimum system cost of $107035 occurs when a population size of 500 was used. The minimum cost solution was achieved in generation 31 after 12400 evaluations.

The same solution was obtained using a population of 1000 with a cross over probability of 0.8, but a larger number of evaluations were required than those shown in Table 9.15. The computer time shown in Table 9.15 is the time required for the evaluation of $Pc \times Pop \times Gen$ evaluations (in this example = 32000).

TABLE 9.15 Influence of population size (Pc=0.8; Pm=0.02; seed number= 1000; tournament size =5)

run	population size (Pop)	best cost ($)	computer time for all evaluations(s)	generations to reach the best	evaluations to reach the best
1	20	113,809	669	73	1168
2	50	114,456	650	46	1840
3	100	112,961	640.2	36	2880
4	200	108,181	631	26	4160
5	300	111,517	642	85	20400
6	400	108,741	638.5	33	10560
7	**500**	**107,035**	**615.4**	**31**	**12400**
8	600	107,304	630	65	31200
9	700	108,698	623.4	23	12880
10	800	110,229	621.7	26	16640
11	900	114,015	620.2	50	36000
12	1000	108,675	611.0	39	30400

Crossover is the partial exchange of bits between two chosen strings to form two new strings. The most common crossover operator (one-point crossover) is applied in this study. As the probability of crossover increases the chance for mixing more strings is increased. The importance of having adequate mixing was addressed in Simpson and Goldberg (1994). The effect of varying the probability of crossover is examined by using a maximum value (Pc= 1.0) and then reducing to a probability of 0.5 in increments of 0.05. The results are summarized in Table 9.16.

In order to examine the effect of Pc the other parameters are kept constant . The model was allowed to run up to 150 generations with a population of 500 which gave the best results from the previous analysis. The minimum cost solution for each particular Pc value and also the time for Pc ×500×150 evaluations plus the generation at which the minimum cost solution is obtained are given in Table 9.16. A lower minimum cost solution of \$106533 is obtained when a probability of 0.75 is applied. This minimum cost occurred in generation 29 after 10875 (0.75×500×29) evaluations. This is a new record compared to the best cost solution obtained in Section 9.7.1. This result indicates that the use of a crossover probability of 0.75 instead of 0.8 and a population size of 500 instead of 1000 leads to much better results as shown in Table 9.16.

TABLE 9.16 Influence of the crossover operator (pop=500, Pm= 0.02; Gen =150, tournament size =5; seed =1000)

run	crossover probability (Pc)	least cost (\$)	computer time for all evaluations (s)	generations to reach the best	evaluations to reach the best
1	1.0	109,874	1143	89	44500
2	0.95	108,426	1145	20	9500
3	0.90	106,976	1139	105	47250
4	0.85	109,579	1138	43	18275
5	0.80	107,036	1140	37	14800
6	**0.75**	**106,534**	**1138**	**29**	**10875**
7	0.70	110,761	1141	33	11550
8	0.65	109,585	1141	33	9900
9	0.60	109,585	1145	33	9900
10	0.50	107,036	1149	123	30750

The run times associated with different values of Pc show that, although the number of evaluations carried out by GA is different for each value of Pc, the total run time is nearly the same. It shows that the value of Pc does not affect the total run time significantly.

A bit-wise mutation was applied in this research. In this, a bit is randomly chosen in a string with a small probability of mutation and switched to another value. For the examination of the probability of mutation, a wide range of values for probability of mutation were used. First, the GA model was allowed to run without applying mutation (Pm =0.0) then increasing values with an increment of 0.2% and 2% were tried.

TABLE 9.17 Influence of mutation operator (Pc=0.75; pop=500; Gen= 150, tournament size= 5; seed=1000)

run	mutation probability (Pm)	least cost ($)	computer time for all evaluations (s)	generations to reach the best	evaluations to reach the best
1	0	127,662	1153.0	18	6750
1	0.002	111,019	1166.6	47	17625
2	0.004	123,595	1147.3	25	9375
3	0.006	122,045	1139.4	26	9950
4	0.008	109,360	1144.0	19	7125
5	0.010	110,221	1144.4	27	10125
6	0.015	110,315	1157.0	61	22875
7	**0.02**	**106,534**	**1178.0**	**29**	**10875**
8	0.04	108,345	1158.0	37	13875
9	0.06	107,034	1160.0	69	25875
10	0.08	106,783	1148.0	77	28875
11	0.1	108,345	1152.0	108	40500
12	0.12	118,891	1163.0	66	24750
13	0.15	128,668	1162.0	124	18600
14	0.18	132,268	1165.0	89	33375
15	0.2	135,792	1165.0	144	54000

The results are tabulated in Table 9.17. Note that for all values of Pm a constant value of 0.75 for Pc and 500 for the population size were used. This analysis shows that a low value of 2% gives a better solution among all the other values of Pm. For Pm=0.02 the same minimum cost ($106,535) which was obtained already in the previous runs with Pc=0.75 occurred at generation 29 after 10,875 evaluations. It took 221 seconds (3.6 minutes) of computer (CPU) time to run. Since for some runs the best cost curve converges very quickly and no progress is achieved, the number of generations was reduced from 150 to 100. This is a simple way to reduce the run time if only the minimum cost solution is investigated. The result given in Table 9.17 indicates that a low value of Pm (but not zero) leads to the best solution.

Since the GA models using tournament selection performs better than those using proportionate selection the sensitivity analyses was carried out on the model with tournament selection. To examine the performance of tournament selection a variety of tournament sizes were examined.

TABLE 9.18 Influence of tournament size (Pc=0.75, Pm =0.02, Pop=500, seed number=1000, Gen= 150)

run	tournament size	least cost ($)	computer time for all evaluations (s)	generations to reach the best	evaluations to reach the best
1	2	109,034	1182	105	39375
2	**3**	**106,534**	**1183**	**105**	**39375**
3	4	109,062	1191	30	11250
4	**5**	**106,534**	**1167**	**29**	**10875**
5	6	108,341	1165	27	10125
6	7	107,036	1160	20	7500
7	8	110,230	1184	68	25500
8	9	109,900	1151	47	17625
9	10	112,284	1163	21	7875
10	20	106,975	1157	65	24375

All other GA parameters were kept constant at the values which gave a better result. The value of tournament size was varied from 2 to 10 with an increment of 1 and at the end a value of 20 was examined. The results shown in Table 9.18 indicate that the range of minimum cost solutions obtained for various tournament sizes is not as large as the range obtained for various values of Pc and Pm or population size. However, there is still a considerable difference in the minimum cost solutions for various tournament sizes. In this analysis a tournament size of 3 or 5 gave the least cost solution. With a tournament size of 3 the minimum cost solution was obtained after 39,375 evaluations (105 generations) and with a size of 5 it was obtained after 10,875 evaluations (29 generations) which needs much less computer time. Using a tournament size of 20 also led to a solution very close to the optimum solution ($ 106,975) after 24,375 evaluations (65 generations).

Since genetic algorithms are stochastic algorithms the use of seed number as a starting point for generating the initial random population and for the subsequent operators is important. Different seed numbers may provide a significantly different initial population. The random number generator used in this model was that one which developed by Barnard and Skillcorn (1989). This guarantees that when using the same seed number the same random number sequences is provided. The algorithm for this random number generator has been used widely (Murphy and Simpson, 1992; Dandy et al, 1993; Connarty, 1995). A number of different seeds as given in Table 9.19 were examined. As mentioned previously, all other parameters remained at a fixed level. This allows a comparison of the effectiveness of the various values of the parameters to be carried out. In this analysis, the model was examined in ten runs each with a different seed number but a constant values of population size, Pc and Pm. Ten runs are used so that if a particular solution is obtained on all ten occasions, it is more than likely that will be the optimal solution. As can be seen from Table 9.19 only the seed numbers 1000 and 1500 lead to the minimum cost solutions. This indicates that it is usually better to rerun GA model a number of times with different starting seeds in order to have a good chance of identifying the optimum solution.

The sensitivity analysis carried out in this section indicates that the least cost solution which is most probably the global optimum (or very close to it) was obtained at a value of $106,534 with the parameters given in the Table 9.20. The variation of the least cost solutions corresponding to this case is illustrated in Figure 9.12. The general trend of variation is similar to the least cost curve shown in Figure 9.8. The final layout and the pipe sizes corresponding to this optimum solution is illustrated in Figure 9.13.

TABLE 9.19 Influence of seed number (Pc = 0.75; Pm= 0.02; Pop = 500; Gen= 150, tournament size =5)

run	seed number	least cost ($)	computer time for all evaluations(s)	generations to reach the best	evaluations to reach the best
1	1	106,975	1157	65	24375
2	100	110,580	1153	68	25500
3	300	107,953	1160	22	8250
4	500	109,572	1251	118	44250
5	**1000**	**106,534**	**1153**	**31**	**12400**
6	**1500**	**106,534**	**1149**	**76**	**28500**
7	2000	111,728	1151	22	8250
8	3500	109,034	1168	88	33000
9	5000	110,353	1165	120	45000
10	7500	111,293	1168	31	11625
11	10000	110,315	1160	26	7750

TABLE 9.20 The parameters that lead to the least cost solution

crossover probability	mutation probability	tournament size	population size	seed number	computer time(s)	least cost($)
0.75	**0.02**	**5**	**500**	**1000**	**221**	**106534**

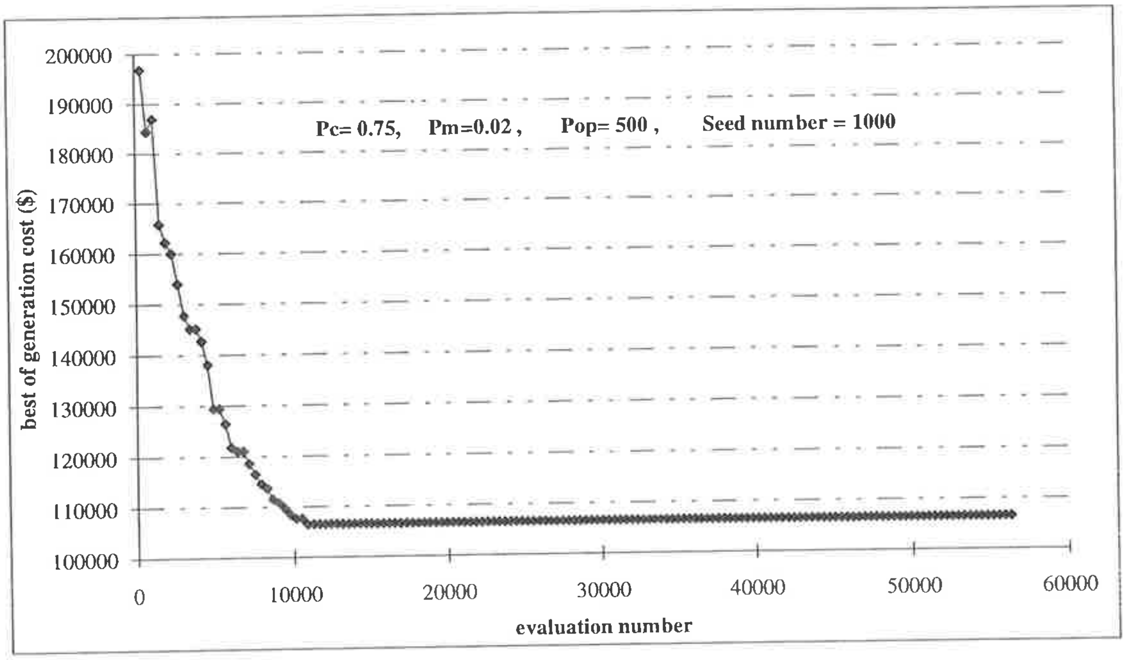


Fig. 9.12 The variation of the least cost solutions using the optimum value of GA parameters (tournament selection)

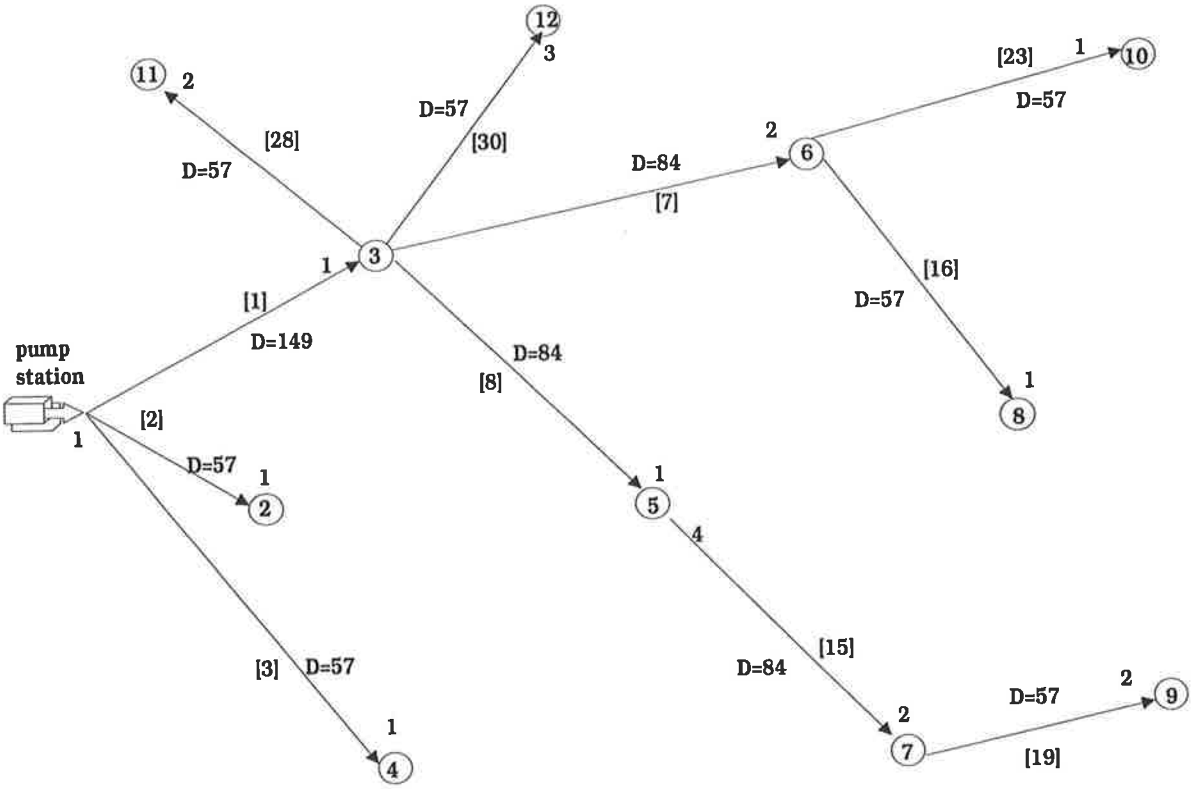


Fig 9.13 The layout and corresponding information for the least cost design ($106,534)

9.10 SUMMARY

A genetic algorithm model (OPIRSYS) for the optimum design of an irregular branched pipe network was developed. Any branched pipe network consisting of a single source node and a number of demand nodes may be formulated and designed using this model. The main aims of this model are to identify the least cost layout for the best connection of demand nodes to the source node, the optimum pipe diameters from a number of discrete sizes and also the selection of an appropriate speed for a turbine pump. The general layout is assumed to be a base graph with a high degree of connectivity and the final selected solution is a branched system in which only one pipe is directed to each demand node. A methodology for the application of the GA technique to branched pipe networks for the above purpose is presented. In this model, the connections (links) between nodes, the diameters of all pipes and also the pump speed are assumed to be decision variables. A turbine pump with appropriate speed for the design discharge is selected to provide adequate pressure to meet the minimum pressure requirements. The standard (simple) genetic algorithm with three traditional operators is employed. The GA codes the links between nodes, the pipe diameters and also the pump speed as an integer string. The total system cost is inversely proportional to the fitness and consists of the cost of pipes, pumping plant, the present value of annual operating cost, accessories and penalty costs. Two penalty costs are considered: one for connectivity and the other for the pressure constraints. By implementing these two penalty costs, infeasible solutions are eliminated from consideration by the GA.

A case study consisting of 12 demand nodes and a source node with a pump station was examined by the model. The results using two selection techniques (tournament and proportionate) are presented. The performance of genetic algorithms can be outlined by observing the progression of cost for successive generations during the evaluation process. The cost of the best solutions declines during the evaluation and it converges to a minimum which is expected to be close to the global optimum. The results obtained using tournament selection improved rapidly during the first few generations and then reached its minimum level of $107,036 at generation 50 after 40,000 evaluations. No improvement was observed with further evaluations for the parameters which lead to the optimum solutions. The results obtained with proportionate selection also showed a good improvement for the first few generations and then changed in an unsteady manner. Some new solutions were created by mutation which were worse than the previous solutions. On average there is a very mild improvement in the cost of the best solutions. The curve of the best cost solution reaches its minimum level of $110,840 at generation of 742 after 593,600 evaluations. When both models were run for the same number of evaluations it was observed that the model with proportionate selection is faster than the other model with the tournament selection.

However, the model with proportionate selection needs a large number of evaluations to find the least cost solution. Thus in terms of time consumed to reach the optimum solution the tournament selection is more efficient. It finds not only the optimum solution with the lowest cost (very close to global optimum) but also reaches the optimum with fewer evaluations. In this study the model with tournament selection found the optimum solution with a cost of $107,036 within 788 seconds (13.13 minutes) while the model with proportionate selection found its minimum cost solution with a cost of $110,840 within 1136 seconds (19 minutes).

In order to find a new set of values of GA parameters that may perform, better a sensitivity analysis was carried out. A significant improvement for the model with tournament selection was observed. The minimum cost solution obtained with the new values of GA parameters at a cost of $106,534. Using the new values of GA parameters the optimum solution that most likely is the global optimum or close to it obtained in generation 29 after 10,875 evaluations. The computer time needed to reach this optimum solution was 221 seconds (3.6 minutes) which is considerably less than the time required for the previous optimum solution.

Chapter 10

Summary and Conclusions

10.1 INTRODUCTION

This investigation has achieved several goals in the optimum design of branched pipe networks for pressure irrigation systems. Various problems which affect the least cost design of a micropipe irrigation system and also the layout and component sizes of major pipes have been considered. The following summarises the work which has been undertaken in this thesis:

- Optimum design of a micropipe (drip) irrigation system for a subunit and analysis of the system cost for different field dimensions, slopes, and working pressures by applying various irrigation intervals and irrigation times (different loading cases).
- Development of a model, OPSHIR, for the optimum division of a field into subunits. This model, examines the effect of different factors on the system cost and searches for the optimum design of a multiple subunit drip irrigation system.
- The methodology of genetic algorithms (GAs) has been explored as a new search and optimisation method for branched pipe networks.
- Development of a model for identifying the optimum connections between nodes in a branched pipe system using GAs.
- Extension of the above model to a drip irrigation system considering the optimum connection between demand nodes and the sizes of the corresponding pipes using

GAs. The product, is an optimisation model, OPDESGA which uses GAs and enumeration techniques, to search for the optimum design of a regular (rectangular) drip irrigation system.

- Generalisation of the above work to an irregular branched pipe network for any irrigation purposes including the pump selection using genetic algorithms. The major product of this modification, is a GA optimisation model which deals with connection between demand nodes (layout), component sizes and pump selection simultaneously.

Two optimisation techniques were used throughout the study. The models dealing with micropipes and multiple subunit systems were formulated using a complete enumeration technique, and the models dealing with the optimum connection between nodes and component sizes (the major) were formulated using genetic algorithms.

10.2 MODELS FOR MICRO IRRIGATION SYSTEMS

In the first part of this study, three optimisation models for the design of micropipes within the field were developed. In the first simple model, the geometry of a field and the operating parameters including irrigation intervals and irrigation times were examined. The results, for a micropipe system, indicate that when the ratio of field dimensions (length parallel to the laterals and width parallel to the manifold) approaches unity the system cost converges towards the global optimum cost. This finding is consistent with the results obtained by Oron and Walker, 1981.

Instead of applying the required irrigation water continuously with a slow rate, various operation schedules to satisfy the irrigation requirements may be considered. However, the operating schedule, which is directly related to the irrigation interval and irrigation duration, is limited by the following constraints: (i) as the irrigation interval increases, and irrigation time decreases, a larger flow rate should be supplied by the piping system; (ii) if the rate of application exceeds the soil infiltration rate, irrigation may cause runoff and consequently soil erosion. (iii) too slow an application rate may cause clogging and siltation problems.

Applying various irrigation intervals and irrigation times provides a wide range of flexibility to the designer to select a more appropriate alternative, considering operation and management constraints. All optimum designs were obtained subject to a desirable pressure variation which in turn, achieves an acceptable water distribution uniformity across the field.

This simple model was extended to sloping lands in which a number of factors, including the effect of slopes on the optimum design, were examined. To keep the pressure variation within a reasonable range, an analysis was carried out considering pressure gains and losses due to slope and fraction. This was performed under various loading cases resulting from different combinations of irrigation intervals and irrigation times. To assess the distribution uniformity for the optimum design the Christiansen coefficient (UC) was computed for various loading cases. The result ranges between 0.959 and 0.996 which is consistent with the findings of Helmi et al, 1993; and Warrick and Yitayew, 1988. The findings indicate that, in order to achieve a reasonably uniformity distribution of water within the field, the following factors should be taken into account:

(1) On steep slopes the use of pressure regulating valves to reduce the pressure at critical points is essential;

(2) For mild slopes, as the slope increases the length of micropipes and also the dimension of subunits should be reduced;

(3) If the condition (2) is not possible the size of pipes on the up slope should be increased. Note that the effect of pipe sizes is limited particularly for steep slopes;

(4) If conditions (2) and (3) are not effective, decreasing the flow rate in pipes could be helpful. This may be achieved by scheduling an appropriate operating program;

(5) Apart from using pressure regulators, moving the position of the manifold and the supply pipe towards the up slope is a useful way to reduce the effect of slope on the pressure variation.

Partitioning of a field into various smaller units (subunits) with different dimensions and also the analysis of optimum design of multiple subunit drip irrigation systems on flat terrain was studied in Chapter 5. This facilitates the agricultural activities such as fertilisation, plowing, fruit picking, spraying, soil treatments, etc.. It also provides more control and more flexibility for irrigation systems. This, in turn, increases the reliability of the system. The model, OPSHIR was developed using an enumeration approach for this purpose. The objective function (system cost) was minimised subject to a number of constraints including: (i) depth of water which can be stored in the root zone; (ii) the rate of irrigation application; (iii) the percentage of wetted area; (iv) the pressure variation within subunits. The lengths of two given diameters of laterals, the diameters of other micropipes within the subunits, the diameters of submain and mainiline pipes, the power of pump as well as the irrigation shift patterns were considered as decision variables. In this study, only contiguous shift patterns were considered.

The findings indicate that a significant saving cost may be achieved using an appropate shift pattern. When the model was applied to a particular case study, it showed that one shift operation was more efficient than multiple shift operation. In general, it would appear that the minimum number of shifts consistent with minimum dripper flow rates should be used. The effect of subunit sizes for three applied irrigation shifts were investigated. The minimum solution in each case was obtained when the area of each subunit was between 1.3 and 3.0 ha. In each case these corresponded to using only the smaller size of laterals. The general conclusions from this section of the study may be summarised as follows:

- Under higher number of shift operations the size of subunits is limited since using a high number of shifts the flow rate and consequently, the head loss in pipes increases. This, violates the pressure unifomity distribution;
- The use of smallest size of laterals (if possible) leads to the least cost solution;
- The lowest system cost is achieved when the shortest irrigation interval and longest irrigation time are used;
- As dimension ratio of subunits approaches 1, the system cost converges towards the minimum;
- A significant saving cost may be achieved through the use of the best shift pattern;

10.3 MODELS FOR OPTIMUM LAYOUT AND COMPONENT SIZES USING GENETIC ALGORITHMS

The second part of this thesis, deals with the application of genetic algorithms to branched piping systems. Firstly, the theory of genetic algorithms (GAs) methodology and a review of their application to pipe network optimisation problems, were examined. To apply this new search and optimisation method to branched piping systems, three GA models were developed in this part of thesis. The first model, deals with layout problems only. The selection of layout is important, since it serves as the foundation of any pipe network design. For simplicity it was assumed that a number of demand nodes are located in a rectangular grid pattern.

The formulation of the model used the directed base graph which was useful to reduce the size of the search space. As the diameters of all pipes were unknown the cost of pipes was modelled as a function of the flow in the pipes. The selection of each link is expected to be performed on the basis of the length and the capacity of the pipe. However, the configuration of demand nodes (regular pattern) and the limited possibility of connectivities (only 2 options to each node) reduce the effect of pipe lengths for optimum solution since the total length of each selected path would not differ significantly. It appears that the flow rate in the pipes is more important in the selection of the least cost paths than the pipe lengths for this configuration in this model. The finding indicates that the optimal solution has the pipes aligned predominantly in one direction. i.e. parallel to one edge of the field. Comparison of two applied selection methods shows that tournament selection reaches an optimal solution more rapidly than proportionate selection for this problem. The least cost solution was obtained using tournament selection.

In order to consider both layout and pipe diameters simultaneously, the model was extended. The product of this extension is a GA model which deals with optimum layout, sizes of major pipes and optimum design of subunits. In this model, each demand node was considered as a supply node for a subunit. The size of the search space in this model, was much larger than the previous model which deals just with layout problems. There are at least 15×10^{26} possible alternative solutions for the network with 20 demand nodes. Tournament selection performs much better than proportionate selection for this model. Not only it does find the least cost solution within a smaller number of evaluations but also it finds a solution with lower cost. The cost savings using tournament selection compared to proportionate selection was 4.66% ($18990). It also appears that using discrete sizes of pipes and cost as function of pipe diameter for controlling the pressure at nodes is more realistic than using the cost as a function of the flows in pipes. The findings indicate that the results obtained by this model is consistent with the result found by the previous GA model.

In order to generalise this work, the model OPIRSYS was developed. This model deals with any branched piping system and searches for the optimum layout, optimum pipe sizes and pump selection simultaneously. To reach an optimum solution, a trade off for the selected pump, pipe dimeters and connection between nodes was carried out. Infeasible solutions were discouraged from the search space by applying two penalty costs: one for disconnectivity and the other for violation in minimum required pressure at nodes. A new set of lower cost solutions was observed using the optimum level of GA parameters resulting from a sensitivity analysis. The least cost solution which seems to be very close to the global optimum was obtained with a probability of 0.75 for crossover, 0.02 for mutation and with tournament size of 5.

The results indicate, that the cost of the best solution declines during the evaluation and it converges to a minimum which is expected to be close to the global optimum. The results from both selection methods indicate that at the start, the curve for the least cost is very steep. At this stage, it is not very difficult to achieve improvement in almost every generation as there are a lot of potential solutions of lower cost. Later on, the search space becomes progressively narrower and the best cost can only be improved occasionally. But this is the range where GAs are actually most effective. The algorithm developed in this model, for optimum layout overcomes the problems stated in the methods suggested by Labye et al, 1988.

The overall conclusions from this section of the study may be summarised as follows:

- Genetic algorithms represent a powerful and robust approach for developing heuristics for pipe network optimisation problems;
- Genetic algorithms are very effective in finding the optimum connection between nodes and are superior to the graphical methods (suggested by Lebye et al, 1988) and the other methods given in the literature (eg. dynamic programming, Walters et al, 1993), particularly, for the networks with a high degree of connectivity;
- The GA with tournament selection was found to be reach an optimal solution more rapidly than proportionate selection for the problems examined in this thesis;
- Although GAs can't guarantee to find the global optimum, they do find a near optimal solution with evaluation of a small fraction of the total search space;

- Using a directed base graph in general, and symmetry in the regular networks make a significant reduction in the size of search space and consequently, in the computer memory and run time;
- For regular pipe networks the optimal solution has the pipes aligned predominantly in the one direction (i.e. parallel to one side of rectangular field).

10.4 RECOMMENDATIONS FOR FURTHER RESEARCH

Many simulation models have been developed in irrigation and water distribution systems in recent years. There are few optimisation models dealing solely with pressure irrigation systems. Most of these are concerned only with the pipe systems without considering agronomical and agrotechnical problems. However, considering all the factors which affect irrigation to maximise the objective function, including all corresponding costs and revenues in a fully comprehensive model, is not a simple issue. In this way, a number of complicated factors such as agronomy, management, soil science and engineering need to be taken into account.

In this thesis, an effort has been made to consider some agronomical and agrotechnical aspects in the optimum design of pressure irrigation systems in such a manner as to minimise the system cost. The analysis of some factors were beyond the scope of this thesis. Also a number of assumptions have been made in the models developed, which limit the general applicability of these models. Thus, there is still a wide range of work in this area that needs to be carried out to achieve a full and comprehensive model for pressure irrigation systems. The further research related to the work outlined in this thesis includes the following:

Although the model for the optimum design of subunits outlined in Chapter 3 was modified for sloping lands in Chapter 4, it is limited to very mild and uniform sloping lands. Considering pressure reducing valves in the system, determining the positions of these valves and the use of pressure compensating emitters for different slopes is recommended. This, will necessitates a new formulation for the model discussed in Chapter 4.

In the design of a multiple subunit systems outlined in Chapter 5, it was assumed that: (i) the soil throughout the field is homogeneous; (ii) the crops in all subunits are the same; (iii) the pumping system is located at the centre; and (iv) only one orifice type emitter is considered for each plant. Further work needs to be carried out to develop a model for various crops considering the possible variety in the soil characteristics and possible use of multi exit emitters for each plant in the system. The source node(s) supplying subunits may be located anywhere within or outside of the field and also may not be limited to only one source node. In the model, dealing with multiple subunit systems only contiguous shift patterns have been considered. Further research on non-contiguous patterns is recommended to examine a wide range of possibilities for the least cost solution.

Investigation to extend the model to irregular fields with varying topography would be necessary to generalise the model from a flat rectangular field to any irregular case with varying topography. Also considering a variable pattern for the layout of pipes instead of a fixed layout could be included in future studies. Leaching requirements in some areas with salinity problems are important. It is recommended that this issue be taken into account in a comprehensive model, to estimate a more realistic gross irrigation depth.

As the number of decision variables increases, the size of the search space increases as well. So using a more efficient optimisation technique instead of enumeration is recommended. For the models in the first section of the thesis, GAs could be used for this purpose. A comparison of the results with other models to verify their accuracy is also recommended.

Although GA models developed in this thesis were tested with different examples and the results were analysed with different views, the calibration or comparison of results with other models was beyond the scope of this research. In addition, it is possible that further improvement of the GA models developed could be obtained. This is an area which should be examined, particularly through the use of other coding schemes and different crossover and mutation operators.

In the model for irregular layouts, discussed in Chapter 9, a network with a maximum of 30 pipes with limited numbers of demand nodes were examined. As the number of demand nodes and the number of possible links to each node increase the length of strings and the size of search space increase. In the problems dealing with layout and component sizes, the size of the search space is much larger than the problems dealing with only layout or component sizes. Further research needs to be undertaken to examine the performance of the model on large networks with high connectivity to evaluate the capability of the developed GA models for large networks. The possible use of messy genetic algorithms (*mGA*, Goldberg et al, 1989) and greedy genetic algorithms (*gGA*, Ahuja et al, 1995) particularly for large networks is an area of new work which is recommended to be carried out. This will give a clear view of the capability of different GAs to see which type of GA works better for branched pipe systems.

The effect of the installation cost of pipes on the optimum layout and also the use of a greater number of discrete pipe sizes with less difference in size for the selected links are factors that need to be analysed in a further study to find a solution closer to the global optimum. The model outlined in Chapter 9, was not formulated for the design of subunits and each node simply assumed to be a hydrant or a supply node. The model could be more elaborate and comprehensive if a subunit with micro pipes is designed for each demand node. In this model, the selection of the pump is based upon the pump speed. For any selected speed from a set of given speeds and discharge, the required pump is found. This is for a fixed pump stage which works for a limited range of discharges and heads. In a future study, it is recommended that the pump stage be also considered as a decision variable in the GAs process.

The strings of GA models examined in this thesis, were divided in two segments: one segment for layout and the other for pipe sizes. It appears that if each pair of adjacent bits represented a nodal connection and the corresponding pipe size, the product of crossover and consequently the final result may be different. This may affect the minimum cost solution and is therefore recommended for further study.

The models developed in this thesis are only for branched systems, this needs to be extended to loops systems particularly where multiple loading cases apply. Also the possibility of having booster pumps or pressure reducing valves at various locations need to be investigated. A hydraulic network solver is needed for this purpose. This should be investigated in further research.

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Appendix A

Development of head loss equation in a multiple outlet pipe with a single size

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The head loss in a small section of pipe of length dx as shown in Figure 4.4, may be calculated using Equation 3.15 as follows:

$$hl_{dx} = \frac{10.68dx\left(\frac{Q_0}{L}(L-x)\right)^{1.852}}{C^{1.852}D^{4.87}} \tag{A.1}$$

Integrating over length x

$$\int_0^x hl_x = \int_0^x 10.68\frac{\left((L-x)\frac{Q_0}{L}\right)^{1.852} dx}{C^{1.852}D^{4.87}} \tag{A.2}$$

$$\int_0^x hl_x = \frac{10.68}{C^{1.852}D^{4.87}}\frac{Q_0{}^{1.852}}{L^{1.852}}\int_0^x (L-x)^{1.852}\,dx \tag{A.3}$$

$$\int_0^x hl_x = -\frac{10.68}{C^{1.852}D^{4.87}}\frac{Q_0^{1.852}}{L^{1.852}}\left[\frac{1}{2.852}(L-x)^{2.852}\right]_0^x \tag{A.4}$$

$$hl_x = -\frac{10.68}{C^{1.852} D^{4.87}} \frac{Q_0^{1.852}}{L^{1.852}} \left[\frac{1}{2.852} (L-x)^{2.852} - \frac{1}{2.852} L^{2.852} \right] \tag{A.5}$$

$$hl_x = 3.745 \frac{Q_0^{1.852}}{C^{1.852} D^{4.87} L^{1.852}} \left[L^{2.852} - (L-x)^{2.852} \right] \tag{A.6}$$

$$hl_x = 3.745 \frac{Q_0^{1.852} L}{C^{1.852} D^{4.87}} \left[1 - \left(\frac{L-x}{L} \right)^{2.852} \right] \tag{A.7}$$

Equation (A.7) may be used to calculate the head loss , hl_x, at any distance like x along a multiple outlet pipe.

Appendix B

Development of head loss equation for a multiple outlet pipe with continuous flow at the end

Referring to Figure 4.6, the head loss , in a small length of pipe d_x , is given by :

$$hl_{dx} = \frac{10.68\left(Q_{ss} + (Q_0 - Q_{ss}) \times \frac{(L-x)}{L}\right)^{1.852} dx}{C^{1.852} D^{4.87}} \quad \text{(B.1)}$$

Integrating over length x:

$$\int_0^x hl_{dx} = \int_0^x \frac{10.68\left(Q_{ss} + (Q_0 - Q_{ss}) \times \frac{(L-x)}{L}\right)^{1.852} dx}{C^{1.852} D^{4.87}} \quad \text{(B.2)}$$

$$hl_x = \frac{-10.68L}{C^{1.852} D^{4.87} (Q_0 - Q_{ss})} \int_0^x \left(Q_{ss} + (Q_0 - Q_{ss}) \frac{(L-x)}{L}\right)^{1.852} \left(-\frac{1}{L}(Q_0 - Q_{ss}) dx\right) \text{(B.3)}$$

$$hl_x = \frac{-10.68L}{C^{1.852}D^{4.87}(Q_0 - Q_{ss})}\left[\frac{1}{2.852}\left(Q_{ss} + (Q_0 - Q_{ss})\frac{(L-x)}{L}\right)^{2.852}\right]_0^x \quad \text{(B.4)}$$

$$hl_x = \frac{-3.745L}{C^{1.852}D^{4.87}(Q_0 - Q_{ss})}\left[\left(Q_{ss} + (Q_0 - Q_{ss})\frac{L-x}{L}\right)^{2.852} - {Q_0}^{2.852}\right]$$

(B.5)

$$hl_x = \frac{3.745L}{C^{1.852}D^{4.87}(Q_0 - Q_{ss})}\left[Q_0^{2.852} - \left(Q_{ss} + \frac{Q_0 - Q_{ss}}{L}(L-x)\right)^{2.852}\right] \quad \text{(B.6)}$$

Appendix C

A typical turbine pump with a vertical hollow-shaft electro motor drive and corresponding characteristic curves

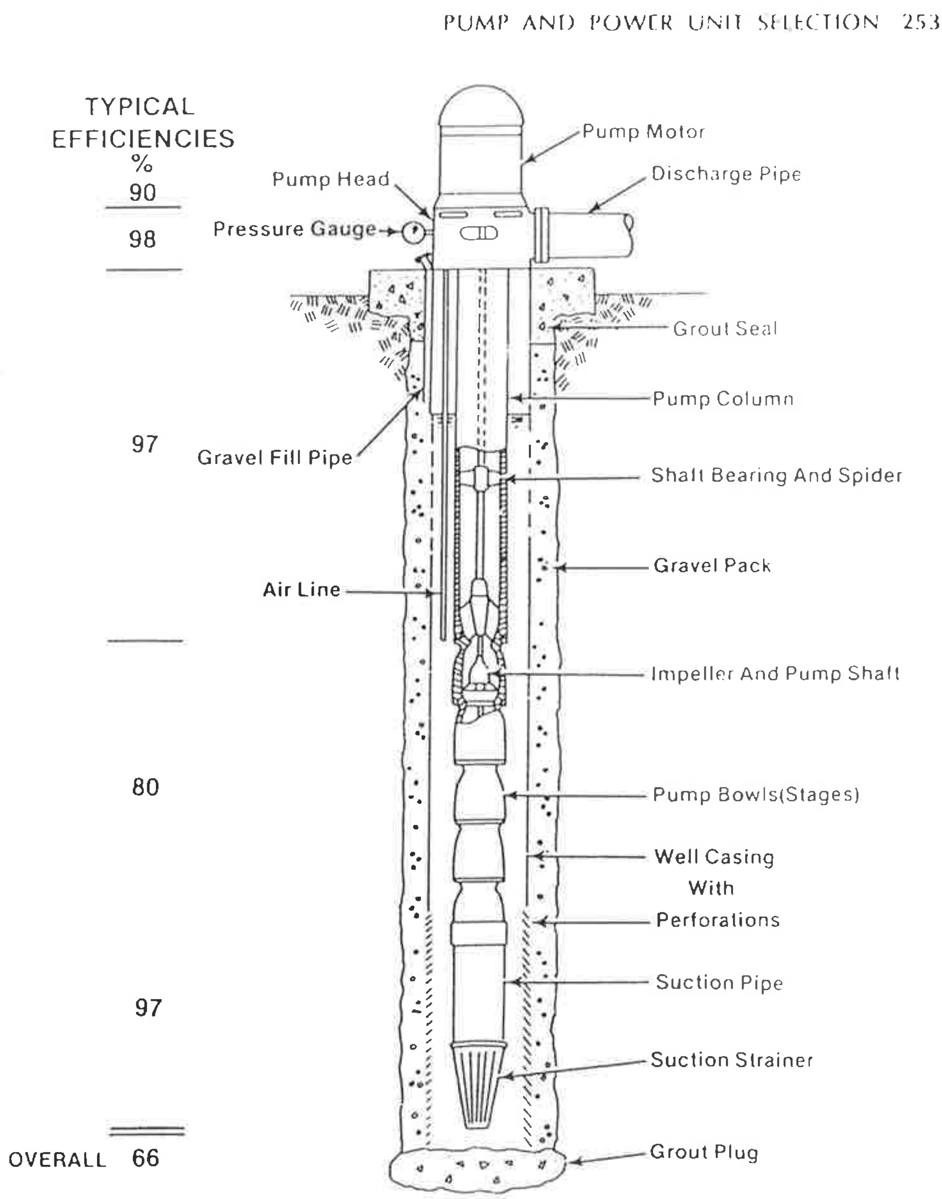


Fig. C.1 A typical turbine pump with vertical hollow- shaft (Keller and Blisener, 1990)

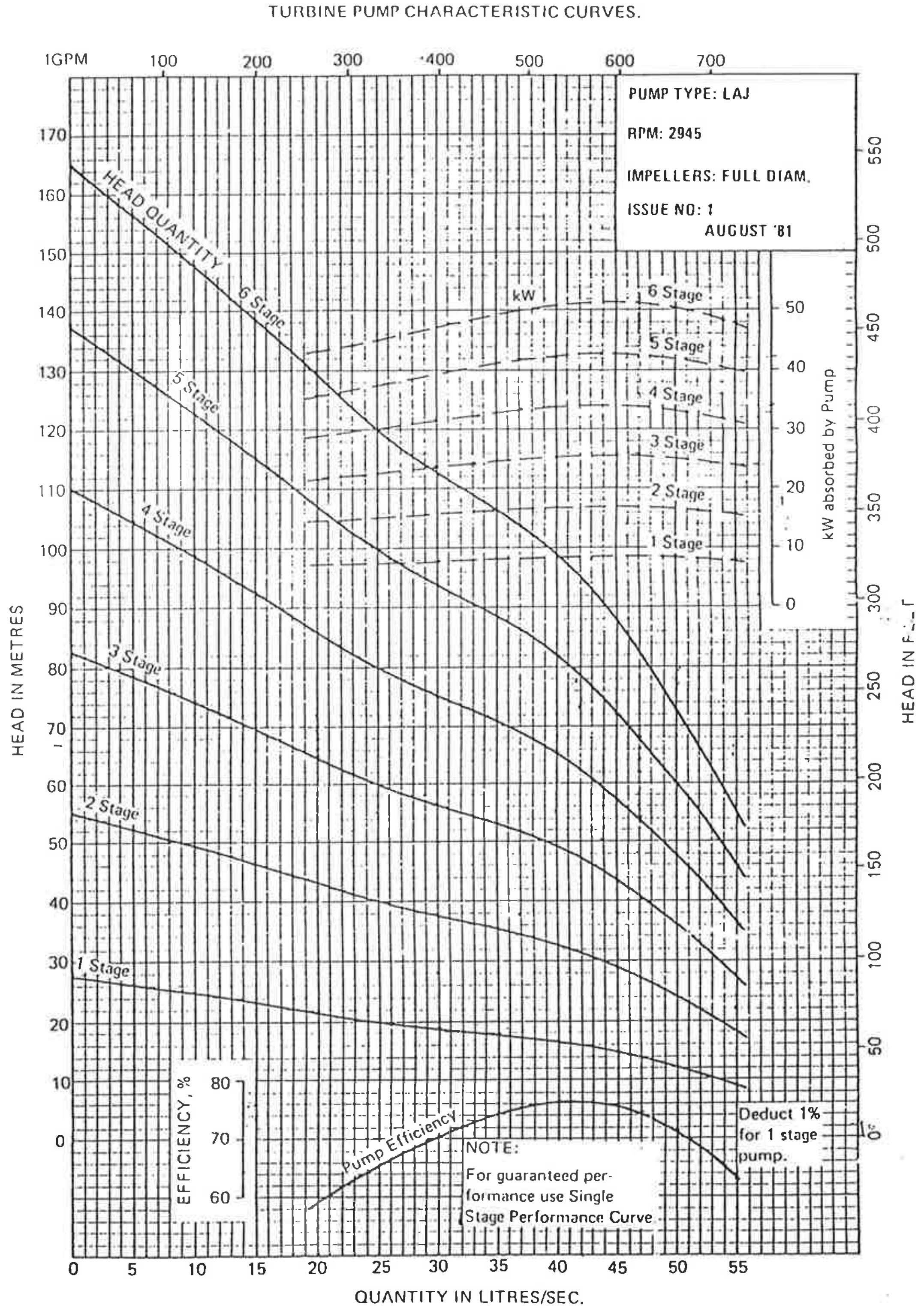


Fig. C. 2 Characteristic curves for LAJ turbine pump type (Southern Cross Manufacturing, 1992)

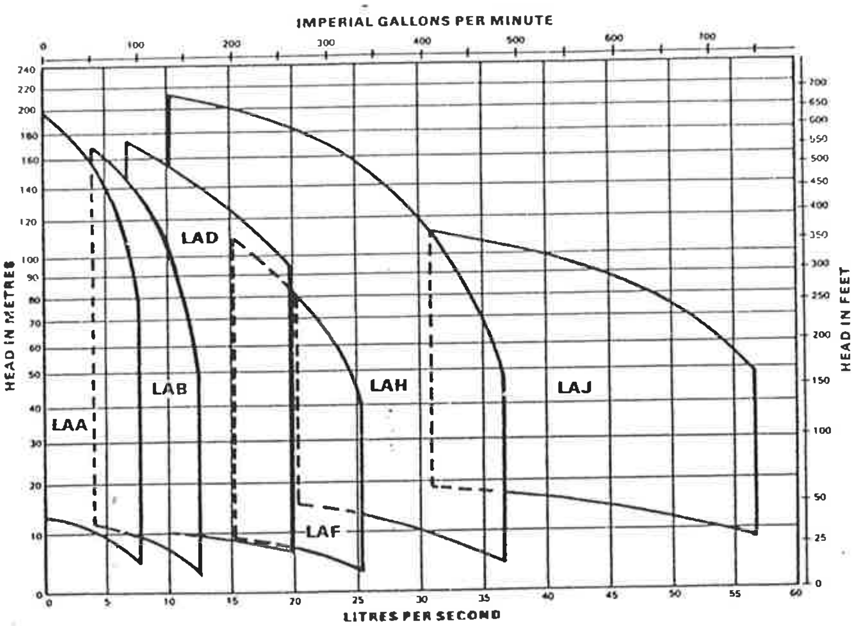


Fig. C.3 Characteristic curves for different type of turbine pumps (Southern Cross Manufacturing, 1992)

Appendix D

The details of system component costs corresponding to different slopes in the X and Y directions (Irrigation interval of 1 day and irrigation time of 12 hours)

slope X direction (%)	slope Y direction (%)	pump flow (m3/s)	design head (m)	pipe cost ($)	pump cost ($)	annual op. cost ($)	total cost ($)
0	0	0.00882	41.98	19936	12428	4923	46702
0	0.2	0.00882	41.98	19936	12428	4923	46702
0	0.4	0.00882	41.98	19936	12428	4923	46702
0	0.6	0.00882	41.98	19936	12428	4923	46702
0	0.8	0.00882	41.98	19562	12428	4923	46328
0	1	0.00882	41.98	19887	12428	4923	46653
0	1.2	0.00882	41.98	21025	12428	4923	47791
0.1	0	0.00882	41.83	19936	12388	4905	46644
0.1	0.2	0.00882	41.83	19936	12388	4905	46644
0.1	0.4	0.00882	41.83	19936	12388	4905	46644
0.1	0.6	0.00882	41.83	19936	12388	4905	46644
0.1	0.8	0.00882	41.83	19562	12388	4905	46270
0.1	1	0.00882	41.83	19887	12388	4905	46595
0.1	1.2	0.00882	41.83	21025	12388	4905	47733
0.2	0	0.00882	41.68	19936	12348	4887	46586
0.2	0.2	0.00882	41.68	19936	12348	4887	46586
0.2	0.4	0.00882	41.68	19936	12348	4887	46586
0.2	0.6	0.00882	41.68	19936	12348	4887	46586
0.2	0.8	0.00882	41.68	19562	12348	4887	46212
0.2	1	0.00882	41.68	19887	12348	4887	46538
0.2	1.2	0.00882	41.68	21025	12348	4887	47675
0.3	0	0.00882	41.53	19936	12308	4870	46528
0.3	0.2	0.00882	41.53	19936	12308	4870	46528
0.3	0.4	0.00882	41.53	19936	12308	4870	46528
0.3	0.6	0.00882	41.53	19936	12308	4870	46528

0.3	0.8	0.00882	41.53	19562	12308	4870	46154
0.3	1	0.00882	41.53	19887	12308	4870	46480
0.3	1.2	0.00882	41.53	21025	12308	4870	47617
0.4	0	0.00882	41.38	19936	12267	4852	46471
0.4	0.2	0.00882	41.38	19936	12267	4852	46471
0.4	0.4	0.00882	41.38	19936	12267	4852	46471
0.4	0.6	0.00882	41.38	19936	12267	4852	46471
0.4	0.8	0.00882	41.38	19562	12267	4852	46097
0.4	1	0.00882	41.38	19887	12267	4852	46422
0.4	1.2	0.00882	41.38	21025	12267	4852	47560
0.5	0	0.00882	41.23	19936	12227	4835	46413
0.5	0.2	0.00882	41.23	19936	12227	4835	46413
0.5	0.4	0.00882	41.23	19936	12227	4835	46413
0.5	0.6	0.00882	41.23	19936	12227	4835	46413
0.5	0.8	0.00882	41.23	19562	12227	4835	46039
0.5	1	0.00882	41.23	19887	12227	4835	46364
0.5	1.2	0.00882	41.23	21025	12227	4835	47502
0.6	0	0.00882	41.08	19936	12187	4817	46355
0.6	0.2	0.00882	41.08	19936	12187	4817	46355
0.6	0.4	0.00882	41.08	19936	12187	4817	46355
0.6	0.6	0.00882	41.08	19936	12187	4817	46355
0.6	**0.8**	**0.00882**	**41.08**	**19562**	**12187**	**4817**	**45981**
0.6	1	0.00882	41.08	19887	12187	4817	46306
0.6	1.2	0.00882	41.08	21025	12187	4817	47444
0.7	0	0.00882	40.93	19936	12147	4799	46297
0.7	0.2	0.00882	40.93	19936	12147	4799	46297
0.7	0.4	0.00882	40.93	19936	12147	4799	46297
0.7	0.6	0.00882	40.93	19936	12147	4799	46297
0.7	0.8	0.00882	40.93	19810	12147	4799	46171
0.7	1	0.00882	40.93	19887	12147	4799	46249
0.8	0	0.00882	40.78	19936	12107	4782	46239
0.8	0.2	0.00882	40.78	19936	12107	4782	46239
0.8	0.4	0.00882	40.78	19936	12107	4782	46239
0.9	0	0.00882	40.63	19936	12066	4764	46181
0.9	0.2	0.00882	40.63	19936	12066	4764	46181

Appendix E

Emission Uniformity

Karmeli and Keller (1975) defined emission uniformity, EU (%) as follows:

$$EU = 100\left(1 - 1.27\frac{\mu}{\sqrt{N_p}}\right)\frac{Q_n}{\overline{Q}_E} \tag{E.1}$$

The discharge from an orifice type emitter Q (L/hr) is given by:

$$Q = K_e(h)^x \tag{E.2}$$

where h= pressure head on the emitter (m); K_e, x constants.

For an average working head of 10m, a minimum value of 9m , and taking x=0.5 for this case, gives:

$$\frac{Q_n}{\overline{Q}_E} = \left(\frac{9}{10}\right)^{0.5} = 0.9487 \qquad ((E.3)$$

Assuming $\frac{\mu}{\sqrt{N_p}} = 0.04$ gives a value of 90% for EU.

Keller and Karmeli (1974) states that under good management the overall application efficiency should approach 0.9 of EU (i.e. 81% in this case).

Appendix F

Derivation of head loss in laterals

Consider a pipe of uniform diameter D (m), length L (m) and spacing of emitters d_x (m). If the discharge from each emitter is Q_E(m^3/s), the discharge along the pipe is shown in Figure F.1. The discharge into the left hand end of the pipe is Q_0(m^3/s). If the spacing of emitters is small, the discharge in the pipe may be considered to be linear function of l, the distance from the left hand end,

$$\text{i.e.} \quad Q_l = \frac{Q_0(L-l)}{L} \tag{F.1}$$

The head loss in any small length of pipe dl (m) is given by applying the Hazen-Williams equation.

$$d(HL) = \frac{10.68}{C^{1.852}D^{4.87}}\left[\frac{Q_0(L-l)}{L}\right]^{1.852} dl \tag{F.2}$$

Where C= Hazen-Williams coefficient for the pipe.

Integrating Equation (F.2) from 0 to l

gives: $$HL = \frac{3.745 Q_0^{1.852} L}{C^{1.852} D^{4.87}} \left[1 - \left(\frac{L - l}{L} \right)^{2.852} \right] \tag{F.3}$$

Clearly if l equals L, Equation (F3) becomes:

$$HL = \frac{3.745 Q_0^{1.852} L}{C^{1.852} D^{4.87}} \tag{F.4}$$

Consider a lateral consisting of a length L_1(m) of diameter D_b(m) and length L_2(m) of diameter D_s(m) with Hazen-Williams coefficient CH_l throughout. Applying Equations (F.3) and (F.4) to find the total head loss in the lateral, Hl_l, gives :

$$HL_l = \frac{3.745}{CH_l^{1.852}} \left[\frac{Q_0^{1.852}(L_1 + L_2)}{D_{bs}^{4.87}} \left\{ 1 - \left(\frac{L_2}{L_1 + L_2} \right)^{2.852} \right\} + \left(\frac{Q_0 L_2}{L_1 + L_2} \right)^{1.852} \frac{L_2}{D_{ss}^{4.87}} \right] \tag{F.5}$$

Substituting $Q_0 = \dfrac{2.78 \times 10^{-7} Q_E (L_1 + L_2)}{d_x}$ and simplifying yields:

$$HL_l = 3.745 \left(\frac{2.78 \times 10^{-7} Q_E}{CH_l d_x} \right)^{1.852} \left[\frac{\left\{ (L_1 + L_2)^{2.852} - L_2^{2.852} \right\}}{D_{bs}^{4.87}} + \frac{L_2^{2.852}}{D_{ss}^{4.87}} \right] \tag{F.6}$$

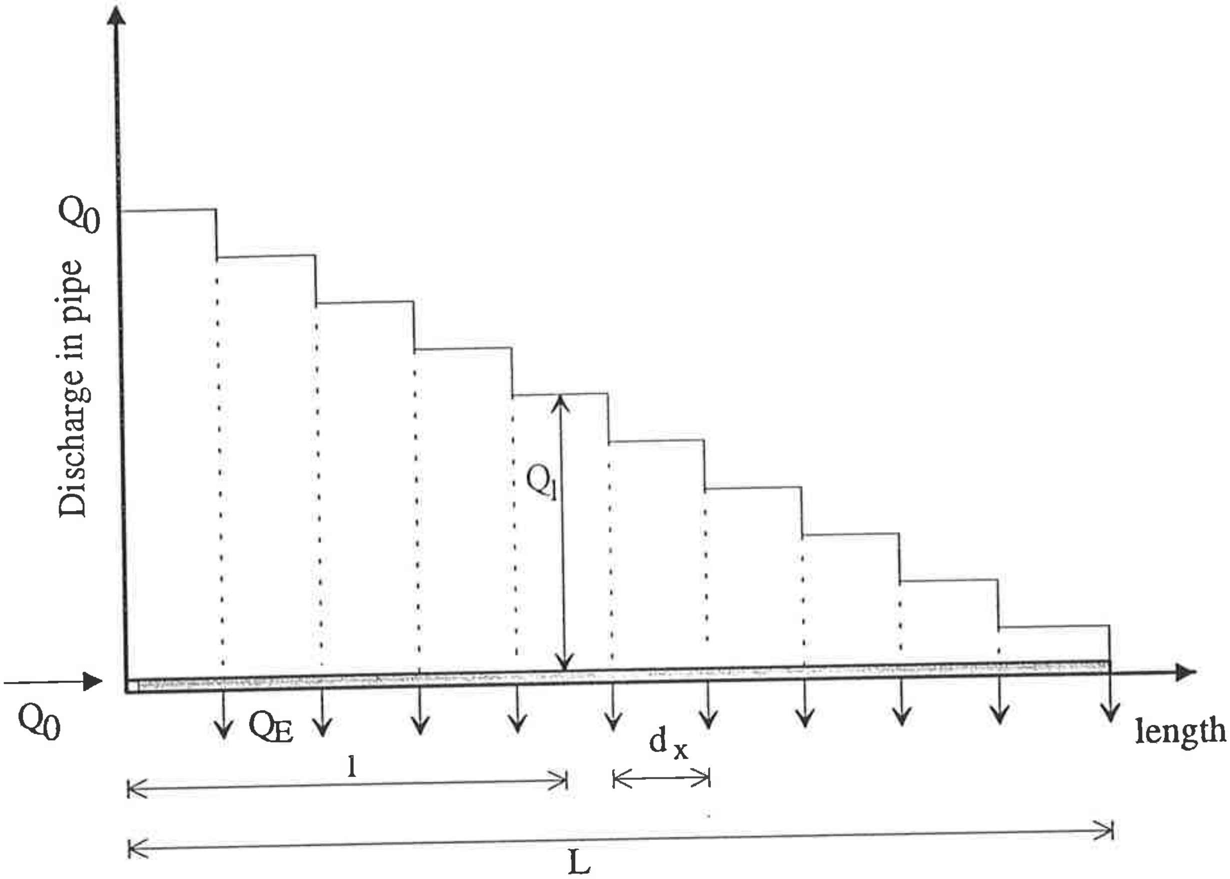


Fig. F.1 Discharge in multiple out let pipes

Appendix G

Flow chart for estimation of irrigation requirements

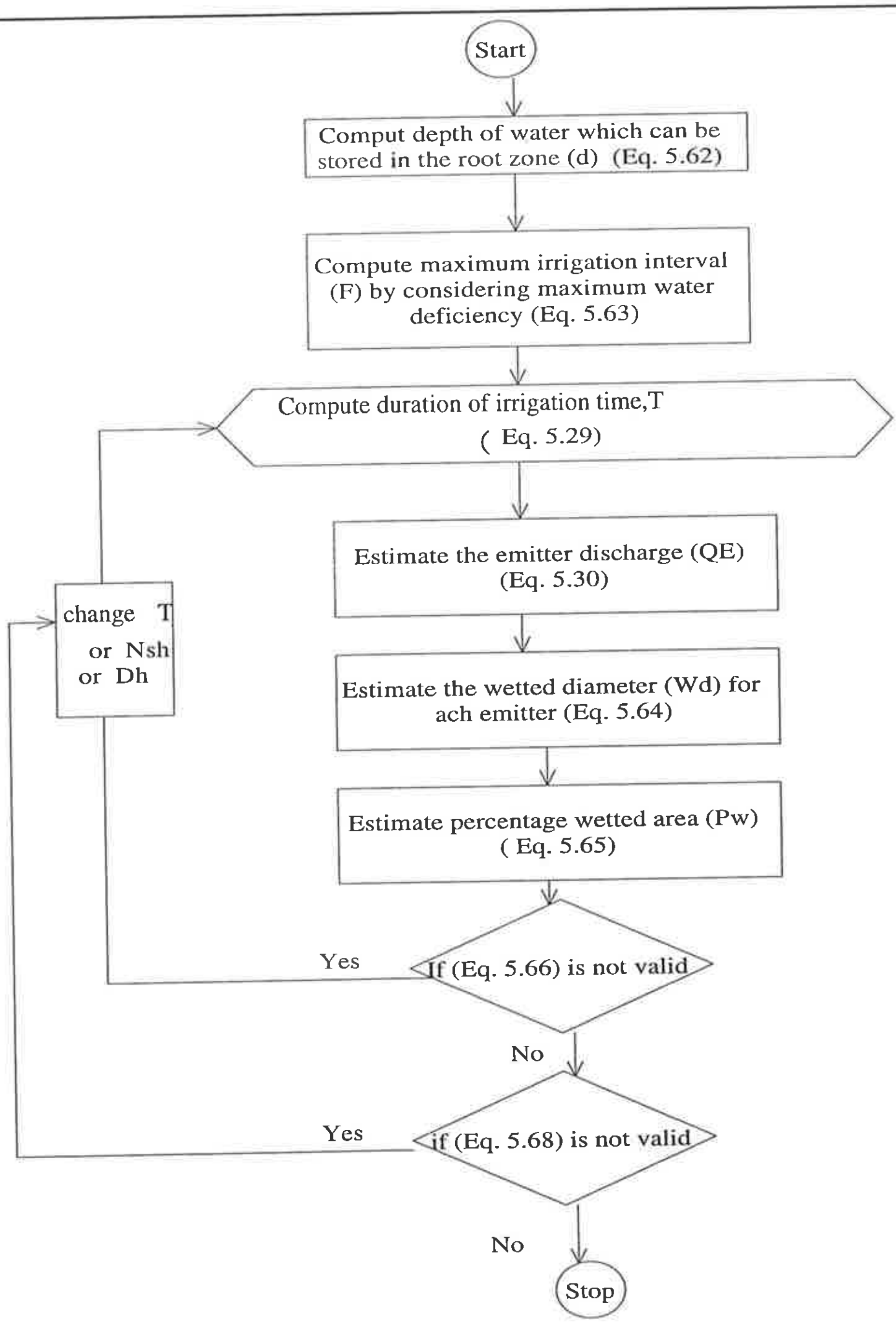


Fig. G.1 Flowchart of irrigation requirement estimating

Appendix H

Typical examples of created strings for least cost layout determination (tournament selection)

No. of strings (trial solutions)	No. of generations	No of evaluations	best cost	average cost	maximum cost
11011111010001010101	1	20.00	51752.0	52783.8	54400.7
11111110111011101111	2	40.00	48606.1	52120.9	53848.5
11111110111010101111	3	60.00	49235.6	51669.8	53514.5
11111110111010101111	4	80.00	49235.6	51322.8	53121.1
11111110111010101111	5	100.00	49235.6	50627.1	52449.5
11111111111010101111	6	120.00	48856.5	50147.0	51540.3
01111111110111111111	7	140.00	48726.1	49930.8	51003.1
11111111110111111111	8	160.00	47980.7	49789.0	51208.2
11111111110111111111	9	180.00	47980.7	49406.7	51294.4
11111111110111111111	10	200.00	47980.7	49276.9	50576.2
11111111110111111111	11	220.00	47980.7	49020.7	50481.7
11111110111011111111	12	240.00	48195.5	48858.0	49894.3
11111111111011111111	13	260.00	47816.5	48785.3	50033.2
11111111111011111111	14	280.00	47816.5	49070.7	51725.9
11111111111011111111	15	300.00	47816.5	48711.3	49994.9
11111111111111111111	16	320.00	47359.5	48783.6	50921.3
11111111111111111111	17	340.00	47359.5	48707.6	49890.9
11111111111111111111	18	360.00	47359.5	48642.7	49894.3
11111111111111111111	19	380.00	47359.5	48647.9	50976.3
11111111111111111111	20	400.00	47359.5	48399.4	51513.8
11111111111111111111	21	420.00	47359.5	48212.5	49486.0
11111111111111111111	22	440.00	47359.5	48158.7	50560.6
11111111111111111111	23	460.00	47359.5	47931.2	49635.1
11111111111111111111	24	480.00	47359.5	48007.6	49102.9
11111111111111111111	25	500.00	47359.5	48133.1	49703.1
11111111111111111111	26	520.00	47359.5	48157.6	49439.1
11111111111111111111	27	540.00	47359.5	48082.7	49492.2
11111111111111111111	28	560.00	47359.5	47950.9	49145.9
11111111111111111111	29	580.00	47359.5	47891.1	49678.5
11111111111111111111	30	600.00	47359.5	47806.1	50131.9
11111111111111111111	31	620.00	47359.5	47882.5	49389.6
11111111111111111111	32	640.00	47359.5	47755.4	49046.3
11111111111111111111	33	660.00	47359.5	47825.1	49054.4
11111111111111111111	34	680.00	47359.5	47682.5	48481.4
11111111111111111111	35	700.00	47359.5	47791.5	48993.7
11111111111111111111	36	720.00	47359.5	47783.8	48989.6
11111111111111111111	37	740.00	47359.5	47804.2	49314.0
11111111111111111111	38	760.00	47359.5	47896.0	49787.7
11111111111111111111	39	780.00	47359.5	47769.9	48463.0
11111111111111111111	40	800.00	47359.5	47739.6	48766.7
11111111111111111111	41	820.00	47359.5	47919.5	49600.3
11111111111111111111	42	840.00	47359.5	47778.3	48965.9
11111111111111111111	43	860.00	47359.5	48026.8	50739.1
11111111111111111111	44	880.00	47359.5	48154.0	50070.2
11111111111111111111	45	900.00	47359.5	47872.5	49567.0
11111111111111111111	46	920.00	47359.5	47998.2	49871.2
11111111111111111111	47	940.00	47359.5	48124.7	50410.5
11111111111111111111	48	960.00	47359.5	47991.9	49779.6
11111111111111111111	49	980.00	47359.5	47973.0	49253.9
11111111111111111111	50	1000.00	47359.5	48020.3	49865.7

proportionate selection

strings ial solutions)	No. of generation	No. of evaluation	best cost	average cost	maximum cost
01110111110001110111	1	200.00	50170.6	53341.1	57708.2
01111100111011111111	2	400.00	49623.8	53252.9	57708.2
01110111111001111101	3	600.00	49931.3	53294.8	57285.7
01101011111110111110	4	800.00	50230.3	53296.2	56780.1
11011111110111011011	5	1000.00	50108.5	53315.1	57515.2
10000111111011111011	6	1200.00	50469.9	53291.4	56732.8
01101110011010111111	7	1400.00	50278.4	53251.0	56673.7
11100111011111110110	8	1600.00	49532.9	53143.4	56673.7
11100111011111110111	9	1800.00	49055.2	53161.1	55985.6
11100111011111110111	10	2000.00	49055.2	53007.3	55885.0
11100111101110111111	11	2200.00	49429.9	52891.1	56587.1
11110111011110011011	12	2400.00	50001.2	52818.8	56305.7
11101001111111111110	13	2600.00	49980.5	52864.3	56305.7
11111111011101011110	14	2800.00	49661.2	52840.4	55922.9
01111111111111110110	15	3000.00	49958.0	52861.8	56376.4
11111111111111011100	16	3200.00	48978.6	52891.3	56725.2
11111111111111011100	17	3400.00	48978.6	52799.9	55614.1
11111111111111011001	18	3600.00	49206.3	52791.7	56209.9
11001110111111011101	19	3800.00	49505.4	52901.1	56793.6
11111110111111011101	20	4000.00	48955.9	53018.9	56497.4
11100111011101101101	21	4200.00	49775.1	53110.0	56613.0
11001100110111101111	22	4400.00	49886.5	53172.3	56472.2
11110010111111111110	23	4600.00	49784.9	53252.6	56472.2
11101100110111111101	24	4800.00	49800.1	53417.7	56146.5
11001110111110101110	25	5000.00	49993.3	53349.5	56475.4
11100111111100101110	26	5200.00	50312.0	53365.5	56475.4
11001011111111101011	27	5400.00	49976.3	53279.7	56531.0
11100111111100100111	28	5600.00	50132.8	53327.0	56403.2
11001111110111001011	29	5800.00	50233.6	53225.6	56465.4
11000111111101011101	30	6000.00	50290.9	53276.6	55743.0
01101111011100111111	31	6200.00	49740.6	53260.7	56764.8
01000110111111101011	32	6400.00	50405.4	53240.7	56548.7
01000110111111101111	33	6600.00	49786.9	53187.9	56651.2
01000111111111110111	34	6800.00	49988.9	53097.9	56288.6
01001111111111101011	35	7000.00	49928.7	53240.3	56633.5
01001111111111101010	36	7200.00	50366.7	53285.9	56934.2
11111011101110011010	37	7400.00	50779.5	53279.7	57372.2
11101111101111110111	38	7600.00	50020.3	53215.8	56853.1
11111111111110001110	39	7800.00	49826.5	53193.3	57061.9
11101110101110111111	40	8000.00	49097.7	53181.2	56472.4
11101110101110111111	41	8200.00	49097.7	53271.0	56616.4
11001111101110001111	42	8400.00	50464.7	53310.9	56616.4
11111001111111001111	43	8600.00	49993.4	53253.0	56338.7
11000111111011101111	44	8800.00	49518.6	53142.2	56944.0
11111110111011001110	45	9000.00	49895.1	53110.7	56394.8
11101101111101111111	46	9200.00	49781.3	52999.5	56394.8
11101010111110111110	47	9400.00	49710.1	52941.6	55838.2
11101010101111101111	48	9600.00	49612.5	52941.4	56902.3
11101010111111101101	49	9800.00	49985.6	52828.5	55823.9
11101010111111101101	50	10000.00	49985.6	52789.9	56364.7
11100111111111011111	51	10200.00	48917.0	52713.6	55334.1
11110110111101011111	52	10400.00	49712.7	52767.0	55357.0
11101100111111011111	53	10600.00	49268.7	52826.0	56179.6
01111111111101111110	54	10800.00	49582.4	52823.2	56179.6
01111101111011011111	55	11000.00	49916.0	52767.0	55694.6

Appendix I

Flow chart of initial population showing the generation of binary and integer numbers for a string with two segments

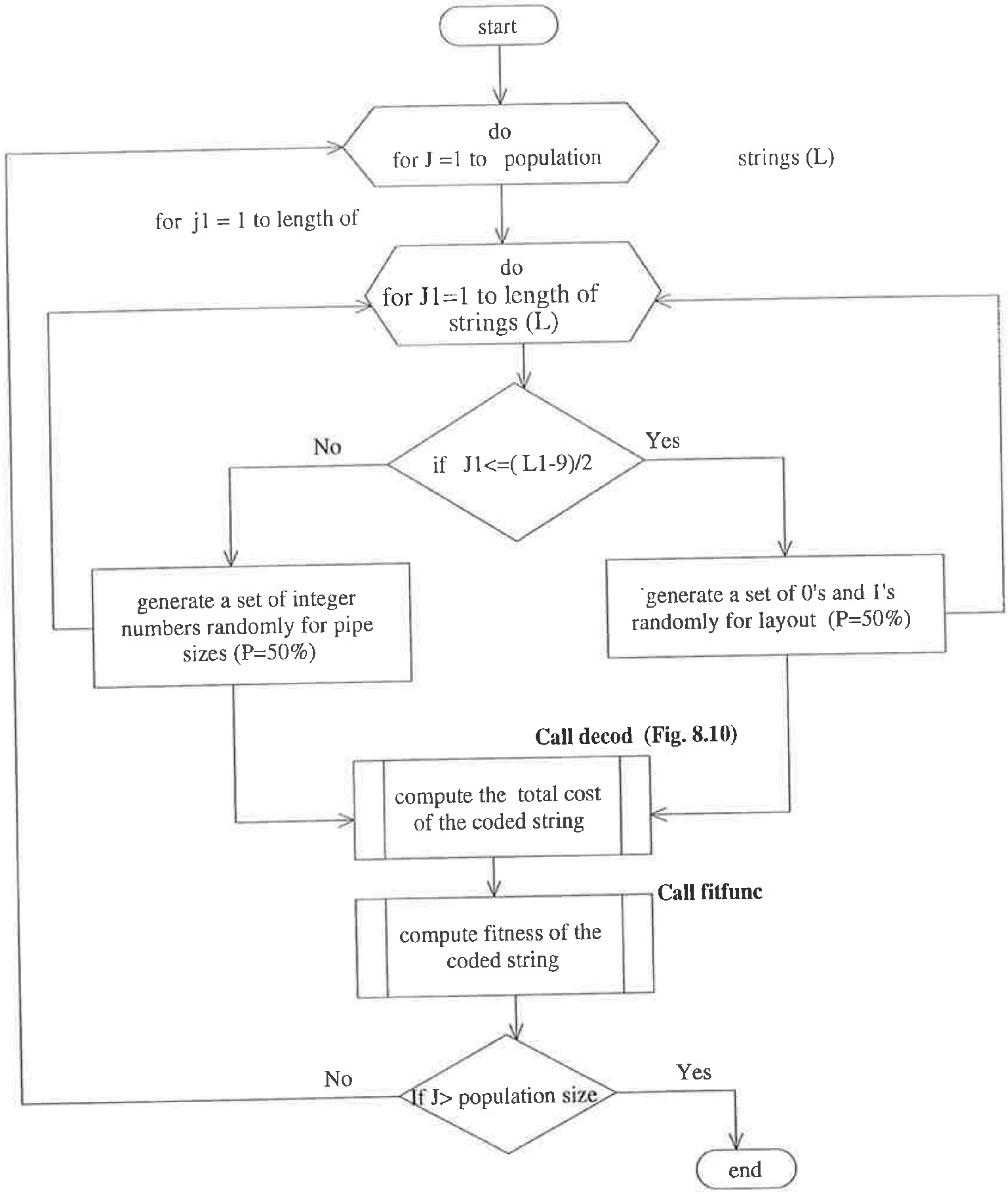


Fig. I.1 Flow chart for generation of binary and integer numbers

Appendix J

Typical examples of chromosomes created by OPDESGA model with binary and integer presentation (tournament selection)

Chromosomes	generation number	eval. number	best costs($)
111011011111101101166568655556565565564578877484	45	16200.0	405452
01111111111111111111665676758565655554565778875864	46	16560.0	405222
0110110110111111111166568655556565565564578855569	47	16920.0	405554
1110110110111111111166568655656565554546698875864	48	17280.0	404548
1110110110111111111166568655556565565564578875743	49	17640.0	403371
1111111111111111111166566655756565565556678975677	50	18000.0	399759
1111111111111111111166566655756565565556678975677	51	18360.0	399759
0111111111111111101166567655565655554565778875746	52	18720.0	400603
0111111111111111101166567655565655554565778875746	53	19080.0	400603
1111111111111111111166667655576555554565778875673	54	19440.0	399682
0111111111111011111166566655656555565454778875864	55	19800.0	398031
1111111111111111111166667655576555554565778875864	56	20160.0	399682
011111111111111110116656665556564565444978875746	57	20520.0	399505
111011011011111110116656665556564565444978875346	58	20880.0	397942
111111111111111110116556665556565565556678975699	59	21240.0	398227
1111111111111111101166566655566565554454698865777	60	21600.0	397760
111111111111111111116656665556555554454698865777	62	22320.0	395499
111111111111111111116656665556555554454698865747	63	22680.0	395499
111111111111111110116656665556655565454778875746	64	23040.0	395669
111111111111111111116656765555555555454778875743	65	23400.0	395385
111111111111111111116656665556564564545778876686	66	23760.0	394890
111111111111111111116656665556564564545778876686	67	24120.0	394890
111111111111111110116656665555555555454778875743	68	24480.0	394439
111111111111111111116656665555555555454778875746	69	24840.0	393848
111111111111111111116656665556565554444778875868	70	25200.0	393610
111111111111111111116656665555555565454778875323	71	25560.0	394273
111111111111111110116656665556564554444778875948	72	25920.0	393285
1111111111111111111166567655555554554454778966684	73	26280.0	393319
1111111111111111101166566655555554554454778966684	74	26640.0	392750
1111111111111111111166566655555554554454778966684	75	27000.0	391780
1111111111111111111165566655555554554454778966778	76	27360.0	391427
1111111111111111111165566655555554554454778966778	77	27720.0	391427
1111111111111111111165566655555554554454778966778	78	28080.0	391427
1111111111111111111165566655555554554454778966778	79	28440.0	391427
1111111111111111111165566655555554554454778966778	80	28800.0	391427
1111111111111111111165566655555554554454778966778	81	29160.0	391427
1111111111111111111165566655555554554454778973686	82	29520.0	391427
1111111111111111111165566655555554554454778973686	83	29880.0	391427
1111111111111111111165566655555554554444778895615	84	30240.0	391057
1111111111111111111165566655555554554444778895615	85	30600.0	391057
1111111111111111111166565655555554554454778875977	86	30960.0	391218
1111111111111111111166566555555554554444678875676	87	31320.0	390512
1111111111111111111166565655555554554444678875676	88	31680.0	390512
1111111111111111111166565655555554554444678875676	89	32040.0	390512

(proportionate selection)

Chromosomes	generation number	eval. number	best cost($)
001111001011000010116636489747388466375735365947 4	1	400.0	535846
0000100001001000001166364897794865696335327885839	2	800.0	516571
0011100101100001011663648974738846637573534868 83	3	1200.0	472558
0000100000001100110187556943984669559776497889674	4	1600.0	468127
0000100001001000001166364897484465696337327885834	5	2000.0	502195
1000101011111001101875569974738844637573536594 74	6	2400.0	483232
0000100001001000001166364897484465696335327885834	7	2800.0	499860
0011100001111100110187556897473884663757355659674	8	3200.0	489640
0000110001000000001166364897473484663757353486883	9	3600.0	503161
1000100001001000001166464896484765696335393759674	10	4000.0	492689
1000100001000000001166464896484765696335353659654	13	5200.0	483008
1000100001110100101166556897453884668757323689474	20	8000.0	479895
1000100001110100101166556897453884668757353689474	21	8400.0	479895
1000000001110100101166556897453884666757323689474	23	9200.0	463057
0000100000000000001066464897794465696357351686876	24	9600.0	474584
1000110001000000001166464896474765667787753659654	29	11600.0	453832
1000110001000000001166464896474765667787753659654	30	12000.0	453832
1000110001010000001166464896474765667787753657778	31	12400.0	447722
1000100000000000001066464897574765664785353679674	32	12800.0	455471
0000110001000000001166634898474465664457593686574	38	15200.0	445559
0000110001000000001166634898474465664457593686574	39	15600.0	445559
0000100001000000001166498896484765664457593686574	40	16000.0	459061
1000100001000000001067666857474487664457393686574	48	19200.0	445810
1000100001000000001067666857474487664457693786874	49	19600.0	439951
1000110001010000001166464855794485666355163689778	52	20800.0	433216
0000001001000000001166464858774485664458593686676	54	21600.0	431760
0000110000110000001196474857745485664458427679666	63	25200.0	432483
0000110000110000001196474857745485664455383679674	64	25600.0	428312
1000110000110000001196474857745485664455383679674	65	26000.0	425173
0000110000110000001196474857745485664455383679674	66	26400.0	428312
0000010101001000001066466757794487664447173686874	67	26800.0	441923
0000000001010001001066566755484487664447173686674	68	27200.0	435211
0000100001010000001166746755494465864457251886674	69	27600.0	430581
1000000001010001001166566755494765669457593686555	70	28000.0	439013
0000110011010000001169464855745485568457697887875	74	29600.0	428538
0000000001000000001066457544454465664557673686676	75	30000.0	427603
0000010001000000001066457544454465665557573696575	85	34000.0	421424
0000001000000000001055654544754465664454315876884	133	53200.0	417289
0000001000000000001055654544754466657455728687874	134	53600.0	413492
0000001000000000001055654544754466657455728687874	135	54000.0	413492
0000001000000000001055654544754466657455728688994	136	54400.0	415593
0000000001000000001056457544754465654574353888656	137	54800.0	420109
0000000110000000001066457544754465655457235787674	144	57600.0	418535
0000000001000000001066546544475465654457553589964	145	58000.0	421185
0000000000000000001056457544754466667454372689964	148	59200.0	415116
0000000000000000001056457544754466667454372689964	149	59600.0	415116
0000000000010001011664575447544666674543294898 84	155	62000.0	413374
0000000000000000001056457544654465646457593487774	164	65600.0	415259
0000000000000000001066455544754465666454486689874	170	68000.0	412123
0000000011000000001058457644454475654454359479674	180	72000.0	413838
0000000000000000001176456544754466646454872289874	197	78800.0	413590
0000000000010001001465555447544666464568321898 84	198	79200.0	411748

Appendix K

Flow chart of algorithm for calculation of hydraulic grade line in the system

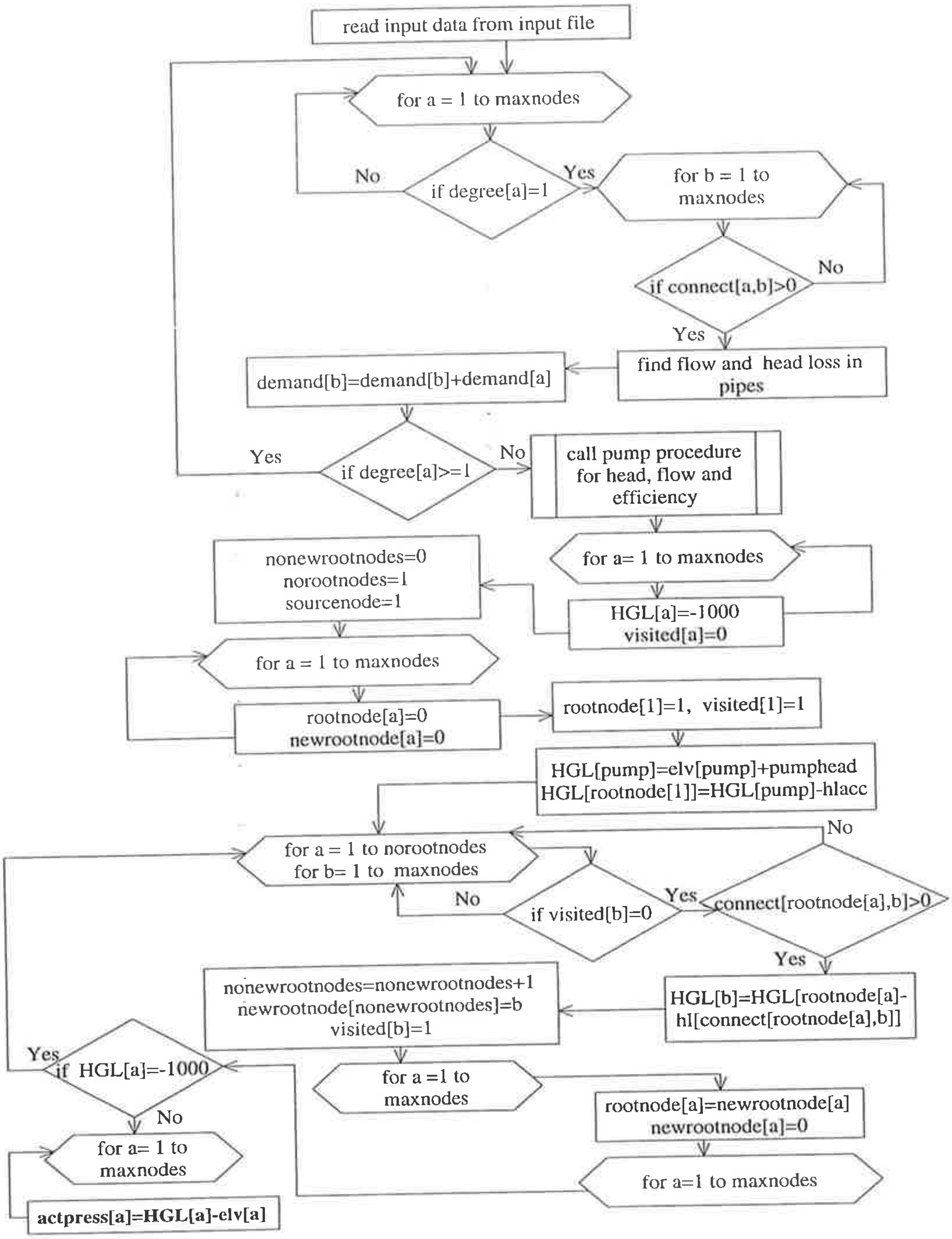


Fig. K.1 Flow chart for hydraulic grade line calculation

Appendix L

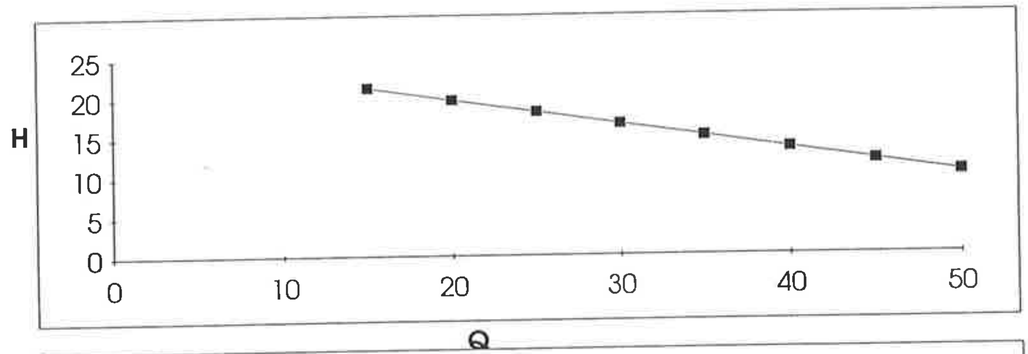
Pump characteristic curves and corresponding equations for pump head and pump efficiency

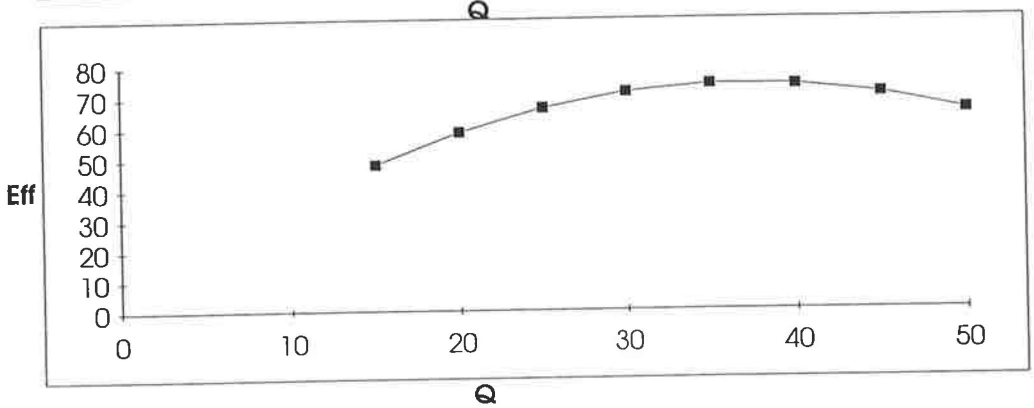
RPM=2800

H	Q	n=1	for H-Q			n=2	for H-Eff	
m	L/s							
15	21	21.25	0.25	25.96		48.15	0.146	1.217
20	19	19.68	0.679	-0.31		58.49	-0.51	3.923
25	18	18.11	0.107	0.845		66.19	0.187	-0.05
30	17	16.54	-0.46	0.025		71.24	0.235	1.728
35	16	14.96	-1.04	0.794		73.63	0.634	0.115
40	14	13.39	-0.61	0.965		73.38	-0.62	0.002
45	12	11.82	-0.18	0.982		70.49	-0.51	0.567
50	9	10.25	1.25	0		64.94	0.438	0.997
						0	0	0.999
						0	0	0

H=0.31Q+25.96

Eff=-0.05Q^2+3.923Q+1.217





CORRIGENDA

For

MODELLING AND OPTIMISATION OF PRESSURE IRRIGATION SYSTEMS

BY

A. HASSANLI

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On page 14, the third line from the bottom: "Kpa" should be "kPa".

On page 17,the first paragraph should be replaced by:

"In Equation 2.3, H_i, ($=Z_i+P_i/\gamma+V_i^2/2$g) is the total head (elevation plus pressure head plus velocity head) at node i; H_k is the total head at node k, ($= Z_k+ P_k/\gamma+V_k^2/2$g); hf_j is the head loss in pipe j; r_j is the resistance for pipe j which depends on the form of the head loss equation, Q_j is the flow rate, and n is an exponent dependent on the form of the head loss equation; n=2 for the Darcy-Wiesbach equation and 1.852 for the Hazen-Williams equation. Z_i is the elevation of the centre of the pipe at node i with respect to a horizontal datum, P_i is the pressure in the pipe at node i, γ is the specific weight of water (typically 9800 N/$m^2$for water at 20°C), V_i is the velocity of flow in the pipe at node i and g is the acceleration due to gravity (9.81 m/s^2). Typically the velocity head is quite small compared to the other terms and may be neglected."

On page 11, line 1: "show" should be "shows".

On page 18, Figure 2.3: "turbulenc" should be "turbulence".

On page 18-20, the term "energy gradient line" should be replaced by "hydraulic grade line".

On page 28, the second paragraph, the term, $\partial(\text{cost }t)/\partial H_j$ should be: $\partial(\text{cost })/\partial H_j$.

On page 60, Figure 3.8 (c): the minimum system cost should be $33,502.

On page 72, line 21: "walker" should be "Walker".

On page 78, equations (4.8) and (4.9): "d_x" should be "dx".

On page 114, paragraph 3: "dy" should be "d_y".

On page 196, paragraph 3, line 3: quotation should be "related genes are close together on the chromosome, while there is little interaction between genes" (Beasley et al, 1993).

On page 208, last sentence: "optimum" should be replaced by "least cost".

On page 230, equation (7.6) should finish with a closing bracket.

On page 242, paragraph 1: "Appendix G" should be "Appendix H".

On page 264, there should be a vertical arrow down from the box entitled " irrigation requirements"to the box entitled " subunit".

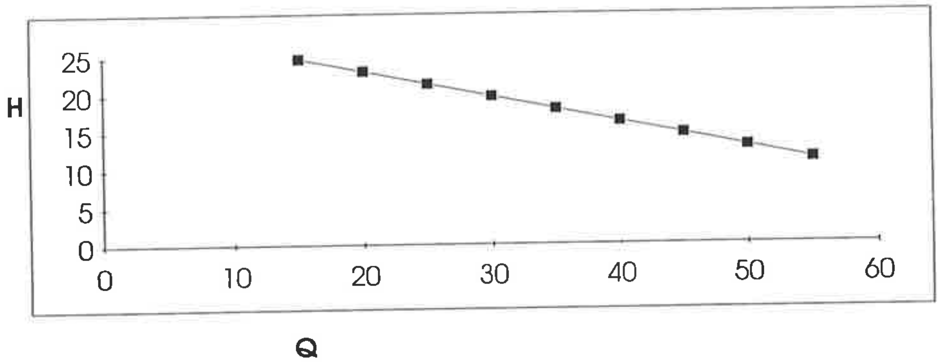
On page 297, the last paragraph: the third sentence should be replaced by " This is based on the fact that the least cost network for a single demand loading case is a treed network".

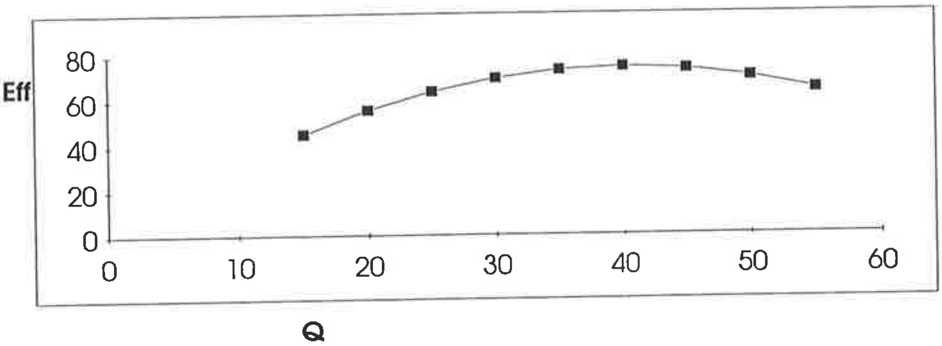
On page 266, Fig.8.10, should be replaced by the Figure on the following page.

RPM=3000

H	Q	Eff	N=1 for H-Q		N=2 for H-Eff		
m	L/s	%					
15	24	44	24.71	0.711	29.76	44.864	0.86364
20	23	56	23.03	0.028	-0.34	55.424	-0.5758
25	21	65	21.34	0.344	0.693	63.658	-1.342
30	20	69	19.66	-0.34	0.019	69.565	0.56494
35	18.8	73	17.98	-0.82	0.719	73.145	0.14502
40	17	74	16.29	-0.71	0.979	74.398	0.39827
45	15	73	14.61	-0.39	0.99	73.325	0.32468
50	13	70	12.93	-0.07	0	69.924	-0.0758
55	10	64.5	11.24	1.244	0	64.197	-0.303

H=0.34Q+29.76 Eff=-0.05Q^2+3.741 Q^-0.78

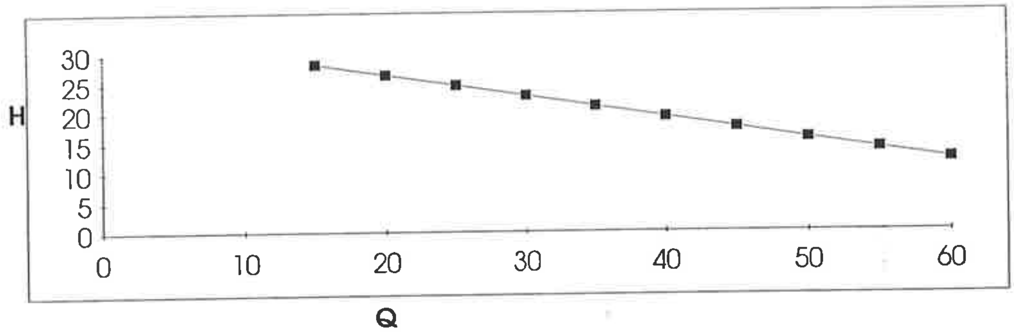


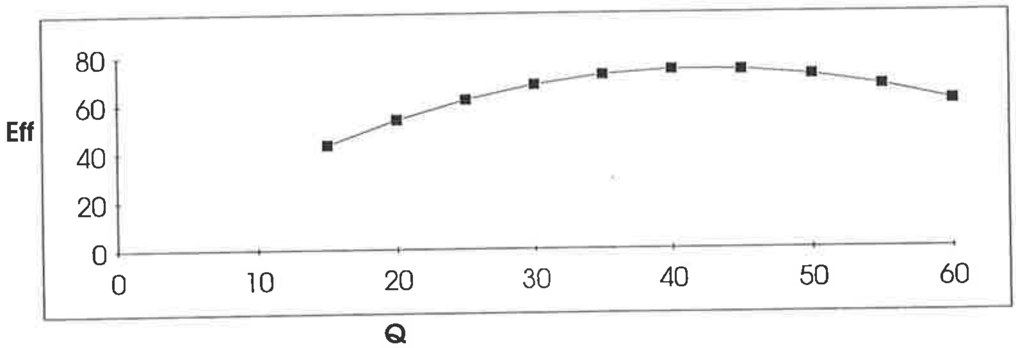


RPM=3200

H	Q	Eff	N=1 for H-Q			N=2 for H-Eff	
15	28	43	28.3	0.3	33.7	0.2364	-0.2455
20	26	54	26.5	0.5	-0.36	-0.452	3.52606
25	24	62	24.7	0.7	0.753	-0.23	-0.0418
30	23	68	22.9	-0.1	0.019	-0.1	1.46384
35	22	71	21.1	-0.9	0.851	0.9394	0.08544
40	20	73.5	19.3	-0.7	0.979	0.3879	0.00112
45	18.5	74	17.5	-1	0.989	-0.255	0.64533
50	16	72	15.7	-0.3	0	-0.488	0.99674
55	14	68	13.9	-0.1	0	-0.812	0.99837
60	10.5	60	12.1	1.6	0	0.7727	0

H=0.36Q-33.7 **Eff=-0.04Q^2+3.526Q-0.25**

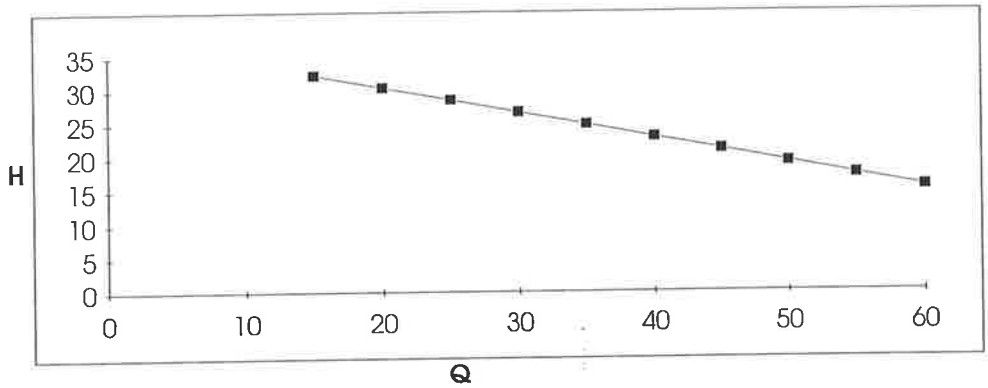


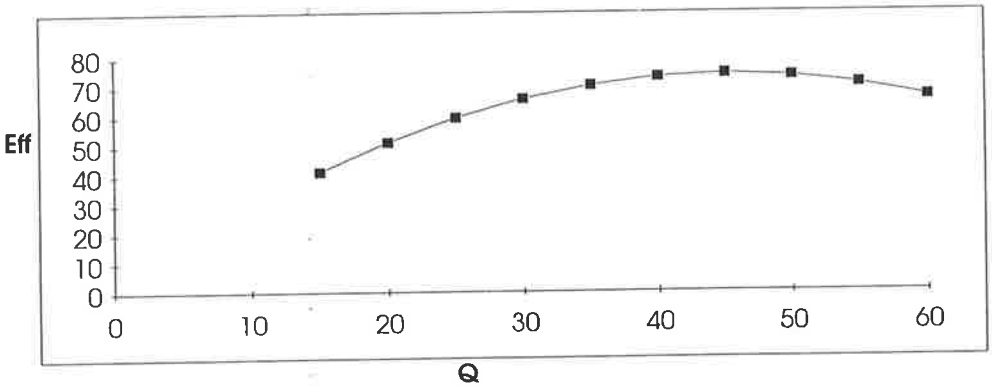


RPM=3400

H	Q	Eff	N=1 for H-Q			N=2 for H-Eff	
15	32	41	32.24	0.236	37.83	41.21	0.5473
20	30	51	30.37	0.373	-0.37	51.17	3.25
25	28	60	28.51	0.509	0.733	59.34	-0.036
30	26.5	66	26.65	0.145	0.018	65.7	0.9307
35	25	70	24.78	-0.22	0.828	70.27	0.0543
40	24	72.5	22.92	-1.08	0.981	73.04	0.0007
45	22	74	21.05	-0.95	0.991	74.02	0.4103
50	20	73.2	19.19	-0.81	0	73.2	0.9989
55	17	71	17.33	0.327	0	70.58	0.9994
60	14	66	15.46	1.464	0	66.17	0

H=-0.37Q+37.83 **Eff=-0.04Q^2+3.25Q+.547**





RPM=3600

H	Q	Eff	n=1 foı	H-Q		n=2	for H-Eff
15	36	39	35.94	-0.06	41.43	39.74	1.04
20	34	50	34.11	0.106	-0.37	49.43	3.0625
25	32	58	32.28	0.276	0.561	57.51	-0.032
30	30	64	30.45	0.445	0.014	63.98	1.6536
35	28.5	69	28.62	0.115	0.635	68.84	0.0965
40	27	72	26.78	-0.22	0.988	72.1	0.0013
45	26	73.4	24.95	-1.05	0.994	73.75	0.729
50	24	74	23.12	-0.88	0	73.79	0.9969
55	21	71	21.29	0.294	0	72.22	0.9985
60	18.5	70	19.46	0.964	0	69.05	0

H=-0.37Q+41.43

Eff=-0.03Q^2+3.063Q+1.04