EVALUATION PARAMETERS FOR COMPUTER AIDED DESIGN OF IRRIGATION SYSTEMS

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To my wife, Melanie

An unfailing source of support and inspiration

SYNOPSIS

The research has entailed the formulation and coding of computer models for the design of pressurized irrigation systems. Particular emphasis has been given to the provision of routines for the evaluation of the expected performance from a designed system. Two separate sets of models have been developed, one for the *block* or *in-field* system and one for the *mainline* network.

The thesis is presented in three sections as follows :

* **Basic theory,** in which the general background to the research is covered.
* **The models,** which includes detailed descriptions of both the design models and the computer programs.
* **Applications,** in which several test cases of both sets of models are reported.

SECTION 1 : BASIC THEORY.

This section contains three chapters as follows :

Chapter 1 : Rationale. The general nature of the design problem is discussed and shortcomings of current manual design procedures are identified. A motivation for the research is proposed. Particular reference is made to philosophies of *computer aided design* and the way in which these concepts can enhance the design process.

Chapter 2 : Systems analysis of the design process. A systematic review is presented of the process required for the complete design of an irrigation system. The components of an irrigation system are categorized in terms of *hardware* and *system* characteristics. The design process, which entails the establishing of these characteristics for a given set of conditions, is structured into three distinct modules, *viz* :

* **Preliminary design ;**
* **Block design** ; and
* **Mainline design.**

Specific aspects of the design problem relating to each of these modules are discussed.

Chapter 3 : Review of irrigation quality analysis. A review of the literature relating to the evaluation of operating irrigation systems is presented, with a view to the formulation of suitable evaluation parameters for the computer design models. A preliminary structure for the proposed evaluation model is outlined.

SECTION 3 : APPLICATIONS.

Once again three chapters are presented, as follows :

Chapter 7 : Applications of the block design and evaluation models. A single apple orchard is used as a test case for a number of alternative designs. Some sensitivity analyses of the design parameters are carried out using the evaluation model to provide an analysis of the various results.

Chapter 8 : Applications of the mainline design model. A series of different applications of the mainline design model are presented. The examples show the variety of applications that are possible with the design model, and also examine the efficacy of the procedures.

Chapter 9 : Summary and conclusions. A summary is given of the principal contributions of the research. Conclusions are drawn about the applicability of the design models and about the nature of the *CAD* process incorporated into the programs.

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Part 3 : APPLICATIONS

PART 1 :

BASIC THEORY

1. RATIONALE

1.1 Introduction

The design of irrigation systems is a multi-faceted, multi-objective problem. Different facets of the problem include :

* selecting emitters to meet the irrigation requirements of the specific design case.
* the laying out and sizing of pipelines on the basis of various hydraulic considerations;
* selecting pumps to meet the hydraulic requirements of the system; and
* determining irrigation operating schedules on the basis of hydraulic, agronomic and climatic considerations.

Objectives of the design problem can range from trying to maximize crop yield per unit of land or per unit of water used, to stabilizing food production and/or social development in a particular region.

System design is normally carried out in a series of **independent** steps, each dealing with a separate aspect of the design problem. The design process at each stage is based on various criteria that have been established over time, through experimentation and observation. In other words the derivation of these criteria has been **empirical.** The work described herein relates to research that has been carried out into the design of irrigation systems, aimed at the development of an **integrated** set of design procedures incorporating **rationalized** rather than purely empirical design criteria.

A methodology has been proposed for the **design of irrigation systems utilizing computer based models.** The major objective in developing the computer models has been to provide the designer with access to measures describing the expected quality of irrigation that will be obtained from the designed system. These measures provide the designer with a set of parameters for making a rational evaluation of the effects of various decisions made in the course of the design process. They are therefore referred to as **"evaluation parameters".**

For example, perhaps the best known design criterion is the so called *"20% rule"*: by which the allowed pressure variation in a network of pipes delivering water to a single field is limited to a maximum of 20% of a predefined nominal pressure value. This value of 20% allowable **pressure** variation is based on an expected resultant **discharge** variation within the field of 10%, which is considered to be within "acceptable" limits. The acceptability of this degree of variation has been established over time through experience rather than through any analytical process. Furthermore, the *20% rule*, is based on an assumed square root relationship between the emitter operating pressure and its discharge. However, some recently developed pressure compensating emitters provide discharges which are less sensitive to pressure variation, and the *20% rule* may therefore be inappropriate for these emitters. The proposed computer models have been structured to enable rapid evaluation of the effects of varying the allowable pressure variation for a given design situation. By so doing, the designer is able to make a rational decision as to what degree of pressure variation should be allowed in the system, on the basis of the results of the evaluation.

This chapter presents an overall rationale for the research. An overview of the design process is presented and some of the shortcomings of current design practices are discussed. This is followed by a review of the impact that **computer aided design (CAD)** has had in the engineering design process. Finally, the underlying philosophy of the proposed design models is discussed, with particular reference to the aims of incorporating the principles of computer aided design.

1.2 Design of Irrigation Systems

Any engineering project will typically undergo three main phases in its development, namely :

* **Planning**
* **Design**
* **Implementation**

In the case of an **irrigated agricultural development project**, these phases are defined more specifically as follows :

Planning. Once a potential site for the development of irrigated agriculture has been identified, decisions have to be made as to what crops will be planted and to which areas each crop will be allocated. These decisions will be based on an analysis of the available resources in relation to the project objectives. The resources to be considered in this analysis include soils, water, energy, labour, management and capital. The objectives will generally concern a maximizing of economic returns from the project, but will usually also include social and ecological objectives such as regional development, food production, creation of employment, distribution of incomes and soil conservation.

Design. Once the primary planning decisions have been made, the irrigation system has to be designed. This includes selecting the type of irrigation to be applied and the operating regime, as well as designing the actual system hardware.

Implementation. With regard to the irrigation system, the implementation phase consists of installation and commissioning of the system and the establishing of real time operating procedures. These procedures include both management practices such as opening and closing valves, moving pipes and flushing filters, as well as irrigation scheduling practices such as evaporation and soil moisture measurement and water balance calculations.

The research described in this report has been concerned with the design phase of a project, as defined above.

1.2.1 Elements in the Design of Irrigation Systems

The design of an irrigation system involves determining the characteristics of the specific system which will deliver water to the plant in the field. In order to design the system, the following factors would have been established during the planning phase of the project :

* the exact geometry and topography of the field;
* the nature of the soils;
* the proposed cropping programme;
* the location and capacities of the water and energy sources;
* the estimated plant water requirements during the season.

The design process then involves the determination of the various system **components,** which include both the actual **hardware** and the **system characteristics.** These two sets of components can be broken down as follows :

Hardware

i) The emitters.
ii) The in-field or block network, which transports water from the supply pipes to the emitters. This network consists of lateral and manifold or branchline pipes.
iii) The mainline or conveyance network, which transports water from the source to the block network. This network consists of main and submain pipelines.
iv) The control components such as valves, regulators, controllers and sensors.
v) The pumps (water and fertilizer injection).

The hierarchical nature of the hardware in an irrigation system is illustrated diagrammatically in figure 1.1.

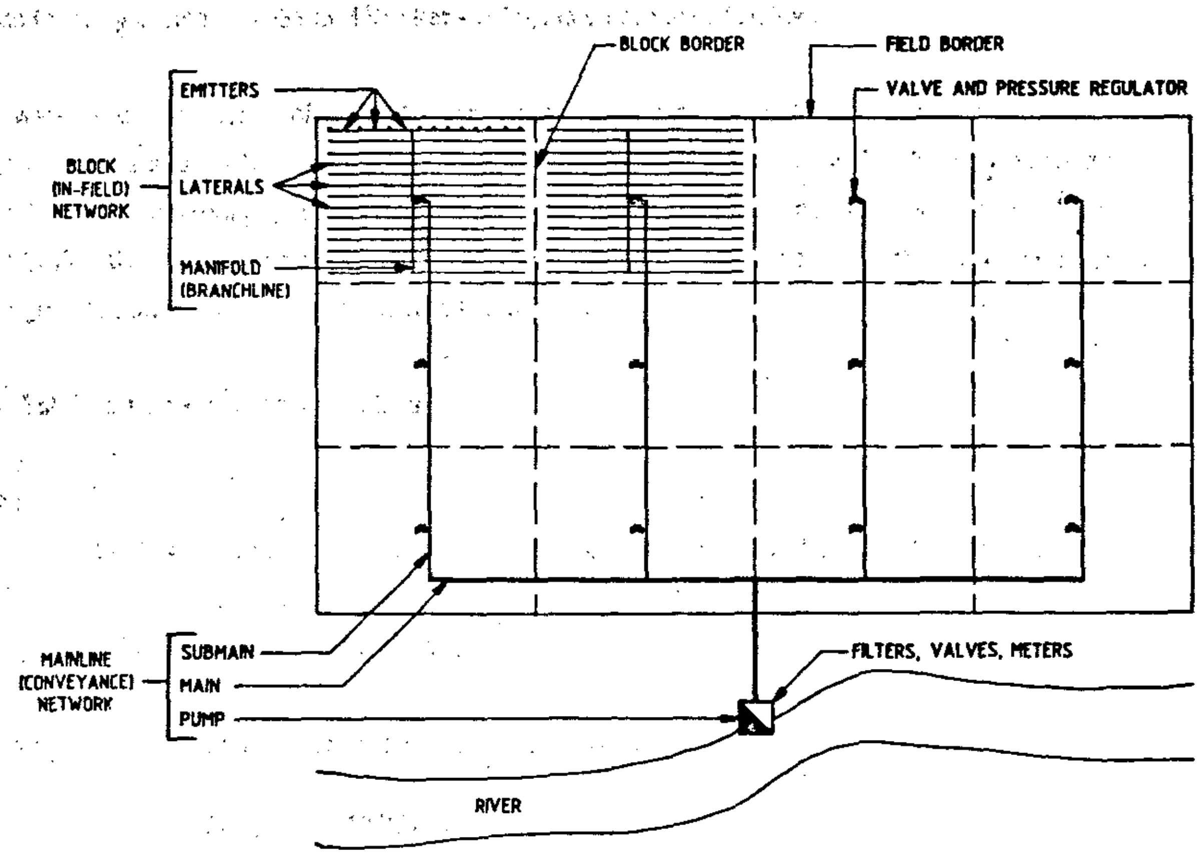

Figure 1.1 Diagramatic representation of a typical irrigation system

System Characteristics

i) Capacity, which is defined by the discharge rate, the flow and pressure distributions and the gross application quantities.

ii) Layout and alignments of both the block and mainline networks.

iii) The nature of the control system (types and locations of the control elements, levels of automation).

iv) The operating regime, which is defined by the length (time) of an irrigation, the irrigation cycle time and the irrigation programmes.

v) The pumping requirements.

vi) The system performance, which relates to the extent to which the design meets the original objectives.

1.2.2 The Generalized Existing Design Procedure

Although specific design procedures vary between different designers, from case-to-case and most significantly for different methods of irrigation, it is possible to classify a generalized procedure which is widely applied. This procedure, which is discussed at length in several texts (notably Jensen - 1981 and Walker - 1978), is summarized below.

All water measurements of rainfall, crop water requirements, soil moisture and irrigation applications are usually specified in terms of volume per unit area; which is expressed as *"depth"* and is normally given in millimeters. The equivalent volumes can be computed by multiplying these depths by the area over which they apply. On the basis of these units, the design procedure involves the following five steps :

(*i*) Establish operating constraints

If :

RAW = the readily available water that can be extracted by a plant from a soil that is wet to field capacity (mm);

Et = the peak daily evapotranspiration (mm/day)

Then the maximum irrigation interval (TI_{max}) is given by :

$$TI_{max} = RAW/Et \text{ (days)} \tag{1.1}$$

And given :

Ea_g = the gross application efficiency of the irrigation system (%); being a measure of the portion of the total water application that becomes available to the plant, rather than being lost through evaporation or deep percolation in the soil.

Then the maximum peak application per irrigation due to agro-climatic constraints (IA_{max}) is given by :

$$IA_{max} = RAW/Ea_g \text{ (mm)} \tag{1.2}$$

Finally, If :

Nb = the number of blocks to be irrigated in a given field; each block being irrigated separately in one irrigation set, so that there are Nb irrigation sets in a complete irrigation cycle.

Td = the time available per day for irrigation (hrs).

Then the maximum system application rate (AR_{max}) is given by :

$$AR_{max} = (Nb \times IA_{max})/(TI_{max} \times Td) \text{ (mm/h)} \tag{1.3}$$

This value of AR_{max} must also be checked against the maximum infiltration rate of the soil, ie. AR_{max} < *maximum infiltration rate.*

(2) Select Emitter (Establish Actual Operating Characteristics)

Given the **operating constraints** established in step (*1*), based on soil, crop and field considerations, the designer now considers the irrigation system itself in order to determine the **actual operating characteristics.** The first step is to select an emitter and the emitter spacings in the field. This is normally done on the basis of experience and trial and error, with reference to manufacturers' recommendations and tables of emitter operating characteristics.

If :

q_{nom} = the emitter discharge at the intended operating pressure (m^3/h or *l*ph)
sl = the spacing of the emitters along the lateral (m)
sb = the spacing between laterals (m)

Then the actual irrigation system application rate (AR_{act}) is given by :

$$AR_{act} = (q_{nom} \times 1000)/(sl \times sb) \text{ (mm/h)} \tag{1.4}$$

Where q_{nom} is given in m^3/h.

The condition $AR_{act} < AR_{max}$ must be checked.

And if :

AT = the total area of the field being irrigated (m^2)
AI_s = the area irrigated per irrigation set $= AT/Nb$ (m^2)

Then the number of emitters operating simultaneously during each irrigation set (Ne) is :

$$Ne = AI_s/(sl \times sb) \tag{1.5}$$

And the discharge capacity of the system (Q_{cap}) is :

$$Q_{cap} = Ne \times qnom \text{ (m}^3\text{/h)} \tag{1.6}$$

(3) Block Design

The next step is the design of the block network. Alignments, layouts lengths and diameters of laterals and manifolds have to be established. Layout and alignment are normally established on the basis of experience and trial and error; pipe sizes are established on the basis of hydraulic calculations relating to the loss of pressure due to friction and emitter discharge along the pipelines. Consideration has to be given to :

* the area to be covered per irrigation set (AI_s).
* the number of irrigation sets per cycle (*Nb*).
* the proposed method of shifting laterals (or other relevant operating units) between irrigations.
* maintaining uniformity of discharge throughout the field. The variation in discharge is normally restricted to a maximum of 10%, which translates to 20% pressure variation (*the 20% rule*) for non pressure compensating emitters.

(*4*) Mainline design

Once the block design is complete, the pressure and flow requirements at each block valve are known. Mainline design entails establishing the layout and sizes of the pipes connecting each block valve to the water source. Design considerations are principally the same as those for the block design.

(*5*) Establish pump and control systems

Finally the designer selects pumps to meet the system supply requirements (head and discharge duty), and establishes the type and positioning of various control elements such as valves, regulators, booster pumps and automatic controllers.

1.2.3 Design Criteria

The procedure outlined in the five steps described above is an iterative one. The designer will normally shift back and forth from step to step in establishing the hardware and system characteristics.

When considering various pipe sizes for alternative network layouts, the respective system discharges are known. Thus the basic consideration is one of establishing the pressure distributions on the basis of the 20% rule. Also, the primary decisions of emitter size and spacing are constrained by the results of the set of calculations in step (*1*). The value of *Nb*, the number of sub-areas or blocks, is determined on the basis of the designer's experience of how many irrigations can feasibly be executed per day and the number of irrigating days per cycle.

Throughout the design process, the principal considerations are generally to maintain costs as low as possible, whilst aiming to achieve the highest possible yield in each specific case.

Thus the criteria used in the design process can be summarized as follows :

* Minimum system and operating costs.
* Maximum yield.
* Uniformity of application defined by adherance to the 20% rule.
* Constraints on the operating regime defined by pre-determined soil, plant and field-geometry characteristics.

1.3 Shortcomings of Existing Design Procedures

The principal shortcoming of existing design procedures is that they do not have a "systems analysis" orientation. Design methods have evolved over time, together with the development of various irrigation technologies. As such, no thorough rationalization of the various design parameters has been carried out in order to relate design practice to a set of clearly stated objectives. This is manifested in the following problem areas :

(1) **Minimum Cost and Maximum Yield.** Perhaps the most well recognized problem is the use of the minimum cost criterion, rather than one of maximum profits. A trade-off exists between cost and the performance of the system, which implies a trade-off between cost and expected returns. Ideally an optimal design will be one in which the marginal costs of improving the system are equal to the marginal revenue from the crop yield. This is illustrated in figure 1.2 (English et. al. 1983), which shows generalized curves relating applied water to income and cost respectively.

Curve #1 illustrates the response of crop yield to water applied from an irrigation system, which can also be equivalenced to gross income since crop yield is directly proportional to income. Crop yield has been shown in a number of studies to be more or less linearly related to plant consumptive use of water. As consumptive use increases however, the amount of applied water lost to evaporation and deep percolation also increases, hence the curvilinear shape of the curve. Once maximum yield has been achieved, excess water can actually have a negative effect on crop yield, through factors such as deteriorating soil structure due to waterlogging, reduced nitrification and constrained movement of nutrients in the soil. This results in the downward slope of the curve beyond the maximum yield position.

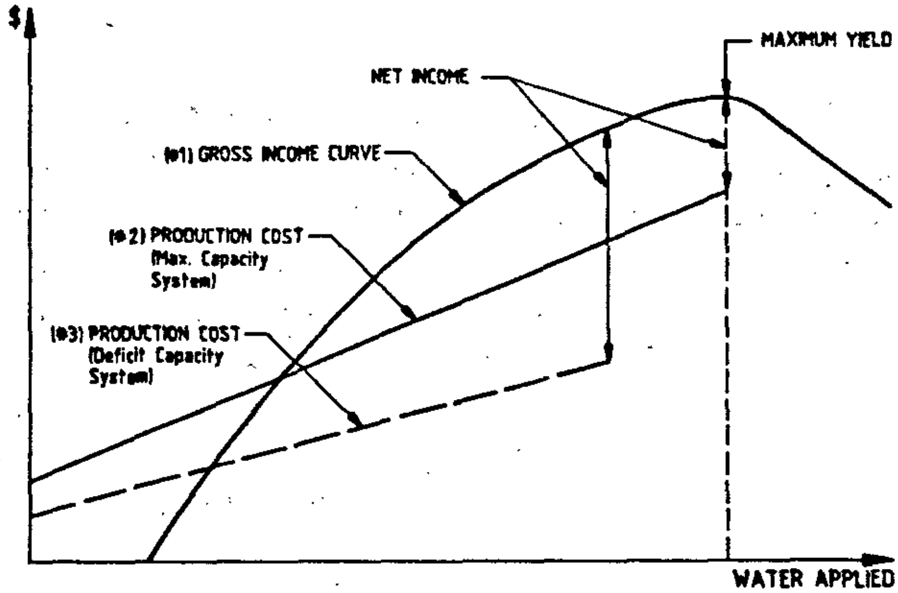

Figure 1.2 Generalized applied water vs cost and income curves for irrigation systems

The relationship between production costs and applied water is a complex one. In its generalized form shown in curves #2 and #3 in figure 1.2, the starting point on the vertical axis represents the the capital and other fixed costs of a system. The total operating costs then increase as the water application increases up to a point representing the maximum capacity of the irrigation system. The solid line (curve #2) illustrates this for normal design practice, in which the irrigation system capacity equals the application required for maximum yield. However, as can be seen in figure 1.2, maximum yield does not necessarily imply maximum nett income (the difference between the two curves at any given applied water value). The dashed line (curve #3) illustrates the potential advantages that can be achieved through so called "*deficit irrigation*", whereby a system is deliberately designed to have a capacity that is less than that required to achieve maximum yield. By so doing, savings in both the capital and operating costs of the system can be affected, with a resulting increase in the nett income achieved.

These curves illustrate an idealized state, and optimization on this basis is difficult. Firstly, the existing design process is not structured to yield an assessment of the cost versus performance relationship of the system being designed. Secondly the cost structure that has to be incorporated into the optimization procedure is complex. As well as the total capital and operating costs of the system, the marginal cost calculations should also include estimates of the additional costs incurred in harvesting and marketing the improved yields. And finally, reliable information on the yield response to water (irrigation) is difficult to establish.

(2) **Design Objectives.** The design problem, like the planning problem, is generally a multi-objective one. Rydzewski (1978) proposed the following list of possible social and economic objectives, a selection of which may be included in both the planning and design framework of an irrigation project :

1. Maximizing the return per unit of capital invested.
2. Maximizing the return per unit of project area.
3. Maximizing the return per unit of water.
4. Maximizing the value of the agricultural output.
5. Maximizing the output of food products.
6. Reaching a target output of food products (possibly linked with National self-sufficiency).
7. Maximizing output of export crops.
8. Maximizing farm-family net income.
9. Maximizing the number of families settled on a project (i.e. minimizing the cost per family settled).
10. Maximizing job creation, at a specified level of skill, for a given expenditure.
11. Minimizing the use of foreign currency in project operations.
12. Maximizing Government revenue (from taxation etc.).
13. Minimizing public expenditure (i.e. encouraging private sector investment).
14. Achieving a re-distribution of income in the region.
15. Generating maximum economic activity in the project area.
16. Settling previously nomadic communities so as to place them within reach of the instruments of social advancement.
17. Establishing social stability.
18. Satisfying political ideals.

Loucks, et. al. (1981) have formulated an objective oriented irrigation planning model, in which they have made provision for multiple socio-economic objectives in the objective function. However, while the irrigation system designer may consider his client's objectives implicitly in designing the system, existing design procedures do not incorporate any explicit formulation of objectives, nor any evaluation of the extent to which the system meets these objectives.

Thus the designer is unable to fully rationalize, either for himself or his client, his recommendations in regard to the generation and selection of alternative designs. As can be seen from the above list of possible objectives, any explicit formulation of the objectives which may pertain to a specific design situation may well incorporate some conflicting objectives. This is not an uncommon situation, and is addressed through the use of multi-objective analyses which assist the decision maker to select a *"best compromise"* solution.

(*3*) **System Performance.** Underlying both of the points discussed above is the fact that the existing design process does not incorporate performance related design criteria. The 20% rule is a surrogate criterion which ensures some unspecified (in terms of irrigation performance) minimum standard. After completing a design, the designer and his client, who is the decision maker, generally know the costs and layout of the proposed system, the energy, labour and water requirements, the operating regime and the pressure and flow distributions within the system. However, they do not have any definite measure of the quality of the irrigation to be expected from the system.

Furthermore, with the development of increasingly sophisticated pressure and flow regulation mechanisms, in particular pressure compensating emitters, the 20% rule is becoming inappropriate, and the need for considerations based on rationalized cost/benefit trade-offs is becoming increasingly critical.

(*4*) **Trade-offs.** The design process is characterized by a number of trade-offs, some of which are listed below:

1. *Cost vs. performance.* As discussed above, greater uniformity of application implies improved crop yields. However it also implies larger pipe diameters and hence greater system costs. Similarly, several other decisions, such as the emitter spacing and the operating regime, are related to a trade-off between overall costs and performance.

2. *System vs. operating costs.* Smaller pipe sizes, which imply lower system costs, result in greater pressure losses and hence greater pumping requirements, which in turn imply greater operating costs. Similarly, designing towards "solid set" (permanent) systems implies more hardware in the field and greater system costs, to be offset by reduced operating requirements and simpler system management. Included in this classification is the perennial question of labour intensive versus automated systems.

3. *Block (In-field) network vs. mainline network.* The costs of the block system can be reduced through the use of shorter laterals, with smaller discharges enabling smaller

diameters. However, this implies a more ramified mainline network and hence greater costs.

4. *Network geometry*. Several trade-offs are required in establishing the various network configurations. For example, a network can often be laid out to exploit the prevailing topography in order to offset pressure losses due to friction, thus enabling the use of smaller diameter pipes. However this will often also imply greater lengths of the various network sections, and hence the trade-off.

5. *Flexibility of operation*. The designer is always faced with the question of how much flexibility to allow his client in the operation of the system. This flexibility manifests in the ability to rearrange operating schedules to suit changing cropping patterns and cultivation practices. However, provision of this flexibility naturally requires a degree of overdesign in aspects such as the system capacity and the pressure and flow distributions in the networks.

Notwithstanding the extent of these trade-offs in the design of irrigation systems, only limited mechanisms have been established in current design procedures for sensitivity analysis of the various relationships. As a result, the designer is required to incorporate a great deal of intuition into the design process.

1.4 Computers In Engineering Design

The rapid development, in recent years, of micro-computer technology has lead to the incorporation of computers into many aspects of our daily lives. This technology has been one of the cornerstones of what Toffler (1980) has termed the "Third Wave". He believes that solid state electronics and computer based technology have largely contributed to the catapulting of Mankind into a third social revolution, following the Agricultural and Industrial revolutions respectively.

For the design engineer, this has manifested itself in the development of **Computer Aided Design** (CAD) or more generically, **Computer Aided Engineering (CAE)**. CAD systems enable the engineer to work interactively with the computer by projecting onto the screen multi-dimensional representations of the system being designed. By sitting in front of this screen and manipulating these projections the engineer is able to contemplate and investigate design aspects that were previously beyond his capabilities and may even have been beyond his cognition.

James and Robinson (1981) have written that "....It is important to realize that the Civil Engineering profession is currently experiencing a major revolution in design, brought about by advances in computer hardware (i.e., equipment) and software (i.e., programming techniques). Four phases may be identified in this revolution :

1. *more "number crunching,"* where limited access to batch-oriented mainframes allowed more calculations ("number crunching"), of the same type, than could previously be carried out on more elementary machines;
2. *better "number crunching,"* where new programs incorporating advances in techniques of numerical analysis allowed more design options to be explored, typically in a remote-batch environment, using for example, design-office terminals;
3. *new kinds of "number crunching,"* where widespread access to inexpensive minicomputers allowed wholly different problems to be investigated and solved; new programs, techniques and machines increased the scope of engineering design; computing became an essential and naturally accepted basis for design;
4. *much more than "number crunching,"* where entirely new approaches to the design problem have taken root; for example, where communication with comprehensive models is through interactive color graphics that allow the design engineer to focus on difficult problem areas using a single keystroke on the terminal...."

In describing the extent of the enhanced design capabilities that can be achieved with CAD, Preiss (1982) has used the analogy of Man's progression from solely oral communication to the development of written communication. This development provided, through paper based information systems, a medium which facilitated conceptual or abstract thinking. Preiss believes that inasmuch as the computer overcomes many of the limitations of the paper based information systems, such as analysis in more than two dimensions, the identifying and notifying of errors and the affecting of corrections and alterations, it represents a quantum leap forward that is comparable to the development of written communication. In discussing the implications of future CAD systems, Preiss believes that they "...will have a capability of checking interrelations between data, and will be able to check implications of proposed decisions, to a degree which is difficult to grasp today."

Thus, using CAD the engineer is able to develop greater understanding and even new perceptions of the system he is designing. The full implications of the effect of CAD on engineering design has been the subject of considerable debate, which is beyond the scope of this report. The interested reader is referred to Cooley (1980). However the nature of CAD systems is discussed in more detail below, in order to illustrate how irrigation systems design can be structured for CAD.

Preiss has classified two generations of CAD systems. The first generation systems are characterized by 2-dimensional drawings that are merely representations in the machine of what used to be on paper. Man/machine interaction is directed and actuated by the user; and the software is "deterministic", in that on receiving input it will either calculate a given output or fail because of the nature of the input. The second generation of systems have an interpretive capability that enables them to generate their own data and to lead the designer through solutions in an interactive "conversation". In these systems the machine can display to the designer, and maintain within its own processor, 3-dimensional representations of the system being designed. It is therefore able to carry out solid geometric modelling which, for example, will identify and preclude infeasible interactions between solid objects.

These second generation systems are imbued with "*artificial intelligence*" through non-deterministic software. This software has alternatively been termed "*knowledge based*" and "*problem solving*". Starting with the initial state, which is the input data, the programs apply a series of rules to transform the data through several intermediate states to a final output state. These rules are governed by a number of logical preconditions which test the current state of the data and enable a decision on which rule to apply on the basis of specified criteria. The dynamic properties of the software emerge from the hierarchical structure of the rules and preconditions. In order to evaluate the preconditions for a specific rule, other rules may be used, which in turn trigger the use of additional rules, and so on recursively. Also, there can be rules to change the preconditions depending on the current state of the data. Thus the user will not generally know, *a priori*, to which state the application of rules will drive the computation. This class of software has been used for the development of applications such as computer chess players, language interpretation, robotics and medical diagnosis.

The nature of the CAD system that has been developed for irrigation design is discussed further in section 1.5 below.

1.5 Basis for the Research

The basic motivation for the research has been to utilize CAD technology to develop a more rationalized approach to the design of irrigation systems. In order to do this, two distinct sets of work have been undertaken.

Firstly, as stated in the introduction of this chapter, the work has attempted to develop a set **of appropriate performance parameters to be used as evaluation tools in the design process.** Much work has been reported in the literature, relating to the development of performance parameters to evaluate **operating** systems in the field. This work has been used as a basis for the development of the proposed **design** parameters, and is therefore reviewed in chapter three of this report.

Secondly, the work has attempted to develop an integrated approach to the overall design process. A considerable amount of work has been done over the years in the development of optimization routines for individual aspects of the design process. This work has been incorporated into the proposed model wherever considered appropriate, and is consequently reviewed in the relevant chapters. The proposed model has been predicated upon a thorough "systems analysis" of irrigation systems themselves. This is described in chapter two.

1.5.1 Structure of the proposed model

The proposed model can be used to generate alternative systems, which can then be evaluated in terms of prestated objectives using the various performance criteria. On the basis of this evaluation new alternatives may be generated. By this process, the designer is able to thoroughly investigate the effects of the various trade-offs that have to be made, and finally to select the design which best meets his objectives.

Thus in terms of classic modelling theory, the proposed model may be classified as :

* an event based (ie. deterministic rather than probabilistic) simulation model;
* with an isomorphic and iterative (multi-directionally) internal structure (ie. it attempts to model the exact processes of cause and effect in irrigation systems, these processes being multidirectional rather than having a fixed path from start to end);
* and its function is predictive (*heuristic*) rather than purely descriptive.

The computer programs can been seen as a hybrid first and second generation CAD package. Inasmuch as the design process is not yet fully rationalized (analytical), it is still directed by the designer. Nevertheless the level of interaction with the computer is high and considerable flexibility is provided in the directing of this interaction. It is hoped that in the future, as experience is gained in using the CAD based model, and insights into the interactions of the various irrigation system components are developed, algorithms for knowledge based software will also be developed.

1.5.2 Summary

In summary, the contribution of the reported research to the current state of the art in irrigation systems design is seen to be:

1. The carrying out of a comprehensive "systems analysis" of the process of design of irrigation systems, leading to a proposed detailed structuring of the design problem. This includes: a listing of all of the irrigation system components to be designed; the formulation of three distinct design modules and the individual routines contained within these modules; and identification of the links governing components and design parameters within and between each module (chapter 2).

2. The formulation of measurable performance and quality of irrigation related design criteria, to be used in the design process for "on-line" evaluation and selection of alternatives (chapters 3, 5 and 6).

3. The development of a suite of programs for a computer based irrigation design model. The programs have been structured to enable the user to carry out an interactive dialogue with the computer, thereby building up experience with the effectiveness of the various evaluation parameters. It is hoped that this will lead in future to the development of more knowledge based algorithms (chapters 4 - 8).

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2. SYSTEMS ANALYSIS OF THE DESIGN PROCESS

2.1 Introduction

The **systems approach** to engineering problems has developed from work in the disciplines of engineering science, operations research, management science and cybernetics. It is a philosophy that perceives processes as systems, consisting of "... a goal-directed collection of interrelated, interdependent parts existing in an environment, with boundaries that are dependent on the purposes of the person defining the system" (Duffy and Assad, 1980). In order to study the interrelationships between the various parts of a system, as well as those between the system and its environment, techniques have been developed for the construction of **models** which can simulate the operation of a system. These models are normally conceptual, rather than physical, and are constructed using various analytical procedures.

In the above context, **systems analysis can have two distinct meanings.** The first refers to the techniques employed, as part of the system model, for the analysis of a given situation. These techniques are often mathematical, and typically include optimization procedures such as *linear and dynamic programming*. The **second meaning** refers to a systematic analysis of a particular process, that is carried out in order to identify and characterize the individual parts of the system being analysed. Such an analysis is normally carried out in order to facilitate construction of the system model. It is this latter meaning that is implied in the title of this chapter (note that in the context of computer science, systems analysis has yet another meaning).

Thus, this chapter provides a detailed review of the design process. The requirements of the process are discussed, with particular reference to identification of the design parameters. This is followed by a presentation of the proposed design procedure, with discussion focussing on the specific design problems in each part of the process. Finally, a review is given of the principles employed in the design of the computer models.

2.2 Requirements of the Design Process

The main purpose of the design process is to establish the irrigation system components. These components can be classified into two groups, namely :

* **Hardware.** This includes all the physical elements of the irrigation system, such as pipes, emitters and accessories. The design involves determining the type and size of these elements, as well as the quantity of each element to be used in the system.

* **System Characteristics.** This includes all of the non-physical attributes of the system that are established during the design process.

The design process involves establishing these components, for a given set of prevailing physiographic conditions and on the basis of a number of predefined objectives. In order to achieve this, a number of different procedures are used for different sets of components. Each of these procedures utilizes a specific set of **design parameters.**

2.2.1 Components

Table 2.1 shows a list of all of the system components that have to be designed. The actual design requirements in each case are listed under the respective component. The right-hand column of the table shows the parameters which affect the design of each component.

As can be seen from the table, there is a considerable degree of interaction and inter-dependence between components and their associated parameters. This is examined further in the description of the actual design procedure (section 2.3).

2.2.2 Design objectives

As discussed in chapter one, existing design procedure normally entails the following objectives :

* maximum yield; and
* minimum cost.

The proposed computer based procedures developed in this research however, attempt to incorporate an orientation towards :

* maximum profit.

In the case of private ownership under ideal conditions, for which economic return is the primary objective and there are no other constraining factors, the abovementioned objectives may be adequate for design purposes. However, there may often be certain limiting factors which require the incorporation of other objectives. For example, if water is limited then the design might be aimed at producing the maximum return per unit of water. Alternatively, in the case of a development agency project, certain social objectives, such as maximum job creation, will have to be incorporated into the design. Such objectives may conflict with those

Table 2.1 System components and associated design parameters

Component	Design Parameters
Hardware	
i. Emitters - type	Spacing; Nominal operating pressure; Costs; Pressure/discharge relationship; Operating regime;
ii. Block network - lateral pipe diameters - manifold diameters	Hydraulic grade line; Allowable pressure variation; Coefficient of uniformity; Pipe alignments; Topography; Pipe costs;
iii. Mainline network - pipe diameters	Hydraulic grade line; Pipe costs; Energy costs; Flow and pressure requirements;
iv. Control elements - valves : type, size; - flow and pressure regulators : if needed, type, size - meters : if needed, type, size - automation equipment : if needed, type - filters : if needed, type, size	Flow and pressure requirements; Hydraulic grade line; Discharge volumes; Water quality; Costs;
v. Pumps - main pump size - booster pump sizes - fertilizer injection equipment : if needed, type, size	Hydraulic grade line; Discharge volumes; Flow and pressure requirements; Costs;
System characteristics	
i. Capacity - maximum system discharge - flow and pressure distribution - system application rate - maximum application depth	Pump size; Pipe sizes; Emitter discharge; Max. block size; Emitter pressure; Emitter spacings; Irrigation set time;
ii. Layout and alignments - division of field into blocks - emitter spacings - orientation of laterals - positioning of manifold - location of block valves - configuration of mainline network	Operating regime; System application rate; Pressure and flow requirements of emitters; Maximum length of a single or double diameter pipe; Topography; Pressure and flow requirements of valves;
iii. Control system - location of control elements - degree of automation	
iv. Operating regime - irrigation set time - irrigation cycle length - timing of irrigation sets (i.e. times of the day, days of the week) - sequencing of block valves - filter flushing programme - fertilizer injection programme	Readily available soil moisture; Peak daily irrigation requirement; Number of irrigation blocks; System capacity; System application rate; Degree of automation;
v. Pumping requirements - maximum pumping capabilities - the pumping regime, including operation of booster pumps	System capacity; Operating regime; Hydraulic grade line; Flow requirements;
vi. Performance - coefficient of uniformity - application and requirement efficiencies - capital and operating costs - return on investment	All design components

aimed at generating maximum economic return from the project. In this case, the onus is on the designer to establish a best compromise design. A list of possible objectives is shown in chapter one.

The proposed design models have been structured to provide a full economic evaluation, thereby enabling the designer to assess the performance of a given design in terms of its specific economic objectives.

2.3 The Design Process

The overall design process incorporates three distinct phases, as shown diagrammaticallly in figure 2.1. These are respectively, *basic input, design* and *final output.*

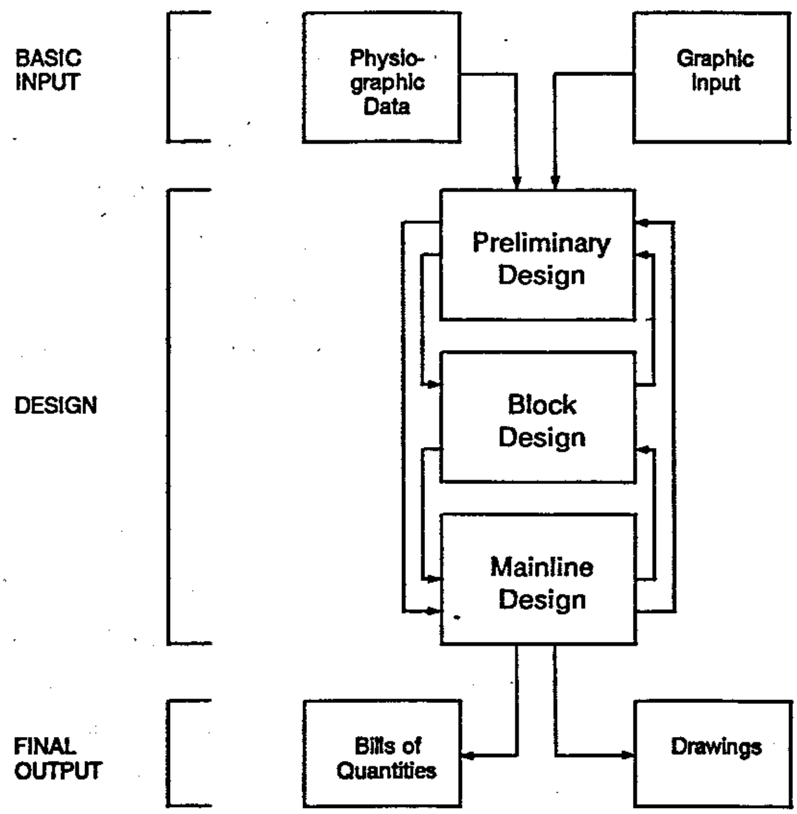

Figure 2.1 : Phases in the Design Process

Basic Input. This phase involves the accumulation of all relevant physiographic data, which includes the prevailing soil, plant and climatic characteristics, as well as the field topography. The topographic data may be incorporated into the computer models either digitally from the keyboard, or via a graphics tablet using computer aided draughting techniques.

Design. This phase incorporates three modules, namely preliminary design, block design and mainline design. As indicated in figure 2.1, the design process involves considerable iteration between all three modules until the design is complete.

Final output. To complete the design process a bill of quantities, together with a set of drawings to be used for installation and future management of the system, are needed.

Three design modules are discussed in more detail below. This discussion aims to identify the individual design problems within each module; to provide a perspective on the relationships between these problems within the overall design process; and to introduce the basic approaches to solving each of the individual problems. Figure 2.2 shows the main elements of each module. The required input data for each module are shown on the left of each respective block, and the components that are designed in each module are listed on the right of each block.

The research has concentrated on the **block** and **mainline** design modules.

2.3.1 Preliminary Design

This module centres around three principal design problems, namely: establishing the operating regime; selecting the emitter on the basis of required operating pressure, discharge and spacings; and sub-division of the field into blocks.

The first step of the process entails calculating the basic soil/plant/water relationships from the input data. These include the soil moisture holding capacity and infiltration rates and the plant water requirements. These relationships are then used as constraints in the ensuing trial and error process for emitter selection and determination of the operating regime. A set of required operating and capacity characteristics is calculated; a number of emitters are then considered and their performance in relation to the required characteristics is examined. This process is repeated for several sets of required characteristics and in this way a matrix of possible emitters and related operating characteristics is developed. The designer is then able to make a selection that best suits the prevailing circumstances.

Division of the field into blocks is carried out principally on the basis of the designer's intuition, with due consideration of the following factors :

* the chosen operating characteristics limit the maximum number of blocks that can be irrigated within the complete irrigation cycle;

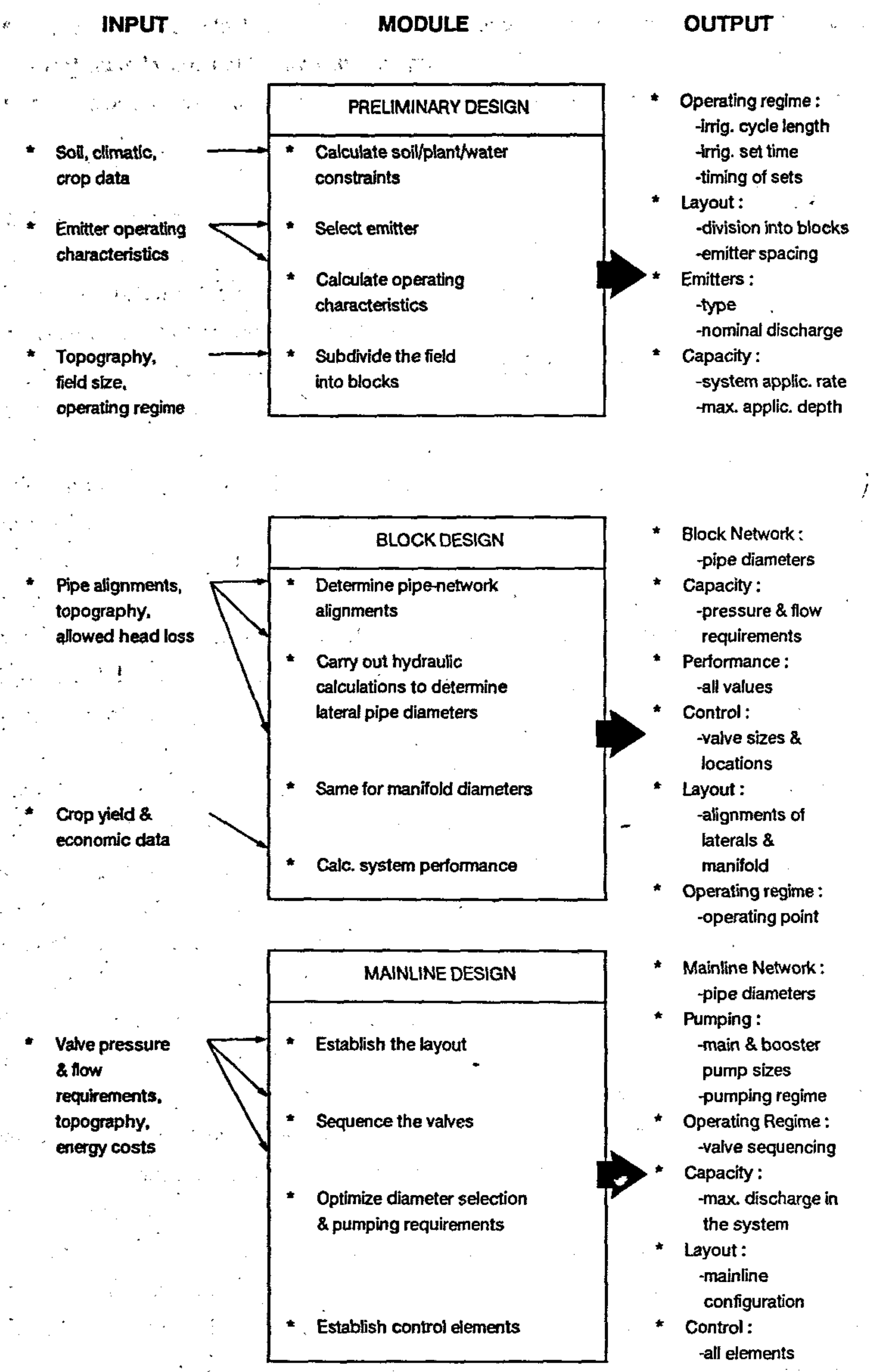

Figure 2.2 : The Design Modules

* the size of each block determines the flow requirement for the block, which must be practicable in terms of the available supply;
* the block dimensions determine the block pipe-network dimensions, which must be economical.

2.3.2 Block Design

This module entails :

* determination of the block pipe-network alignments;
* determination of the pipe diameters; and
* an assessment of the system performance for the block being designed.

This is carried out for each irrigation block in turn.

Pipe alignments. The lateral pipes are aligned parallel to the planted rows. The position of the manifold, which transects each of the laterals, is then established on the basis of both practical convenience and hydraulic considerations. These hydraulic considerations arise out of the fact that the manifold divides the laterals into two sets, lying on either side of the manifold. Ideally, the manifold should be positioned such that the lengths of the laterals running uphill away from the manifold are maximized, within the constraints of the allowable pressure loss in the system.

Pipe diameters. The pipe diameters are established using the allowed pressure variation in the system as a design parameter. The general procedure for a given pipe involves first of all establishing an allowed pressure envelope, defined by the topographic elevations along the length of the pipe and the allowable pressure variation within the pipe. The upper and lower limits of the envelope represent the maximum and minimum allowable hydraulic grade lines respectively along the pipe being designed. Then, starting at the furthest end of the pipe with the smallest available diameter, the pressure head in the pipe is calculated for points along its length, working back towards its inlet. This pressure head will increase exponentially as the flow in the pipe increases, because more and more outlets are included along the length being considered. Considering this curve in the other direction (i.e. in the direction of flow in the pipe), the exponential shape represents the decreasing rate of head loss due to friction, per unit length, as the flow in the pipe decreases. As soon as the actual hydraulic grade line for the diameter of pipe being considered rises steeply towards the upper limit of the allowed envelope, the pipe is replaced by a larger diameter, thereby reducing the rate of pressure loss due to friction. The process is continued until the inlet is reached. The pipe will then have been designed with the smallest possible diameters (and therefore the cheapest) that will keep the pressure variation within the allowable limits. A more detailed discussion of this design process is given in chapter 4.

This process is carried out for each lateral in turn, and then using the results from the lateral design, it is repeated for the manifold. At the end of this process, the pressure and flow required at the block valve, for irrigation of the block, are known.

Evaluation. The last step of the block design process involves a calculation of various performance parameters for the designed block. These parameters include indices of both the quality of the irrigation that will be applied by the designed system, and the expected economic benefits that will result from the use of the system.

The system is designed for a pre-specified capacity, which depends on the expected crop water requirements. However it may in fact be operated at different levels of intensity up to a maximum level defined by its capacity. Maximum benefits may not necessarily accrue from operating the system at its full capacity, which implies always providing all of the plant water requirement. In some cases, the nett benefits may be increased by operating the system below capacity, thereby providing the plant with less than its full requirement (so called **deficit irrigation**). The evaluation process includes an analysis of the effects of operating the designed system at different levels of intensity. A procedure for determining the optimal level of operation of the system by dynamic programming has been developed and is discussed in chapter 5.

2.3.3 Mainline Design

This module entails the following design problems :

* establishing the pipeline layout;
* establishing the operating sequence of the valves (irrigation blocks) within the irrigation cycle;
* determining the diameters of the pipes; and
* establishing the pumping requirements for the system.

Layout. The routing of the pipelines from the water source to the block valves is straightforward to achieve, but difficult to optimize. Since there are normally an infinite number of alternative routes, any one of them will generally satisfy the requirements of the problem. However, it is difficult to establish which of these routes involves the minimum cost in terms of both capital and operating expenses. For example, the topography of the site may be such that minimizing the total length of pipes results in the need for larger diameter pipes in order to avoid excessive pressure loss due to friction. In addition, there may be certain practical considerations which influence the route of the pipelines. For example, in an established farm it may be preferable to align the main pipelines alongside the existing roads rather than through already planted fields.

required function, it is essential that these programs should operate efficiently and accurately. Structuring of the computer programs has therefore constituted an important aspect of the research effort.

Three components to this problem can be identified. Firstly, the actual analysis routines, which carry out the mathematical design procedures, have to be formulated. The success of these routines is measured by the speed and accuracy with which they generate results. Accuracy in this sense is determined by their ability to handle all design problems, including those of an irregular nature, without resulting in a software failure.

Secondly, routines have to be designed for the validation of the input data, prior to analysis. Once the required data have been supplied by the user, they should be checked by the computer to ensure that they conform to both the format and the limits that can be handled by the analysis routines. This prevents the generation of unnecessary errors during computation.

The third aspect of the software structuring problem relates to the nature of the user-machine interaction. The development of so called *demand-mode computing* has enabled the user to interact with the computer, during the design process, via a specifically designed dialogue. The most appropriate nature of this dialogue, for specific problems, has been the subject of a considerable amount of study, much of which is summarised by James and Robinson (1982). They believe that the importance of formulating a good man-machine dialogue cannot be over-emphasized. Newstead and Wynne (1976), and Roy (1980), found that suitably designed interactive procedures can assist the user in making the judgements necessary for the solution of multi-objective problems by providing more complete information. However, James and Robinson believe that "...if the method of communicating with the computer is complicated and exacting, or the dialogue ambiguous, the positive aspects of computing will be nullified."

The interactive dialogue has two principal functions, firstly to assist the user to input the required data for the model, and secondly to guide the user through the design process. In this latter regard the interaction should not be an inflexible step by step process, whereby the user's role is a passive one of merely inputting the data and then reading the results of the analysis. Since ultimately all design decisions should be made by the designer, the interactive procedure should ensure that the user receives full information regarding all options in a multi-objective problem, and in such a form that he is able to make a correct and well informed decision.

James and Robinson have formulated a set of nine criteria and suggestions for the appropriate design of interactive systems, aimed at achieving the above-mentioned objectives:

1. The dialogue should be terse, coherent, and unambiguous, yet should be conducive to a cooperative attitude. The dialogue should flow smoothly from one concept to the next, following a logical sequence that is clear to the user.
2. Prompts should be concise and should always be presented in the same manner and position on the screen. Each prompt should be numbered so as to enable reference to a pocket manual for aid.
3. The user should not be required to respond to more than one idea at a time.
4. The input translation routine should accept free format data.
5. The computer should always respond to the user. Some indication should be given that the user's response is being processed.
6. The user must be able to observe and control the procedure and be able to abort the current state of the system and/or reset the procedure to the initial state, an earlier state, or a new, user specified, local state.
7. Data entered should be validated by checking syntax and comparing with reasonable limits.
8. Error messages should be designed to convey information in a manner that is concise yet does not antagonize the user.
9. Results relayed to the terminal should be ordered and easily read and interpreted, using graphics or clear print-outs.

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3. REVIEW OF IRRIGATION QUALITY ANALYSIS

3.1 Introduction

Since one of the primary objectives of the research relates to the development of evaluation parameters for incorporation into the design process, it is necessary to review the current state of the art.

A substantial amount of work has been done in establishing methods to measure and evaluate the performance of irrigation systems. Meriam et.al. (1981) have identified four purposes for this work as follows :

1. To determine the efficiency of the system as it is being used.
2. To determine how effectively the system can be operated and whether it can be improved.
3. To obtain information that will assist engineers in designing other systems.
4. To obtain information for comparing various methods, systems and operating procedures as a basis for economic decisions.

As can be seen from these purposes, the orientation of this work has been in the evaluation of existing systems, rather than in the design of new systems. The various performance parameters that have been proposed in the literature have been determined in each case from field measurements made on operating systems. Notwithstanding this, several authors (Karmeli et.al., 1978 and Walker, 1979) have identified the potential advantages of incorporating these performance concepts into the design process. This chapter therefore presents a review of past approaches to the evaluation of irrigation system performance, together with discussion on the possible adaptation of these approaches for use in the design process.

In order to evaluate an irrigation system, some measure of the quality of the irrigation delivered by the system is needed. Ultimately, since the purpose of irrigating is to improve production, this quality must be measured in terms of the yield attained from the crop. In other words, some measure of the extent to which the irrigation has influenced the yield is needed. Considerable experimental and theoretical research has been done in investigating yield/water relationships for various crops. This work has focused on the relationships between the amount of water supplied to the crop and the resultant yield, and also on the effect of various irrigation scheduling practices on the yield. However, these relationships are

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Since the dispersion of water from an irrigation system is not uniform, the general approach in the design of systems is to attempt to supply "*...an adequate average irrigation depth, ... with reasonably high uniformity ... in order to minimize the reduction in crop yield as a consequence of the non-uniformity of the irrigation*" (Chaudhry 1978). However, improved uniformity implies increased system and operating costs, and therefore an optimal system is not necessarily one in which uniformity is maximised. Ideally, in order to establish a profit maximising objective function for the design of irrigation systems, the relationship between the costs of the system and the expected yield is required. This aspect of system design is discussed further in chapter 5.

Given the difficulties inherent in determining the abovementioned relationships, past work has concentrated on the definition of parameters which measure certain aspects of the water dispersion in the field. Once the amount of irrigation water required in the field has been established on the basis of knowledge of the plant evapotranspiration and the moisture condition of the soil, the irrigation quality can be defined in terms of the extent to which it meets this requirement. For any point in the field the requirement (depth) at the time of irrigation, and the amount of water applied (depth) during irrigation can be measured. Thus the extent to which the given point has been over or under irrigated can also be calculated. If this measurement is integrated over the whole field, then the following parameters can be determined :

1. The extent (area) of the field which was under irrigated (deficit);
2. The extent (area) of the field which was over irrigated (excess);
3. The respective deficit and excess water volumes.

If it were required, it would also be possible to identify the specific regions in the field which are respectively in deficit or excess. However, in the overall evaluation of the irrigation this is generally not done. Instead the data are aggregated and expressed in the form of the cumulative depth versus area irrigated relationship, which is discussed in section 3.2.3 of this chapter. From this relationship several different quality parameters can be defined. These can be classified into two principal groups :

1. **Efficiency measures** which give an indication of the extent to which the irrigation has been usefully utilised. These measures are based on the extent and volume of the deficit and the excess.
2. **Uniformity measures** which give an indication of the *evenness* with which the irrigation was applied in the field. These measures are based on the extent to which the application depths vary from the overall average application depth.

This chapter consists of firstly a review of the various methods which have been used to describe the distribution of water resulting from an irrigation; followed by a review of the numerous efficiency and uniformity measures respectively that have been developed in the literature. Finally some discussion is presented of the appropriateness and possible adaptation of these measures for use in the design process.

3.2 Water Distribution Functions

3.2.1 Single Emitter

The description of the overall field distribution begins with the analysis of the distribution from a single emitter. Three possible situations can be identified :

1. A sprinkler or sprayer that is stationary during the irrigation;
2. A sprinkler or sprayer that moves during irrigation;
3. A stationary drip (trickle) irrigation emitter.

Stationary sprinkler

In order to be able to analyze and compare distributions of water obtained from single stationary sprinklers, the American Society of Agricultural Engineers (ASAE) (1969) established a standard testing procedure. This procedure entails the definition of a square grid matrix which is superimposed over the sprinkler being tested. "Catch cans" are then placed in the field at the nodes of the grid to measure the precipitation resulting from the operating sprinkler over a given period of time.

The results of this testing procedure provide point values on a square pattern of the depth of water being spread from the sprinkler in a given time. The size of intervals on the grid is not specified in the ASAE standard, and is therefore selected by the tester, and will naturally depend on the purpose of the test. A sprinkler manufacturer wanting to test the operation of one of his sprinklers would probably select a relatively fine grid, whereas a farmer evaluating his system in the field would probably utilize a more coarse grid in which the intervals were some whole fraction of the sprinkler spacing of his system.

Other salient aspects of the testing procedure include rigorous measurements of the prevailing operating conditions, such as the sprinkler flow rate and pressure, the general climatic conditions at the time of the test and wind measurements.

Alternatively, simple geometric shapes which will approximate the distribution pattern expected from a single sprinkler have been proposed. Instead of the point measurements obtained from the testing procedure, mathematical expressions can be derived from the assumed shapes for the precipitation expected from an operating sprinkler at any point within its circular distribution pattern. Bittenger and Longenbaugh (1962) developed analyses for both triangular and elliptical distributions as shown in Figure 3.1.

In the figure, r is the wetted radius of the sprinkler and h is the precipitation rate (mm/h) at the sprinkler. Considering a point p at a distance s from the sprinkler, then the precipitation rate, P_p, at this point is given by :

$$P_p = h[(r-s)/r] \quad \text{(mm/h)} \tag{3.1}$$

for the triangular pattern, and

$$P_p = h[(r^2-s^2)/r^2]^{½} \quad \text{(mm/h)} \tag{3.2}$$

for the elliptical pattern.

Moving sprinkler

With the development of mechanical irrigation systems in which the sprinkler moves continuously during irrigation, appropriate methods for describing the distribution have had to be developed. Special test conditions can be set up in order to make field measurements. However, this is not as practicable as in the case of the stationary sprinkler. For the moving systems, the mathematical analyses developed by Bittenger and Longenbaugh have proved to be more useful than the experimental methods, in describing the distribution patterns. Heermann and Hein (1968) found that both the triangular and the elliptical models gave figures which were not significantly different from those obtained experimentally in tests on two different center-pivot systems.

Equations 3.1 and 3.2 can be extended for both lateral-move systems (straight line path of sprinkler) and center-pivot systems (circular path of sprinkler) as follows :

a) **Lateral-move systems.** With reference to Figure 3.2a, s is given by :

$$s = (m^2r^2+y^2)^{½} \tag{3.3}$$

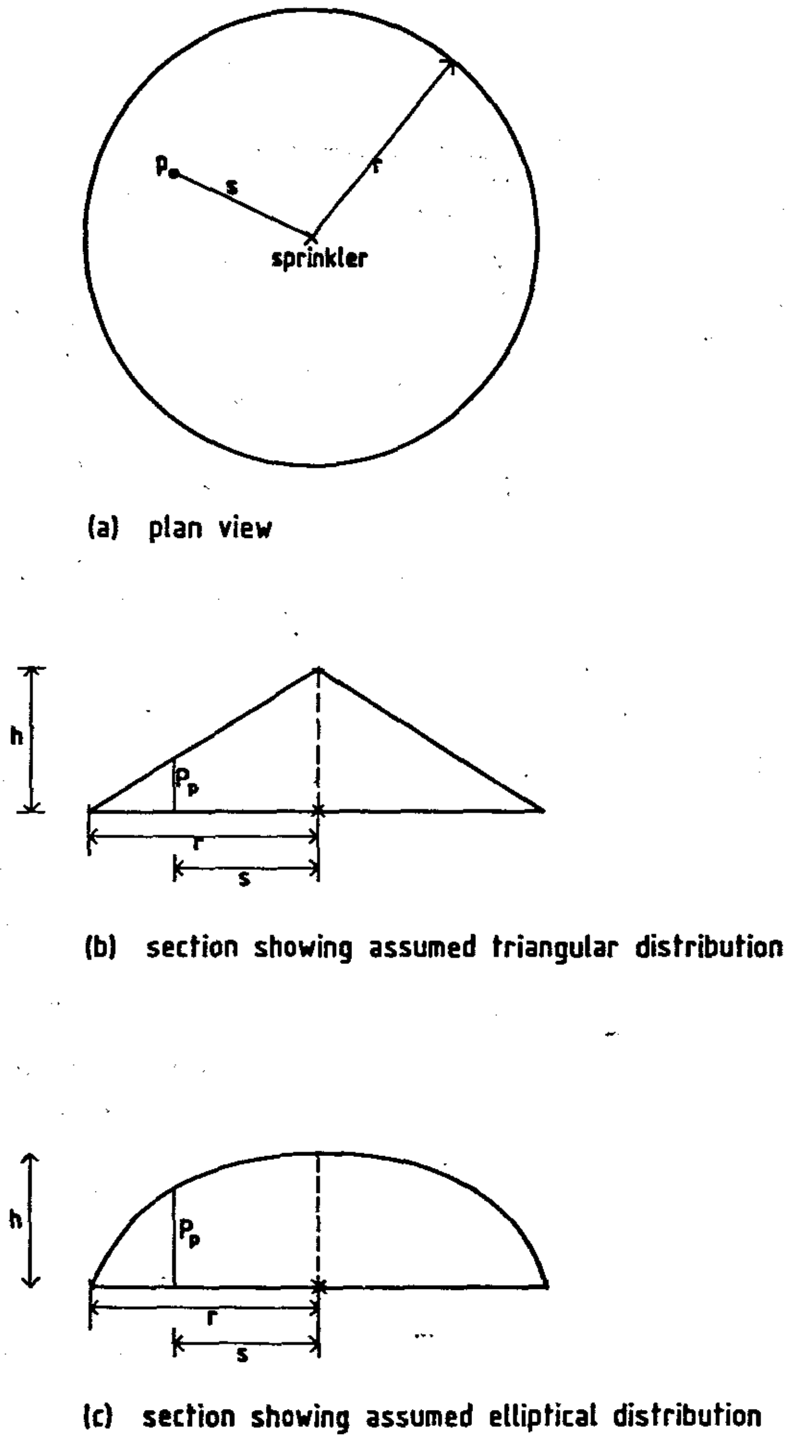


Figure 3.1 Assumed geometric patterns of water distribution from a single operating sprinkler (r = wetted radius on the ground; h = application rate in mm/h)

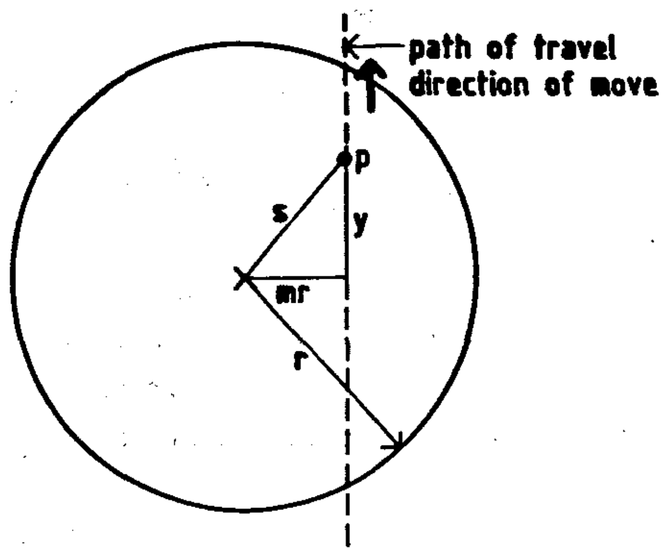


Figure 3.2(a) Analysis of distribution pattern for a moving sprinkler : lateral move

And if the travel speed of the sprinkler is v, and it takes time t to move from its present position to a point perpendicular to p, then :

$$y = vt \tag{3.4}$$

By substitution in equations 3.1 and 3.2, the precipitation rate at point p and time t after the sprinkler was perpendicular to p, is given by :

$$P_p = h\,[r - (m^2r^2 + v^2t^2)^{½}]/r \quad \text{(mm/h)} \tag{3.5}$$

for the triangular pattern, and

$$P_p = h\,[r^2 - (m^2r^2 + v^2t^2)]^{½}/r \quad \text{(mm/h)} \tag{3.6}$$

for the elliptical pattern.

Thus, if the sprinkler takes time T to move from a point perpendicular to p to a point where p is on the circumference of the wetted circle formed by the sprinkler, then the total depth of precipitation, D_p, at point p for a single pass of the sprinkler is :

$$D_p = 2\int_0^T P_p\,dt \quad \text{(mm)} \tag{3.7}$$

Substituting equations 3.5 and 3.6 respectively into equation 3.7 and solving, gives :

$$D_p = \left[(1-m^2)^{1/2} - m^2 \ln\left[\frac{(1-m^2)^{1/2}+1}{|m|}\right] \right] hr/v \qquad (3.8)$$

for the triangular pattern, and

$$D_p = (1-m^2)\pi hr/2v \qquad (3.9)$$

for the elliptical pattern.

It can be seen that hr/v and $\pi hr/2v$ give the depth of precipitation along the sprinkler's path of travel for the triangular and elliptical patterns respectively. Thus the terms in the brackets of equations 3.8 and 3.9 are dimensionless, and represent the fraction of the maximum depth which is received along a path which is fraction m of the wetted radius r away from the sprinkler's path.

b) **Center-pivot systems.** With reference to figure 3.2b, s is given by :

$$s = [(R+mr)^2 - 2R(R+mr)\cos\alpha + R^2]^{1/2} \qquad (3.10)$$

By substituting equation 3.10 into equations 3.1 and 3.2 respectively, and each of these into equation 3.7, relatively complex expressions for the depth of application at point p are obtained. Nevertheless, these expressions are readily evaluated using numerical methods and hence lend themselves to solution by computer.

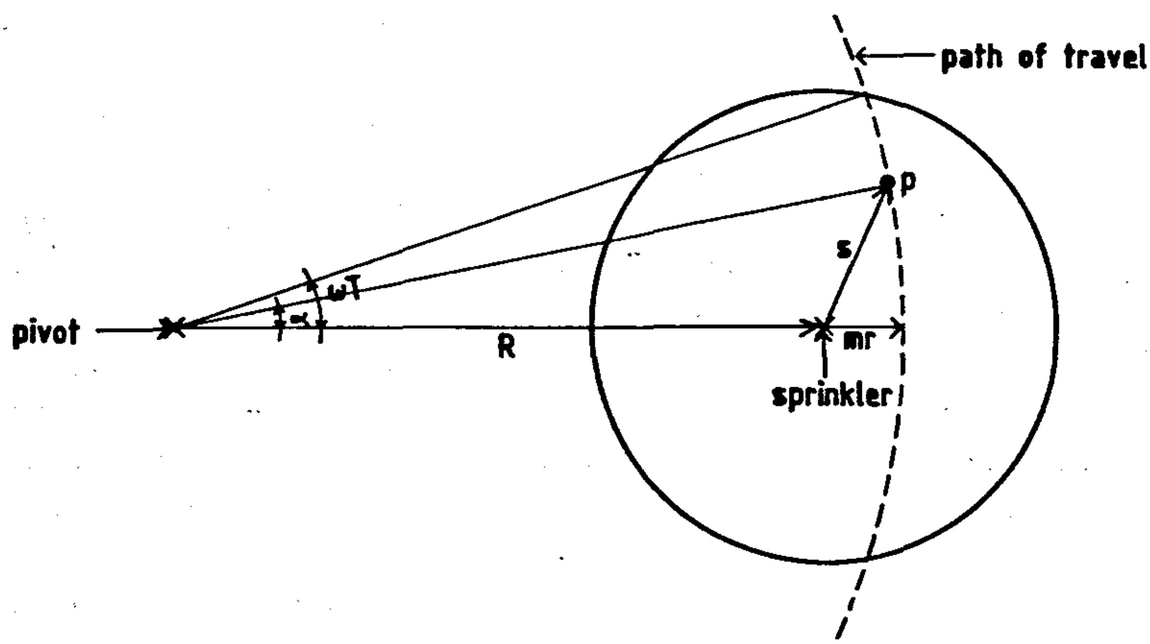


Figure 3.2(b) Analysis of distribution pattern for a moving sprinkler: center-pivot.

Single dripper

The emitters in a drip irrigation system provide a point source for water, which infiltrates directly into the soil at the point of emission. Any lateral spreading of the wetted area on the soil surface is due to forces within the soil matrix, rather than forces imparted by the emitter. Thus in assessing the distribution characteristics of drip irrigation systems, measurements of the application depth of various points over the irrigated area are no longer meaningful. The principal parameter related to distribution, from the point of view of the actual emitter, is its discharge rate, which is in turn dependent on the physical characteristics of the emitter and on the operating pressure. Karmeli and Keller (1975) conducted extensive analysis of the discharge characteristics of individual drippers. They classified the emitters in terms of the generalized flow/pressure relation :

$$q_e = kP^x \tag{3.11}$$

Where q_e = the emitter flow (*l*ph).
k = a proportionality factor, characteristic of each emitter.
P = the operating pressure head (m).
x = the emitter discharge exponent, characteristic of the water flow regime within the emitter.

The lower the value of x, the less the discharge will be affected by pressure variations in the irrigation system. For laminar flow $x = 1.0$, wheras for fully turbulent flow in the emitter, $x = 0.5$. In practice, for the most common long path emitters, x is somewhere between 0.5 and 1.0. Some pressure-compensating emitters have been developed, for which $x < 0.5$. In fully compensating emitters, the discharge will be constant regardless of the pressure variations, implying a value of $x = 0.0$.

In designing drip irrigation systems it is important to have an assessment of the water distribution pattern in the soil profile. As the water infiltrates the soil, it spreads to form a '*wetted bulb*' radiating from the source. Eventually a steady flow situation is reached in the relevant area around the emitter. The zone immediately around the source is saturated and the dimensions of the wetted bulb are constant. The extent of this bulb depends on the discharge rate and on the soil characteristics. In general, the higher the discharge rate and the lower the infiltrability of the soil, the larger the wetted area. Also, the higher the discharge rate, the greater the affect of gravity, resulting in an elongation of the bulb in the vertical direction (i.e. deeper area of wetting) together with a narrowing in the horizontal direction.

3.2.2 Field distribution pattern

The assessments of the distributions from individual emitters are used to develop overall patterns of the irrigation distribution in the field. Once again three different situations can be identified. These are loosely classified in terms of dimensions as follows:

1. Three-dimensional distribution in which the wetted area is continuous throughout the field. This is normally associated with closely planted field crops under sprinkler irrigation.

2. Two-dimensional distribution in which the wetting pattern consists of continuous rows, with a dry area between each row. This is normally associated with wider spaced "row crops" such as some vegetables and orchards. In these cases the most common irrigation systems are drip or micro-jet (sprayer) with the lateral lines running parallel to the rows.

3. One-dimensional distribution in which individual plants are irrigated and only the area around the plant is wetted. This occurs in orchards with large planting distances, and is normally associated with drip systems, but may also occur with sprayer or even sprinkler systems.

In order to develop an overall field distribution pattern, measurements can be taken over the whole area, in the same way as was described for single emitters. Alternatively, the patterns can be built up using a simple overlapping procedure developed by Hart and Heerman (1976). This procedure assumes that the distribution pattern obtained from a single emitter is replicable for the other emitters in the field. Thus, starting with the point values on the grid matrix for the single emitter, the matrix is shifted left and right, and up and down, sufficiently to provide a new, composite pattern with the sprinkler position repeated at the appropriate spacing. The sprinkler spacing must be some whole integer multiple of the grid intervals, so that the values on the grid of each sprinkler coincide with each other in the area where they overlap. Then the overall pattern is developed by simply adding together all the values occurring at each point of the new composite grid.

This process can be described mathematically :

Given an m x n matrix consisting of elements a_{rs} describing the distribution from an individual emitter

Then within the p x q matrix describing the composite distribution pattern within the rectangle described by the four positions of adjacent sprinklers in the field the elements b_{kl} are given by :

$$b_{kl} = \sum_{r=k}^{m} \sum_{s=l}^{n} a_{rs} \tag{3.12}$$

where $r = k,\ k+p,\ k+2p \ldots r_{max}$ and $r_{max} < m$

$s = l,\ l+q,\ l+2q \ldots s_{max}$ and $s_{max} < n$

Hart and Heerman investigated the reliability of this process in describing real irrigation situations. In particular they considered the effects of the variations in winds during and between irrigations. They compared the pattern formed by the results from a single sprinkler, with patterns obtained from overlapping the results from a series of sprinklers on a lateral line which is moved with each irrigation. They found the single sprinkler analysis to be satisfactorily accurate.

The same overlapping analysis can be carried out using the mathematical expressions derived from the assumed geometric shapes of the distribution pattern of individual emitters, as discussed in the previous section. Furthermore, these expressions provide an approach for describing the overall patterns obtained from continuous move systems, for which the summation of stationary point values procedure is not valid.

Given the generalized integral shown in equation 3.7, for the depth of application at point p from a single continuously moving sprinkler:
Then an expression for the depth of application, D_s at point p due to irrigation from several sprinklers on a continuous move system, is given simply by:

$$D_s = \sum_{i=1}^{n} D_p^i \tag{3.13}$$

Where n = the number of sprinklers in the system that irrigate point p.

D_p^i = the depth of application from one pass of the system at point p from sprinkler i, as derived from solving expression 3.7.

Once the data on application depths in the field have been collected, they are arranged in the form of a frequency histogram, as shown in Figure 3.3.

The histogram shows application depth versus area receiving the given depth or greater, for a hypothetical 20 Ha plot. This frequency distribution provides the basis for extensive analysis of the quality of the irrigation. These methods of analysis are discussed in more detail in the following sections of this chapter. It is important to note that arranging the data in this form provides an indication of what fractions of the field being irrigated receive specific depths. However, it does not indicate where these specific fractions are located in the field. It is assumed that the quality of the irrigation can be assessed through analysis of this frequency distribution, without reference to location in the field (See section 3.3).

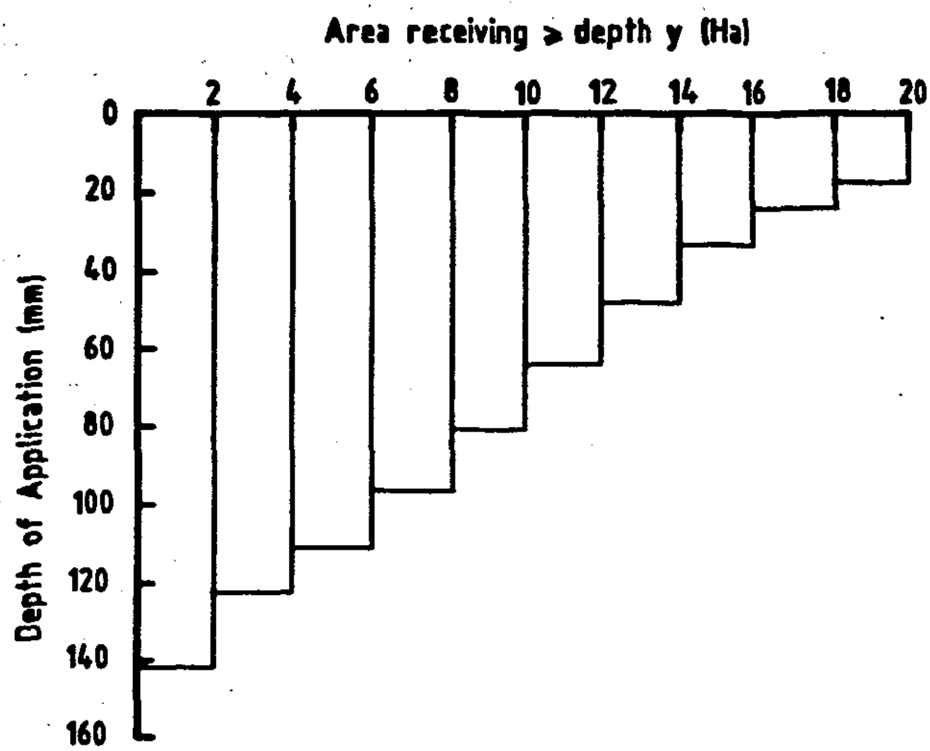


Figure 3.3 Example of an application depth vs area irrigated histogram for a 20Ha plot.

3.2.3 Functional forms of the distribution pattern

The arrangement of the distribution pattern data in the form of a frequency histogram lends itself to the development of mathematical functions to describe the distribution. From these functions, several parameters that describe the distribution pattern and provide a measure of the quality of the irrigation can then be identified.

The first step in this process is to form the normalized dimensionless distribution curve shown in Figure 3.4 by dividing each application depth by the average depth, and each area by the total area. The curve is then drawn by joining the midpoints of each frequency in the histogram. Note that the average depth of application is 1.0 on the new scale and that the total area bound by the two axes and the curve is equal to 1.0. This latter property implies that any function, $f(x)$, used to describe this distribution curve will be a normalized density function.

Several functional forms have been proposed. Howell (1964) suggested the process of determining as many moments as possible to characterize the distribution. However Hart and Heerman (1976) have shown that the first two moments (i.e. the mean and variance of the distribution) are sufficient for most purposes, particularly when a standardized function is used.

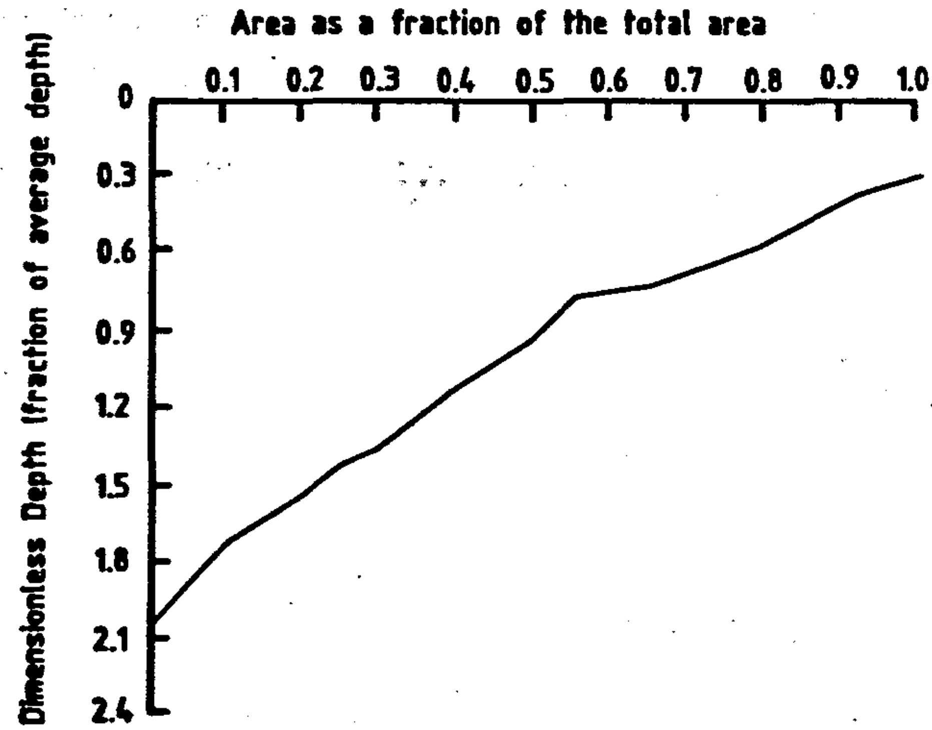


Figure 3.4 Normalized dimensionless distribution curve

Since the application depth frequency curve generally approximates an "S" shape, Hart (1961) proposed a normal distribution model to describe the distribution pattern. Once the mean and standard deviation of the data are known, then the distribution is completely defined by the expression:

$$Q(y) = (1/s\pi\sqrt{2}) \int_{Y}^{\infty} exp\,[-½\{(Y-\bar{y})/s\}^2\,]\,dy \tag{3.14}$$

Where $Q(y)$ = The area under the normal distribution curve from y to ∞, which in this case represents the fraction of the area which receives dimensionless application depth y or greater;

s = The standard deviation of the application depths;

$\bar{y}$ = The average application depth;

Y = The dimensionless depth for which the following holds : $y =< Y =< \infty$

The coefficient of variation $v = s/\bar{y}$ provides an indication of the uniformity of the distribution. A small value of v implies a highly uniform irrigation, characterized by a concentration of the application depths around the mean, and hence a frequency distribution curve tending towards the horizontal.

A larger value of v implies a less uniform irrigation with a greater spread of application depths giving a more "stretched out" distribution curve. This is illustrated in Figure 3.5.

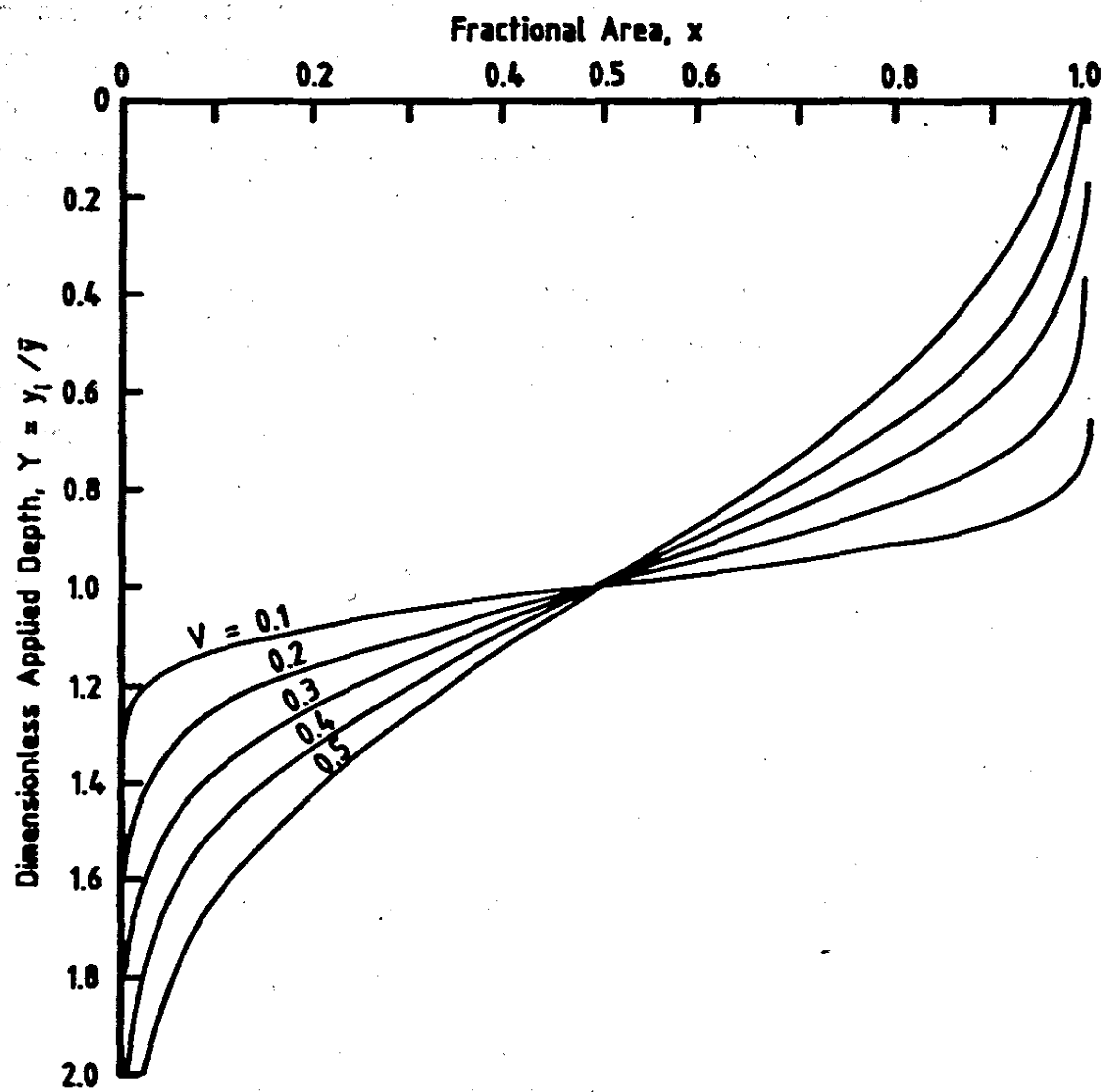


Figure 3.5 Examples of the S shaped cumulative frequency curve for varying values of the coefficient of variation v.

On the basis of above, Karmeli (1977) has proposed an alternative linear model to describe the distribution pattern. In this model, the distribution is given by:

$$Y = a + bX \tag{3.15}$$

Where Y = dimensionless precipitation depth

X = fraction of total area

a,b = linear regression coefficients

The constants a and b are determined by least squares regression. Karmeli argues that for distributions where v is large, the linear model provides a better fit than the normal distribution model, since the errors at both extremes of the frequency curve will be smaller for the linear model. Also, whereas the normal model might be expected to provide a better fit than the linear model in cases where v is small, in fact the linear model should provide a good fit since in these cases most of the values are concentrated around the mean and therefore the errors at each extreme of the curve will be limited.

Karmeli calibrated his model for several different distribution patterns, with a wide range of values of v. The regressions yielded intuitively sound models with highly significant goodness-of-fit statistics in each case, indicating most satisfactory models. the principal advantage of Karmeli's model is its simplicity, particularly in developing the irrigation quality parameters which are discussed in the following sections.

The gamma and beta distribution models

Several variations of these two basic models have been proposed. Chaudhry (1978) examined the effects of the skewness of observed distribution patterns, together with the exclusion of negative observations, on the fit of the normal model. He proposed an alternative gamma distribution model, which can account for the observed skewness and eliminates the negative observations. It also provides more accurate evaluations, than the normal model, of some of the irrigation quality parameters discussed in the following sections. By this model the area fraction, a, which receives depth y_a or greater is given by :

$$a = 1 - [\, \gamma_{\sqrt{rt_a + r}}(r) \, / \, \Gamma(r) \,] \tag{3.16}$$

Where
$\gamma_m(r)$ = the incomplete gamma function.
$\Gamma(r)$ = the complete gamma function.
r = the parameter of the gamma distribution, which in this case is $4/C_s^2$.
C_s = the coefficient of skewness = $[\, \Sigma\ (y_i - \bar{y})^3 \,] / s^3 N$
t_a = $[y_a - \bar{y}\,] / s$

Elliot et al (1980) proposed an alternative beta distribution model which they believed was more flexible than the gamma distribution in modelling any skewness of the application depth data. They calibrated the model using a simple transformation to constrain the dimensionless depth values between 0 and 1, and method-of-moments estimates to calculate the parameters α and β. They compared the fits of normal, linear and beta models on 2 450 different overlapped sprinkler patterns, using the RMS error statistic as a means of comparison. They found that the beta model consistently gave a better fit than the other two models. However they concluded that the normal and linear models are more practical than the beta model since they are simpler to calibrate and provide adequate results. They found further that the linear model provided a better fit for non-uniform patterns, but that the reverse was true for more uniform patterns. Since the most common patterns fall into the more uniform category, they recommend the general use of the normal model.

Seniwongse et al (1972) used the Chi-square test to compare the goodness of fit of the gamma and normal distributions. They found that in fact the normal model generally provided better fits than the gamma model, and they determined further that for highly uniform patterns, the skewness and kurtosis of the distributions had a negligible effect on the values of the various irrigation performance parameters.

Evaluating the distributions

Once the dimensionless depth distribution function has been determined, several evaluative interpretations can be made. In Figure 3.6, the dimensionless depth of water required throughout the field at the time of irrigation is given by the broken horizontal line dividing area A from area B. Then, given the dimensionless depth frequency curve shown in the figure, area A represents the volume of water which was used to satisfy the irrigation requirement; area B represents the volume of water lost to deep percolation in the soil; and area C represents the shortfall volume of water in areas of the field that were under-irrigated. The intersection of the frequency curve with the irrigation requirement line indicates that approximately 55% of the field was adequately irrigated and that the remaining 45% was under-irrigated.

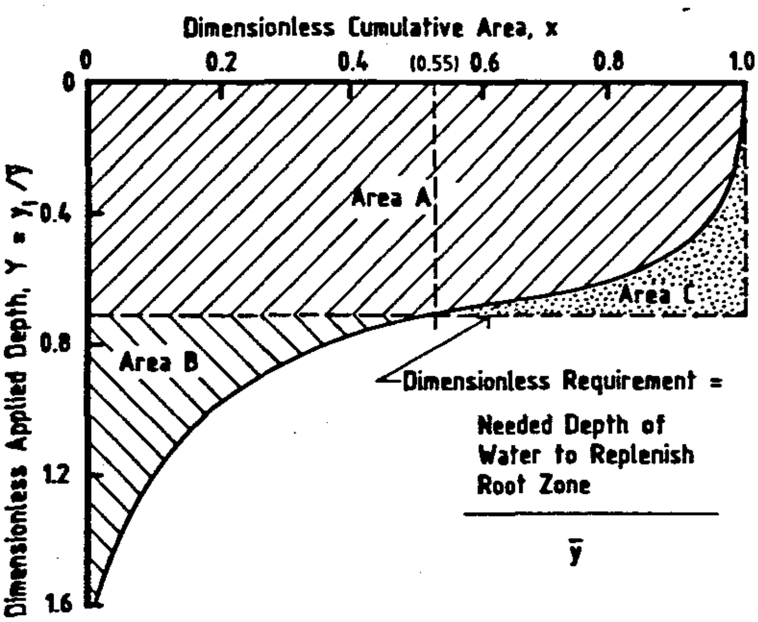


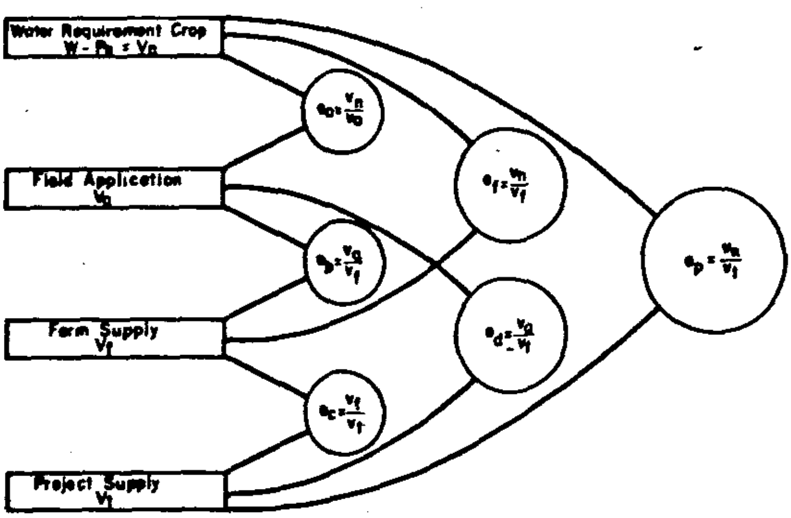
Figure 3.6 Analysis of the cumulative frequency distribution curve.

On the basis of these analyses several parameters measuring the efficiency of the irrigation can be calculated. Furthermore, the shape of the frequency curve provides a measure of the uniformity of the distribution. These parameters are discussed further in the following sections.

3.3 Efficiency

In the engineering context, the term *"efficiency"* generally refers to the ratio of output to input. This context is applied to irrigation, where the inputs and outputs are quantities of water. Numerous different efficiencies have been defined in the literature, differing from each other on the basis of what each of the inputs and outputs represent.

Bos and Nugteren (1974) defined a set of efficiencies based on the four basic water quantities as shown schematically in Figure 3.7. These efficiencies were used to evaluate irrigation data form several different countries with greatly varying crops, climatic and other physiographic conditions and irrigation methods. The set proved to be adequate for developing a composite picture of the efficiency of different irrigation methods over a wide range of operating conditions. However, this efficiency was measured in terms of overall water utilization, rather than actual irrigation efficiency.



V_n, V_a, V_f, V_t = water volumes
w = crop requirement at time of irrigation
P_e = effective rainfall
e_a = application efficiency
e_b = farm ditch efficiency
e_c = water conveyance efficiency
e_d = distribution efficiency
e_f = farm efficiency
e_p = overall project efficiency

Figure 3.7 Set of efficiencies defined by Bos & Nugteren (1974)

Several investigators have developed measures of irrigation efficiency by defining a fifth basic water quantity, viz. the quantity of water which is beneficially used during one irrigation cycle. Although this is a more difficult parameter to measure or determine than the other four, it does enable a more rigorous analysis of the irrigation performance than the set defined by Bos and Nugteren.

Many of the efficiency measures are very similar to each other, but are defined in the literature slightly differently and may therefore have different names. A list of the principal parameters is provided on the following pages on the basis of the terms that will be adopted for this chapter. In cases where alternative terms have been used for these parameters in the literature, these terms are listed together with their respective sources.

The following measures are expressed in terms of depths and volumes defined in figure 3.8 :

Maximum and average deficit (Hart and Reynolds 1965)

The average deficit is given by Volume-A/a, and the maximum deficit by $H_R - H_{min}$.

Application efficiency (Hart and Heerman 1976; Karmeli et al 1978; Walker 1979; Chaudhry 1978)

This is the fraction of the total application that is made available to the plant, as given by Volume-B/Volume-D. It has been alternatively referred to as :

Water storage efficiency (Hart and Reynolds 1965)

Effective water application (Howell 1964)

Deep percolation efficiency (Hart, Peri and Skogerboe 1979)

Requirement efficiency (Walker 1979)

Defined as the fraction of the total requirement that is met, as given by Volume-B/Volume-E. It has been alternatively referred to as :

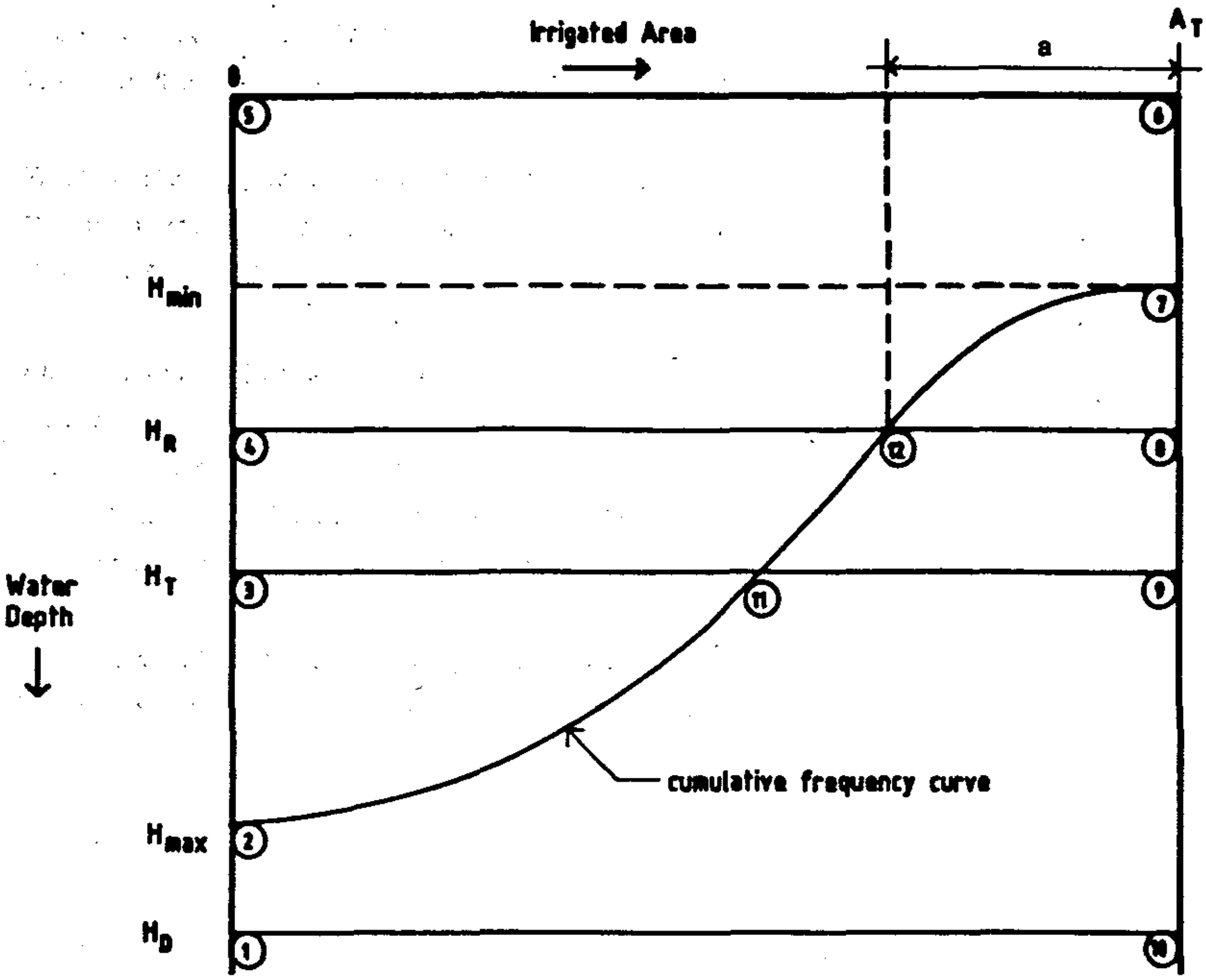
Availability factor (Hart and Reynolds 1965; Chaudhry 1978)

Storage efficiency (Karmeli et al 1978; Hart et al 1979)

1 - deficiency coefficient (Peri and Skogerboe 1978)

Delivery efficiency (Hart et al 1979)

If we have a measure of losses, other than deep percolation, due to factors such as wind drift, evaporation and runoff, then this parameter is defined as the fraction of the total volume of



a = area of the field which is deficiently irrigated

A_T = total area of the field

H_{min} = minimum application depth

H_{max} = maximum application depth

H_R = requirement at the time of irrigation

H_T = average application depth

H_D = average delivery depth

The numbers below refer to points on the above diagram, and the areas they enclose designate volumes as follows :

Volume-A =	12,7,8,12	(deficit volume)
Volume-B =	4,5,6,7,12,4	(volume received by the plant)
Volume-C =	2,4,12,11,2	(excess volume)
Volume-D =	2,5,6,7,12,11,2	(total applied volume)
Volume-E =	4,5,6,8,4	(required volume)
Volume-F =	1,5,6,10,1	(total delivered volume)

Figure 3.8 Cumulative depth vs irrigated area curve, showing measurements defining the various efficiency and uniformity parameters.

water delivered that is effective in the irrigation. It is given by Volume-*D*/Volume-*F* and has been alternatively referred to as :

Tailwater efficiency or 1 - proportion of tailwater (Karmeli et al 1978)

Fraction of deep percolation (Karmeli et al 1978)
This the fraction of the total application which is lost to deep percolation, and is given by Volume-*C*/Volume-*D*.

Application ratio (Chaudhry 1978)
This is the ratio of the average application depth to the required depth, as given by H_T/H_R. It has been alternatively referred to as :

The application coefficient (Peri and Skogerboe 1978)

The distribution coefficient (Hart and Reynolds 1965)

Delivery coefficient (Peri and Skogerboe 1978)
This is the ratio of the delivery depth to the required depth, as given by H_D/H_R.

3.4 Uniformity

Similarly, numerous concepts of uniformity have been defined, and these are reviewed below (summarised in Karmeli et al 1978 and Hart and Heermann 1976).

Christiansen Uniformity Concept - UCC (Christiansen 1942)
Christiansen first suggested a uniformity coefficient which gave the absolute mean deviation of the various application depths from the average depth, over the whole field, expressed as a fraction of this average depth. It was expressed mathematically as follows :

$$UCC = 100\,[\,1 - (\,\Sigma\left|X_i - \overline{X}\right|)/N\overline{X}\,] \qquad (3.17)$$

Where N = the number of observations X_i.
$\overline{X}$ = the average application depth.

Wilcox and Swailes - UCW (1947)
Wilcox and Swailes replaced the absolute mean deviation from the mean, with the standard deviation (sum of squares deviation from the mean), to give the following formula :

$$UCW = 100\,(1 - s/\overline{X}) \qquad (3.18)$$

Where s = the standard deviation.

Hawaiian Sugar Planters Association - UCH (Hart 1961)

If the distribution in the field is normal, then the absolute mean deviation from the mean is equal to $\sqrt{2/\pi}s = 0.798s$. Thus Hart proposed the following uniformity coefficient :

$$UCH = 100\,(1 - 0.789s/\bar{X}) \tag{3.19}$$

Karmeli et. al. (1978) - UCL

In the linear model proposed by Karmeli et al, the slope of the regression line (coefficient b in Figure 3.9) gives an indication of the uniformity of the irrigation. The smaller the value of b, the more uniform the irrigation. For the dimensionless model, as shown in Figure 3.9, the average deviation from the mean is given by $2[0.5 \times 0.5 \times (Y_{max} - \bar{Y})]/1$. And since $Y_{max} - \bar{Y} = 0.5b$, this average deviation is given by $0.25b/1$.

Thus since the average dimensionless depth = 1, Karmeli et. al. have proposed a uniformity coefficient (UCL), equivalent to UCC, given by :

$$UCL = 1 - 0.25\left|b\right| \tag{3.20}$$

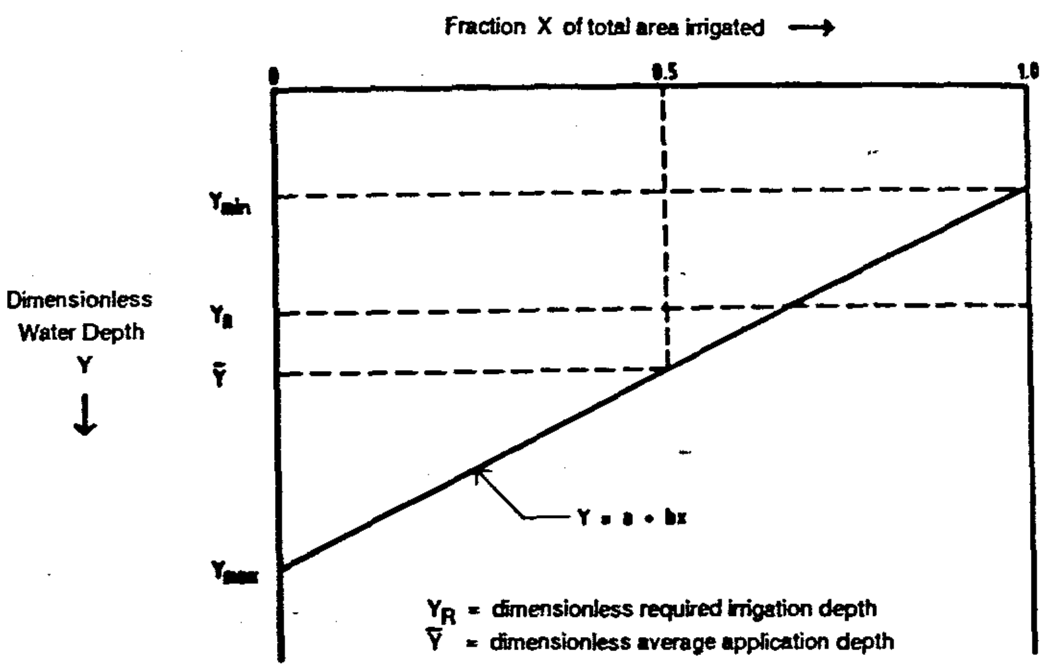


Figure 3.9 Dimensionless linear model of cumulative depth vs area irrigated (after Karmeli et. al. 1978)

Benami and Hore (1964) - A

These authors proposed an alternative coefficient which considers deviations of observations greater than the mean separately from deviations of observations less than the mean. Their coefficient is given as :

$$A = 1.66 \frac{\overline{X}_b - (\Sigma|X_{i,b} - \overline{X}_b|)/N_b}{\overline{X}_a - (\Sigma|X_{i,a} - \overline{X}_a|)/N_a} \tag{3.21}$$

Where Subscript a refers to observations greater than the mean.
Subscript b refers to observations less than the mean.
$X_{i,a}, X_{i,b}$ are the individual observations.
$\overline{X}_a, \overline{X}_b$ are the means of the two groups.
N_a, N_b are the number of observations in each group.

Pattern Efficiency - PEU (The USDA; Criddle et. al. 1956)

The United States Department of Agriculture proposed a pattern efficiency coefficient which expressed the average of the lowest 25% of the observations as a fraction of the overall average. This is expressed mathematically as :

$$PEU = \Sigma X_{i,q} / N_q \overline{X} \tag{3.22}$$

Where $\Sigma X_{i,q}$ = the sum of lowest 25% of the observations.
N_q = the number of observations in this group.
$\overline{X}$ = the mean of all observations

Hart and Heermann (1976) showed that given a normal distribution of depths in the field, an equivalent pattern efficiency can be calculated as follows :

$$PEH = 1 - 1.27\,(s/\overline{X}) \tag{3.23}$$

3.5 Appropriate evaluation parameters for design

3.5.1 Qualitative analysis

In order to utilize some of the measures of irrigation quality for design purposes, the multitude of parameters described above must be screened in order to arrive at the most appropriate ones to provide a composite picture of the irrigation performance, and to reflect the effects of the changes made to the design.

As far as efficiency is concerned, it can be seen that all of the parameters listed above can be derived from six basic measurements, *viz :*

1. The total volume of water delivered by the system.
2. The total volume of water usefully applied to the field.
3. The total crop requirement at the time of irrigation.
4. The volume of the deficit.
5. The volume of the excess (deep percolation).
6. The minimum depth of application.

Hart et al 1979 showed further that all of the parameters described above, apart from the maximum and average deficits, can be readily derived from the three basic parameters, viz.:

* Water application efficiency
* Water requirement efficiency
* Delivery efficiency

The delivery efficiency is principally a measure of the effect of factors related to the actual operation of the system in the field. Typical losses that reduce the delivery efficiency are those due to wind drift, evaporation, runoff and leakage. Thus, whilst delivery efficiency is affected to some extent by design decisions such as irrigating at night and relating the emitter spacing to expected wind conditions, it is difficult to measure *a priori* on the basis of an assumed distribution pattern. For design purposes, an average, empirically established value is usually assumed for the whole field. This value will generally be the same for all design alternatives of a given method of irrigation. This implies that for the purpose of evaluating a design, the irrigation efficiency can be adequately characterized by the application and requirement coefficients.

As far as uniformity is concerned, each of the parameters described above expresses some measure of the deviation of parts of the distribution from the mean. In considering the pattern efficiencies (PEU and PEH) it is obvious that any number of parameters can be derived by simply defining different fractions of the distribution. In this sense, the parameters of Christiansen, Wilcox and Swailes, Hart and Karmeli respectively must be considered as being the most general, since they all give a measure of the deviation of the total distribution.

Benami and Hore (1964) believe that their coefficient (A) is more sensitive than others, because it stresses the deviations of deficient observations and also because it is more sensitive to the larger deviations from the mean than other parameters. Hart and Reynolds (1976) however, have shown that both of these assumptions are not necessarily correct. They showed further that for a given sample data set, a simple linear relationship expressing A in terms of UCC could be established by least squares regression, yielding a correlation coefficient of 0.883. They concluded that there appeared to be no apparent advantage in using A rather than any of the other parameters.

The parameters UCC, UCH and UCL all purport to measure the mean deviation of observations from the overall mean. The latter two assume different functional forms of the distribution. Several researchers (notably Hart and Heerman 1979, Seniwongse et al 1972 and Karmeli et al 1978) have investigated the correlation between these parameters for numerous different sets of data. In all cases high correlation was obtained for simple linear relationships between any two of the parameters. Furthermore, it can be seen from the definitions of UCH and UCW that a simple linear relationship exists between these two parameters. Thus it can be seen that all the uniformity parameters discussed above are very much equivalent to each other. In the case of computer-based design procedures the distribution data will be generated in the computer from calculated discharge rates at each emitter. In this case there is no point in assuming any functional relationship to represent the distribution, since the uniformity can be calculated directly from these numerical data, using the formulation for UCC.

A composite picture of the irrigation performance can therefore be developed for alternative designs from the application and requirement efficiencies and the Christiansen uniformity coefficient. As long as either the required depth or the average application depth are given, then these parameters fully characterize the irrigation. (Note: Chaudhry 1978 believes that the skewness coefficient should also be added as a measure of the extent to which the distribution is not normal).

The uniformity coefficient is dependent only on the shape of the cumulative frequency curve, and is independent of the actual depths applied. In other words, it is independent of the vertical alignment of the point H_T on the "*water depth*" axis in Figure 3.8. In this sense, the uniformity coefficient is independent of the two efficiency coefficients. These efficiency parameters however, are determined both by the uniformity (shape of the cumulative frequency curve) and by the application ratio.

Hansen (1960) and Hart et al (1979) have illustrated this for three different cases as shown in Figure 3.10. If H_R is greater than H_{max} (Figure 3.10a), then regardless of the uniformity there will be no deep seepage, and the application efficiency will therefore be 100%. However, in this case the requirement efficiency will be the inverse of the application ratio. Alternatively, if the required depth is less than H_{min} (Figure 3.10b), then the requirement efficiency will be 100% and the application efficiency will be equal to the application ratio. In

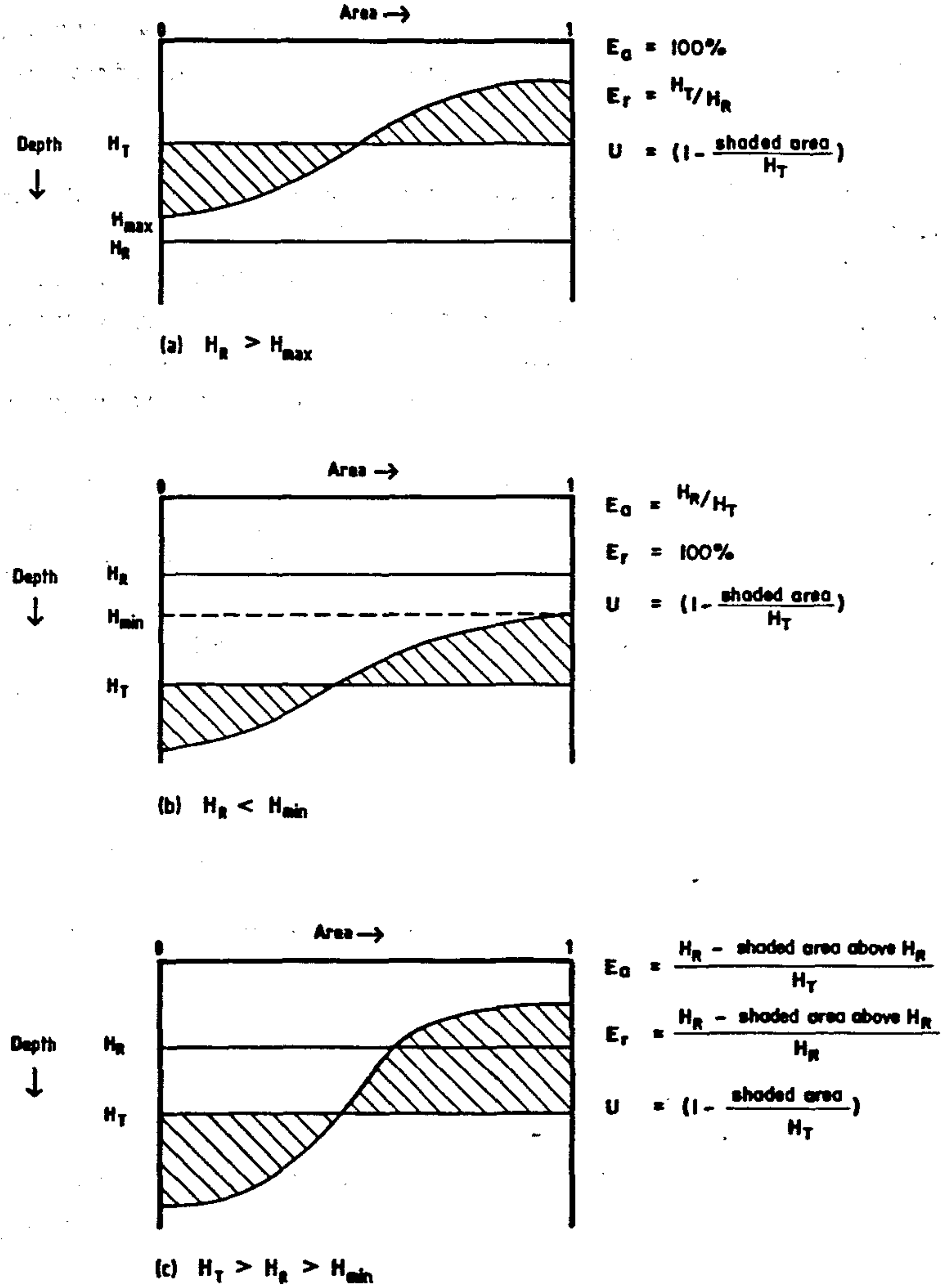


Figure 3.10 Cumulative frequency curves in three extreme cases illustrating the relationship between application efficiency, requirement efficiency and uniformity (symbols as defined in figure 3.8 and the text)

between these two extremes, when the required depth is greater than H_{min} but less than H_T (Figure 3.10c), which is the most common situation in practice, then the requirement efficiency can be increased either by improving the uniformity or by increasing the application ratio.

Conversely, once the uniformity and average application depth are established, then a given value of either of the efficiencies will also determine the application ratio. This implies that if predetermined minimum acceptable values of the efficiencies are specified, then for a given uniformity the application ratio can be determined. In other words, for a given distribution pattern in the field (which fixes the uniformity) the application ratio can be established by specifying the required values of the efficiencies.

Thus it can be seen that these three parameters can be used to provide a complete description of an irrigation. Hart and Reynolds (1965) produced a complete table of feasible values of these parameters, thereby characterising all possible irrigations within this feasible range. Hart et al (1979) illustrated this characterisation graphically for a given uniformity. Walker (1979) and Chaudhry (1978) developed analytical solutions for the relationships characterizing an irrigation. Both of these models also yield the size of the area which will be deficiently irrigated.

3.5.2 Economic analysis

Whereas a number of researchers have investigated the inter-relationships between all of the abovementioned parameters, in order to characterize an irrigation qualitatively, limited work has been done to investigate their use in any economic evaluation of irrigation performance. Hart et. al. (1979) have proposed the possible formulation of a composite objective function Q given by :

$$Q = C_1E_r + C_2E_a + C_3E_d + C_4U_d \tag{3.24}$$

where E_r, E_a and E_d are the requirement, application and delivery efficiencies respectively, U_d is the uniformity coefficient and C_1 ... C_4 are weighting factors giving the relative importance of each quality parameter in the overall evaluation. These weights reflect the economic factors related to each of the parameters. Hart et. al. have suggested that C_1 and C_4 are both related to crop yield and water quantity and the associated earnings and costs; C_2 is related to the cost of deep percolation; and C_3 is related to the cost of distribution losses. They have not however, suggested how Q can be evaluated, nor how it can be related to any absolute measure of irrigation quality.

If, within the framework of the normal *minimum cost* design practice, the irrigation quality is to be maximised, then equation 3.24 can be reformulated as an objective function as follows:

$$Min\ [Q] = K_s + K_o + C_1(1\text{-}E_r) + C_2(1\text{-}E_a) + C_3(1\text{-}E_d) + C_4(1\text{-}U_d) \qquad (3.25)$$

Where K_s = the irrigation system capital costs.

K_o = the system operating costs.

C_i = the respective costs minus earnings associated with each quality parameter.

Note that the quality measures have been expressed in the form : 1 - the parameter, in order to maintain the consistency of the minimizing objective function. It can be seen that the last four terms of equation 3.25 represent the *"cost of non-uniformity"*. If the irrigation was perfectly uniform ($U_d = 1$) and exactly the right amount of water was applied, so that $H_R/H_T = 1$, giving $E_r = E_a = E_d = 1$, then these four terms would all be equal to zero.

If the system capital and operating costs, rationalized over the expected life of the system, can be expressed in dollars (or other monetary units) per cubic meter of water delivered by the system, C_w, then :

$$C_3 = V_d C_w \qquad (3.26)$$

$$\text{and } C_2 = V_a C_w \qquad (3.27)$$

Where V_d = the total volume delivered.

V_a = the total volume applied in the field.

C_1 and C_4 however are more difficult to establish, due to the fact that the parameters E_r and U_d are aggregate measures. In this regard they echo one of the classic problems in the use of mathematical modelling for systems analysis. The derivation of these measures results in the discarding of data regarding the spatial relationships between *individual* observations. From E_r it is possible to calculate the average deficit, D, in the field from :

$$D = V_R(1 - E_r)/A_d \qquad (3.28)$$

Where V_R = the volume required.

A_D = the area in deficit.

And the loss of earnings due to this deficit from :

$$L_D = (P_y - K_y)\, Y_D \qquad (3.29)$$

Where L_D = loss of earnings/unit area/unit depth of deficit
P_y = price earned/unit of yield.
K_y = production cost/unit of yield.
Y_D = loss of yield/unit area/unit depth of deficit.

Then the constant C_1 in the objective function is given by :

$$C_1 = L_D \times A_D \times D = L_D V_R (1 - E_r) \tag{3.30}$$

Hart and Reynolds (1965) proposed a similar model to calculate the expected earnings from an irrigation characterized by E_r.

Similarly, for C_4, Seginer (1978) has proposed a model giving the relationship between crop yield and hence net earnings on the one hand and irrigation uniformity on the other. The model is based on the assumptions that :

1. The yield/water function can be divided into two linear segments, with no loss of yield due to excess irrigation.
2. The irrigation distribution can be represented by Karmeli's linear model.
3. The cost of water is independent of the uniformity, i.e. that an increase in uniformity does not imply an increase in water cost due to the increase in system cost.

Ideally, given a yield/water function, $y(i)$, which expresses the yield per unit area achieved as a function of the irrigation depth i for the crop being considered, then a *profit-maximizing design objective function* can be derived as shown below :

Given the frequency distribution function of irrigation depths for a given irrigation, $f(i)$, then the expected total yield, Y, for a field of Area A_T is given by the integral :

$$Y = A_T \int_{i_{min}}^{i_{max}} y(i)\, f(i)\, di \tag{3.31}$$

When a design alternative is simulated by computer, the irrigation depths throughout the field are generated as part of the design. In this case integral 3.31 can be solved numerically, without any explicit evaluation of $f(i)$.

The profit-maximizing objective function, is given by :

$$Max\,[NE] = Y(P_y - K_y) - V_a C_w \tag{3.32}$$

Where NE = The Net Earnings achieved through irrigation.

V_a = The total volume of water applied.

All other terms are as defined previously.

The constraints required to formulate the full optimization problem consist of the complex set of functions determining the cost C_w and the volume V_a together with the integral 3.31 above.

The most significant problem related to the models expressed in equations 3.25 and 3.31 respectively is the obtaining of sufficiently accurate yield/water functions. It is believed, however that with the ongoing research and development of these relationships, various adaptations of these models will increasingly be incorporated into the design process. Several papers relating to the solution of equation 3.31 are analysed further in chapter 5, leading to the proposed evaluation model which has been formulated as part of this research.

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PART 2 :

THE MODELS

4. BLOCK DESIGN

4.1 Introduction

An **irrigation block** consists of the components of the system that operate downstream of the valve; it is alternatively known as the **in-field** system because it is usually located above ground in the actual field being irrigated, as opposed to the **mainline** which is normally underground and conveys water from the source to the field. The principal hardware components of the block system are the emitters, the lateral pipes and the manifold, as shown in figure 1.1 in chapter 1 and discussed further in chapter 2.

The block design process consists of three distinct phases, *viz* :

* **Establishing the pipe alignments;**
* **Sizing of the pipes;** and
* **Detailed evaluation of the expected system performance.**

As shown in figure 2.2, the output from the block design process consists of :

* The lengths and diameters of all pipes in the block;
* The specifications needed for the layout of the block system;
* The pressure and flow requirements at the valve;
* The required size of the valve; and
* An assessment of the expected costs and benefits that will result from the use of the system.

This chapter presents a detailed analysis of the first two phases of the block design process, namely *pipe alignment* and *pipe sizing*; the third (*evaluation*) phase is discussed in chapter 5.

4.2 Pipe Alignments

This aspect of the design process is influenced largely by local conditions which affect the block layout on a case-by-case basis. Consequently, it is normally done by trial and error, based on the designer's experience. Two distinct aspects of the design problem can be identified:

* Establishing the block dimensions; and
* Establishing the pipe layout within the block.

4.2.1 Block dimensions

This aspect of the design problem is strongly related to the preliminary design process, since the size of the block is determined by constraints related to the operating regime, the emitter selection and the available water supply, as follows :

* The operating regime determines the number of irrigation shifts in a complete cycle; the area to be irrigated in each shift is then given by the total area divided by the number of shifts per cycle.
* The emitter selection determines the irrigation application rate and thus establishes the relationship between block size and associated discharge requirement. The total discharge required in each shift is given by the product of the area to be irrigated per shift and the application rate.
* The maximum available water discharge may be limited by a number of factors such as existing pumps and pipelines, borehole capacities or regulations governing extraction rates from irrigation schemes or public rivers. The total discharge required per shift must not exceed the available maximum.
* In addition, practical considerations related to the most economical valve sizes also limit the available supply rate to each block. This maximum available supply determines the maximum block size; the total discharge required per shift, divided by the maximum available block supply, gives the number of blocks to be irrigated per shift and hence the total number of blocks required for the whole scheme.

Once the block sizes have been established, the actual dimensions are related to the following factors :

* the maximum allowable lengths of the lateral and manifold pipes, which in turn are related to hydraulic factors such as the maximum allowable pressure losses in the pipes;
* the economical trade-off between lateral length and manifold length, which is not easy to establish; and most importantly
* the constraints of the prevailing land geometry.

Oron and Walker (1981) have proposed an algorithm based on non-linear, mixed-integer mathematical programming techniques, for the optimization of the block dimensions. This procedure assumes a regular, rectangularly shaped field and determines the optimum length to width ratios for all blocks in the field. However, in practice irrigation fields are often irregularly shaped and the block dimensions are influenced by a number of other factors such as existing farm infrastructure, the location of the water source and peculiarities in the local topography. For these reasons, as mentioned above, the block dimensions are probably best determined by trial and error based on experience.

4.2.2 Pipe layouts

Once the boundaries of the block have been determined, then the alignments of the pipes can be established. This is also normally done on the basis of the designer's intuition.

The laterals are positioned parallel to the planted rows and the manifold then transects the laterals. The exact positioning of the manifold requires further consideration, since it may split the laterals into two sets lying on either side of it. Ideally, the manifold should be positioned such that the lengths of the laterals running uphill away from the manifold are maximized, within the constraints of the allowable pressure loss in the system, as shown in figure 4.1 below. However, once again, local conditions may mitigate in favour of alternative alignments. For example, if the block has a natural ridge running through it, then it may be desirable to position the manifold along the ridge so that the laterals run downhill on either side of the manifold.

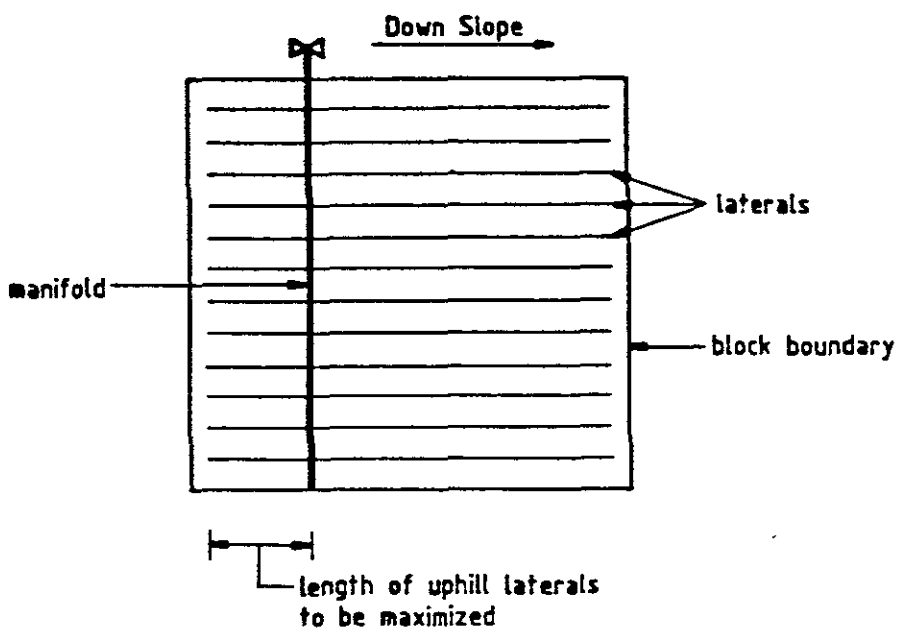


Figure 4.1 Layout of laterals and manifold

The positioning of the valve is often determined by the layout of an existing mainline. However, if its position can be determined quite freely, then similar considerations to those governing the positioning of the manifold apply to the positioning of the valve; in other words, the valve may split the manifold in two, in which case this should be done such that the length of the uphill section is maximized.

The computer programs developed through the research incorporate a pop-up "*maximum length calculator*", which is a rapidly accessed utility enabling calculation of the maximum allowable length of one or two diameter pipes on given slopes with given emitter characteristics and predefined allowable pressure losses. This utility can be used as an aid to establishing the valve and manifold positions for a current design (see section 4.4).

4.3 Determination of Pipe Sizes

This aspect of the design problem is currently perceived as constituting the **core** of the whole irrigation systems design process. It involves working with each pipe in turn and selecting diameters that maintain the variation in pressure, and hence discharge, along the length of the pipe within predetermined limits.

Calculation of the pressure losses in the pipe is done by any one of a number of empirical relationships that have been developed for the different pipe materials operating under various flow conditions. The most general form of these relationships is given by :

$$J = \alpha\,(Q/c)^{\beta} D^{\gamma} \tag{4.1}$$

Where J = the headloss per unit length of pipe;
Q = the flow rate in the pipe;
D = the diameter of the pipe; and
α, β, c, γ = parameters dependent on the pipe material, the flow regime in the pipe and the units of measurement of the variables.

The relationship between the pressure head in the pipe, H, and the emitter discharge, q, is given by :

$$q = k H^{x} \tag{4.2}$$

Where k and x are constants dependent on the emitter type and the units of q and H.

Several different design procedures have been proposed in the literature, and these are reviewed in section 4.3.1 below.

4.3.1 Existing manual solution procedures

The general principles of each of these methods are similar :

* A trial and error procedure is used for selecting either feasible pipe diameters for a given lateral length, or a feasible length for given diameters.

* For a given diameter, over a specified length of the pipe, the decreasing discharge from the pipe and hence the flow in the pipe are known. The headloss due to friction in the pipe over the specified length can therefore be calculated (equation 4.1) and the hydraulic grade line established.
* The variation in pressure and/or discharge along the length of the pipe is used as an indicator to test and modify the diameter or length selection.
* In each case the main objective is to establish a **minimum cost** set of diameters that meet the requirements regarding the maximum allowable variation of the discharge along the pipe.

The different solution procedures can be classified broadly into two main groups :

Design nomographs. The first set of procedures were proposed in a series of papers by Wu and Gitlin (1974, 1975, 1983) and Wu *et al* (1979). They are based on the determination of four dimensionless parameters, shown in figure 4.2 and defined as follows :

* **the friction loss ratio** $\Delta H_i/\Delta H$
* **the slope ratio** $\Delta Z_i/\Delta Z$
* **the total dimensionless friction loss** $\Delta H/H$, and
* **the total dimensionless slope** $\Delta Z/H$

Where ΔH_i = the friction loss in the pipe from the inlet to point i along the length of the pipe;

ΔH = the total friction loss over the full length of the pipe;

ΔZ_i = the head gain or loss due to land slope from the inlet up to point i;

ΔZ = the total headloss or gain due to slope over the full length of the pipe;

H = a known fixed pressure head, either the pressure at the pipe inlet or the emitter nominal design pressure.

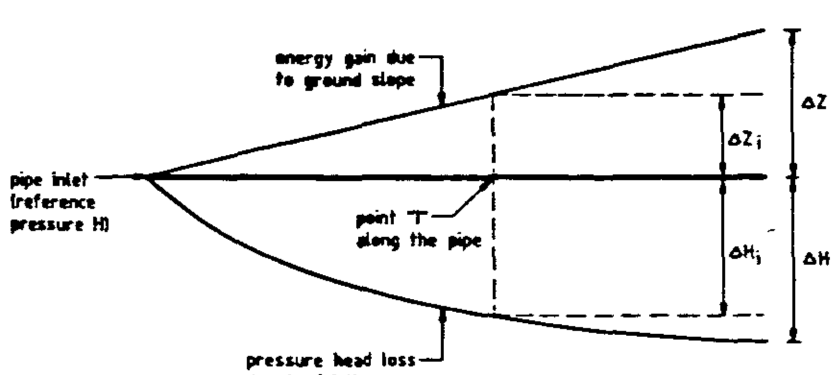


Figure 4.2 The parameters defined for irrigation pipe design nomographs

The four parameters can be derived from equation 4.1 and from information on the prevailing topography. They can be calibrated for a series of different operating conditions (i.e. pipe lengths and diameters, total discharge requirements and land slopes); and can then be used to characterize either the pressure or the flow variations (by incorporating the emitter pressure/discharge relationship - equation 4.2) for various combinations of the operating conditions. The designer can then test selected pipe diameters or lengths against the resultant pressure or flow variations or associated irrigation performance values related to these variations.

Wu *et al* developed a set of design charts based on calculations of the dimensionless parameters relating different operating conditions to :

* the coefficient of uniformity (equation 3.17);
* the maximum percentage pressure variation;
* the maximum percentage discharge variation;
* the irrigation application efficiency expressed as $q_{min} / q_{average}$; and
* possible variation of the emitter pressure/discharge exponent (x in equation 4.2) for given values of the total discharge variation.

Perold (1977) proposed an iterative solution procedure based on the four dimensionless parameters, that can be applied using a programmable pocket calculator. The procedure entails the use of the coefficient of uniformity as the criterion for acceptability of trial designs. It is applicable for the single diameter pipe/uniform slope case. However modern systems, particularly those designed for micro-sprayer emitters, generally incorporate telescopic (multi-diameter) pipes and they are often installed on sites with varying topography. Wu *et al* showed how their design charts can be adapted for the multi-diameter and non-uniform slope cases.

The "poly-plot" procedure. The major limitation of the procedures described above is that they do not incorporate an explicit algorithm for determining the lengths of successive diameters in a multi-diameter pipe, that will satisfy the design requirements at minimum cost. The widely used poly-plot procedure, proposed by Herbert (1971), is a graphical method which overcomes this limitation. The principles of the method are as follows :

* The criterion for acceptable design is set as a chosen value of the **maximum pressure variation** along the pipe. Then for a given nominal design pressure the maximum variation defines the maximum and minimum allowable pressure heads in the pipe respectively. These values can in turn be used together with the prevailing topographic

data to define an allowable pressure envelope along the length of the pipe. The upper and lower boundaries of this envelope are the maximum and minimum hydraulic grade lines respectively. A sectional representation of this is shown in figure 4.3(b).

* The pressure headloss due to friction in a pipe with a series of emitters along its length is a function of: firstly the size and type of emitter and its spacing along the pipe, and secondly the diameter of the pipe. If the discharge from each emitter is assumed to be constant along the length of the pipe, then the emitter size and spacing determine the **specific discharge rate (SDR)**, which is defined as the discharge through the emitters per unit length of the pipe.For example, a 50*l*/h sprayer operating at 2m spacing along the lateral has an SDR of 25*l*/h/m. For any given SDR and pipe diameter, a characteristic headloss curve can be produced. It will typically have an exponential shape, representing the decreasing rate of headloss per unit length in the direction of the flow, as the flow along the pipe decreases down to zero at the end.
  Herbert produced several sets of curves for different SDR's. Each set consisted of a curve for each different available pipe diameter, for the given SDR. Typical curves for an SDR of 30*l*/h/m are shown in figure 4.3(a).

* The design procedure entails drawing the allowable pressure envelope on transparent paper and then aligning it over the headloss curves for the appropriate SDR. The envelope is shifted vertically until the characteristic headloss curve fits within the boundaries of the pressure envelope.
  The designer normally starts with the smallest available diameter, working progressively from the closed end of the pipe back towards the pipe inlet. The envelope is shifted until the bottom of the headloss curve is tangential with the lower boundary of the pressure envelope. This ensures that the longest possible length of the smallest diameter is used before its headloss curve moves above the allowable pressure envelope. At a point where the headloss curve moves steeply up towards the upper boundary, it is replaced by the curve for the next bigger diameter, which is flatter at this point. The point of intersection of the two curves gives the length of the small diameter pipe that will be used. This procedure is continued until the inlet is reached, as shown in figure 4.3(b).

Jobling (1972) simplified the procedure by producing a **single universal set of curves** that could be used, together with a key diagram indicator to the appropriate curve, for all SDR and diameter values. Wu (1985) proposed a **uni-plot** procedure, based on the poly-plot principle, which fits a feasible energy grade line within the allowable pressure envelope, and then calculates the pipe diameter associated with the energy grade line from equation 4.1.

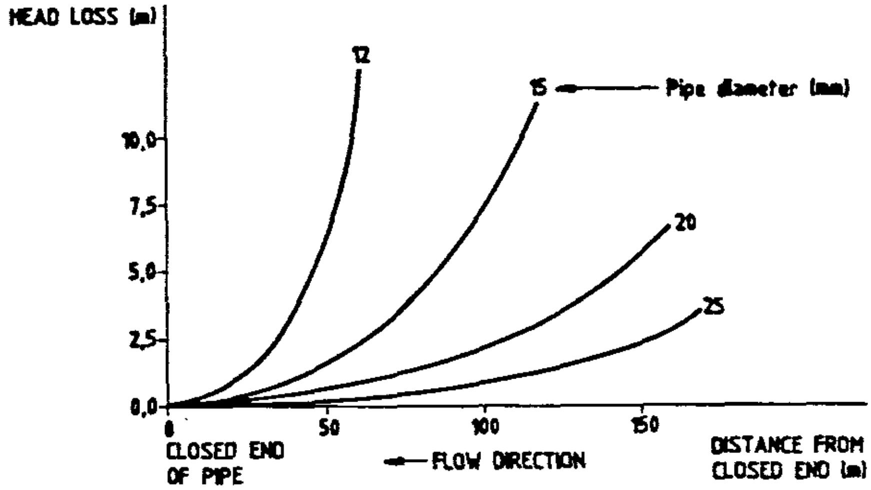


Figure 4.3(a) : Head loss curves for SDR = 30*l*/h/m

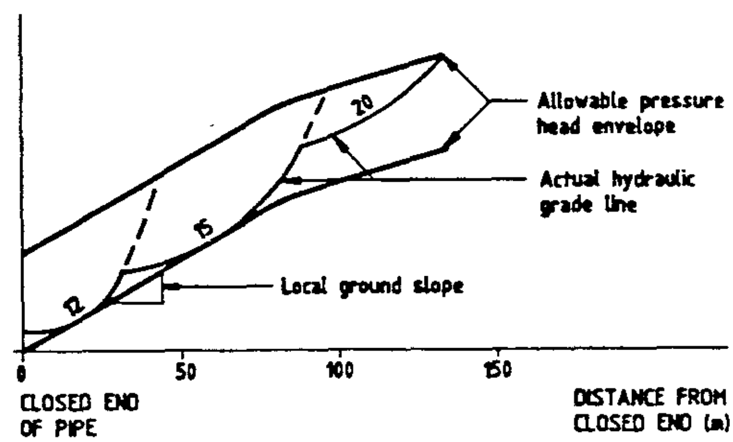


Figure 4.3(b) : Poly-plot design for pipe with SDR = 30*l*/h/m

A limitation of the poly-plot procedure is that the point of change from one diameter to the next is not determined by any fixed algorithm, it is normally done on the basis of the designer's experience. After a lot of repetitive use of the curves, experienced designers develop a good "*feel*" for the most economical policy regarding the positioning of the curves within the envelope and the optimum point at which to change diameters. Lateral pipes seldom consist of more than three different diameters, so that the general policy that is adopted is normally :

a) to keep the first two curves (of the smaller diameter pipes) tangential to the bottom envelope; and then

b) to raise the final curve as high as possible without lifting it above the upper limit at any point, thereby minimizing the length used of this diameter.

The abovementioned process is illustrated in the design shown in figure 4.3(b) above.

Pleban *et al* (1984) proposed a more rigorous mathematical optimization procedure for determining a least cost design within the constraints of the allowable pressure envelope. The solution utilizes the method of Legrange Multipliers, and has been programmed for execution by computer. It is however limited to the uniform slope case.

A further limitation of all of the methods described above is that they are based on an assumption of constant discharge from the outlets along the pipe. In other words, the exponent x in equation 4.2 is assumed to be zero. The rationale for this assumption is that if the maximum pressure variation is kept within *acceptable* limits, then the error in assuming constant discharge will be small. A widely accepted industry standard for the allowable pressure variation is 20%. This is based on a typical x exponent of 0.5, which, when applied in equation 4.2, will result in a discharge variation of approximately 10%. The extent of the possible error due to this assumption is in fact very small, as shown in chapter 7 using results from the computerized design procedure described below.

4.3.2 The proposed computer based algorithm for pipe sizing.

The proposed computerized solution procedure which has been developed as part of this research is based directly on the poly-plot algorithm; with the exception that the pipe hydraulics are calculated on a step by step basis incorporating calculation of the varying discharge along the length of the pipe, rather than assuming a constant SDR.

Development of the algorithm for the computer programs entailed formalizing the rules governing the graphical poly-plot process. In other words, the conditions for shifting a headloss curve up or down within the allowed envelope had to be stated mathematically. In addition, an algorithm for establishing the point of change-over from one diameter to another had to be developed.

Computer algorithms for this process were originally proposed by Perold (1979), and these algorithms have been adopted for this research. As experience was gained in using the algorithms, some modifications were developed to overcome specific cases that were found to cause the design process to break down. These modifications related to the adjustments after a shift or change of diameter, and the handling of the cycling condition. In addition, the process was adapted to incorporate a pressure/discharge relationship for each designed lateral in the manifold design process.

The Pascal listing of the computer routines for lateral design is given in Appendix 2a, and the full algorithm is described on the following pages.

Basic hydraulic calculations. The emitters in the pipe are numbered from 1 at the inlet through to l, the last emitter at the closed end of the pipe. The hydraulic calculations are carried out on a step-by-step basis, starting at the last emitter and working back towards the pipe inlet in steps of one emitter at a time. See figure 4.4

The calculations are initiated with an assumed value of the pressure head, H_l, at the last emitter. At each subsequent step the pressure at the current emitter, H_i, is known from the calculations of the previous step. The discharge from the current emitter, q_i, is calculated from equation 4.2; the flow along the pipe in the next section towards the inlet from the current emitter, Q_i, is calculated from the flow along the previous section by :

$$Q_i = Q_{i+1} + q_i \tag{4.3}$$

and finally the headloss due to friction, δHl_i, in the next pipe section is calculated using equation 4.1.

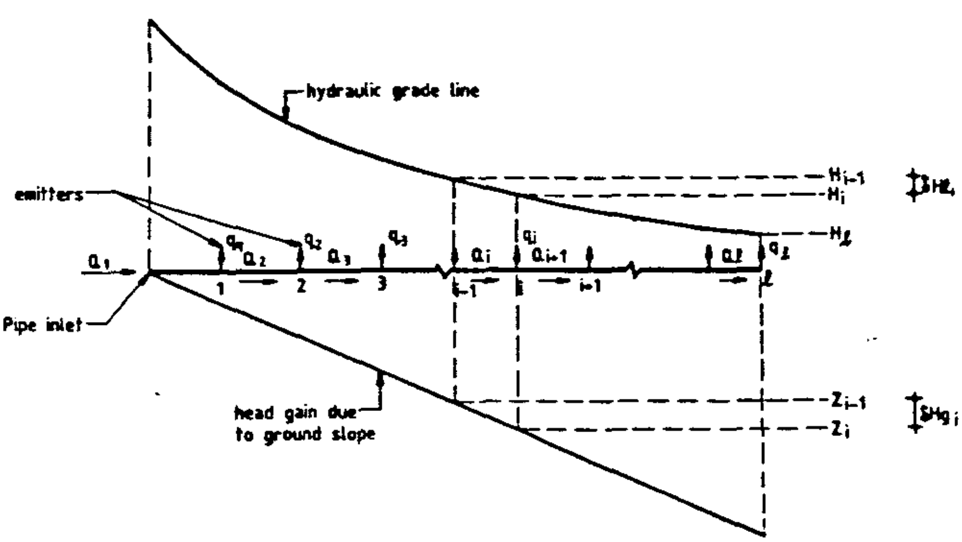


Figure 4.4: Elements in the basic hydraulic calculation procedure of the computer design algorithm

The change in elevation between the current emitter, z_i, and the next emitter, z_{i-1}, gives the loss or gain of head due to slope :

$$\delta Hg_i = z_{i-1} - z_i \tag{4.4}$$

over section i. And hence the pressure at the next emitter is given by :

$$H_{i-1} = H_i + \delta Hl_i - \delta Hg_i \tag{4.5}$$

In this way, the headloss curve along the length of the pipe is gradually established, from the closed end back towards the inlet. At certain points during this procedure the curve must be shifted vertically, in the same way as is done in the poly-plot process.

Shift routine. A vertical shift of the headloss curve is carried out in order to move the curve to a position where it is tangential to one of the envelope boundaries, thereby ensuring that it can continue through as far as possible before a change in diameter is needed. This is illustrated in figure 4.5, where :

* curve 1 is in the initial position,
* curve 2 shows the result of a downwards shift, and
* curve 3 shows the result of a subsequent upwards shift.

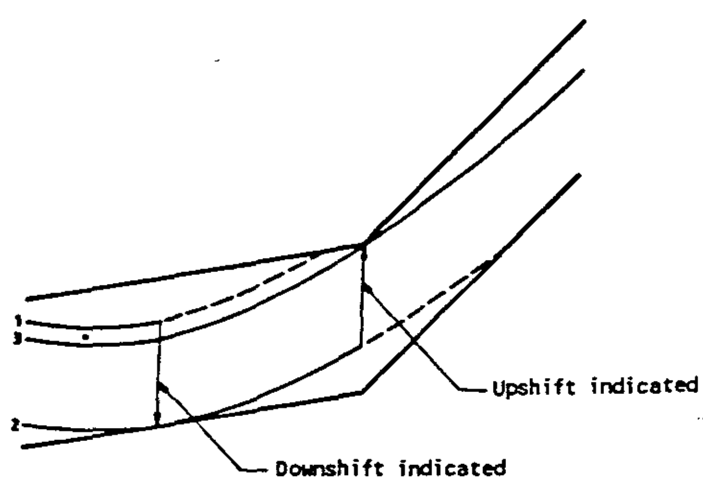


Figure 4.5: The basic shift procedure

The point of tangency is determined by the difference between δHl_i and δHg_i. For example, if the headloss due to friction at emitter i is less than the head gain due to the slope (i.e. $\delta Hl_i < \delta Hg_i$), then the headloss curve is moving towards the lower boundary. As the flow in the pipe increases, the headloss due to friction will increase. At a certain point it will be equal to the head gain due to the slope and thereafter will exceed it. At the point where $\delta Hl_i = \delta Hg_i$, the headloss curve is parallel to the pressure envelope. If the headloss curve is **shifted down,** it will touch the lower envelope boundary tangentially at this point, as shown in figure 4.5 for the shift from curve 1 to curve 2.

Similarly, if the curve is moving up towards the upper boundary ($\delta Hl_i > \delta Hg_i$) and there is a sudden sharp increase in the ground slope such that beyond this point $\delta Hg_i > \delta Hl_i$, then if the curve is **shifted up** it will touch the upper envelope boundary tangentially at the point of the sharp change in the ground slope. This is shown in figure 4.5 for the shift from curve 2 to curve 3.

In the computer algorithm, the amount of any shift is constrained by the fact that the starting point of a curve (i.e. the point nearest the closed end of the pipe) that is being shifted must coincide with the position of an emitter, since the hydraulic calculations are carried out in steps of one emitter spacing at a time. If the curve being shifted is not the first curve, in other words it does not start at the closed end of the pipe, then the pressure at the starting point of the curve is determined by the pressure at the end of the previous curve. This means that the starting point of any curve can only be shifted to the emitter on the previous curve that results in the shifted curve being still within the envelope and as close as possible, but not necessarily exactly tangential, to the envelope boundary. This effect can be seen in figure 4.6 and is discussed further below.

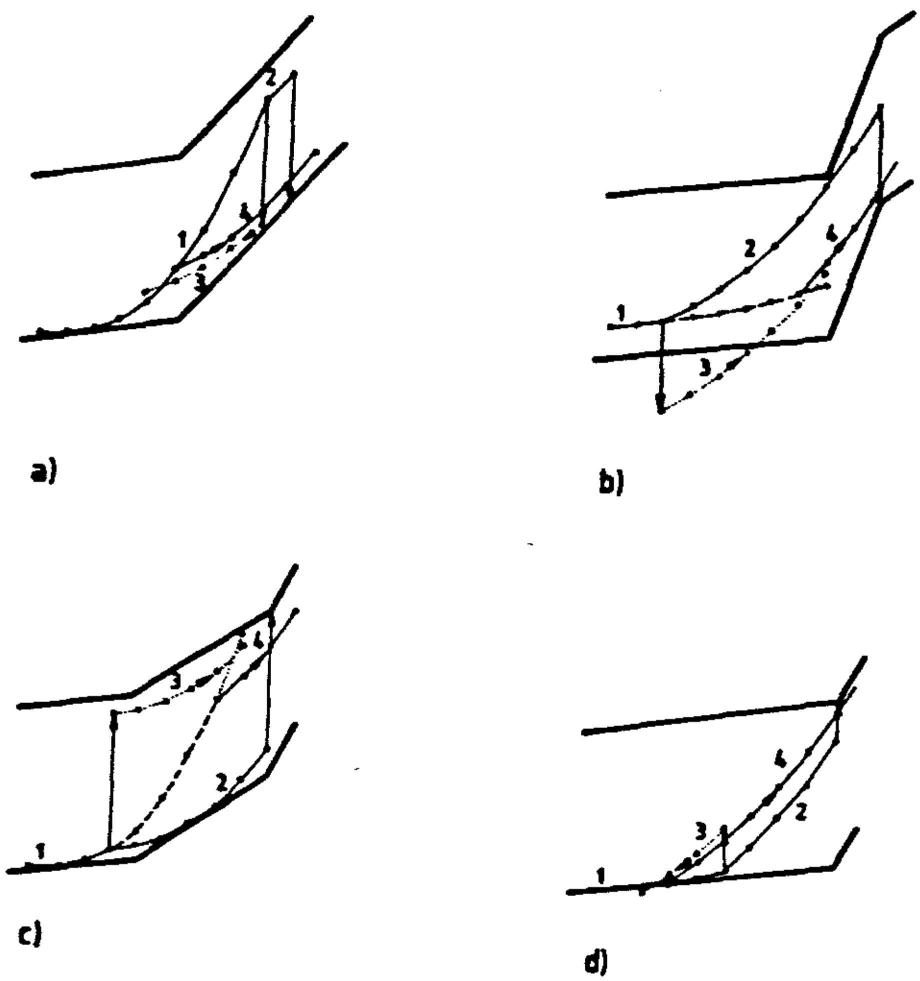
Perold identified two so called "*modes*" of operation, defined by the envelope boundary (upper or lower) to which the current headloss curve will be shifted, and four different shift situations as shown in figure 4.6 :

a) **Calculations in the down mode and the current diameter larger than the previous one.** In this case, the amount of the required shift is indicated by the arrow at the end of curve 2; the starting point of curve 2 is shifted down by this amount and then the curve is calculated in the **reverse direction** (curve 3) until it intersects curve 1; the starting point becomes the first emitter **after** the point of intersection, and curve 4 is calculated on from this point.

b) **Calculations in the down mode and the current diameter smaller than the previous one.** As in the previous case, the amount of the down shift is indicated by the arrow at the end of curve 2; the starting point of curve 2 is shifted down by this amount and in this case the curve is calculated forwards (curve 3) to the point of intersection of an **extension** of curve 1; the starting point is taken as the first emitter **before** the point of intersection, and curve 4 is calculated on from this point.

c) **Calculations in the up mode and the current diameter larger than the previous one.** This is similar to case b, except that the shift is up instead of down.

d) **Calculations in the up mode and the current diameter smaller than the previous one.** This is similar to case a, except that the shift is up instead of down.



a) Down mode, larger diameter
b) Down mode, smaller diameter
c) Up mode, larger diameter
d) Up mode, smaller diameter

Figure 4.6 : The shift cases (after Perold 1979)

Change diameter routine. The design routine is initiated with the smallest available diameter. Two specific cases for the subsequent changes of diameter can be identified :

a) **Change to larger diameter.** This occurs when the calculations are in the **up** mode and the headloss curve reaches the upper boundary before a shift up is indicated.

b) **Change to smaller diameter.** This occurs when the calculations are in the **down** mode and the headloss curve reaches the lower boundary before a shift down is indicated.

Once again the constraint relating to the calculations being carried out in steps of one emitter at a time applies to the point of change of diameter. It will always be at the emitter which is closest to the envelope boundary on the previous curve.

Change of calculating mode. The design process is initiated with a value of H_l which is assumed to be equal to the maximum allowed pressure. In other words, the first headloss curve starts from the upper boundary, at the last emitter. The calculations begin in the down mode.

After a down shift, the calculating mode is set to up; similarly after an up shift the mode reverts to down.

After a change to a larger diameter the calculations continue in the down mode; after a change to a smaller diameter the calculations continue in the up mode.

Adjustments. Three distinct sets of adjustments may be required at various stages during the design process :

Firstly, after any vertical shift of the headloss curve the new curve will not be exactly parallel to the previous curve because the discharge from each emitter is dependent on the pressure (eq. 4.2). Since the pressure at each emitter is either increased or decreased after a shift, the discharge from each emitter will change accordingly and hence the flow in the pipe and the headloss due to friction will also be different in each section. In the up shift case the increased pressures will result in greater headlosses and hence a steeper headloss curve than before; in the down shift case the reduced pressures will result in lower headlosses and hence a flatter headloss curve than before. This means that in both cases the amount of the shift should have been reduced to account for this effect. The extent of this effect is directly proportional to the size of the shift and the value of the x coefficient in the emitter pressure/discharge relationship (eq. 4.2). Thus whenever a shift is indicated, the size of the shift is adjusted by a factor, F, given by :

$$F = [1 - (x * shift\_amount/allowed\_pressure\_variation)] \tag{4.6}$$

This is an arbitrarily determined correction factor, but it is regulated by the second adjustment case discussed below.

Secondly if the amount of a shift has still been overestimated (even after application of the first adjustment factor), then the curve may move outside of the envelope boundary before getting to the previously determined point of tangency. Also, after a diameter change the new curve may move immediately outside of the envelope boundary at which the change was made before getting a chance to be shifted. In either of these cases, the starting point of the affected curve is adjusted one position forward or back, depending on the case as defined in figure 4.6, and the calculations are re-started from this new point.

The third adjustment is made on the final curve after the pipe inlet has been reached. If the pressure at the inlet is less than the maximum allowable pressure, then the final curve is shifted up so that the inlet pressure is as close as possible to the maximum. This reduces the length of the last diameter and increases the length of the previous smaller and hence cheaper diameter.

Minimum length. It is not practical to use short lengths of several different diameters for a lateral or manifold. Designers generally have a minimum practical length for any section, determined mainly through experience. The algorithm includes provision for this constraint. Whenever a change of diameter is indicated, the length of the previous section is checked. If it is less than the specified minimum, then it is discarded and the new diameter is moved across to start at the starting point of the previous diameter.

A similar check is done after a shift up or down, and after an adjustment, since these may have reduced the length of a previous section to below the minimum.

Cycling. In the case of a very narrow tolerance band, particularly for manifold design, the curve may get into a repetitive loop whereby: a change to a larger diameter is indicated but this takes the curve immediately below the lower boundary; and changing back to the smaller diameter takes the curve above the upper boundary, as shown in figure 4.7. In this case the width of the envelope is temporarily increased by 5% until the curve has moved beyond this point.

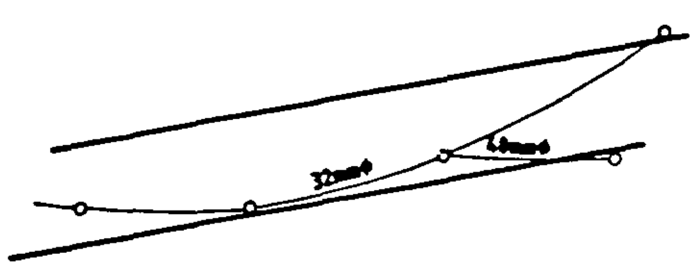


Figure 4.7: The cycling condition (Perold 1979)

4.3.3 The overall computer based solution procedure.

The overall block design procedure incorporates the diameter design procedure as detailed above, together with the following additional algorithms that enable the complete design of the block :

The pressure envelope. The maximum and minimum allowable pressures within the block are normally expressed in terms of a percentage allowable tolerance around the nominal design pressure. The computer algorithm allows the designer to specify the tolerance above and below the nominal value as two separate design parameters.

A second decision is required to establish the split of the total allowable headloss between the manifold and the laterals. This will vary from case to case and depends mainly on the slopes of the respective pipes. The manifold is most often on a steeper slope than the laterals and this slope can therefore compensate for headloss to a greater extent in the manifold than in the laterals. For this reason the split is normally set to around 20% / 80% for manifolds and laterals respectively; based purely on experience. The computer algorithm allows this split to be specified as a separate design parameter.

Lateral vs. manifold design. The design process described in section 4.3.2 above relates principally to the laterals, where the variation in discharge with pressure at the emitters is given by equation 4.2. In the case of the manifold, the design process is a similar one with the laterals taking the place of the emitters along the pipe length. However, the discharge/pressure relationship for each lateral will vary with the lateral length and the type of emitters. An equivalent relationship to that of equation 4.2 cannot be established *a priori*. The computer algorithm deals with this problem in the following way :

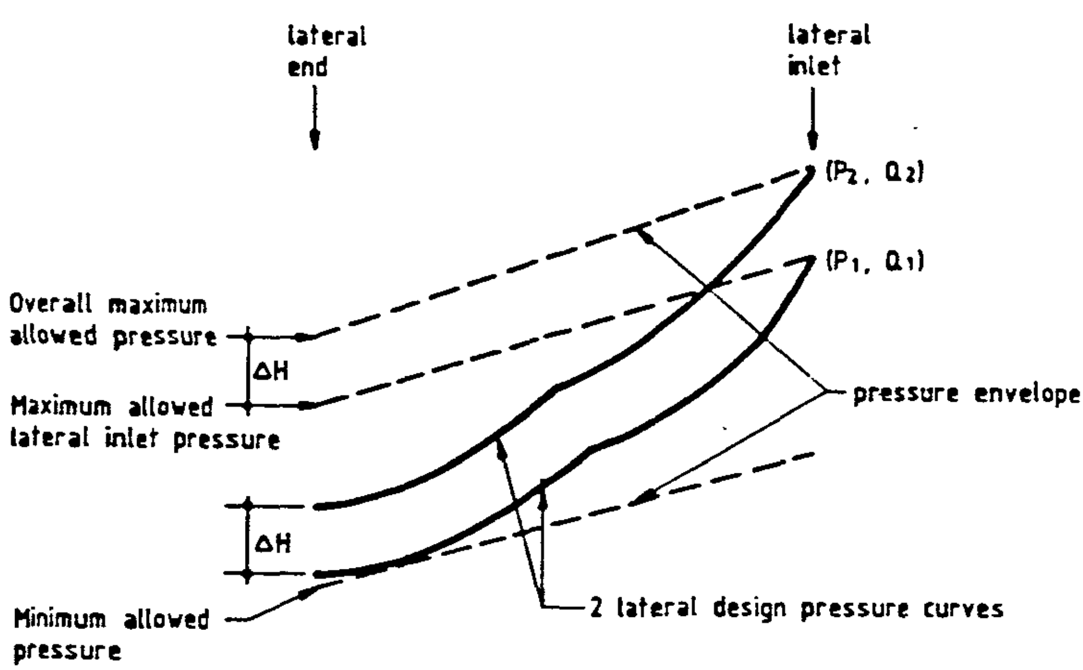


Figure 4.8: Calculation of the lateral pressure/discharge relationship

* After the design has been done, two sets of headloss curves are calculated for each lateral: the **first** with the lateral inlet pressure set to the allowed maximum, taking into account the split of headloss between manifold and laterals; the **second** with the inlet pressure set to the overall maximum allowable pressure (i.e. the likely **manifold** inlet pressure). These two curves are shown in figure 4.8.
* The two sets of values of lateral inlet pressure and discharge (P_1, Q_1 and P_2, Q_2) are then used to calibrate a linear pressure/discharge relationship for each lateral. In fact, the actual relationship will be closer to a power function because of the power relationship which applies to the emitters. However, it is impractical to calculate sufficient sets of values to calibrate a power function and the error in assuming a linear function over the small range between the maximum and minimum pressures will be small.

This algorithm means that all the laterals have to be designed first, before the manifold can be designed.

Extrapolation. Experience has shown that when installing a designed system it is impractical to have completely differing lengths of the various diameters for each lateral. Designers normally design two or three laterals in a block and then draw a straight line from the point of diameter change on the first designed lateral to the equivalent point on the second designed lateral, and so on, as shown in figure 4.9. In this way the lengths of each diameter for the pipes in-between the designed laterals are determined by the values for the designed laterals. When installing the system, the contractor or the farmer himself need only measure out the extreme laterals. He can then join up the respective points of diameter change with a piece of string, and the intersection between the string and the line of each in-between lateral determines the point of diameter change in each case.

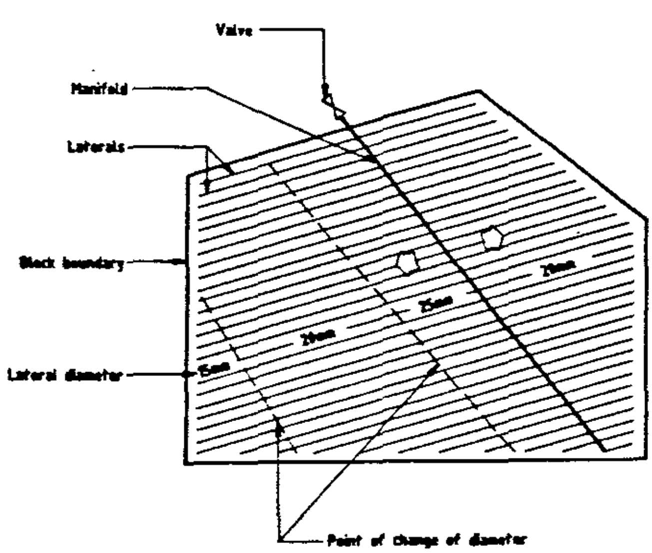


Figure 4.9 : Block design showing extrapolation of lateral diameter changes

The computer algorithm allows for this by incorporating an **extrapolate** function. The designer specifies any two extreme laterals that have already been designed and then the computer extrapolates the lengths of each diameter for the intermediate laterals. The extrapolation is done linearly between lengths of like diameters. This means that if the length of any particular diameter is greater than zero on one of the laterals and equal to zero on the other one, then the lengths of this diameter on the intermediate laterals will gradually decrease down to zero.

4.4 The Computer Programs

The computer models for block design have been written in Turbo-Pascal (Borland Int. 1985) for MS-DOS (Microsoft Inc. 1983) based personal computers. They incorporate a number of different functions that have been integrated into a single package, as shown in Appendix 1a. The package is menu driven and consists of four different modules, *viz* :

* **data base maintenance;**
* **the layout module;**
* **the design module; and**
* **the evaluation module.**

The first three of these modules are discussed in sections 4.4.1, 4.4.2 and 4.4.3 respectively. The evaluation module is discussed in chapter 5.

In addition, four *utility* functions have been programmed to operate as so called **pop-up** windows. These are windows that can be popped onto the screen temporarily over any current screen and then popped down, leaving the screen as it was previously. These utilities include :

* **Pipe data base look-up table;**
* **Emitter data base look-up tables;**
* a program **help facility**; and
* a **maximum length calculator.**

These utilities are discussed further in section 4.4.4 below.

4.4.1 Data base maintenance.

Maintenance of the data base is carried out via on-screen editing of the input tables for both the pipes and the emitters.

Pipes. The pipe data base consists of two different tables :

* A general table listing :
  - all the available pipe materials;
  - the available classes for each material; and
  - the values of the four parameters in the headloss formula (eq. 4.1) for each material.
* A set of tables, one for each pipe material, showing :
  - the nominal diameters that are available;

  And for each available class for each of these nominal values :
  - the internal diameter; and
  - current list price per meter length.

Emitters. The emitter data base consists of a single "*page*" display for each emitter, as shown in the lower right corner of figure 4.14. The user can page through this data base using the "Pg Up" and "Pg Dn" keys on the computer keyboard. The display for each emitter shows :

- an identification code;
- the manufacturer;
- the emitter name;
- its size;
- the k and x coefficient values for the pressure/discharge equation (eq. 4.2);
- the list price of the emitter; and finally
- five sets of pressure/discharge operating points taken from information supplied by the manufacturers.

Functions at the bottom of the emitter display page enable the user to :

* calibrate the k and x values from a minimum of three operating points;
* calculate the pressure value from any given discharge, or vice versa, using the calibrated k and x values;
* store the emitter information in the data base; and
* delete the currently displayed emitter from the data base.

4.4.2 Layout.

The layout module enables the user to specify the details about the system alignments that are required for the actual design. These include :

* the emitter and lateral spacings;
* the lengths of each lateral and the manifold; and
* the elevations of the pipes.

Provision is made for the possible layout of two laterals per row ("*tramlines*") and for the possible installation of two emitters per tree in orchards.

<Proj - 1> * MBB 30/09/86

LATERAL DESIGN (lengths / elevations / pressure heads in meters)

<Quad a> <Quad b> <Quad c> <Quad d>

Lat	15(mm)		20(mm)		25(mm)		32(mm)		40(mm)		Totals		
#	Len	#Em	Len	#Em	Len	#Em	Len	#Em	Len	#Em	Len	#Em	Cost
1	24	8	10	3							34	11	25.64
2	27	9	10	3							37	12	26.73
3	30	10	10	3							40	13	27.82
4											00	00	00.00
5											00	00	00.00
6											00	00	00.00
7											00	00	00.00
8											00	00	00.00
	81		30		00		00		00		111		R80.19

[* 3/ 20]

Figure 4.10 : Table of results of the lateral design process

4.4.3 Design.

This module controls the pipe designs, based on the algorithms described in section 4.3 above. The module is presented on two main display pages, one for lateral design and one for manifold design.

The lateral design page is shown in figure 4.10 above. It shows a window of up to eight laterals at a time, for each of which a table of the lengths (Len) and equivalent number of emitters (#Em) of up to five different diameters is shown. The three rightmost columns of the table show totals of the length, number of emitters and cost of each lateral.

The *lateral design* process is operated from a pop-up menu which presents the following functions :

* **Select a quadrant.** Any block may be divided into up to four quadrants, depending on the locations of the valve and manifold within the block. This function enables the designer to specify which quadrant he is working in at any one time.

* **Design parameters.** Selection of this function presents a **pop-up** window on top of the design page, in which the user specifies the following parameters :
  - the **emitter**, extracted from the data base;
  - the **nominal operating pressure**, in which case the nominal operating discharge will be displayed, or vice versa;
  - the **upper percentage tolerance on the nominal pressure**, in which case the maximum allowable pressure will be displayed, or vice versa;
  - the **lower percentage tolerance on the nominal pressure**, in which case the minimum allowable pressure will be displayed, or vise versa;
  - the **maximum lateral inlet pressure**, which establishes the split of the allowable headloss between the laterals and the manifold;
  - the **lateral pipe material, class, minimum diameter and minimum length per diameter**;
  - the **manifold pipe material, class, minimum diameter and minimum length per diameter**; and finally
  - the **discount** on the list prices stored in the data base that the designer expects to be able to obtain.

* **Edit diameters.** The designer will have specified a minimum allowable diameter in the list of design parameters discussed above. The computer then selects the minimum diameter plus the next four diameters in the data base for use in the design process. These are displayed at the top of the display table, as shown in figure 4.10. The user is however free to select the **edit diameters** function which enables him to change any of the five diameters to alternative values, if so desired.

* **Edit table.** This function enables the designer to move the cursor into the main display table and make manual changes to any of the lateral designs. Any change that is made to the designed values of a lateral will result in an automatic adjustment of the other values for the changed lateral, such that the total length remains correct. All the other displayed totals are also automatically updated.

  Whilst operating in this function, the user is able to scroll the display window up or down in order to display any particular lateral in the set that was defined in the layout phase.

<Proj - 1> * MBB 30/09/86

LATERAL DESIGN (lengths / elevations / pressure heads in meters)

<Quad a> <Quad b> <Quad c> <Quad d>

Lat	15(mm)		20(mm)		25(mm)		32(mm)		40(mm)		Totals		
#	Len	#Em	Len	#Em	Len	#Em	Len	#Em	Len	#Em	Len	#Em	Cost
1	24	8	10	3							34	11	25.64
2	27	9	10	3							37	12	26.73
3	30	10	10	3							40	13	27.82
4											00	00	00.00
5											00	00	00.00
6											00	00	00.00
7											00	00	00.00
8											00	00	00.00
	81		30		00		00		00		111		R80.19

[* 3/ 20]

Lat No. 3 <From 0 Elev 96.5> <To 13 Elev 94.2> <Slope 5.8%>

End H(m)	Min H(m)	Max H(m)	Inlet H(m)	Inlet Q(m3/h)
<12.3 - 12.7>	<10.4 - 11.0>	<13.6 - 14.1>	<13.6 - 14.1>	<1.245 - 1.546>

<Calc> <Design>

Figure 4.11 : Table showing design results for lateral 3

* **Design.** On selecting the design function, an input box appears below the main table (see figure 4.11). The designer specifies the lateral number that is to be designed. He can then either calculate the hydraulic characteristics (pressure and flow distribution in the lateral) for a prespecified set of diameter lengths, or design the lateral from scratch.

  The results of the design are displayed in three different forms :

  a) A summary of the flows and pressures is given in the design box (figure 4.11) for the two design curves, as discussed in section 4.3.3. This includes :
  - the pressure at the end of the lateral;
  - the minimum pressure in the lateral;
  - the maximum pressure in the lateral;
  - the pressure at the lateral inlet; and
  - the flow requirement at the lateral inlet.

b) A synopsis of the shifts, diameter changes and adjustments carried out in each design operation is written to a text file and can be displayed on the screen or printed out. An example of this output is shown in figure 4.12. At each step, information is given on the hydraulic calculations as follows :

- the current section number and diameter;
- the starting point of the current section and the pressure at that point;
- the current point of the calculations and the pressure at that point;
- the next point and pressure, where applicable;
- the action taken;
- the tolerance on the pressure envelope;

This enables the designer to follow the poly-plot procedure through the various stages in its execution.

DESIGN OF LATERAL 2 IN QUAD a Date : 28/01/87

STEP #	SECT	DIA(mm)	Start Pt & P		Current Pt & P		NEXT Pt & P		ACTION	TOLERANCE
1	1	12	21	157	14	150	13	150	Shft Dn	(0.00)
2	1	12	21	144	6	153	5	158	Chng Up	(0.00)
3	2	15	6	153					ChkMinLen	(0.00)
4	2	15	6	153	5	153	4	153	Shft Dn	(0.00)
5	2	15	10	140					ChkMinLen	(0.00)
6	2	15	10	140	0	139			End Shift Up	(0.00)
7	2	15	6	153	0	155			Design Done	

Figure 4.12 : Example of the design synopsis

c) A graphical display of the final headloss curves as shown in figure 4.13 can be obtained using the **view** function described below.

* **View.** This is the function which the user selects in order to get the graphical presentation of the headloss curves, as shown in figure 4.13. The display shows :
  - the allowable pressure envelope running from the closed end of the lateral on the left to the inlet end on the right, with the elevation of the lower boundary given on the left axis of the graph;
  - the two headloss curves for the designed lateral, with the actual pressure values given on the right axis;
  - a summary table of the designed lengths of each of the five diameters.

b) A synopsis of the shifts, diameter changes and adjustments carried out in each design operation is written to a text file and can be displayed on the screen or printed out. An example of this output is shown in figure 4.12. At each step, information is given on the hydraulic calculations as follows :

- the current section number and diameter;
- the starting point of the current section and the pressure at that point;
- the current point of the calculations and the pressure at that point;
- the next point and pressure, where applicable;
- the action taken;
- the tolerance on the pressure envelope;

This enables the designer to follow the poly-plot procedure through the various stages in its execution.

DESIGN OF LATERAL 2 IN QUAD a Date : 28/01/87

STEP #	SECT	DIA(mm)	Start Pt & P		Current Pt & P		NEXT Pt & P		ACTION	TOLERANCE
1	1	12	21	157	14	150	13	150	Shft Dn	(0.00)
2	1	12	21	144	6	153	5	158	Chng Up	(0.00)
3	2	15	6	153					ChkMinLen	(0.00)
4	2	15	6	153	5	153	4	153	Shft Dn	(0.00)
5	2	15	10	140					ChkMinLen	(0.00)
6	2	15	10	140	0	139			End Shift Up	(0.00)
7	2	15	6	153	0	155			Design Done	

Figure 4.12 : Example of the design synopsis

c) A graphical display of the final headloss curves as shown in figure 4.13 can be obtained using the **view** function described below.

* **View.** This is the function which the user selects in order to get the graphical presentation of the headloss curves, as shown in figure 4.13. The display shows :
  - the allowable pressure envelope running from the closed end of the lateral on the left to the inlet end on the right, with the elevation of the lower boundary given on the left axis of the graph;
  - the two headloss curves for the designed lateral, with the actual pressure values given on the right axis;
  - a summary table of the designed lengths of each of the five diameters.

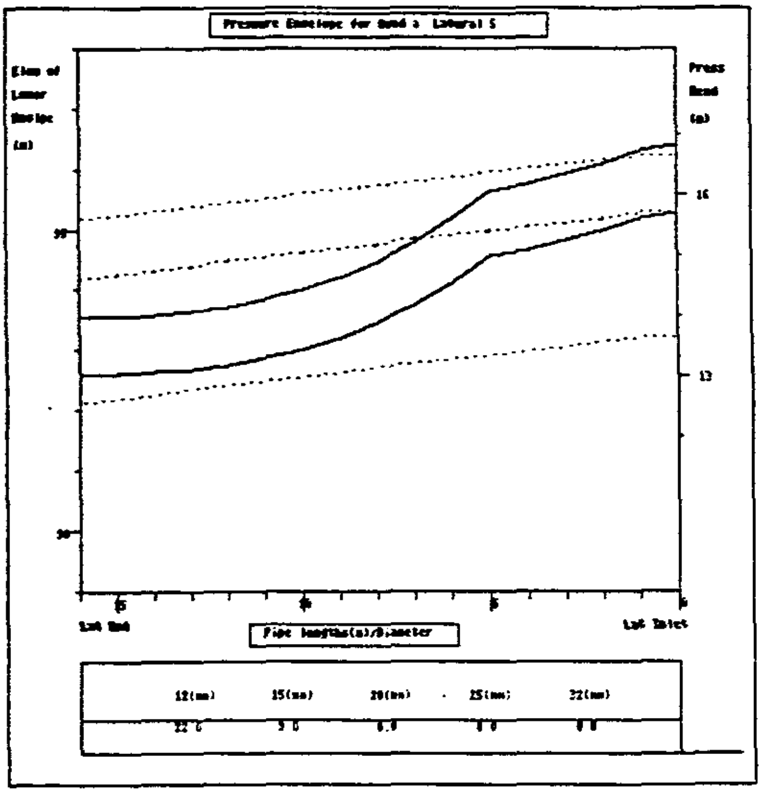


Figure 4.13 : Graphical plot of the pressure profile resulting from the lateral design process

* **Extrapolate.** Selection of this function enables the designer to extrapolate the design of a set of laterals using the results of two previously designed laterals, as discussed in section 4.3.3.

The manifold design programs are similar to those for lateral design, the only differences being the display of up to ten different diameters for the manifold and the absence of an extrapolate function.

4.4.4 The pop-up utilities.

The four pop-up utilities are accessed from function keys on the computer keyboard, as follows :

* **F1-Help.** This is an on line help facility which gives several pages of information and assistance in working the program. On selecting this function, the page which discusses the current activity on the screen will be popped up. The user can then scroll through the other pages using the "**Pg Up**" and "**Pg Dn**" keys.

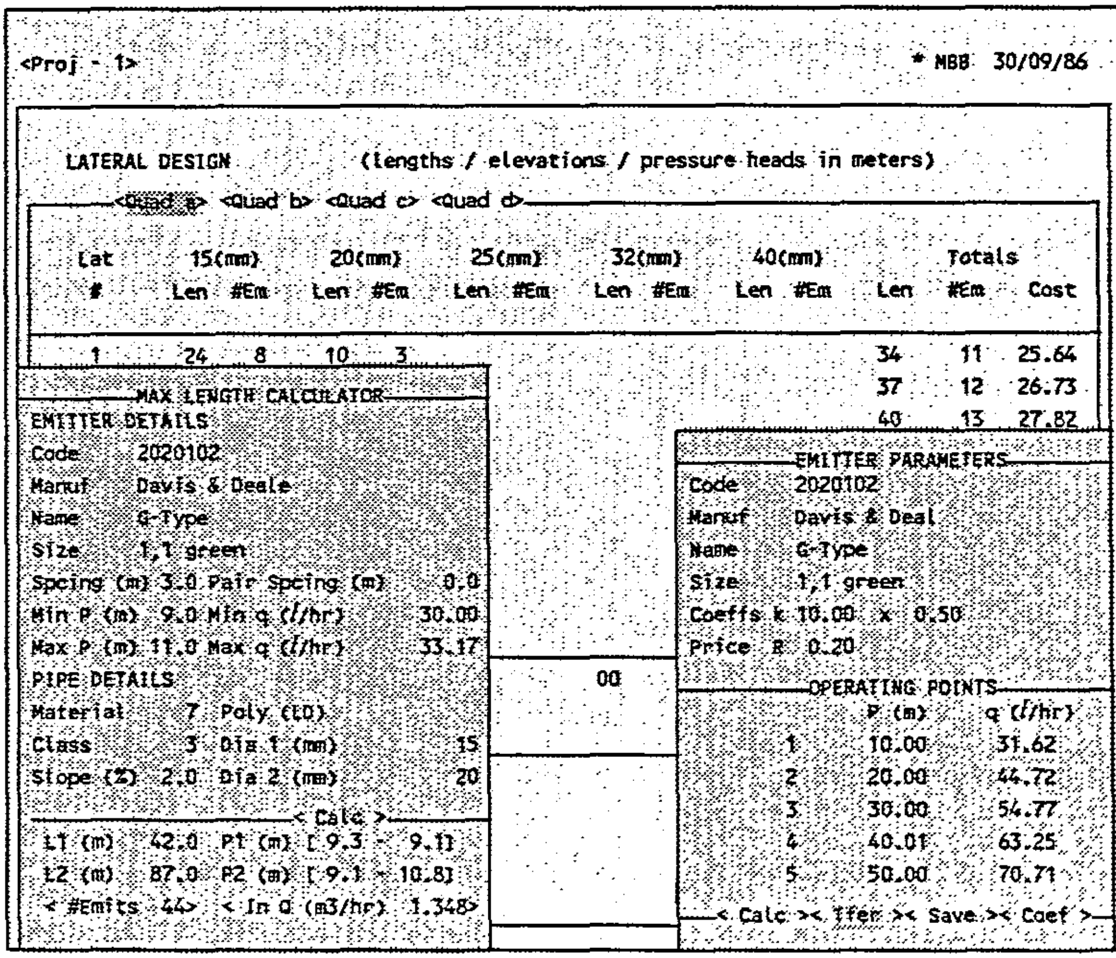


Figure 4.14 : The maximum length calculator and emitter database "pop-ups"

* **F5-Maximum length calculator.** This utility provides a quick calculation of the maximum length that can be installed of a one or two diameter pipe on a given slope, with given emitter, spacing and allowed pressure variation. An example of this screen is shown overlaying the lateral design screen in the lower left corner of figure 4.14. The results of a calculation show the lengths of the two diameters; the starting and end pressure in each section; the total number of emitters in the pipe; and the flow required at the inlet to the pipe.

* **F6-Pipe data base look-up.** This utility enables the user to page through the pipe data base as described in section 4.4.1 above.

* **F7-Emitter data base look-up.** This utility enables the user to page through the emitter data base as described in section 4.4.1 above. An example of the display is shown overlaying the lateral design screen simultaneously with the maximum length calculator in figure 4.14.

Both the emitter and pipe pop-up tables have a transfer facility, which enables the designer to select a required component and then transfer this selection to either the maximum length calculator or the table of design parameters.

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5. EVALUATION OF BLOCK DESIGN

5.1 Introduction

Evaluation is carried out once the design has been completed, in order to assess the expected performance of the irrigation system. The aim of the evaluation process is to give the designer some measure of the quality of the design, in order to :

a) Consider possible improvements through changing one or more of the values of the various design parameters; and

b) Provide the farmer with an assessment of the expected return on his investment.

During the development of the systems analysis of the design process (chapter 2) the evaluation was initially considered as a separate module, coming after completion of all the blocks and the mainline design. However it was found that this rendered the process inflexible when working with individual blocks within a large system. In order for the designer to evaluate and then modify a single block it would be necessary to first complete the initial design of all the other blocks and the mainline. In addition, testing of any changes made to a single block would entail re-evaluation of the whole system.

Therefore, since the mainline evaluation can readily be separated from the block evaluation, it was decided to incorporate the block evaluation model as the last step in the block design process, to be applied as an integral part of the design of each block. This means that if the mainline has not yet been designed then some estimates have to be made of the costs and operating characteristics of the mainline in order to carry out the block evaluation process. If these estimates prove to be too inaccurate, then an iterative process between the block and mainline designs may ensue.

The block evaluation model has been formulated principally on the basis of the work that was reviewed in chapter 3. In addition, it also includes further analysis relating to the *level of operation* of the system in the field. Once an irrigation system has been installed, it can be operated to apply any irrigation depth ranging from zero up to the system capacity. However it will not always be most economical to operate the system at full capacity. The evaluation process therefore includes an analysis of the **operating point** of the system, which is defined as the depth of application applied by the system in response to the varying plant requirement during the season. One of the results of the evaluation process is a calculation of the optimum operating point; both an overall seasonal value and values for individual applications during the season.

This chapter presents a detailed analysis of the model structure, together with a review of some additional relevant literature. This is followed by a description of the computer programs.

5.2 Basic Model Structure

The proposed evaluation model is divided into two distinct phases. The first looks at the **water distribution patterns** that result from the designed system in a given block and the second carries out an **economic cost/benefit analysis** of the system. An outline of the basic input and output in each of these analyses is shown in figure 5.1.

5.2.1 Water distribution analysis.

This analysis entails calculation of the parameters listed under the "OUTPUT" column in figure 5.1. Several different procedures for the calculation of these parameters from **field data** are shown in chapter 3. Since it is impractical to measure the discharge of every emitter in an operating system in the field, the calculation procedures are based on assumed statistically determined distribution functions. In the case of the evaluation model however, it is not necessary to assume any mathematical function for the distribution pattern since the discharge at each emitter can be calculated directly. Hence each of the evaluation parameters can be calculated numerically from the emitter discharge values.

The **coefficient of uniformity** is calculated as the Christiansen coefficient, given by equation 3.17 :

$$UCC = 100\,[\,1 - (\,\Sigma|X_i - \bar{X}|)/N\bar{X}\,] \qquad (3.17)$$

Where N = the number of observations X_i.

$\bar{X}$ = the average application depth.

It should be noted that the Christiansen coefficient was originally formulated as a measure of the water distribution in the field, determined on the basis of the variation in spread around each sprinkler (the "**within emitter variation**"). In this case however, it represents a measure of the variation in **discharge** from each emitter (the "**between emitter discharge**" variation). It has been used interchangeably in the literature and has become commonly accepted as representing both of these concepts, even though they have quite different applications.

The **maximum discharge, minimum discharge** and **percentage variation** also provide an indication of the efficiency of the design algorithm. The percentage discharge variation, *Var*, is given by :

$$Var = (q_{max} - q_{min})/q_{av} \tag{5.1}$$

Where q_{max} = the maximum emitter discharge in the system;
q_{min} = the minimum emitter discharge in the system;
q_{av} = the average discharge.

The final parameter that is determined by the distribution analysis is a list of any lateral containing one or more emitters operating at a pressure value that is outside of the limits defined for the pressure envelope. This enables the designer to identify any specific areas in the field which might be inadequately designed, possibly as a result of having been established by using the extrapolate function rather than the actual design function.

After completing the block design, the required pressure at the valve will be known. However for a number of reasons related to the nature of the system and the prevailing topography, the designer may wish to operate the block valve at a different pressure, which will affect the performance of the system. The evaluation model therefore calculates all the abovementioned parameters for a series of different operating pressures. Four sets of values are presented, being respectively 0.8, 1.0, 1.2 and 1.4 times the design value. The designer may also specify any other valve pressure for which he might want to calculate the evaluation parameters.

Comparison of the results for the different operating pressures provides an indication of the sensitivity of the system to either over or under-pressure operation. These effects can be further considered by studying the results of the economic analysis for each different operating pressure. In this way the designer is able to make a rational decision on the most appropriate operating pressures in the system and the extent to which any pressure regulation may be required in the mainline.

5.2.2 Economic cost benefit analysis.

Once the water distribution analysis has been completed the designer must specify a valve operating pressure for which the ensuing economic analysis will be carried out.

This analysis entails estimating the yield that will result from the designed system and then comparing the income resulting from the yield with the various costs involved in generating this yield. The evaluation is carried out on the basis of the economic return, both per unit of cultivated area and per unit of water used. The specific parameters generated by the analysis are shown in figure 5.1.

The main measure of financial return generated by the analysis is the **equivalent annual worth** (EAW) of the project, which is established as follows: A conventional **Nett Present Worth** (NPW) analysis of the system is calculated using different inflation rates for **energy costs**, other **production costs**, and **producer prices** respectively. The EAW is then obtained by converting the NPW value back to an annual figure using a capital recovery calculation with the prevailing **project discount rate**, which reflects the overall cost of money. Thus the EAW value represents a measure in current financial terms of the average annual return that will be obtained from the system over its expected lifespan.

The input required for the model includes :

* the emitter discharge values;
* a breakdown of the expected seasonal plant water requirements;
* the price earned for the produce per unit of yield;
* details of the **capital** costs of the system, which include the cost of the all the block-system elements and the value of the mainline which can be apportioned to the block being evaluated;
* details of the **operating** costs of the irrigation system;
* a breakdown of other non irrigation-related operating costs, separated into yield dependent and yield independent components;
* prevailing rates of inflation for the producer price, energy costs, production costs and the overall market rate; and the project discount rate.

A more detailed mathematical derivation of the model is given in section 5.4 below.

5.2.3 System operation.

In order to carry out the economic analysis, the depth of water to be applied by the system (its "*operating point*") during the season must be known. It has previously been common practice, for the purpose of this type of analysis, to assume that the average application depth is always equal to the expected plant requirement. However, the system can in fact be operated to apply any depth between zero and the maximum design capacity. If the distribution uniformity of the system is low, then it may be desirable to generally over-irrigate in order to ensure that as much of the field as possible obtains the amount of water required for maximum yield. Alternatively, it may be more economical to deliberately under-irrigate in order to reduce operating costs and/or conserve water.

This latter practice, which is known as **deficit irrigation**, has been increasingly considered in recent years. It entails a trade-off of reduced yield and hence reduced earnings for a reduction in the operating costs and a saving in water usage. If the system is deliberately designed for deficit irrigation practice, then its maximum capacity can be reduced and this may also generate a reduction in the capital costs of the system.

The evaluation model includes an analysis of this aspect of the design problem. A dynamic programming algorithm has been developed to determine the optimal seasonal operating point for the designed system, based on the expected water requirement in individual periods during the season. The mathematical derivation of the optimization model is shown in section 5.5 below.

5.2.4 Application of the model

Thus by using the model described above, the designer is able to evaluate a given design in terms of its expected distribution and economic performance characteristics. Through iterative use of the model, it will be possible to generate a three dimensional sensitivity analysis of **EAW vs UCC vs operating point.**

Similarly, the relationships between any of the design parameters and the expected system performance can also be investigated. One of the most commonly considered of these relationships is that between allowed pressure variation and UCC. The results of some of these investigations are shown in chapter 7.

5.3 Yield Estimation

The core of the economic evaluation model is the yield estimation equation that was given in chapter 3 :

$$Y = A_T \int_{i_{min}}^{i_{max}} y(i)\, f(i)\, di \tag{3.31}$$

Where Y = the expected total yield from a field of area A_t;

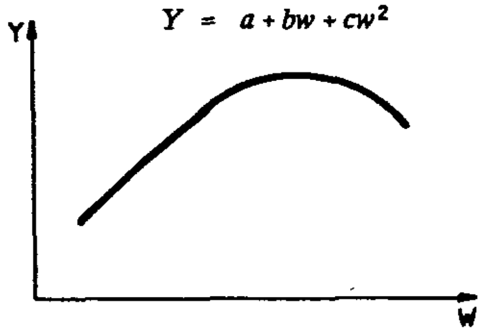
$y(i)$ = the yield per unit area as a function of the irrigation depth, i;

$f(i)$ = the frequency distribution function of irrigation depths

In order to evaluate this integral, both the frequency distribution of irrigation depths function, $f(i)$, and the water/yield function, $y(i)$, must be known. As discussed above, $f(i)$ can be established directly from calculations of the discharges at each emitter. The question of the water/yield function is however a sensitive one since it has not been fully resolved in the literature.

The basis for the formulation of the water yield functions is the relationship between irrigation and plant water deficits on the one hand and actual and maximum crop yields on the other. Several approaches have been used in the quantification of this relationship. The most common of these has been the derivation of purely empirical functions of yield versus applied water, based on experimental results. These functions typically take the form shown in figure 5.2 below :

$$Y = a + bw + cw^2 \tag{5.2}$$



Where Y = crop yield per unit area;
w = water supplied to the plant;
a,b,c = constants.

Figure 5.2 : The general form of experimentally derived water/yield relationships

The value of the constant c in equation 5.2 may vary from zero, implying a linear function, to either positive (curvilinear concave function), or negative (curvilinear convex function) values.

The principal short coming of this approach is the fact that the crop yield is also dependent on numerous other factors apart from the amount of water applied. These include *inter alia* : the crop variety; soil types; prevailing climatic conditions such as wind and temperatures; and prevailing cultural practices such as the fertilization rates. Since these factors are not isolated in the abovementioned models, it is impossible to produce any universal functions.

A second approach that has been used, which attempts to overcome this difficulty, is to investigate the relationship between **relative yield** and **relative plant water use**. The term "*relative*" implies the ratio between **actual** and **potential maximum** yield and water use respectively. Several researchers have published experimental results indicating a strong linear correlation between crop dry-matter yield and cumulative seasonal evapotranspiration.

A review of this research is given by Stegman, *et al* (1980). The most generalized form of this relationship is given by plotting the relative dry-matter yield, Y/Y_{max}, against relative evapotranspiration, ET/ET_{max}. Then for any given situation in which prevailing local conditions and cultural practices enable predetermination of the maximum yield, Y_{max}, and evapotranspiration, ET_{max}, values, the actual yield can be estimated for lower values of ET.

Doorenbos and Kassam (1979) have formalized the relative yield/relative evapotranspiration model as follows :

$$(1 - Y_a/Y_m) = ky\,(1 - ET_a/ET_m) \tag{5.3}$$

Where Y_a = actual yield;

Y_m = maximum yield;

ET_a = actual evapotranspiration;

ET_m = maximum evapotranspiration;

ky = the crop or yield response factor.

This can be expressed in words as follows : the expected yield loss is directly proportional to the evapotranspiration deficit. The function is shown graphically in figure 5.3.

5.3.1 Maximum yield.

The maximum potential yield of a crop is determined by its genetic characteristics and by the prevailing environmental conditions in which it is planted. The climatic factors which affect Y_m are temperature, radiation, day length and length of the total growing season. Most crops are available in a number of varieties, which differ in their climatic requirements for attainment of maximum yield. This enables a given crop to be adapted for high-yielding production under several different sets of growing conditions.

In general, the maximum attainable yield for a given crop, grown under farming conditions, can be estimated from existing local records in the area under consideration. In many cases, local government agencies will have conducted controlled experiments to establish maximum yields and the cultural practices required to attain these yields. Some empirically derived functions for calculating Y_m have been proposed in the literature and are reviewed in Doorenbos and Kasam (1979).

5.3.2 Maximum evapotranspiration.

The rate of evapotranspiration, ET, of a plant is an expression of the crop water requirements needed for unrestricted growth. Maximum evapotranspiration represents the maximum rate of transpiration of a healthy crop grown in a large field under ideal agronomic conditions and in particular with adequate water. Determination of ET_m is an important aspect of the design problem since it is equivalent to the plant water requirement and hence determines a number of factors relating to the capacity of the system. A general function for ET_m is given by :

$$ET_m = k_c ET_o \quad \text{(mm/day)} \tag{5.4}$$

Where ET_o = the reference evapotranspiration;
k_c = a crop factor.

The **reference evapotranspiration**, ET_o, represents the rate of evapotranspiration, under the given climatic conditions, of an extended surface of an 8 to 15 cm tall green grass cover, actively growing with adequate water supply and completely shading the ground. There are several methods for the calculation of ET_o. The "*Penman*" and "*radiation*" methods provide accurate calculation of ET_o from measurements of climatic data including vapour pressure, wind speeds, temperature, sunshine duration and radiation. The more commonly used "*pan evaporation*" method is simple and adequately accurate for most farming requirements. By this method, ET_o is calculated from :

$$ET_o = k_p E_p \quad \text{(mm/day)} \tag{5.5}$$

Where E_p = the evaporation from an unscreened class A evaporation pan (mm/day);
k_p = the pan coefficient which can be obtained from reference tables (see Doorenbos and Kassam) for given estimations of the prevailing wind, relative humidity and surrounding ground cover conditions.

The **crop factor**, k_c, varies with the stage of growth of the plant and is dependent on the humidity and wind conditions under which the crop is being grown. General values for most crops are readily available in a number of references (Doorenbos and Kassam) and more specific values for local conditions are usually published by government agencies.

5.3.3 Actual evapotranspiration.

The crop water demand, ET_m, must be met by the available water in the soil, via the root system of the plant. The actual rate of water uptake by the plant plus evaporation, ET_a, is determined by the level of **available water** in the soil.

The amount of water that can be stored in the soil and then extracted by the plant is dependent principally on the physical properties of the soil and also on the type of crop under consideration. A model that has been widely accepted for estimating ET_a is as follows :

Given the upper and lower limits of soil moisture availability, **field capacity** and **permanent wilting point** respectively, there is a *"critical point"* between these two levels at which the rate of evapotranspiration is affected. The position of this point in relation to the two limits differs with crop type. If the moisture content of the soil is maintained above the critical point then the actual rate of evapotranspiration is equal to the maximum rate, $ET_a = ET_m$. However as soon as the soil moisture content falls below the critical point then ET_a begins to slow down. It reduces linearly with the reduction of moisture content, down to zero at the permanent wilting point.

Thus the actual evapotranspiration can be calculated from measurements of the soil moisture content.

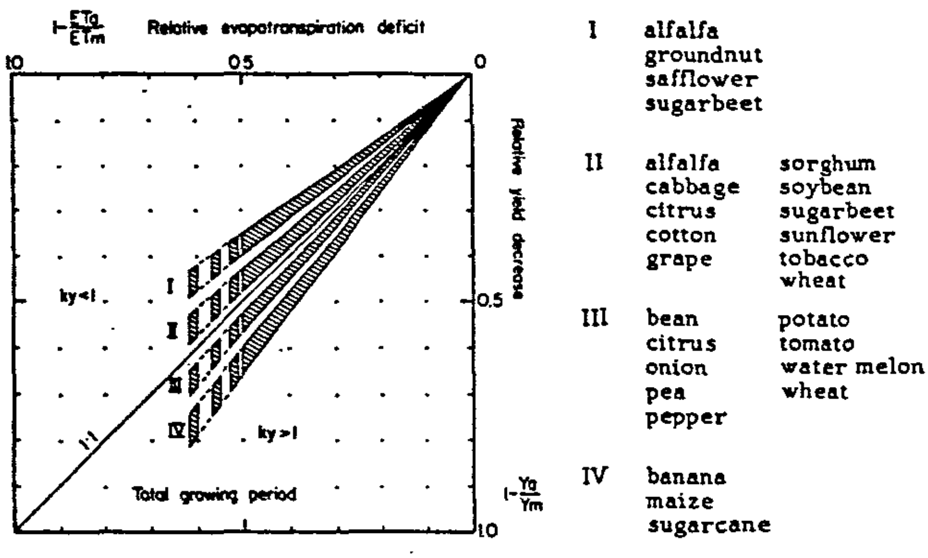
5.3.4 Yield response factor.

The yield response factor, *ky*, defined by the water/yield function (equation 5.3) is an expression of the sensitivity of a given crop to water deficits. The larger the value of *ky*, the larger will be the loss of yield due to a given water deficit.

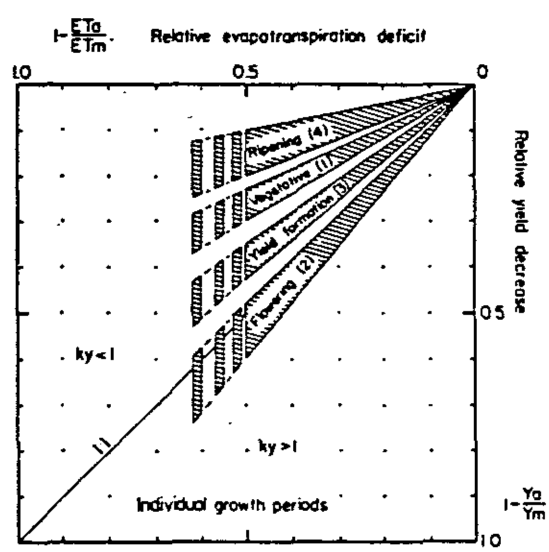
In an analysis of a large volume of experimental data, Doorenbos and Kassam found that representative *ky* values could be expressed for a large number of crops. They reported values both for the total growing season, representing the overall yield response to seasonal water deficits, and for individual growth periods within the season, representing the sensitivity of a given crop to water deficits occurring within the individual periods during its growth and maturation. The individual periods considered were: vegetative growth; flowering; yield formation; and ripening.

Figure 5.3 shows a graphical representation of the water/yield model. The first figure (5.3a) shows four groups of crops having different *ky* values for the total growing period, as classified by Doorenbos and Kassam, and the second figure (5.3b) shows typical *ky* values as they vary during the growing season. It can be seen from this latter figure that plants are most sensitive to water deficits during the flowering period.

The *ky* values reported by Doorenbos and Kassam were based on experimental results for high-producing crop varieties, well adapted to the growing environment and grown under a high level of crop management. There will naturally always be variations in the *ky* values due to peculiarities in the prevailing agro-technical conditions. There is considerable experimental evidence that the *ky* value will also vary with the type of irrigation method being used and the irrigation regime.



a) Total growing period



b) Individual growing periods

Figure 5.3 : Diagramatic representation of the relative yield/relative evapotranspiration model (Doorenbos and Kassam 1979)

Young *et al* (1985) found that whereas Doorenbos and Kassam reported *ky* values of 1.2-1.35 for bananas under long cycle sprinkler irrigation, the value for the same crop under short cycle drip irrigation came down to 0.63. The value of Ym increased from 40-60 tons/ha under sprinkler irrigation to 100-120 tons/ha under drip. These results illustrate the adaptability of the relative yield/relative evapotranspiration model to local conditions. As experimental results are built up, the values of the input parameters will become increasingly accurate and the model itself will become increasingly reliable.

5.3.5 Actual yield and applied water.

It can be seen that the expected yield loss can readily be calculated for given values of ET_a, ET_m and ky, and that if a value of the maximum potential yield, Y_m is known for the crop under consideration, then the expected actual yield, Y_a, can be established. Y_m, ET_m and ky are dependent on the crop and prevailing soil and cultural conditions, and are therefore constant for a given design situation. ET_a is dependent on the soil moisture condition; which in turn is directly related to the gross water supply, consisting of (*i*) the initial soil moisture content at the start of the season, (*ii*) the water added to the soil due to natural rainfall and (*iii*) the water applied by irrigation, IA.

Thus if a relationship could be established between ET_a and IA then the expected yield could be estimated from the irrigation application. In fact, a number of researchers have proposed a one-to-one relationship between ET_a and IA on the basis that the water applied by irrigation directly replaces the water evapotranspired by the plant. This implies a linear relative yield/relative irrigation application model, equivalent to the relative yield/relative evapotranspiration relationship :

$$(1 - Y_a/Y_m) = ky\,(1 - IA/ET_m) \qquad \text{for } IA/ET_m < 1 \qquad (5.6)$$

In terms of this model, the expected yield will increase linearly with the amount of water applied by irrigation up to a maximum, Y_m, after which it will remain constant. This model is supported by a large volume of research, some of which is reviewed by Warrick and Gardener (1983) in defense of the linear model. Shalhevet *et al* (1976) calibrated a number of linear regression models of yield versus applied water, for several different crops. The models were based on experimental results and gave good goodness-of-fit statistics, thereby supporting the linear response hypothesis.

It has been found that the maximum yield level is maintained for considerable levels of over-irrigation, above which the yield may begin to come down in conditions of poor drainage and excessive nutrient leaching. The model has therefore been considered valid for the type of analysis required for the design problem, where optimal irrigation levels will not be greatly in excess of that required for maximum yield.

5.3.6 Yield loss.

Thus for a given irrigation requirement to achieve maximum yield, IR, and average application, IA, the **expected yield loss** can be calculated as follows :

For each emitter an **application factor** is calculated by :

$$Af_i = (q_i/q_{av}) * (IA/IR)$$
$$Af_i = 1 \qquad \text{for } (q_i/q_{av})*(IA/IR) >= 1 \tag{5.7}$$

Where Af_i = the application factor at emitter i;
q_i = the discharge from emitter i;
q_{av} = the average emitter discharge.

Then the **yield loss for each emitter,** YL_i, is given by :

$$YL_i = ky\,(1 - Af_i)\,Ym \tag{5.8}$$

And the total **yield loss for the block,** YLb is given by :

$$YLb = Ym\,(\sum_i YL_i)/\,ni \qquad \text{(tons/Ha)} \tag{5.9}$$

Where ni = the total number of emitters in the block.

5.4 Economic calculations

5.4.1 Inflation.

Inflation can have a significant effect on the viability of a development project, particularly when it is evaluated over a long period. The lifespan of irrigation development projects is typically taken as between 10 and 20 years. Furthermore, the economics of agricultural projects are affected by several different cost and income parameters, each of which might be inflating at a different rate. These parameters can be grouped as follows :

* production costs;
* energy costs;
* general cost of money; and
* the producer price (the price paid to the farmer for his produce).

The economic evaluation model incorporates four different inflation rates, one for each of the abovementioned factors, as input variables to be specified by the designer.

A number of different approaches have been proposed to account for the effects of inflation, in the economic evaluation of development projects. The method used in the proposed model has been modified from Hanke *et al* (1975) and is given by :

$$(1+i) = (1+I)/(1+r) \tag{5.10}$$

Where i = the rate of interest to be used in ensuing discounted cash flow analyses;

I = the rate of inflation;

r = the discount rate.

5.4.2 Discount rate.

The **discount rate** is the rate used in the discounted cash flow analyses to relate future worth to present value. The question of the most appropriate rate to use for agricultural development projects is a complex one. For most commercial projects of a private nature, the opportunity cost of the investment represents a reasonable measure for calculating the economic worth of the project. In other words, the discount rate can be taken as being equal to the rate that could be earned from an alternative investment, commonly the prevailing bank rate. However, there is varying opinion with regard to the evaluation of agricultural projects.

On the one hand, experience has shown that farmers do not generally perceive the opportunity costs of a project as being important. They are often strongly motivated by the desire to develop the land as fully as possible, so as to foster a feeling of permanent ownership and stability. In other words, they take a long term view of their operation, and any proposed development cannot therefore be evaluated purely in terms of the immediate return on investment that will accrue. Also, farmers are inclined to attach more weight to capital costs which require immediate expenditure, than to future operating costs, even when these operating costs form the dominant portion of the total costs. This is because capital expenditure normally has a greater impact on the farmer's cash-flow situation than operating costs. These factors all mitigate for a low discount rate to be used.

Similarly, in the case of agricultural projects that are initiated by development agencies the need for a rapid return on investment is often supplanted by the aim of stimulating rural development. This also mitigates for a lower than normal discount rate.

On the other hand, Keller (1983) argues that agricultural development projects are naturally of a high risk nature, and economic evaluations of these projects should therefore reflect this through the use of a discount rate in the region of 10% higher than the prevailing interest rates.

It can be seen therefore that the value of the discount rate to be used in the analysis should be decided subjectively to reflect the preferences and objectives of the developer. It is therefore incorporated into the evaluation model as a user specified variable.

5.4.3 Costs.

The different cost elements incorporated into the model are as follows :

* **Fixed costs.** These include :
  - equipment and installation costs of both the block and a portion of the mainline equal to the ratio of the block area to the total scheme area.
  - system maintenance costs, again for both the block and a representative portion of the mainline.
  - fixed production costs, representing those costs of production other than direct irrigation costs, that are fixed and not related to the size of the yield. These typically include tractor and machinery costs, land preparation costs, chemicals, labour and interest on operating finance.
* **Yield related production costs,** which typically include items such as harvesting, packing and in some cases fertilization.
* **System operating costs,** which include energy and direct water costs.

The cost of water has two possible components. Firstly the **basic cost** of the actual water used for irrigation. This may be charged directly to the farmer, in which case it is straight forward to calculate. In some cases however, the water supply may be subsidized and then the rate to be included in the calculations depends on the purpose of the analysis.

The second water cost component is an **opportunity cost** which may be applicable when the available water supply is limited and there are a number of alternative uses competing for the water. It represents the potential additional benefits that could be realized by using the water elsewhere. The opportunity cost of water is particularly relevant in the case of deficit irrigation, where reduced application will result in a saving of water.

5.4.4 Return on investment calculations.

The **equivalent annual worth, EAW,** is derived in the following steps :

1) Present value factors for the four groups of parameters discussed above are calculated using the conventional present value formulation :

$$PVF = [(1+i)^N - 1]/i(1+i)^N \qquad (5.11)$$

Where PVF = the present value factor;

i = the interest rate (%/100);

N = the period of analysis (years).

The value of the interest rate, i, is calculated for each factor from equation 5.10, using the different inflation rates. Four present value factors are calculated, being respectively :

"*PVF_prod*" for production costs;

"*PVF_energy*" for energy costs;

"*PVF_earning*" for the producer price;

"*PVF_general*" for the general cost of money.

2) The cost elements are then calculated as follows :

$$k\_fixed = tot\_captl\_costs + (tot\_maint\_costs + fixed\_prod\_costs) * PVF\_prod \qquad \text{(R/Ha)} \qquad (5.12)$$

$$k\_yield = yield\_prod\_costs * PVF\_prod \qquad \text{(R/ton)} \qquad (5.13)$$

$$k\_operat = [(tot\_energy\_cost * PVF\_energy) + ((water\_base\_cost + water\_opport\_cost) * PVF\_general)] * Ab * Ea_g * 10 \qquad \text{(R/mm)} \qquad (5.14)$$

Where *k_fixed* = the fixed costs over the life of the project, ammortized to present value terms;

k_yield = the yield related operating costs over the life of the project, ammortized to present value terms;

k_operat = the non yield related operating costs over the life of the project, ammortized to present value terms;

tot_captl_costs, *tot_maint_costs* and *fixed_prod_costs* are the equipment, maintenance and fixed production costs respectively, as defined in section 5.4.2;

yield_prod_costs are the yield related production costs given in R/ton of produce;

tot_energy_cost = the total energy cost per cubic meter of water

= $base\_energy\_rate * XP/367 * 0.75$ (R/m^3 of water);

base_energy_rate is given in R/kWhr;

XP = the pump operating head (m);

water_base_cost and *water_opport_cost* are given in R/m^3 of water;

Ab = the area of the block (Ha);

Ea_g = the gross efficiency of the system, reflecting losses in the system and to the atmosphere.

3) The present value of **nett earnings from production**, *nett_earn*, is given by :

$$nett\_earn = (prod\_price * PVF\_earning) - k\_yield \qquad \text{(R/ton)} \qquad (5.15)$$

Where *prod_price* = the producer price paid to the farmer (R/ton).

4) For a given average irrigation application, IA, the expected **yield loss**, YL_b, in tons/ha is calculated from equation 5.9. And the **present value cost of this loss** is given by :

$$NPV\_cost = (nett\_earn * YL_b * A_b) + (IA * k\_operat) \qquad \text{(R)} \qquad (5.16)$$

5) Thus the **nett present worth of the project** is given by :

$$NPW = nett\_earn * Y_m * A_b - NPV\_Cost \qquad \text{(R)} \qquad (5.17)$$

6) Finally, a capital recovery factor, CRF, is calculated using the conventional formulation (the inverse of the PVF expression in equation 5.11) with the interest rate equal to the project discount rate. And the **equivalent annual worth** is then given by :

$$EAW = NPW * CRF \qquad (5.18)$$

A sample set of all the input parameters required for the economic evaluation model is given in table 5.1.

5.5 Operating Point

The economic analysis described above was specified for an average irrigation application, IA. As discussed in section 5.2 above, this average application can be varied between zero and the maximum system capacity in response to the plant requirement. It is therefore necessary to determine the optimal operating point (i.e. the optimal IA) corresponding to a given seasonal requirement, IR. The value of IA/IR is termed the "*application ratio (ar)*", as defined in chapter 3.

Furthermore, since the requirement varies during the season, the optimal application ratio may also vary. For example, in the case of a period of low irrigation requirement it will be feasible to raise the application ratio so that all or most of the field receives the required application. This will not be possible during the peak requirement period since the upper limit of the ratio will be constrained by the system capacity. The optimum point in each period will depend on the economic factors affecting the evaluation model. For example, if the yield response factor, ky, varies during the growing season, then it would intuitively make sense to raise the application ratio during the period when the ky value is highest (the crop is most sensitive to water deficits). However there is a trade-off with the higher operating costs that ensue from a higher application.

If the total available seasonal water supply is limited but may be allocated in any way during

Table 5.1 Sample set of the input parameters for the economic evaluation model

Design Data

1) Capacity/Operating regime:

Average emitter discharge	*(48.6 lph)	Gross application rate	(6.9mm/h)
Emitter spacing	(2.0 x 3.5m)	Gross system efficiency	96%
Total available water	800mm	Nett application rate	(6.7mm/h)
		Maximum irrigating time/set	6hrs
		Maximum application/cycle	(40mm)

2) Area:

Area of block	(0.2Ha)	Area of scheme	5Ha

3) Agronomic data:

Maximum crop yield 53tons/Ha

Period	Nov/Dec	Dec/Jan	Feb	March	Total
Crop water requirement (mm/cycle)	25	40	30	20	(590)
Duration of period (cycles)	6	6	4	4	(20)
ky value for each period	0.8	0.8	0.8	0.8	

4) Application fractions for operation point optimization: 0.90 1.00 1.05 1.10 1.15

Economic Data

1) Capital costs:

Material costs	
-block piping and emitters	(R 343.07)
-other block materials	R 34.30
-estimated mainline cost	R 2500.00
Installation costs	
-block	R 100.00
-mainline	R 500.00
Other capital items (fees,etc)	R 50.00
Total capital costs	(R 3162.54/Ha)

2) Maintenance costs:

Block	5%	(R 91.68/Ha/yr)
Mainline	3%	(R 15.00/Ha/yr)

3) Fixed production costs: R 2500/Ha/yr

4) Yield related production costs: R 75.00/ton

5) Operating costs:

Energy cost	5c/kWh
Estimated headloss in mainline	200kPa
Pump delivery pressure	(362kPa)
Overall energy cost	(0.66c/m^3)
Water base cost	10c/m^3
Water opportunity cost	0c/m^3

6) Discounted cash flow analysis:

Period	15years
Discount rate	13%
Inflation rates	
-energy	18%
-production costs	18%
-earnings	14%
-general	19%

7) Producer price to farmer: R 253.00/ton

*All values within ( ) are calculated by the computer from the design and other input data.

the season, as is typically the case when the supply comes from a storage dam, then there is a further problem to allocate the operating points during the season in such a way that the overall return on investment is optimized.

The optimization problem defined above lends itself to solution by **dynamic programming**, if the evaluation model is applicable for individual periods during the growing season. This in turn is dependent **firstly** on the validity of the crop water/yield model over the individual periods, and **secondly** on being able to specify the required input data for each period. These individual *"periods"* would typically be portions of the growing season during which the crop water demand remained the same. Each period may incorporate one or more irrigation cycles.

5.5.1 Defining the evaluation model on an individual period basis.

In order to fulfill the first condition mentioned above, an additive water/yield function is proposed. This implies that the total yield loss over the full season will be equal to a **weighted** average of the water deficits incurred during the season, where the weighting factors are the product of: *(i)* the *ky* values in each period, and *(ii)* the fraction of the total growing season taken by each individual period. If the length of each period is expressed in terms of the number of irrigation cycles, then the additive model is given by :

$$(1 - Ya/Ym) = \underset{p}{?}\, ky_p\,(Nc_p/Nc_t)\,(1 - IA_p/ETm_p) \qquad (5.19)$$

Where ky_p = the crop response factor during period p;
Nc_p = the number of irrigation cycles in period p;
Nc_t = the total number of irrigation cycles in the season;
IA_p/ETm_p = the application ratio in period p.

On the basis of the underlying theory of the linear model, this additive model is intuitively reasonable. No reported experimental evidence has been found to support it, but it is used by Doorenbos and Kassam (1979) in several of their worked exercises illustrating the applicability of the linear model. It has also been proposed in the literature by Jensen (1968), and has been used in similar design analysis models by Allen (1986) and Tsakiris (1985).

Given the additive model, corresponding theoretical yield losses for each individual period can be defined by :

$$(1 - Ya/Ym)_p = ky_p\,(Nc_p/Nc_t)\,(1 - IA_p/ETm_p) \qquad (5.20)$$

Where $(1 - Ya/Ym)_p$ = the yield loss incurred during period p.

And similarly the evaluation model can be applied to calculate the nett present value cost of the yield loss in each period by modification of equation 5.16 :

$$NPV\_cost_p = (nett\_earn * YLb_p * Ab) + (IA_p * k\_operat) \quad \text{(R)} \quad (5.21)$$

Where the terms of equation 5.21 are the same as those defined for equation 5.16 with the subscript p referring to an individual period in the growing season.

The second condition stated above for the applicability of the evaluation model to individual periods in the growing season was the availability of data on an individual period basis. The specific parameters needed for this are shown in table 5.1, *viz:* the crop water requirement in each period, the number of fixed irrigation cycles (of equal length) in each period and the corresponding *ky* value for the period.

The crop water requirement is normally estimated from daily evaporation data, and it is fairly common practice to group the data into periods of the same average daily requirement. These are typically, but not necessarily, monthly periods. Thus there should not be any difficulty in defining the crop requirement for a number of different periods during the growing season. The length of each period will also be known.

The most difficult parameter to establish for each period is the crop response factor, *ky*. Doorenbos and Kassam have defined factors for a number of different crops for four different growth periods in the plant cycle: *"vegetative"*, *"flowering"*, *"yield formation"* and *"ripening"*. If these are available for the crop under consideration and they can be correlated to the periods defined by the varying water requirement, then the corresponding *ky* values can be used. If no individual *ky* values are available then the total seasonal value can be used for each period.

5.5.2 The proposed dynamic programming model

On the basis of the foregoing, the proposed **dynamic programming (DP) solution** to the problem of optimizing the system operating point is derived as follows :

The *stages* of the DP are the periods defined during the growing season, and the *decision variable* is the operating point, or application, in each period, IA_p. The solution is best derived from a backward-moving DP. In other words the solution procedure starts by calculating the cost function at the last period in the season and then continues by working back through each stage to the first period. The cost component of each stage is a function of the decision variable given by :

$$R_p(IA_p) = NPV\_cost_p \quad (5.22)$$

And the objective function is then : **Min** $[\Sigma R_p(IA_p)]$

subject to : $\Sigma IA_p < Q$

Where Q = the total available water supply.

The state variable, s_p, represents the amount of water remaining out of the initial total, at stage p. It is determined from the sum of the allocations in each period from the last, back to p. For any feasible state in period p a function of the cumulative cost up to the current stage, $f_p(s_p)$, is given by :

$$f_p(s_p) = R_p(IA_p) + f_{p+1}(s_{p+1}) \tag{5.23}$$

Where $f_{p+1}(s_{p+1})$ = the cumulative cost up to the previous stage, which was at period p+1.

And then if the notation, $f^*_p(s_p)$, is used to denote the minimum cumulative cost at stage p for state s_p, then the recursive equation of the DP is given by :

$$f^*_p(s_p) = \underset{0<IA_p<s_p}{\text{Min}} [R_p(IA_p) + f^*_{p+1}(s_{p+1})] \tag{5.24}$$

Thus by calculating f^*_p for each stage working back to period 1, the value of $f^*_1(s_1)=f^*_1(Q)$ gives the total *NPV_cost*, and the optimal operating point at each stage can be found by following the optimal allocation policy through the DP solution.

Both the decision variable, and consequently the state variable, are continuous over the range of feasible operating points. In other words, literally any depth of water can be applied by the irrigation system, within the limits of the system capacity. This implies an infinite number of feasible values of the decision variable and a corresponding infinite number of values of the state variable. In order to set up the DP it is therefore necessary to specify a series of discrete values within the feasible range of the decision variable, for each stage of the DP. This in turn will generate a limited number of feasible states at each stage. This *discretization* is achieved by specifying a constant set of application ratios, from which the actual applications can be calculated for each period.

5.5.3 Example of the DP application.

A hypothetical example is presented in order to illustrate the setting up of the DP and the calculations carried out at each stage :

Assume :

- the water requirement, ETm, for three periods during the season is 300, 400 and 200 mm respectively;
- the total available water supply = 800 mm; and
- the discrete values of the water application, IA, to be used in each period are determined by application ratios of 0.7, 0.8, 0.9, 1.0 and 1.1 respectively.

Then :

The actual values of the decision variable to be applied at each stage are calculated by multiplying the requirement by each application ratio in turn. The values are shown in table 5.2.

Table 5.2 Values of the decision variable, IA_p, for each period

ar	IA_1	IA_2	IA_3
0.7	210	280	140
0.8	240	320	160
0.9	270	360	180
1.0	300	400	200
1.1	330	440	220

For each of these applications, a hypothetical value of the cost function, $R_p(IA_p)$, which is equal to NPV_cost_p, is shown in table 5.3.

Table 5.3 Values of the cost function, $R_p(IA_p)$, for each period

ar	$R_1(IA_1)$	$R_2(IA_2)$	$R_3(IA_3)$
0.7	1500	1650	1400
0.8	1350	1500	1200
0.9	1050	1100	900
1.0	950	1000	900
1.1	950	950	1000

The calculations made using the recursive equation (5.22 and 5.23), for each stage, are shown in tables 5.4 to 5.6 respectively. For example, an application of 400mm in period 2 will have an associated cost, given in table 5.3, of 1000. If the current state (amount of remaining water) in this period (table 5.5) is 560mm, then the application of 400mm will leave 160mm remaining in period 3. From table 5.4, the optimal application in this case is the full 160mm with an associated cost of 1200. Therefore the minimum cost at stage 2 of an application of 400mm from a total of 560mm is 1000+1200=2200.

Similarly, an application of 300mm in period 1 has a cost of 950. This leaves 500mm remaining in period 2, for which the optimal application is 320mm at a cumulative cost of 2400 (1500 from applying 320mm in period 2 and 900 from applying 180mm in period 3). The total cost of this strategy is therefore 3350.

Once the calculations are complete, the optimum strategy can be identified by working through the results. From table 5.6, the minimum cost for this system is 3300, and this is incurred from applying 240mm, 360mm and 200mm in periods 1, 2 and 3 respectively.

Table 5.4 Values of the function $f_3(s_3)$

State	$R_3(IA_3)$						
s_3	IA_3: 140	160	180	200	220	$f^*_3(s_3)$	IA^*_3
140	1400					1400	140
160	1400	1200				1200	160
180	1400	1200	900			900	180
200	1400	1200	900	850		850	200*
220	1400	1200	900	850	1000	850	200

Table 5.5 Values of the function $f_2(s_2)$

State	$R_2(IA_2)$						
s_2	IA_2: 280	320	360	400	440	$f^*_2(s_2)$	IA^*_2
470	2550	2900				2550	280
500	2500	2400	2500			2400	320
530	2500	2350	2300			2300	360
560	2500	2350	1950	2200		1950	360*
590	2500	2350	1950	1900	2350	1900	400

Table 5.6 Values of the function $f_1(s_1)$

State	$R_1(IA_1)$						
s_1	IA_1: 210	240	270	300	330	$f^*_1(s_1)$	IA^*_1
800	3400	3300	3350	3350	3500	3300	240*

5.6 Comparison With Other Models

Several researchers have proposed models to solve the yield-distribution integral (equation 3.31) with the specific purpose of considering system design rather than in-field evaluation. Some of these are reviewed below for the sake of comparison with the proposed model.

The following papers are reviewed :

Allen (1896);

English *et al* (1983);

Hill and Keller (1980);

A series of three papers by Peri, Norum and Hart published with each author listed first, in turn (1979, 1979, 1980);

Seginer (1978);

Tsakiris (1985);

Warrick and Gardner (1983).

5.6.1 Water distribution function.

All of the abovementioned authors, with the exception of Allen, have used a functional analysis of the water distribution pattern. In other words, they have assumed that the distribution pattern can be adequately represented by a statistically derived mathematical function. There is however, considerable disparity between the functions and solution procedures that have been used.

Allen does not consider the water distribution function directly in his model, he simply designs for acceptable limits of pressure variation within the system.

English *et al* use the analysis of Hart and Heerman (1976) that was discussed in chapter 3, with the assumption of a normal distribution of the watering pattern. Their analysis requires estimation of the mean and variance of the ET_a/ET_m ratio, on the basis of assumed values of the coefficient of uniformity and the requirement and application efficiencies.

Hill and Keller present an analysis for three different irrigation systems, *viz* : sprinkler, drip and furrow. A normal distribution characterized by the coefficient of uniformity is assumed for the sprinkler system; similarly a normal distribution characterized by the *emission uniformity* (Karmeli and Keller 1975) is assumed for the drip system; and empirical formulae for the water time advance and infiltration respectively are used for the furrow system. Statistical analysis, published by Hart and Reynolds (1955), relating irrigation depth received per increment of area to the uniformity coefficient is used for the sprinkler system.

Peri *et al* develop their own analytical solutions to the yield-distribution integral for both normal and linear functions of the water distribution pattern. Similarly, Seginer develops a solution based on an assumed uniform distribution function. Warrick and Gardner show mathematical formulations for five different statistical functions, in which they include consideration of the effects of both the non-uniformity due to the system design and that due to the heterogeneity of the soil.

Tsakiris, in part 1 of his paper, formulates three "*application pattern parameters*" to characterize the distribution pattern. These parameters are calibrated from experimental catch-can measurements of the spread of water from a single sprinkler. In part 2 of his work, an alternative analysis is presented using both normal and Pearson type 3 distributions. A graphical (numerical) solution procedure is used to relate the coefficient of variation to the relative mean deficit for varying application ratios.

5.6.2 Water/yield function.

Several different approaches to the water/yield function are also reported.

Warrick and Gardner, Allen, Tsakiris and English *et al* all use the linear relative yield/relative evapotranspiration model formalized by Doorenbos and Kassam. English *et al* use the specific curve for winter-wheat, which in fact has a correction term for apparent non-linearity of the response curve close to the maximum point. Tsakiris discusses the validity of the model in estimating the seasonal evapotranspiration deficit from data on the deficit at each irrigation during the season. He considers both the additive and multiplicitive models.

Allen uses an additive form of the linear water yield function for individual periods of the growing season. His model is mathematically equivalent to the model proposed in this research (equation 5.19), but excludes the second weighting factor (ie. fraction of the total growing season taken by each period). Also, he does not use a direct relationship between ETa and IA. He proposes rather, multiple linear regression equations expressing ETa for each period p as a function of IA_p and the antecedent soil moisture content at the start of each period.

Seginer uses a linear function of similar shape to the Doorenbos and Kassam function, but with absolute values of yield and seasonal water application rather than the dimensionless (relative) values.

Hill and Keller use a graphical curvilinear plot of experimental data relating absolute yield to seasonal infiltrated water depth.

Peri *et al* present an analysis that is essentially independent of the water/yield function. They postulate parameters a and b representing the economic loss incurred due to areas receiving deficit irrigation and excess irrigation respectively. Their analysis is based on the ratio b/a, and on the assumption that the magnitudes of these cost parameters are directly proportional to the amount of the deficit, or excess respectively. In their third paper they do present a case specific analysis based on a quadratic absolute yield/seasonal water application function.

5.6.3 Operating point.

All of the authors consider the operating point as a decision variable, however they relate to it in different ways. Only Allen considers the possibility of varying the operating point during the season. Peri, Tsakiris, and English *et al* make a stated assumption that the application ratio remains constant throughout the season.

Allen uses the effective irrigation application rate as a measure of the operating point. It is expressed as a series of decision variables, one for each identified growing period in the season. These variables are evaluated within a linear objective function which is solved by conventional linear programming methods.

English *et al*, Tsakiris, and Warrick and Gardner all do not consider the question of optimizing the operating point, but rather incorporate it as a decision variable that is established on a case-by-case basis as part of the input data. English *et al* specifically examine the feasibility of deficit irrigation.

The analyses of Peri *et al* are based entirely on optimizing the operating point; whilst both Seginer, and Hill and Keller present methods of finding the optimal operating point as part of their overall analyses.

5.6.4 Model structure and solution procedure.

Only three of the seven papers present a full economic analysis including consideration of complete system designs. These are the papers of Allen, English *et al*, and Hill and Keller.

Allen derives a single linear objective function of "expected equivalent annual net after-tax profit" for an irrigation development project, which is maximized by standard linear programming. The decision variables are the capital, energy and labour requirements of the system; expected yield; production costs; and the effective irrigation application rates for different periods in the growing season. The model is based on linearized expressions of various costs and labour and energy requirements versus the application rate. These expressions are in turn derived by linear regression from a series of simulated hydraulic designs for given system configurations. The hydraulic designs are carried out by a "critical branch approach" for blocks and "life cycle costing" for mainlines. The model is best suited to coarse level design of very large projects. The example shown by Allen in his paper is a 2 700ha. multi-crop development.

Hill and Keller consider the irrigation of a single field of sugar cane to be irrigated alternatively by a sprinkler, a drip and a gravity (furrow) system. For each alternative method, a series of designs are developed for a range of uniformities. The capital costs associated with each of these uniformities are identified. Then for each design, the expected yield and hence profit are calculated using the functions discussed above with an assumed seasonal depth of application. A search technique to find the optimal seasonal application depth is described. The process, which is similar to that of the model proposed in this research, entails evaluating the model for discrete increments of the application depth between prespecified limits; as the application depth appears to be approaching the optimum, the size of the increments is decreased. The profit calculation is done by amortizing the capital costs using a capital recovery factor based on a fixed discount rate and zero inflation. The results are presented in tabular form for each system, for varying uniformities, pumping requirements and producer prices.

English *et al* present a case study on a field of winter-wheat to be irrigated by a moving lateral system. Two alternative designs are presented, one for full irrigation and the other for deliberate deficit irrigation. For each design, a system uniformity coefficient and operating point are assumed and the expected yield is determined analytically using the models discussed above. The water supply is from wells, so that the total supply is limited by the capacity of the wells. The water savings realized for the deficit design are utilized by irrigating a larger area than is possible under full irrigation. The economic analysis incorporates equivalent annual worth calculations using two different inflation rates, 15% for energy costs and 10% for all other factors. Considerably better returns are realized from the deficit system than from the full system.

The models of Seginer and Peri *et al* are independent of any specific design, with economic results presented as a function of hypothetical uniformity coefficients. Both papers show calculations of the expected net income for two different uniformities; pointing out that the difference between the two results represents the amount that can be afforded in upgrading a proposed irrigation system from operating at the lower uniformity to operating at the higher uniformity.

Seginer solves the yield-distribution equation analytically using the functions mentioned above. He develops an expression for the mean expected yield, $\overline{Y}$, in terms of the coefficient of uniformity and the mean seasonal application depth, $\overline{w}$. The optimal application depth is then found from solving $d\overline{Y}/d\overline{w} = P_w$, where P_w = the unit price of water. The model is illustrated for cotton irrigation and the results are presented graphically through a family of curves of $\overline{Y}$ vs $\overline{w}$ for varying *UCC* and P_w.

Peri *et al* define the "*system optimal depth*" as the irrigation depth that minimizes the losses due to deficit and excess irrigation respectively, for a particular irrigation system. They define the total loss function, *LT*, as the sum of the deficit loss function, *LD*, and the excess loss function, *LE*. These latter functions are in turn given as linear functions of the loss per unit volume of deficit and excess, b and a, and the extent of the deficit and excess respectively. The extent of deficit and excess are functions of the distribution pattern and time of application respectively. Thus, since the distribution pattern for a given system is assumed known, the optimal time of application, *t*, and hence the optimal average application depth, are given by solving $dLT/dt = 0$. Solutions are generated for varying values of b/a and *UCC*.

Both Tsakiris and Warrick and Gardner do not attempt any economic analyses, but rather develop solutions to the yield-distribution integral. Warrick and Gardner acknowledge that for the more complex forms of the distribution function, the integral is difficult to solve analytically and must therefore be evaluated numerically. They outline a method utilizing Monte Carlo simulations. Solutions are shown in the form of graphical contours of expected yield for varying application ratios and coefficients of variation of the distribution function. Tsakiris develops an analytical solution for the specific functions discussed above and discrete solutions are generated for a range of application ratios.

5.7 The Computer Models

The computer model for evaluation has been designed to provide the user with full flexibility to consider all the aspects of evaluation discussed above. The computer program forms part of the block design package described in the previous chapter and is operated from within the overall block design model. The evaluation function is accessed from the main menu, after completion of the design phase, and the evaluation is then carried out in a number of sub-functions as shown in appendix 1a.

5.7.1 Program operation.

The program operates from six different pages, or "*screens*", which the user can select at will from the evaluation menu.

The first two pages present the results from the *uniformity* evaluation, as shown in figures 5.4 and 5.5 respectively. Page 1 presents a table of the various parameters discussed above, and page 2 shows a list of any laterals in which one or more of the emitters will operate at pressures outside of the defined limits. The calculations are carried out for four different valve inlet pressures, as can be seen in the displays. Initially, the four pressures are respectively 0.8, 1.0, 1.1 and 1.2 times the required pressure determined in the design phase. However the user may subsequently specify any desired pressure and the evaluation will be carried out for this pressure.

<Proj - 1> * NBB 30/09/86

Evaluation of Block J1 : Page 1 - Uniformity Analysis

Inlet Pressure (m)	14.3	16.1	17.9	19.7
Inlet Flow (m3/hr)	12.4	14.5	16.9	19.5
Uniformity (%)	84.1	87.1	85.3	85.1
Ave q (l/hr)	26.4	30.7	33.0	34.8
Ave q/q nom	0.88	0.96	1.04	1.10
q min (l/hr)	21.4	24.7	32.7	31.0
q max (l/hr)	28.8	33.6	36.4	38.4
q variation (%)	18.1	19.2	20.3	21.4

Figure 5.4 : Results from the uniformity analysis of the block evaluation model

<Proj - 1> * MBB 30/09/86

Evaluation of Block J1 : Page 2 - High/Low Pressures

Laterals with Pressure Profiles Beyond Allowable Limits

Inlet Pressure (m)			
14.3	16.1	17.9	19.7
18a	None	1a	1a
19a		2a	2a
20a			3a
			4a
			5a
			6a

Figure 5.5 : Results from the uniformity analysis of the block evaluation model

The remaining four pages relate to the economic evaluation. On initiating this function, the user will have either page 1 or page 2 displayed on the screen and will be requested to specify which of the four displayed inlet pressures should be used for the ensuing calculations. All the input parameters required for the economic evaluation, as listed in table 5.1, must then be specified. This is done in pages 3, 4 and 5. Any of these parameters which are determined by the design process or the uniformity evaluation, such as the block equipment costs, the layout characteristics and the average emitter discharge, are carried through and displayed on the relevant pages automatically. On initiating page 6, the computer checks that all the required data have been supplied and then carries out the economic evaluation. The results are displayed in both tabular and graphical format, as shown in figure 5.6.

At any stage in the process, the user may move interactively from page to page.

GRABOUW APPLES 28/01/87
EVALUATION OF BLOCK (3 :) PAGE 6 ANALYSIS FOR CU = 98% : AV q = 48 l/hr

DEPTH (mm)	530	590	604	625	640
VOL ('000 m3)	1.06	1.18	1.21	1.25	1.28
YIELD					
Tons/ha	48.6	52.5	52.8	52.9	52.9
kg/m3	9.2	8.9	8.7	8.5	8.3
EFFICIENCY					
Applic - Ea (%)	99	98	96	93	91
Reqmnt - Er (%)	89	98	99	99	99
EQUIV.ANNUAL WORTH					
EAW (R/ha)	7348	8607	8672	8604	8549
EAW (R/m3)	1.39	1.46	1.44	1.38	1.34

OPTIMAL OPERATING POLICY DURING SEASON

PERIOD	1	2	3	4	5	6	7	8	TOTAL
Reqmnt (mm/cycle)	25	40	30	20					590
Applic (mm/cycle)	26	40	31	21					604

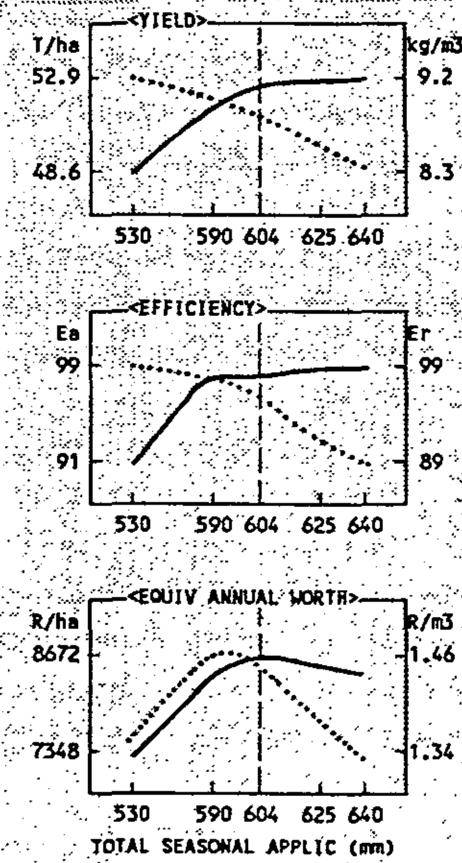


Figure 5.6 : Results from the cost/benefit analysis of the block design evaluation model

5.7.2 The algorithms.

Complete PASCAL listings for a number of the evaluation model algorithms are shown in appendices 2b and 2c. These algorithms are described below :

Calculate the emitter discharges. In order to carry out the uniformity calculations, the actual emitter discharges have to be calculated. Information stored in the design phase includes all the pipe diameters and the coefficients of the assumed linear discharge/pressure relationship for each lateral on the manifold (as defined on p4.16 - "Lateral vs. mainfold design"). An iterative procedure for the calculation of the actual discharges, for a given inlet pressure, is carried out as follows :

- The calculations start at the manifold end, with an assumed end pressure, and work back to the inlet.

- For the given lateral inlet pressure, the expected lateral inlet flows are calculated from the discharge/pressure relationship. The actual emitter pressures, and hence discharges, are calculated in turn from the lateral inlet to its end, using the given lateral inlet pressure and inflow, the known emitter pressure/discharge relationship and a point to point calculation of the head losses in the lateral pipes.
- On reaching the end of the lateral, the calculated remaining lateral flow is checked to ensure that it is equal to zero. If not, then the estimated inlet flow is adjusted, and the calculation is repeated.
- Once these calculations are satisfactorily concluded, the head loss in the manifold up to the next lateral is calculated, using the corrected lateral inflows. This in turn enables calculation of the inlet pressure at the next lateral.
- The process is repeated for each lateral in turn, working back to the manifold inlet. A cumulative total of the manifold flow and its pressure at the current lateral are maintained throughout the calculations.
- Once the valve is reached, the calculated pressure at this point is compared with the specified inlet pressure for which the calculations are being done. If the calculated pressure is not within 1% of the specified pressure, then the end pressure that was used to initiate the calculations is adjusted by the amount of the error minus a correction for the non-parallel effect. The whole process is repeated until the correct inlet pressure is reached.

Calculate the uniformity parameters. The uniformity parameters are calculated by looping through the emitter discharges in a two-pass process. The first is used to calculate the average emitter discharge, as well as the maximum and minimum values, and the laterals in which one or more emitters operate outside the design limits. The second pass uses the average value to calculate the mean deviation from the mean (coefficient of uniformity), as well as the maximum percentage variation.

Perform the dynamic programming optimization. Prior to initiating the economic evaluation, the inputted irrigation requirement values are all rounded to the nearest 5mm, and all subsequent calculations are carried out using 5mm increments. The DP is evaluated in three phases :

- The cost matrix for each stage of the DP and each possible application (table 5.2 in the example presented in section 5.5) is calculated. The program loops through each emitter discharge and calculates the expected yield loss; this is then used to calculate the associated *NPV_cost*.

- Maximum and minimum feasible states (remaining available water) are then calculated for each stage, by subtracting the minimum and maximum applications respectively, starting from the initial total at stage one. Then the cumulative cost matrices at each stage of the DP (tables 5.3 - 5.5 of the example) are calculated for each possible state, in 5mm increments, between the established feasible maximum and minimum limits. This is done working back from the last period to the first.
- Finally, the DP results are determined from the matrices, and the economic evaluation parameters are calculated.

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6. MAINLINE DESIGN

6.1 Introduction

The **mainline** is the pipe network that distributes water from the source to the block valves. It consists of the pipes themselves which may ramify from a single main into several sub-mains, as shown in figure 6.1, and various control elements such as pumps, filters, fertilizer injectors and pressure regulating valves. In general, irrigation networks have a **branching** structure, implying that the network branches out from the source without ever joining back up with itself; unlike urban water supply networks which are normally **looping**. This branching structure simplifies the design problem as is shown further on in this chapter.

At the start of the mainline design process the location and operating requirements (discharge and pressure) of each block valve are known. In most cases, the location of the pumps (water source) is also known. The design problem consists of three distinct elements :

* determining the network **layout**;
* establishing the **sequencing** (or **scheduling**) of the block valves during operation; and
* **sizing** of the pumps and pipes.

These three problems are inter-dependent, so that the solution to any one of them will affect the results of the other two. It is therefore difficult to establish a globally optimal solution procedure for the whole design. For this reason, the approach adopted has been to develop locally optimized solutions for each module. If necessary, each solution can then be modified by a manually directed iterative process, based on the results obtained for each of the other design modules.

This chapter provides a detailed description of each of the three solution processes, together with a description of the computer programs that have been developed to generate these solutions.

6.2 Network Layout

6.2.1 The design problem

The layout problem consists of establishing the lowest cost arrangement of pipes that will link the water source to each of the block valves.

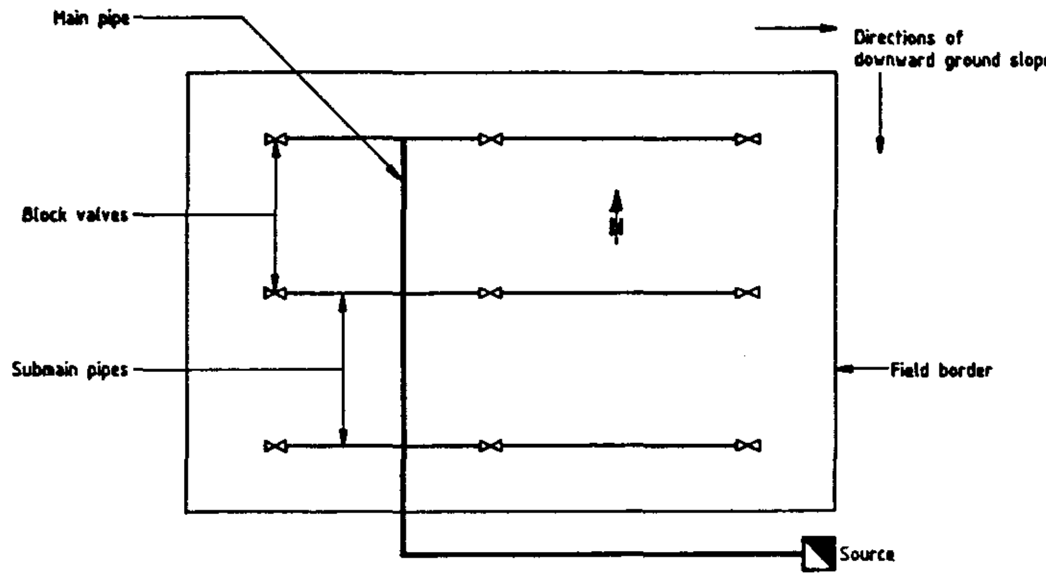


Figure 6.1 : Schematic example of a mainline layout

In its simplest form, the solution to this problem might be seen to be the layout for which the total length of piping is minimized. However, there are trade-offs in the pipe sizing problem which involve exploiting the prevailing topography to offset pressure losses due to pipe friction. These may mitigate against selecting the shortest path. For example, in the network shown in figure 6.1 the length of the main pipe could be reduced by leading it up to the sub-mains directly above the water source. However, this would result in the water in the sub-mains having to be pumped up against the slope. The current arrangement allows for the headloss due to friction in part of the sub-mains to be offset by the gain due to topography. This means that a higher headloss can be tolerated in the sub-main sections below the main pipe, which in turn means that smaller diameter pipes can be used over these sections.

A further consideration in determining the network layout is the operating sequence of the block valves. In general this sequence is established in conjunction with the network layout in such a way as to distribute the flow in the network as widely as possible. In this way the maximum discharge carried by any single branch of the network is minimized, and hence the diameters required are also minimized.

Thus it can be seen that the optimal layout is one for which both the capital costs of the pipes and the energy requirements for operation are minimized. A practical optimization procedure has proved difficult to establish, since the number of possible layouts for any given situation is theoretically infinite. Stephenson (1984) has proposed the following solution : For a given situation, a full *"grid"* network is defined incorporating all of the valves and the source. The network will contain multiple loops and will generally be over-specified for the requirements of the irrigation system. However it will incorporate a large number of alternative layouts. The network is then trimmed by solving a linear programming problem given by :

$$\text{Min} \left[\sum_{i=1}^{N} q_i L_i \right] \tag{6.1}$$

$$\text{subject to} \qquad \Sigma q_{i \in j} = Q_j \qquad \text{for all } j \tag{6.2}$$

Where q_i = the flow in each section i of the network;
L_i = the length of each section i of the network;
N = the total number of pipe sections defined for the network;
$\Sigma q_{i \in j}$ = the sum of flows in all sections i leading water directly into or out of valve or intersection j of the network;
Q_j = the discharge flow out of the network at point j.

Thus the decision variables in this formulation are the flows in each pipe section. Any flow that is set to zero in the solution implies that the section carrying that flow can be discarded. The product $q_i L_i$ represents an expression for the *"cost"* of the pipe section in terms of its flow volume and length, so that the minimization gives a minimum cost solution. In order to solve this problem, assumptions must be made of the flow directions in any sections that form part of a loop. Also the solution does not consider the trade-offs related to the prevailing topography, that were discussed above.

Buras and Schweig (1969) proposed a dynamic programming procedure for the optimal routing of an aqueduct planned for installation in Iran. However their solution is specific to various unique features of the design case, in particular the *"herring bone"* structure of the network.

6.2.2 Proposed solution procedure

Notwithstanding the potentially infinite number of layouts for any given system, most networks will generally have a rectangular, or close to rectangular, alignment that will facilitate the formulation of alternative layouts. For example, in the system shown in figure 6.1, once the direction of the sub-mains has been decided (east-west rather than north-south) then alternative layouts are generated by moving the alignment of the connecting main. Thus if a method for rapid evaluation of each alternative layout can be developed, then the designer can work interactively with the computer generating a number of alternative layouts to be compared on the basis of the evaluation criterion. In this case it is important that the alternative layouts can be specified simply and quickly on the computer, and that the evaluation is also produced quickly, without requiring lengthy computer time.

The proposed evaluation criterion is a preliminary cost estimate, established as follows :

* An acceptable average headloss for the network (say 2% in terms of length) will be preselected by the designer. This is used to calculate the expected headloss in the network from each valve back to the source. This in turn enables estimates of the required pipe-class at each section, the required pumping head for each valve, and the pump duty.
* A first estimate of the pipe diameter at each section is then established on the basis of the flow velocity in each section. Experience has shown that flow velocities in optimally designed irrigation networks vary between a narrow range of ±0.9 to ±1.8 m/s, depending on the size of the flow and the slope of the pipe section. Thus the values used for the diameter estimates have been set as follows :
  - Flow less than $50m^3/hr$: uphill 0.9m/s; downhill 1.2m/s
  - Flow between $50\text{-}150m^3/hr$: uphill 1.0m/s; downhill 1.3m/s
  - Flow greater than $150m^3/hr$: uphill 1.3m/s; downhill 1.8m/s

  For sections at the ends of each branch of the network, these allowable velocities are increased by 67%. Thus for each section the maximum flow is used to calculate the velocity that would result from the various available diameters in the class of pipe established in the first step described above. The smallest diameter that results in a velocity not exceeding the allowed velocity is selected as the first estimate.

In this way the estimated cost of the network, including both the capital costs of the pipes and the operating costs, can be calculated. The optimal layout is then the one with the lowest expected cost.

This layout evaluation model acts as a form of screening model, before the application of a more rigorous optimization model for the sizing of the pipes and pumps. In fact, it provides a starting point for the optimization model, as discussed in section 6.4.4.

6.3 Valve Sequencing

Any given network will contain a number of block valves. Since it is uneconomical to design the network for all the valves to operate simultaneously, the irrigation cycle is divided into a number of irrigation "*shifts*" (or "*sets*") and each valve is allocated to operate in one of these shifts. The valves must be operated in a specified schedule, such that the hydraulic characteristics of the network are maintained within the limits of its capacity.

The number of shifts within an irrigation cycle is established during the preliminary design phase, and the valve operating schedule consists of a specification of which valves operate together in each shift. It can be seen that for each alternative layout the designer will have to specify the valve operating sequence in order to calculate the flows throughout the network.

6.3.1 The design problem

The valve sequencing problem therefore entails allocating the irrigation of each block to be done in one of a given number of available shifts. The optimal schedule will be one for which :

a) the energy required in each shift is minimized; and

b) the maximum flow in each pipe section is minimized.

The second of these optimization objectives relates to the minimization of the capital costs of the pipes through minimizing the required diameters. It implies spreading the required flow in each shift as widely as possible throughout the network.

This is a difficult optimization problem, which is not readily solved by any known techniques. Although there has been considerable work done in optimizing *real-time* responses to the operating requirements of urban water networks, very little work has been reported on the irrigation valve sequencing problem as it is defined above.

The two sets of problems are quite different. On the one hand, urban domestic water networks tend to be multi-sourced and multi-looped. The optimization procedure revolves around establishing the hydraulic characteristics of the network using a computer-based network solver and then developing operating strategies for differing operating requirements. In particular, the *real-time* nature of the problem implies that the network is already designed, and therefore the second of the abovementioned optimization criteria does not apply. The emphasis of the problem relates to optimizing the **operation** of the network.

Irrigation networks on the other hand are generally single-sourced and branching. Also, the different valve pressure requirements are usually constrained within a narrow range, and the duty point on the pump characteristic curve remains fixed during each shift. This enables certain simplifying assumptions to be made. The emphasis of the problem shifts to one of optimizing the **design** of the network.

Sabbagh (1986) has proposed an optimization model for the *real-time* case, in which an objective of minimizing the overall energy consumption of the system is formulated, and the pressure and flow requirements of the valves form the constraints. The model uses a network solver and standard curve fitting techniques to calibrate the pump and system characteristic curves. A search algorithm is then used to establish the optimal operating strategy.

Gofman and Rodeh (1978) also formulated the valve sequencing problem in terms of the single minimum energy objective. They concluded that in this form the problem is equivalent to the so called "*independent task scheduling problem*", which has been shown (Garey et al 1978) to be NP-complete for more than two valves. This is a condition implying that no efficient algorithm for solving the optimization problem is known or ever likely to be discovered. They conclude therefore that the problem is best tackled by using an "*approximation algorithm*", which in mathematical terms is considered satisfactory if it generates solutions that are guaranteed to be close to the optimum.

Their proposed solution procedure employs a computer based hydraulic network solver for interactive use in constructing the valve sequencing schedules. The user plays an important role in directing a search for admissible schedules. Once the system has been defined, the approximation algorithm generates a trial solution on the following basis : An upper bound, B, is set on the total discharge allowed in a single shift. The valves are then scheduled by the *first fit decreasing method*. This method treats the valves in order of decreasing discharges, assigning each valve in turn to the earliest shift in which it will not cause the total discharge to exceed B. For a particular value of B the algorithm will either succeed or fail. Binary search techniques are used to find the smallest possible value of B and the algorithm has been proved to be highly computationally efficient.

6.3.2 The proposed solution procedure

The proposed solution procedure addresses both of the two abovementioned objectives (i.e. energy and pipe flow minimization). The energy requirements of the network are a function of the **pressure** and **discharge** requirements of the valves. However, in single source irrigation networks the operating pressure at the pump is normally determined by the maximum valve pressure requirement, and the system sizing is optimized on this basis. In this case the energy minimization objective can be simplified by expressing it in terms of **discharge** only. And since the total discharge over all shifts is fixed, minimizing the discharge in each shift implies minimizing the deviation of the discharge in each shift from the average discharge per shift. The two design objectives can then be formulated as follows :

a) $$\text{Min}\,[D_j] = \left|\overline{Q}_j - \sum_{v \in j} q_v\right| \qquad \text{for all } j \tag{6.3}$$

Where D_j = the deviation of the discharge in shift j from the average discharge per shift;

$\overline{Q}_j = (\sum_{v=1}^{V} q_v) / J$ = the average discharge per shift;

V = the total number of valves;

J = the number of shifts per cycle;

q_v = the discharge from valve v;

$v \in j$ = all valves v that operate in shift j.

b) $$\text{Min}\,[Q_{k,j}] = \sum_{\substack{v \in j \\ k \in FP_v}} q_{k,v} \qquad \text{for all } k, j \tag{6.4}$$

Where $Q_{k,j}$ = the flow in pipe section k during shift j;

$q_{k,v}$ = the flow in pipe section k due to valve v;

FP_v = the flow path in the network from the source to valve v.

As mentioned above, the second objective stems from consideration of the pipe sizing problem. In the solution to the valve sequencing problem it is incorporated as a constraint which facilitates the formulation of a successful approximation algorithm.

The algorithm works incrementally with each shift in turn. A tolerance level, δ_j, for the maximum allowable deviation of the total shift discharge from the average discharge is set for shift δ_j. Valves are then allocated to this shift on a basis that attempts to minimize the flows in each section and simultaneously bring the total shift discharge to within δ_j% of the average. The value of δ_j is initially set to 5%. If this tolerance cannot be satisfied, then it is increased iteratively until the lowest value that can be satisfied is established.

When a valve is being considered for allocation to a specific shift, the algorithm calculates a measure which has been termed the "**degree of coincidence**" of the valve under consideration with all other valves that have already been allocated to the shift. This is a measure of the extent to which the valve under consideration will add to the maximum flows in the network pipe sections. It is defined as follows :

For each section there is a minimum flow that will be required, irrespective of the valve sequencing programme. This is determined by the largest discharge requirement of all valves whose flow paths include the section in question. The degree of coincidence is calculated as the product of the length of a section times the flow in that section in excess of the established minimum flow, summed over all sections in the network. This is expressed algebraically as follows :

$$Q_{k,j} = \sum_{\substack{v \in j \\ k \in FP_v}} q_v \tag{6.5}$$

Where $v \in j$ = the valves thus far allocated to shift j, plus the valve under consideration for addition to the shift.

The excess flow in each section, $EF_{k,j}$, is calculated as :

$$\begin{aligned} EF_{k,j} &= 0 && \text{for } Q_{k,j} <= Q_{min,k} \\ EF_{k,j} &= Q_{k,j} - Q_{min,k} && \text{for } Q_{k,j} > Q_{min,k} \end{aligned} \tag{6.6}$$

Where $Q_{min,k}$ = the minimum flow in section k.

Then the degree of coincidence, $DC_{v,j}$, of valve v in shift j is :

$$DC_{v,j} = \sum_{k=1}^{K} (EF_{k,j} \times L_k) \tag{6.7}$$

Where K = the total number of sections in the network;

L_k = the length of section k.

The algorithm follows a series of steps which are described below :

1. The valve with the largest discharge requirement is allocated to the first shift.
2. For each remaining valve its degree of coincidence with the allocated valve is calculated and these valves are then ranked in order of increasing coincidence.
3. One valve is allocated to each remaining shift, starting from the valve that has the highest coincidence with the allocated valve and working down in decreasing order of coincidence.

An iterative procedure is now followed, starting with the first shift and then working with each shift in turn :

4. The tolerance, δ_j, by which the total discharge in shift j may deviate from the average shift discharge is set to 0.05 (i.e. 5%).

5. The degree of coincidence of each of the remaining unallocated valves with the valves already allocated to shift j is calculated and the valves are ranked in order of increasing coincidence.
6. An allocation is attempted on the basis of :

   Working from lowest to highest coincidence;

   Keeping the total shift discharge less than $(1 + \delta_j)$ x the average shift discharge.
7. If no allocation was possible, then the value of δ_j is increased by 0.05, and the process returns to step 5.

   Alternatively if an allocation was made and $j>1$, then a backwards checking function is performed as follows :

   For each valve already allocated to a previously completed shift, the possibility is examined of improving the allocations by swapping with the valve that was allocated to shift j in step 6. A swap is made if :

   * the discharges in each shift after the swap remain within the respective allowed tolerances of the average; and
   * the combined degrees of coincidence of the swapped valves with the other valves in the respective shifts is less than for the swapped valves in their original shifts.

   If a swap is made, the backwards checking function continues through the remaining allocated valves, with the last swapped valve in shift j as the candidate for the next swap.
8. If the discharge in shift j is now greater than $(1 - \delta_j)$ x the average, then the shift is fully allocated. The algorithm returns to step 4 and continues for the next shift. If the shift is not yet complete, then the algorithm returns to step 5 and the process is repeated.

The whole process continues until all the valves are allocated.

Although this algorithm yields a good approximation of the optimum as defined by the abovementioned objectives, the design problem is exacerbated by the fact that practical considerations may often override the results of the solution procedure. For example, it may be required to have blocks of the same soil type grouped together in each schedule. Alternatively, the schedule may be dictated by a pesticide spraying program which has an associated drying-off requirement. In the case of manually operated systems, it may not be practical to have valves at distal ends of the network operating together due to the distances between them. In this case it will be preferable to group the valve schedules on a geographical basis, which conflicts with the objective of trying to spread the flow as widely as possible throughout the network.

Consequently it is important to provide the designer with a facility to either allocate the valve schedules himself from scratch, or to manually overide the schedules established by the algorithm. The computer program that has been developed to carry out the valve sequencing provides this facility. It gives a continuously updated display of the discharges in each shift, thereby enabling the designer to evaluate the affect of any changes he might make.

6.3.3 Example of the solution procedure

The network shown in figure 6.2 is a hypothetical example presented for the purpose of illustrating the working of the solution algorithm. The sketch shows a 12 valve network, with the required discharges indicated alongside each valve and the lengths of each pipe shown on each respective section.

The solution is generated for 4 shifts as follows :

* The valve with the largest discharge (valve 9) is allocated to shift 1;
* The degrees of coincidence of the remaining valves with valve 9 are calculated and ranked. For example considering valve 8 with valve 9, these two valves have two pipes sections in common, both of which are 20m long. The minimum flow in these sections will be 100m^3/hr due to valve 9. If valves 8 and 9 are together in the same shift without

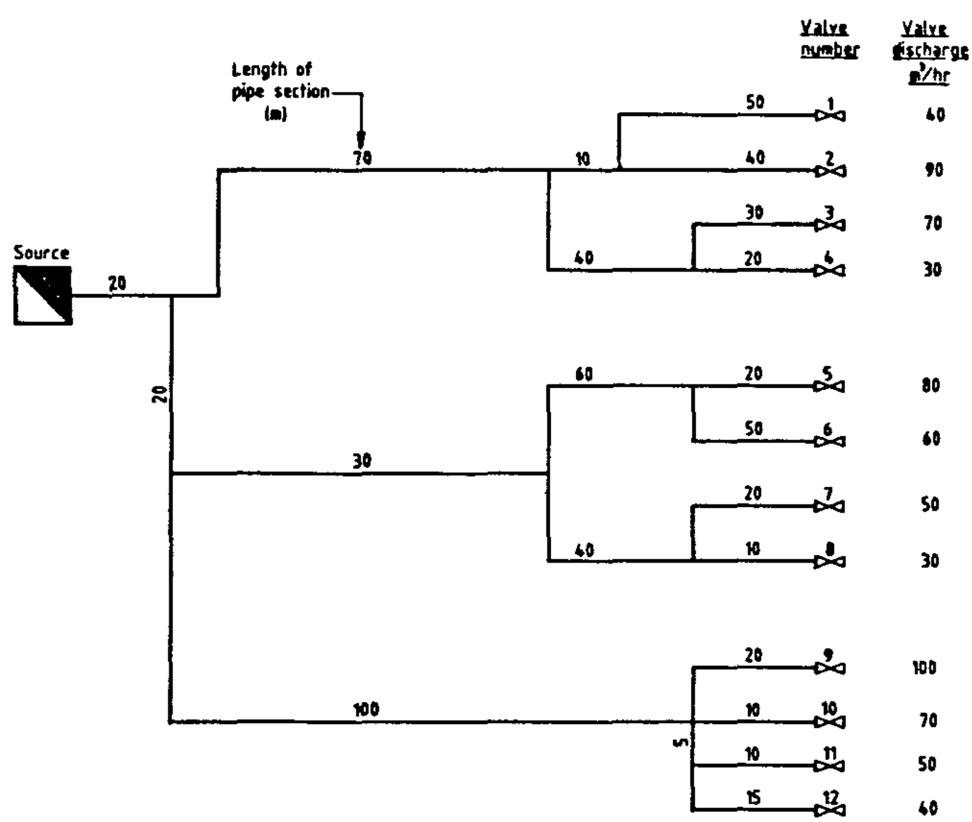


Figure 6.2 : Hypothetical mainline network used in the example of the scheduling algorithm

any other valves, then the flow in these sections will be 30m^3/hr greater than the minimum, and the flows in all other sections will be equal to or less than the minimum. The degree of coincidence is therefore (30x20)+(30x20)=1200;

* Working from highest to lowest coincidence, allocations to the remaining shifts are made as follows :
  shift 2 : valve 10
  shift 3 : valve 11
  shift 4 : valve 12

The total system discharge is 710m^3/hr, which gives an average of 177.5m^3/hr for each shift.

* Starting with shift 1, the allocations made on the basis of minimizing the coincidence level are valves 4 and 1 respectively. At this stage the discharge in shift 1 is 170m^3/hr, which is within 5% of the average, so the allocation is complete.
* In shift 2, valves 8 and 3 are allocated before the shift is full, and after each allocation no improving swaps are found with the valves in shift 1.
* In shift 3, valve 7 is allocated, leaving valves 6, 5 and 2 unallocated. From the point of view of spreading the flow in the network, valve 2 would be the best allocation from these remaining valves. However this would bring the total shift discharge to 190m^3/hr, which is more than 5% greater than the average. The next best allocation, in terms of the flow minimization objective, is valve 6, which has a lower discharge requirement than valve 5. This would bring the shift discharge to 160m^3/hr. Although this is not within 5% of the average, it is less then 5% greater than the average and so passes the second condition of step 6, described above, and the allocation is therefore made. No improving swap can be found.
* The allocation to shift 3 is not yet complete, since the discharge is still less than (1 - δ) x the average. If either of the two remaining unallocated valves are added to shift 3 however, the total discharge will exceed (1 + δ) x the average. No allocation is therefore possible at the current δ level. δ is increased to 10%, which then means that the existing allocation is within (1 - δ) x the average and the shift is considered fully allocated.
* This leaves the overall allocation at this stage as follows :
  shift 1 : valves 9, 4, 1; Q = 170m^3/hr;
  shift 2 : valves 10, 8, 3; Q = 170m^3/hr;
  shift 3 : valves 11, 7, 6; Q = 160m^3/hr;
  shift 4 : valves 12, 5, 2; Q = 210m^3/hr;

  which is not a particularly well optimized allocation, in terms of minimizing shift discharge (shift 4). However, in making the allocations to shift 4, the algorithm finds suitable swaps, which improve the overall allocation. These are valve 10 for valve 5, and valve 6 for valve 2 respectively, leaving the final allocation as follows :

shift 1 : valves 9, 4, 1; Q = 170m^3/hr;
shift 2 : valves 5, 8, 3; Q = 180m^3/hr;
shift 3 : valves 11, 7, 2; Q = 190m^3/hr;
shift 4 : valves 12, 10, 6; Q = 170m^3/hr;

This example illustrates how the algorithm tends to utilize the smaller discharges to fill the first few shifts, leaving the larger discharges, and hence a less flexible situation, for the filling of the last shifts. This is a classic problem with allocation algorithms which can only be alleviated to some extent by the backward seeking function. The algorithm has been tested on a number of real networks, with favourable results, as reported in chapter 8.

6.4 Pump and Pipe Sizing

A trade-off exists in establishing the pipe and pump sizes. The higher the pump pressure (higher pumping costs), the greater the amount of pressure that can be lost through friction in the pipes, and hence the smaller the diameters (and the cheaper the cost) of the pipes needed in the network. By carrying out a present value analysis on the cost of pumping over the expected life of the system, the energy costs can be expressed in terms of current Rands per meter of pumping head supplied at the source. The cost of the pipes is a capital item, which can be expressed in terms of current Rands per meter length of each diameter used in each pipe section. The design problem can then be formulated as an optimization problem, with several alternative methods for its solution. These solution procedures are reviewed below.

6.4.1 Generalized formulation of the optimization problem

In order to discuss the optimization routines further, it is necessary first to outline the conventions that have been developed for the mathematical representation of pipe networks.

Networks are represented mathematically by defining a series of **links** and **nodes**. The links represent the pipelines which connect one node to another, and the nodes represent specifically defined points in the network. These may be points at which the network branches, or other points such as reservoirs, pumps, valves or outlets. **Each link is characterized** by the change in head (energy per unit weight) of the water along its length.

Several empirical formulae have been established to determine the loss of head due to friction in a pipe. A generalized form of these relationships is given in chapter 4 (equation 4.1). **Each node is characterized** by the flow entering or leaving it, either along the various links attached to the node, or due to inputs and consumptions at the node.

Thus, for a network operating at a given time under *steady-state* flow, for which a fixed set of boundary conditions can be specified (i.e. inflows, outflows and fixed heads at specific nodes), two basic types of equations can be formulated to define the flow, *viz* **node and link equations.** The **node equations** are given by the **conservation of flow law** :

For node j : $\sum_{i} Q_{ij} + I_j = 0$ (6.8)

Where Q_{ij} = the flow from node i to node j (for a flow from node j to node i, Q_{ij} will be negative);

I_j = input or consumption (i.e. negative input) at node j;

$\sum_{i}$ = all nodes i linked directly (i.e. via a single link) to node j.

The **link equations** express the total difference in head between the end nodes of a path (b_p) in the network as the sum of head gains and losses (δH_{ij}) in each of the links along the path :

For path p : $\sum_{(i,j)\in p} \delta H_{ij} = b_p$ (6.9)

These equations represent a set of simultaneous non-linear equations expressing the flow and pressure distribution throughout the network. They can be solved iteratively using techniques such as the *Hardy-Cross* and *Newton-Raphson* algorithms.

Solution of these equations enables a full specification of a given flow condition in a network. The solution techniques form the basis of computer "*network solvers*" that have become increasingly widely available. These solvers utilize rapidly converging algorithms (e.g. a modified Newton-Raphson method employing sparse-matrix solution procedures as described by Brailovsky and Rodeh - 1978, and Gofman and Rodeh - 1980), which enable large networks to be solved very rapidly on relatively small computers. The solvers are useful for monitoring and adapting **existing** networks, however they do not provide a process for explicit optimization of the network design.

Several different **design** optimization procedures have been proposed in the literature, utilizing both linear and non-linear methods.

Mandl (1981) formulated a **generalized linear optimization model** for the design problem, which reduces to the *"trans-shipment problem"* for typical pressurized pipe networks. The decision variables are the capacities of each link and external source, as shown below :

The objective function is given by :

$$\text{Min } [\sum_{(i,j)\in A} C_{ij} Q_{ij} + \sum_{j\in A} C_{ja} QP_{ja}] \tag{6.10}$$

Subject to the following constraints :

* The conservation law: $\sum_{\substack{i \\ (i,j)\in A}} \beta_{ij} Q_{ij} + QP_{ja} = \sum_{\substack{l \\ (j,l)\in A}} Q_{jl} + \delta_j$ for all $j\in A$ (6.11)
* $Q_{ij} >= 0$ for all $(i,j)\in A$ (6.12)
* $QP_{ja} >= 0$ for all $j \in A$ (6.13)
* $\delta_j >= 0$ for all $j \in A$ (6.14)

Where Q_{ij} = the flow into node j through pipe i,j (equivalent to its capacity);
QP_{ja} = the flow generated at node j (e.g. by a pump);
Q_{jl} = the flow from node j through pipe j,l (equivalent to its capacity);
δ_j = the flow consumed at node j (e.g. at a sprinkler outlet);
C_{ij} = the unit cost of producing flow in pipe i,j;
C_{ja} = the unit cost of generating flow at node j;
A = the set of all nodes in the network;
$0 < \beta_{ij} <= 1$ = the fraction of water that is not lost on its way through pipe i,j. Typically = 1 in pressurized networks.

Shamir (1974) has proposed a **generalized non-linear optimization** procedure for the design and operation of water distribution networks, incorporating a vector of decision variables which may include either design factors such as pipe diameters or control factors such as heads and flows in the networks :

The objective function is given by :

$$\text{Min } [F(d, u, x, s) = f(d) + \sum_{l=1}^{L} w^l c^l(d, u^l, x^l, s^l)] \tag{6.15}$$

Subject to the following constraints :

* $d \in D$ (6.16)
* $u^l \in U^l$ for all l (6.17)
* $[G^l(d, u^l, x^l, s^l)] = 0$ for all l (6.18)
* $x^l = \{x/[G^l(d, u^l, x^l, s^l)] = 0\} \in X^l$ for all l (6.19)

Where ι = the l th loading condition from the set L;

d = the design variables (pipe diameters, pump capacities) which belong to the set D;

u^{ι} = the operation variables from the set U^{ι} (valves and pumps - on/off) for the l th loading;

x^{ι} = the dependent variables from the set X^{ι} (heads, consumptions) of the l th flow solution;

s^{ι} = the independent (fixed) variable in the l th flow solution;

f = the design cost function;

c^{ι} = the operating cost for the l th loading;

w^{ι} = weighting factors related to the loading;

$[G^{\iota}] = 0$ = a set of simultaneous node continuity equations for the l th loading.

The solution is generated by an iterative, two-stage process. At each iteration, a flow solution for each loading condition is calculated by a modified Newton-Raphson process, and a gradient is then computed from :

$$\nabla F = \begin{bmatrix} \nabla d \\ \nabla u \end{bmatrix} = \begin{bmatrix} \partial \mathcal{L}/\partial d \\ \partial \mathcal{L}/\partial u \end{bmatrix} = \begin{bmatrix} \partial F/\partial d \\ \partial F/\partial u \end{bmatrix} + [\partial G/\partial d; \partial G/\partial u]^T [\ \lambda\ ] \tag{6.20}$$

Where λ = the Legrange multipliers, which are obtained from the solution of :

$$[\partial L/\partial x] = 0 = [\partial F/\partial x] + [\partial G/\partial x]^T [\lambda] \tag{6.21}$$

The step size along the gradient to get to the next iteration is determined by the constraints on d, u^{ι} and x^{ι}.

The examples given above, in particular the non-linear one, have been presented to illustrate the extent of the generality that can be incorporated in formulating the optimization problem. However, several aspects of the design problem as related to irrigation networks render it more specific, and in many ways simpler to solve than in the case of typical domestic water distribution networks. The major factor in this regard is the fact that **irrigation networks are normally of the branching type**, and not looped as in the case of domestic water supply networks. This implies that for a given loading condition the solution of the flows in each of the links is trivial. Furthermore, the network loadings are pre-determined by the valve scheduling process. Thus for a given layout, the optimization problem is reduced to one of selecting the pipe diameters and pumping head that will give the minimum capital and operating costs for the whole network.

Both **linear and dynamic programming** solutions have been proposed for this problem, and these are reviewed below.

6.4.2 *The linear programming (LP) solution*

The **LP** solution to the network design optimization problem has been presented in slightly different forms by Karmeli et. al. (1968), Gupta (1969), and Kinney and Moncrief (1980). It is derived as follows :

The links of a network are defined by the upstream node, i , and the downstream node, j , respectively. Then for each link ij in the network being designed, a set of candidate diameters, m , is to be considered. One set of decision variables is given by the length of pipe for each of the candidate diameters to be used in each link ij. These lengths are expressed as X_{ijm} and they have associated costs given by C_{ijm} per unit length.

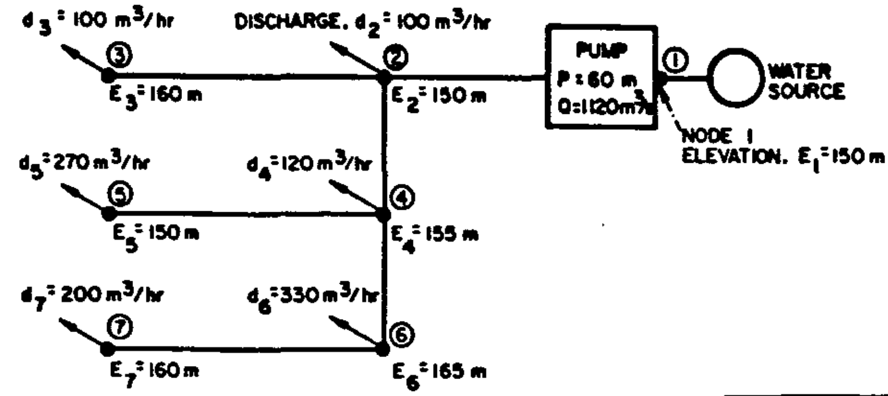
A second set of decision variables is given by the operating head of the pump at the source of the network, for each different loading condition, l. These operating heads are expressed as $XP(l)$. The discharge at the pump for loading l is $QP(l)$, and $k_o(l)$ is the present value of the operating cost of the pump per unit of head and discharge that it delivers, multiplied by : *(a)* coefficients reflecting the units in which the head and discharge are expressed; *(b)* the efficiency of the pump; and *(c)* the fraction of the total pumping time during which loading l is operative. The cost of operating the pump during loading l is given by [$k_o(l)\, XP(l)\, QP(l)$]. Similarly, assuming that the capital cost of a pump is directly proportional to its maximum operating head, XPM, the overall cost of the pump at the source is given by [$k_c\, XPM$], where k_c is the capital cost per unit of head produced by the pump.

These variables are illustrated for a hypothetical network in figure 6.3.

The objective function of the LP is given by :

$$\text{Minimize } [K] = \sum_{ij} \sum_{m} C_{ijm} X_{ijm} + \sum_{l} k_o(l)\, XP(l)\, QP(l) + k_c XPM \tag{6.22}$$

Where
$\sum_{l}$ = the summation of all loadings l;
$\sum_{m}$ = the summation of all candidate pipe diameters m in link ij;
$\sum_{ij}$ = the summation of all links ij in the network;
K = the total (present value) cost of the system.



Link ij	Length (m)	Candidate Diameter 1	2	3
12	1 000	500	450	400
23	1 000	250	200	150
24	1 000	450	400	350
45	1 000	300	250	200
46	1 000	450	400	350
67	1 000	300	250	200

Pipe Cost Dia (mm)	Cost (R/m)
100	16
200	23
250	32
300	50
350	60
400	90
450	130
500	170

Examples of LP variables:

(i) X_{243} = the length of candidate diameter 3 (350 mm diameter) that will be used in link 24.

(ii) C_{243} = The cost of this diameter = R60/m.

(iii) The network has only one loading, $l = 1$, for which QP(1) = 1 120 m³/hr and XPM = 60 m.

(iv) **Pumping costs:**
Capital:
say kc = R200/m of pumping head.
Therefore capital cost = R200 × XPM = R12 000

Operating:
say 4c/Kwhr; 2 000 hrs/year; 1 m³/hr raised 1 m is 0.004 Kwhrs and present value factor for 25 years at 12% is 7.843.

Therefore $k_o(l) = 0.04 \times 2\,000 \times 0.004 \times 7.843$
$=$ R2,51 per m per m³/hr.

So operating cost = 2.51 × 1 120 = R2 811 × XP (1)

(v) If the pressure at node 5 must be ≥ 30 m, then its head loss constraint is given by

$$180 \le 210 - \sum_m J_{12m} X_{12m} - \sum_m J_{24m} X_{24m} - \sum_m J_{45m} X_{45m}$$

(vi) The length constraint for link 12 is given by

$$X_{121} + X_{122} + X_{123} = 1\,000 \text{ m.}$$

Figure 6.3 A hypothetical network illustrating the variables and constraints used in the linear programming solution procedure

Four sets of constraints can be be derived for the LP problem as follows :

1. **The non-negativity constraint.** The decision variables must be non-negative, so that a set of constraints can be stated as :

$$X_{ijm} >= 0$$
$$XP(l) >= 0 \qquad \text{for all } ij,\ l \qquad (6.23)$$

2. **The headloss constraint.** Assume that the head at a specific reference node, s, is fixed and known for loading l, and that either the maximum or minimum (or both) allowable heads at a second node, n, downstream of the reference node are also known for loading l. Then the total headloss in the network along the path from the reference node to the second node, must not result in the head at the second node being either greater than the allowable maximum, or less than the allowable minimum. This is expressed by :

$$HMIN_n(l) <= H_s(l) \pm \sum_{ij}\sum_{m} J_{ijm}(l)\, X_{ijm} <= HMAX_n(l) \qquad \text{for all } l \qquad (6.24)$$

Where $H_s(l)$ = the head plus elevation at the reference node, s, for loading l;

$HMIN_n(l)$; $HMAX_n(l)$ = the minimum and maximum allowable head plus elevation at the second node, n, for loading l;

$J_{ijm}(l)$ = the headloss (or gain depending on the direction of flow) per unit length of pipe of diameter m in link ij under load (flow) l. This is calculated using one of the empirical headloss formulae;

$\sum_{ij}\sum_{m}$ = the summation of all segments X_{ijm} in all links ij along the path from node s to node n.

3. **The length constraint.** The lengths of each diameter of pipe selected for a given link must add up to the length of the link. This is stated by :

$$\sum_{m} X_{ijm} = L_{ij} \qquad \text{for all } ij \qquad (6.25)$$

Where L_{ij} = the length of link ij

4. **The pumping constraint.** Since the operating head of the pump for each loading is a decision variable in the LP, it must be constrained not to exceed the maximum possible head that the pump is capable of producing, XPM. This is stated by :

$$XP(l) <= XPM \qquad \text{for all } l \qquad (6.26)$$

Thus when the LP defined by the objective function and the four sets of constraints described above is solved for a specific network, the solution provides the designer with the pipe diameters and pumping head at the source that should be used so as to give the cheapest possible network that will satisfy the pre-determined hydraulic requirements. It should be noted that the LP solution provides a consistent hydraulic solution for the network at the same time as it optimizes the design. In effect, it solves the **link equations** defined by equation 6.9.

In order to render the LP practically applicable for real problems, a number of additional factors should be addressed :

Source pumps. In formulating the objective function of the LP, two assumptions were made about the capital cost of installing pumps in an irrigation system. Firstly it was assumed that the cost increases linearly with increasing power of the pumps; and secondly it was assumed that the dominant parameter determining the required power is the maximum head required at the pump.

The first assumption is in fact an incorrect one, since the capital cost per unit of power decreases with increasing power, due to the various economies of scale. This would suggest the use of *separable programming*, whereby linear approximations for for separate portions of the cost vs. power curve would be used. However Alperovits and Shamir (1977) have proposed a simple iterative algorithm based on the LP solution. They assume a value for k_c in the objective function and then run the LP. From the solution and available data, they then calculate the actual value of the cost per unit of power, and compare it with the assumed value. If there is a significant difference, then an adjusted value of k_c is assumed and the LP is rerun. The authors report that this process generally converges within 2 to 5 iterations and that its simplicity renders it highly practical.

The second of the abovementioned assumptions implies that the discharge at the pumps will be the same for all loading conditions. Whilst there are several advantages in trying to achieve this for an irrigation network, it is normally not entirely feasible, as discussed in section 6.3 of this chapter. The objective function is therefore only valid when XPM is associated with the maximum power requirement at the pumps, i.e. when :

$$XPM\ QP(l_m) >= XP(l)\ QP(l) \qquad \text{for all } l \qquad (6.27)$$

Where l_m = the loading condition in which the pressure requirement = XPM.

Booster Pumps. Another consideration in relation to the designing of the pumping capabilities for irrigation systems is the possibility of including booster pumps in the network. This problem can be addressed through the LP by assuming possible locations for the boosters and including their operating pressures as decision variables as follows :

If $XP(t,l)$ is the head added by pump number t during loading l, then for any pump in the path from the reference point s to the node n, the headloss constraint (eq. 6.24) becomes :

$$HMIN_n(l) <= H_s(l) \pm \sum_{ij}\sum_{m} J_{ijm}(l)\, X_{ijm} \pm \sum_{t} XP(t,l) <= HMAX_n(l) \quad \text{for all } l \qquad (6.28)$$

Where $\sum_{t}$ = the summation of all pumps, t, in the path from s to n.

And the objective function becomes :

$$\text{Minimize [K]} = \sum_{ij}\sum_{m} C_{ijm}X_{ijm} + \sum_{t}\sum_{l} k_o(t,l)\, XP(t,l)\, QP(t,l) + \sum_{t} k_c(t)\, XPM(\mathrm{t}) \qquad (6.29)$$

In the solution of the LP some of the variables $XP(t,l)$ may be zero for all values of l, which will imply that no pump should be installed at the particular location t. The same analysis applies to the possible inclusion of pressure reducing valves, which are the opposite of booster pumps. This is the reason for the $\pm$ in the $\sum XP(t,l)$ term in equation 6.28.

Candidate diameters. In setting up the LP for a given network, a set of *candidate diameters*, m, is specified for each link ij. In so doing, an *implicit* constraint is introduced into the LP, since the solution cannot include any segments, X_{ijm}, of a diameter m that was not included in the initial candidate set. Alperovits and Shamir (1977) have proposed an iterative procedure that does not limit the diameters to the initial candidate set. Their procedure is based on the fact that as long as the cost of a pipe is a convex function of the diameter (as is normally the case), then the optimum solution of the LP will contain at most two segments for each link, with their diameters being adjacent in the candidate set for the given link. They therefore, in the interests of limiting the number of variables in the LP, select an initial candidate set of only three diameters for each link. This is done purely on the basis of the designer's experience and intuition. The LP is then solved, and the solution is examined for links containing only one diameter which is at either extremity of the candidate set. This would imply that in generating the optimal solution, the LP may have been constrained that no smaller or larger (whichever the case may be) diameter was specified. Wherever this situation exists, the candidate for the affected link is shifted to include the next available diameter in the direction of the possible constraint, and the LP is rerun. This process is continued until the implicit constraint is no longer binding for all links.

6.4.3 The dynamic programming (DP) solution

An alternative **dynamic programming** optimization procedure for the design of branching mainline networks has been proposed by Karmeli et.al. (1968), Kally (1969) and Liang (1971). It is derived as follows :

As was the case for the LP, the *decision variables* are the pipe diameters in each segment and the pump capacities. The *state variables* are the heads at the nodes, and the *stages* of the DP are the nodes themselves. For a pipe of diameter D_k in segment k between nodes j and j+1, where j+1 is upstream of j, the cost of the pipe is given by $g(D_k)$. If a link contains a pump which raises the head from H_{j+1}, at its intake to H_j at its outlet, then the capital and operating costs of the pump can be expressed as a function of these heads, $g(H_{j+1}, H_j)$.

Now for a given head at node j+1, H_{j+1}, the head at node j, H_j, can easily be calculated for a known flow Q_k and for a selected pipe diameter D_k. Thus if we express the minimum cost of the portion of the network downstream of node j, for different discrete levels of the head H_j, as $F^*_j(H_j)$, then the recursive equation of the DP is given by :

$$F^*_{j+1}(H_{j+1}) = \min_{D_k, H_j} \left[g(D_k) + g(H_{j+1}, H_j) + F^*_j(H_j) \right] \qquad (6.30)$$

The minimization is generally carried out for all possible diameters, D_k, for a set of discrete values of H_{j+1} within the admissible range for node j+1. Whenever $H_j = f(H_{j+1}, Q_k, D_k)$ is outside the admissible range for node j, then the examined D_k is discarded. In the case of a pump between nodes j+1 and j, the minimization is carried out over a set of discrete values of H_j.

Shamir (1979) considers the DP solution to be "free of certain shortcomings present in the LP procedure". In particular, it enables, through adequate investigation of alternative D_k's, an optimization encompassing the class and type of pipe to be used, as well as the diameter. However, Shamir points out that the generalized formulation is computationally more involved, and hence less computer-efficient, than the LP, particularly when a highly ramified network is considered.

6.4.4 The proposed solution procedure

In view of the foregoing discussion, the LP solution procedure has been adopted for the proposed design model. A number of modifications to the process described above have been incorporated into the model, and these are discussed on the following pages :

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Candidate diameters/Pipe class. The layout evaluation screening model described in section 6.2.2 generates a first estimate diameter for each pipe section, on the basis of resultant flow velocities. These diameters are used by the optimization model as a starting point for the candidate set.

The full candidate set is made up of the estimated diameter, plus the next two smaller and the next two larger diameters in the data-base. Thus a five diameter set is defined for each section.

In running the model, the onus is left to the designer to check whether the implicit constraint is binding in the solution. If so, a manual adjustment is carried out by changing the estimated diameter in the layout evaluation table and then rerunning the model.

Pumping costs. The proposed model utilizes a composite value of the capital and operating costs of pumping. In other words, the $k_o(l)$ and k_c terms in the objective function (eq. 6.22) are combined into a single term. Also, since the discharge in each shift, $QP(l)$, is known, it is incorporated into this term and the composite cost factor is therefore given in Rands per meter of pumping head. This is derived as follows :

* The following input parameters are specified by the designer :
  1. The capital cost of the pumps per required unit of power, C_c (R/kW).
  2. The number of hours of irrigation per day, *HPD* (Hours).
  3. The number of days per irrigation cycle, *DPC* (days).
  4. The number of cycles per season, *CPS*.
  5. The unit cost of energy, C_e (R/kWh).
  6. The power supply cost, C_p (R/KVA/month).
  7. Fixed energy or power supply costs, e.g. power line extension fee, C_f (R/month).
  8. Analysis period, N (years).
  9. Bank discount rate for present value analysis, I (%).
  10. Rate of inflation of energy costs, r (%).

* A present value factor, *PVF*, is calculated from the analysis period, N, and the interest and inflation rates, I and r respectively. This is done using the relations given in equations 5.10 and 5.11 for the block evaluation model.

  The number of hours of irrigation per season, *HPS*, is given by the product of *HPD*, *DPC* and *CPS*; and assuming an average of 25 irrigating days per month, the number of months of irrigation in a season, *MPS*, is given by *HPS* / (25 *HPD*).

For each shift, a load factor, $lf(l)$, is defined, representing the fraction of the total irrigation time during which shift l is operative.

* The discharge required at the pump for each irrigation shift, $QP(l)$, is known; and the pumping head required for each valve has been estimated by the layout evaluation process described in section 6.2.2. The *estimated* pumping head for each shift, $XP'(l)$, can therefore be taken as the maximum of the required pumping heads for each valve in the given shift. If $QP(l)$ is given in cubic meters per hour, and $XP'(l)$ in meters, then the *estimated* power requirement for each shift, $P'(l)$, is given by:

$$P'(l) = QP(l)\, XP'(l)/367\,\eta \quad \text{(kW)} \tag{6.31}$$

Where η = the efficiency of the pump

And maximum power requirement, MAXP is given by **Max** [$P'(l)$] for all l. This represents the estimated power rating required at the pump.

* The **operating cost component,** $k_o(l)$, is calculated as follows : The estimated seasonal energy consumption for shift l, $EPS'(l)$, is given by :

$$EPS'(l) = (\, lf(l)\, HrPS\,)\, P'(l) \quad \text{(kWh/season)} \tag{6.32}$$

And cost of this energy, $k_e(l)$, is :

$$k_e(l) = C_e\, EPS'(l) \quad \text{(R/season)} \tag{6.33}$$

The "fixed energy costs, C_f" are assumed to be levied for a full 12 months of the year; whereas the "power supply costs, C_p" are levied on the basis of the maximum power load, *MAXP*, each month during the irrigation season. Thus the sum of these two cost factors, for each shift, $k_s(l)$, is given by :

$$k_s(l) = lf(l)\, [\, (MPS\ MAXP\ C_p) + (12\ C_f)\,] \quad \text{(R/season)} \tag{6.34}$$

And finally, the present value of the full operating cost for shift l, over the analysis period, $k_o(l)$, is given by :

$$k_o(l) = PVF\, [\, k_e(l) + k_s(l)\,]\, /\, [\, QP(l)\ XP'(l)\,] \quad \text{(R/m/m}^3\text{ph)} \tag{6.35}$$

* The **capital cost component** apportioned to each shift, $k_c(l)$, is calculated as follows :

$$k_c(l) = lf(l)\ C_c\ MAXP\, /\, XP'(l) \quad \text{(R/m)} \tag{6.35}$$

* The **composite cost** factor, $K(l)$, is then given by :

$$K(l) = [k_o(l)\ QP(l)] + k_c(l) \quad \text{(R/m)} \tag{6.37}$$

The objective function therefore has a single term relating to pumping costs, and the full function is given by :

$$\text{Minimize } [K] = \sum_{ij}\sum_{m} C_{ijm}X_{ijm} + \sum_{l} K(l)\,XP(l) \tag{6.38}$$

Possible error in the composite cost factor stems from the following :

- the assumed linear relationship between pump capital cost and power rating, as discussed in section 6.4.2 above;
- the difference between the *estimated* pumping heads in each shift, $XP'(l)$, and the final optimal values, $XP(l)$, determined by the LP;
- the difference between the *estimated* maximum power requirement, *MAXP*, and the actual value.

These errors affect the capital cost component and the load cost portion of the operating costs. Since the errors due to each of these factors are expected to be small, and the capital cost is generally a small portion of the total pumping cost (typically less than 10%), it is felt that the total error will not significantly affect the results of the LP process.

In the case of booster pumps, the costs are more difficult to establish since no prior estimate of the pumping head of the boosters is available. The booster cost factor is assumed to be directly proportional to the sum of the composite cost factor for all shifts, weighted by the shift load factor $lf(l)$; with the proportionality constant being the product of (*i*) the ratio of the maximum flow through the booster pump to the maximum discharge through the source pump, and (*ii*) the fraction of the total irrigating time during which the booster will operate.

This assumption implies that the principal component of the total pumping costs is the operating cost. This is considered to be valid since the capital costs are typically only a small fraction of the total pumping costs and this will be particularly true for booster pumps which are normally relatively small and therefore do not require expensive installation structures.

Pump optimization. In the case of irrigation networks, the source pump installation often consists of one or more centrifugal pumps connected in parallel, and therefore with limited flexibility of operating pressure. It is therefore common design practice to assume a constant operating pressure in all shifts (determined by the maximum requirement) and to design the pipelines accordingly. For this reason the optimization model has been structured to operate in two alternative modes :

* **Either** optimizing the pumping head separately for each irrigation shift. This is done using the objective function as defined in equation 6.38, where each loading l represents an irrigation shift.
* **Or** optimizing on the basis of a fixed pumping head which is the same for each shift. In this case the objective function becomes :

$$\text{Minimize } [K] = \sum_{ij}\sum_{m} C_{ijm}X_{ijm} + K_t XP_t \tag{6.39}$$

Where $K_t = \sum_{l} K(l)\, lf(l)$ (6.40)

XP_t = the fixed source pump delivery head.

The headloss constraint. The relation given in 6.24 is simplified by removing the maximum pressure condition (RHS of the relation) thereby reducing it to a single constraint. The reference node is taken as the source and each valve in turn is taken as the downstream node. Thus the set of headloss constraints is given by :

$$XP(l) - \sum_{ij}\sum_{m} J_{ijm}(l)\, X_{ijm} <= VP_n - \Delta Z_{s,n} \qquad \text{for all } n \in l \tag{6.41}$$

Where VP_n = the minimum pressure required at valve n;

$\Delta Z_{s,n}$ = the elevation at the source minus the elevation at valve n;

6.5 The Computer Programs

The computer programs for mainline design have been written in standard BASIC for use on MS-DOS or PC-DOS driven personal computers. The programs have been structured into five different modules as follows :

1. **Layout specification,** in which the alternative layouts are specified.
2. **Valve sequencing,** which carries out the valve sequencing algorithm.
3. **Layout evaluation,** which performs the rapid layout evaluation and simultaneously establishes the candidate diameters and pumping costs for the optimization model.
4. **Optimization,** which sets up the input matrix for the LP procedure and then solves it.
5. **Result interpretation,** which converts the results from the LP solution into a meaningful format and then enables the designer to consider the affect of alterations to these results.

The programs are operated interactively from a series of menus. Appendix 1b shows a synoptic map of these menus, illustrating the logical flow of the program operation.

Layout specification. This is done on three separate tables, *viz* :

* **Valves.** Each valve is listed together with its topographic elevation and the required discharge and pressure.
* **Nodes.** These are all points, *other than the valves*, at which the network branches, or at which there is some other change in the network characteristics. The node number is listed together with its elevation.
* **Links.** The input required for each link in the network includes : the start and end points (node and/or valve numbers) ; the length of the link ; and the pipe material. An additional column allows for optional specification of the pipe class. If the class is left unspecified then it will be determined during the layout evaluation calculations.

The tables can be displayed on the screen in any sequence and edited freely by moving the cursor around within the diplay.

Valve sequencing. The valve sequencing option can only be initiated if the layout has been fully specified. At the start of the sequencing process the user will be asked to input the number of shifts into which the valves are to be scheduled. The scheduling algorithm is then initiated and the results are displayed in a table which lists each valve together with the shifts in which it operates. The bottom of the table shows a summary of the total discharge in each shift.

This table can then be edited by the user. In other words, any of the schedules can be changed manually. The summary table showing the shift discharges is automatically updated as any changes are made.

Layout evaluation. The layout evaluation can only be selected if the layout is fully specified and the valve sequencing has been completed. The evaluation is done in two phases, namely calculation of the capital costs and then calculation of the pumping costs.

On initiating the evaluation function, the pipe class and diameter estimation calculations are carried out. The results are displayed in a table showing, for each section :

- the start and end points;
- the section length;
- the pipe material and class;
- the maximum flow in the pipe;

PROJ : GO 824 EVALUATION OF LAYOUT 1 : PIPING COSTS

Sect.	From	To	Length (m)	Material	Max Q (m3/hr)	Max H (m)	Class	Diam (mm)	Cost (R)
1	Srce	V 4	10.0	PVC	47	42	6	160	172.60
2	V 4	V 2	50.0	PVC	47	42	6	160	863.00
3	V 2	V 1	82.0	PVC	14	41	6	63	387.04
4	V 2	V 3	72.0	PVC	44	41	6	140	1 110.24
5	V 3	V 5	82.0	PVC	44	39	6	125	1 116.02
6	V 5	V 6	10.0	PVC	44	38	6	140	154.20
7	V 6	V 7	50.0	PVC	33	38	6	125	680.50
8	V 7	V 8	142.0	PVC	33	37	6	110	1 817.60
9	V 8	V9A	142.0	PVC	27	35	6	110	1 817.60
10	V9A	V9B	25.0	PVC	27	33	4	110	230.75
11	V9B	V10	187.0	PVC	18	33	4	90	1 170.62
12	V10	V11	84.0	PVC	18	32	4	90	525.84
13	V11	V12	84.0	PVC	18	32	4	75	440.16
TOTAL			1,020.0						10 486.17

Figure 6.4 : Results of the layout evaluation process in mainline design (Estimated pipe costs)

- the estimated maximum pressure head in the pipe;
- the selected diameter ; and
- the pipe cost.

An example of this table is shown in figure 6.4. The total length of the network and the estimated cost of the piping are shown at the bottom of the table. The class and diameter values for each section can be edited manually by the user, and the changes in cost due to any such editing will be displayed automatically.

Once this is complete, the user then initiates the pumping cost calculations. The results of this are displayed in a table showing, for each shift :

- the load factor $lf(l)$;
- the estimated pump operating efficiency;
- the estimated power requirement $P'(l)$;
- the estimated seasonal energy requirement $EPS'(l)$;
- the annual cost of this energy at current prices; and
- the product of the operating cost component and the shift discharge ($k_o(l)\ QP(l)$) in Rands per meter pumping head.

An example of this table is shown in figure 6.5.

PROJ : GO 824 EVALUATION OF LAYOUT 1 : ENERGY COSTS

Sched.	Load Factor	Effic. (%)	Power (KW)	Energy (KWHrs/Season)	Seasonal Cost (R/Season)	Discounted Cost (R/m)
1	0.50	75	7	4 832	149	152
2	0.50	75	6	4 227	158	163
TOTAL (MAX) :			7	9 059	297	
CAPITAL/PRES. VAL. COST (R)			2 148		10 563	315

Power Capital Cost (R/KW)	300.00	Load Cost (R/KVA/Mnth) :	15.00
Pumping Hrs/Day :	18.	Fixed Costs (R/Mnth) :	0.00
Pumping Days/Cycle :	5.	Analysis Period (Yrs) :	25.
Pumping Cycles/Season :	15.	Interest Rate (%) :	15.0
Energy Cost (c/KWHr) :	8.	Energy Cost Inflation (%) :	18.0

Figure 6.5 : Results of layout evaluation process in mainline design (Estimated energy costs)

The totals at the bottom of the table include :

- the *MAXP* value;
- the total seasonal energy requirement;
- the total seasonal cost at current prices;
- the total capital cost of the pump;
- the total present value cost of pumping over the full analysis period; and
- the total present value cost of pumping per meter head of pumping.

A second table showing each of the ten pumping cost parameters is displayed beneath the main *results* table.

The load factors and the estimated pump efficiencies can be edited in the main table, and all of the cost parameters can be edited. The costs are automatically recalculated as soon as any changes are made.

Optimization. *The optimization can only be initiated once the layout evaluation has been* completed.

At the start of the process the user is asked to specify whether the pumping head is to be optimized on a per schedule basis, or on the basis of a fixed head for all shifts. An input table is then provided for specifying the booster pump data.

Once this is complete, the computer continues, without input from the user, preparing the initial solution matrix for the LP and then solving the LP.

Result Interpretation. Once the LP process is complete, the computer translates the results from the solution matrix into a more meaningful format. The results are displayed in two tables. The **first table** shows for each section, the selected diameters and their corresponding lengths and costs. Totals are given for each section and for the whole network. A subsidiary table provides a summary of the pipe costs by diameter. The **second table** shows a summary of the operating pressures at each valve, in each shift.

The length and diameter selections can be edited manually by the user. If any changes are made, the various totals and the pressures table are updated automatically.

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PART 3 :

APPLICATIONS

7. APPLICATIONS OF THE BLOCK DESIGN AND EVALUATION MODELS

7.1 Introduction

Figure 7.1 shows a 0.2Ha apple orchard in the Grabouw region (Western Cape, South Africa), that has been used as a test case for the design and evaluation models. A number of different design cases are analysed in order to demonstrate the capabilities of the design models.

The orchard is on a hill-side, and has been laid out with the manifold running down a steep ridge (27% slope), and the laterals, on either side of the manifold, running down more gentle slopes (±4% in Quad *a* and ±8% in Quad *b*). Irrigation is by micro-sprayers, with a spacing of 2m along the laterals and 3.5m between the laterals.

Most of the rain in the area falls in winter, with highly variable rainfall occurring during the summer growing season. The average seasonal irrigation requirement is 490mm. However, in a year of one standard deviation less than average rainfall, the irrigation requirement increases to 740mm. Consequently, irrigation systems in this area are generally designed for a higher peak requirement than the average, but the question of the most appropriate design peak is unresolved.

The farmer irrigates the entire farm on a weekly cycle, using four 6-hour shifts per day, six days per week. All the orchards on the farm have been standardised on a sprayer delivering 49lph at 150kPa pressure, with full wetting. For the given spacings, this implies a gross application rate of 7mm/h, giving a maximum gross application of 42mm in a 6-hour shift. The irrigation system is operated by a time-based automatic controller.

Irrigation scheduling is normally done on the basis of well tried rules of thumb, for various periods in the growing season. The farmer knows that each 1mm of required irrigation needs a setting of 8.5 minutes on the controller. The forthcoming week's requirement is set accordingly, every Monday. Efforts have been made to improve scheduling practices in the region through experimentation to produce appropriate, local crop factors for use with weekly climatic data from a centrally located weather station.

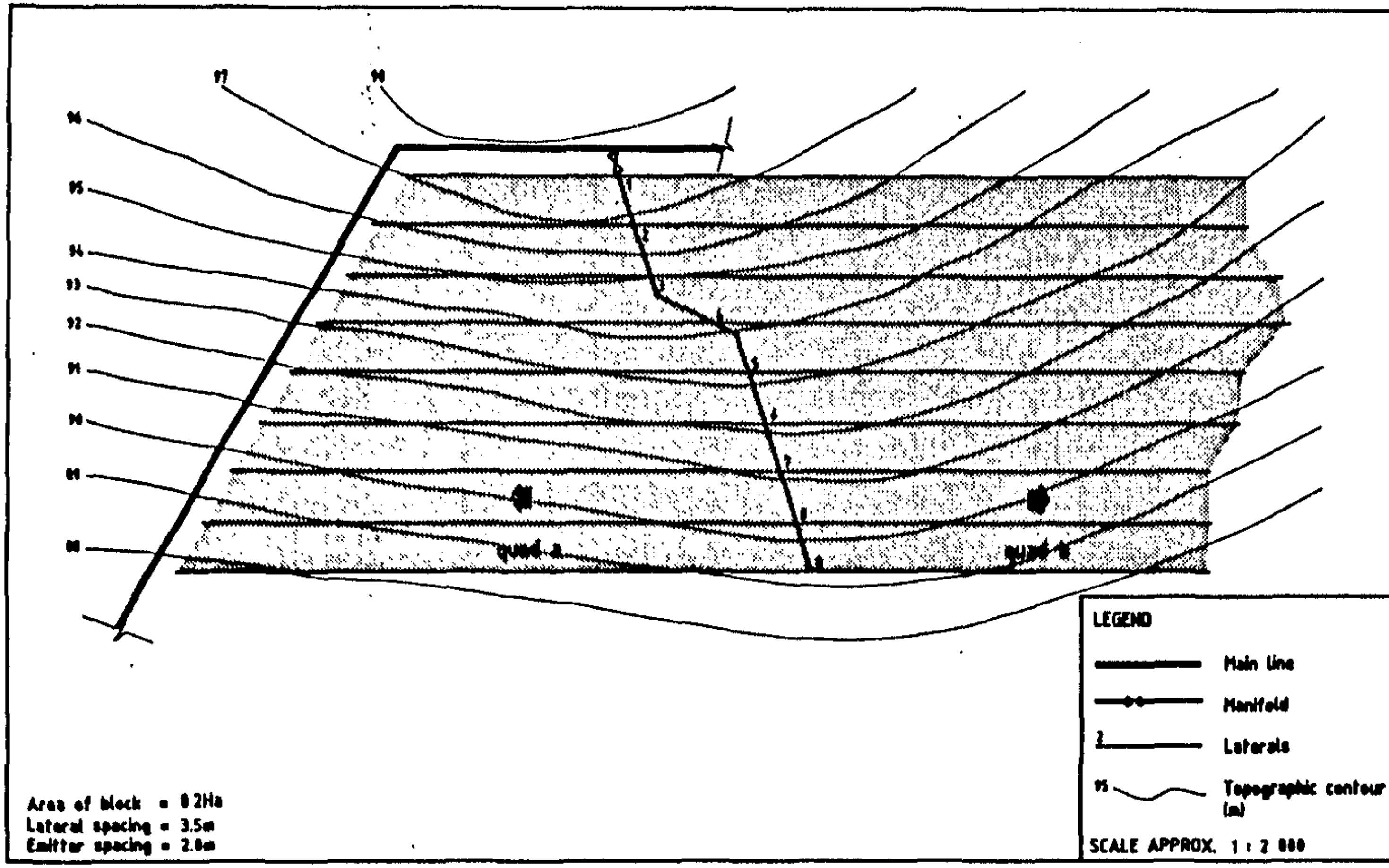


Figure 7.1 Layout of micro irrigation scheme on Grabouw Orchard

7.2 Basic Design

In a moderately dry year, the average weekly irrigation requirement throughout the season, based on historical climatic data, can generally be categorized into four periods as shown in table 7.1 below. It can be seen that if evaporation losses are around 5%, then the existing 42mm/week gross system capacity will supply exactly the 40mm **nett** requirement.

Table 7.1 Irrigation requirements for apples in Grabouw (South Africa) (in a moderately dry rainfall season)

	Nov/Dec (6 weeks)	Dec/Jan (6 weeks)	Feb (4 weeks)	March (4 weeks)	Total (20 weeks)
Weekly requirement (mm)	25	40	30	20	
Total for period (mm)	150	240	120	80	590

The design was carried out on the computer as follows :

7.2.1 Design Parameters

The base set of design parameters was as shown in table 7.2 below :

Table 7.2 Design parameters for Grabouw apple orchard

Emitters	
Type	D & D 1.1mm GT-JET micro sprayer
Coefficients of pressure flow relationship :	k=11.34 x=0.54
Operating Flows and Pressures	
Nominal	49lph @ 151kPa
Upper tolerance	10% (167kPa)
Lower tolerance	10% (136kPa)
Maximum lateral inlet pressure	157kPa
Lateral Piping	
Material	Low density polyethylene, class 3
Minimum diameter	12mm
Minimum length/section	0m
Manifold Piping	
Material	Low density polyethylene, class 3
Minimum diameter	20mm
Minimum length/section	0m

As can be seen in the above table, the allowable pressure variation was specified as the traditional 20%, which gave an absolute allowable variation of 167-136=31kPa. This was divided roughly 2/3 in the laterals and 1/3 (10kPa) in the manifold.

7.2.2 Design results

The results of this design, as produced by the computer, are shown in table 7.3 below :

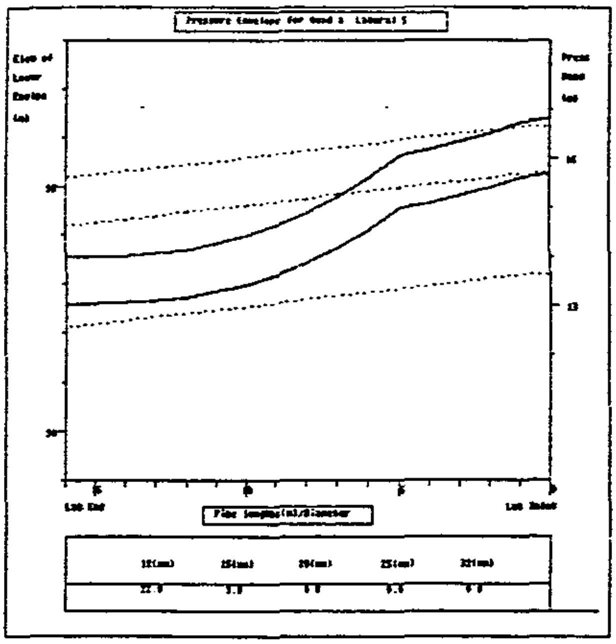
Table 7.3 Results of design of Grabouw apple orchard (20% allowed pressure variation)

Lat #	Lateral lengths (m) 12mm	15mm	Cost (R)
	Quad a		
1	15.0		7.75
2	17.0		8.75
3	21.0		10.75
4	24.0	3.0	14.20
5	22.0	9.0	17.10
6	22.0	11.0	18.40
7	20.0	17.0	21.30
8	18.0	21.0	22.90
9	14.0	29.0	26.10
	Quad b		
1	28.0	13.0	22.70
2	28.0	13.0	22.70
3	26.0	15.0	23.00
4	28.0	9.0	20.10
5	26.0	7.0	17.80
6	28.0	3.0	16.20
7	26.0	3.0	15.20
8	27.0		13.75
9	27.0		13.75
	Totals		
	417.0	153.0	312.45

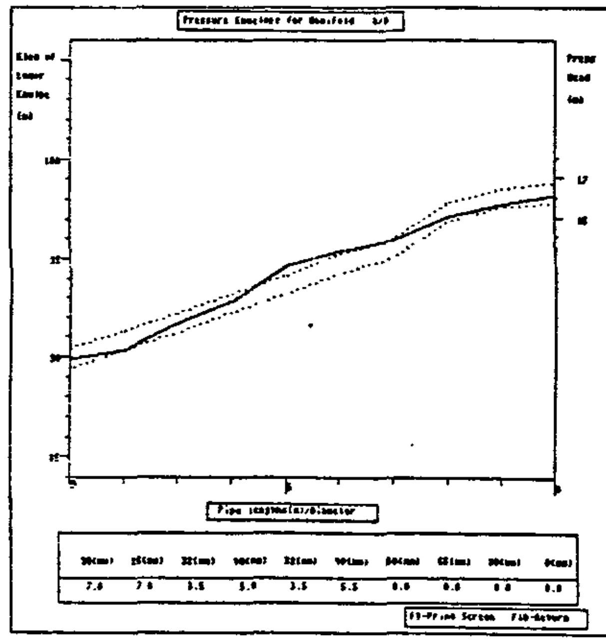
Manifold lengths (m) (0m min length/section) 20mm	25mm	32mm	40mm	32mm	40mm	Cost (R)
7.0	7.0	3.5	5.0	3.5	5.5	31.29

Graphical plots produced by the computer as part of the design process, are shown in figure 7.2. The figure shows the pressure profiles for one of the designed laterals and for the manifold design in two different cases, which are discussed further below.

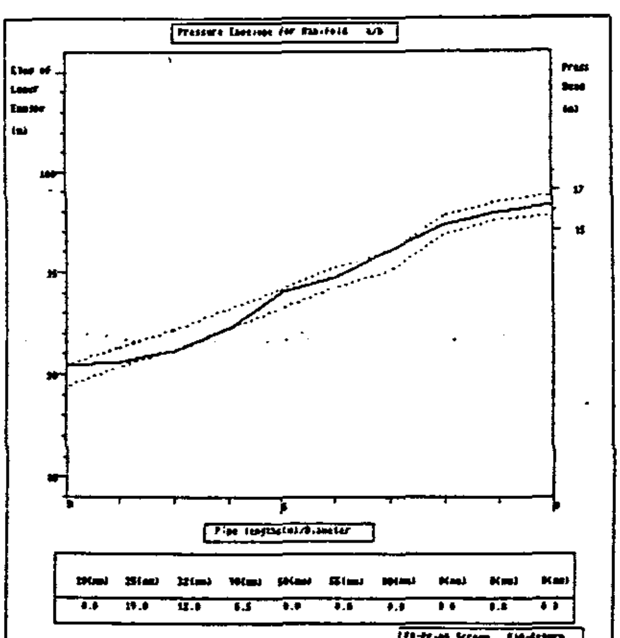
As discussed in chapter 4 (section 4.3.3), two design curves are established for each lateral, in order to calculate coefficients of the pressure/discharge relationship for use in the manifold design. The two curves for lateral 5 in Quad *a* can be seen in figure 7.2a. the second curve is established by moving the end pressure up by an amount equal to the width of the manifold pressure variation envelope (10kPa in this case) and calculating the pressures on a point to point basis back to the inlet. The fact that the second curve moves up above the upper envelope boundary as it gets towards the inlet, as seen in the figure, is indicative of the difference in head losses in the two curves due to the different operating pressures of the emitters. This gives an indication of the error in assuming a constant discharge from each emitter along the lateral.



a) lateral design



b) manifold design
(0m min. length/section)



c) manifold design
(10m min. length/section)

Figure 7.2 Graphical output of results from the computer design model

The design of the manifold is of interest, since it is representative of a number of notoriously difficult factors. Firstly, manifolds are generally more difficult to design than laterals because of their narrower allowable pressure envelopes; secondly this manifold is on a particularly steep slope; and thirdly the hydraulic grade line has a characteristic "kink" between outlets 2 and 3 due to this section being on a steeper gradient than the rest of the manifold (see figure 7.2 b & c).

It can be seen from the results that the designed manifold starts at the inlet with a 40mm diameter pipe, and then decreases to a 32mm "choke" over the steeper section between inlets 2 and 3, in order to counteract the effects of increased head gain due to the steeper topography. As the manifold flattens out, the diameter is increased back to 40mm, and it then "telescopes" normally down to 20mm at the last outlet.

It can be seen further in figure 7.2b and c that the manifold pressure moves out of the envelope boundaries over part of its length. This is a typical result for manifolds on steep slopes, caused when the design routine falls into the cycling condition described in section 4.3.2 of chapter 4. In order to resolve the cycling problem, the design algorithm temporarily widens the pressure envelope by defining a tolerance amount by which the curve may move beyond the envelope boundaries. Once the cycling has been bypassed, the tolerance is reset to zero.

During the design process, each step of the algorithm is summarized in the synopsis file, as discussed in section 4.4.3 of chapter 4. This file is written onto the computer data disk, so that it does not interfere with the results seen on the screen. However, if the designer wishes to review the file, it can be popped-up over the design screen or printed out. Table 7.4 shows extracts from this file for the manifold design.

Case 1 in this table shows the design described in tables 7.2 and 7.3 and shown in figure 7.2b. With reference to table 7.4, the steps of the design process are as follows :

* Starting in step 1, with section 1, a 20mm pipe, at outlet number 9 (the closed end of the manifold) and an assumed pressure of 167kPa (the top envelope boundary value). At outlet 8, in step 1, the pressure is 162kPa, and at the next point the pressure goes back up to 167kPa. A "down shift" is indicated;
* The process starts again in step 2, after the shift, with a starting pressure of 162kPa. This time the calculations go through to outlet 7, where the pressure has come back up to 162kPa. The pressure at the next point will be 180kPa. Since this is above the upper envelope limit, a change to a larger diameter ("Chng Up") is indicated;

* Step 3 shows that a 25mm pipe has been selected for section 2, and the calculations continue from point 7, where the previous section ended. The first action is to check that the previous section is longer than the specified minimum length;
* Step 4 shows that the new section runs out of the top boundary almost immediately, so that an adjustment is indicated;
* The curve for section 2 is moved back 1 outlet, to outlet 8, which means that section 1 must be re-checked for the minimum length criterion (step 5);

Table 7.4 Extracts from the design synopsis file

Case 1 : 0m minimum length/section

DESIGN OF MANIFOLD a/b Date : 28/01/87

STEP #	SECT	DIA(mm)	Start Pt & P		Current Pt & P		NEXT Pt & P		ACTION	TOLERANCE
1	1	20	9	167	8	162	7	167	Shft Dn	(0.00)
2	1	20	9	162	7	162	6	180	Chng Up	(0.00)
3	2	25	7	162					ChkMinLen	(0.00)
4	2	25	7	162	6	163	5	171	Adjust	(0.00)
5	2	25	8	157					ChkMinLen	(0.00)
6	2	25	8	157	8	157	7	153	Chng Dn	(0.00)
7	3	20	8	157					ChkMinLen	(0.00)
	Minlen failed: length = 0.0									
8	1	20	9	167	8	162	7	167	Shft Dn	(0.05)
9	1	20	9	162	7	162	6	180	Chng Up	(0.05)
--										
11	2	25	7	162	5	171	4	187	Chng Up	(0.00)
--										
--										
17	4	40	4	168	3	165	2	152	Chng Dn	(0.00)
18	5	32	3	165					ChkMinLen	(0.00)
19	5	32	3	165	2	160	1	169	Chng Up	(0.00)
20	6	40	2	160					ChkMinLen	(0.00)
21	6	40	2	160	0	161			Design Done	(0.00)

Case 2 : 10m minimum length/section

DESIGN OF MANIFOLD a/b Date : 28/01/87

STEP #	SECT	DIA(mm)	Start Pt & P		Current Pt & P		NEXT Pt & P		ACTION	TOLERANCE
--										
13	2	32	5	165	4	162	3	167	Chng Up	(0.00)
14	3	40	4	162					ChkMinLen	(0.00)
	Minlen failed: length = 3.5									
15	2	40	5	165	4	157	3	155	Chng Dn	(0.00)
16	3	32	4	157					ChkMinLen	(0.00)
	Minlen failed: length = 3.5									
17	2	32	5	165	4	162	3	167	Shft Dn	(0.05)
18	2	32	5	165	3	167	2	162	Shft Up	(0.00)
19	2	32	5	165	2	162	1	170	Shft Dn	(0.00)
20	2	32	5	165	2	162	1	170	Chng Up	(0.00)
--										
--										

* From outlet 8, the curve for section 2 runs straight out below the bottom envelope, indicating a change back to a smaller diameter (step 6);
* The 20mm pipe is therefore re-introduced at outlet 8 for section 3. Section 2 is checked against minimum length and fails since its length is actually zero (step 7);
* The process is back at the same point that it was at in the first step and a cycling condition has occurred. The tolerance is set at 5% and the process starts over from the point of cycling (step 8);
* Steps 8, 9 and 10 are a repeat of steps 1, 2 and 3. However, in step 11, section 2 now gets through to outlet 5 because of the tolerance which allows the curve to move 5% above the upper boundary. The tolerance is reset to 0, and the process continues with a change up to a larger diameter;
* Steps 17 to 20 show the process of inserting the 32mm choking section between outlets 2 and 3;
* The process ends at step 21, when the calculations get to the pipe inlet within the pressure envelope.

The manifold was designed a second time with a stipulated minimum length of 10m for any section. The result of this design is shown in figure 7.2c and in table 7.5 below:

Table 7.5 Alternative manifold design of Grabouw apple orchard

Manifold lengths (m) (10m min. length/section)						Cost
20mm	25mm	32mm	40mm	32mm	40mm	(R)
	14.0	12.0	5.5			30.62

As expected, in this case the design algorithm rejects the 20mm section at the end of the manifold and starts with a 25mm section. The envelope is opened 5% at the start of the process in order to get through with 14m of the 25mm pipe. At this stage the process cycles again, this time over outlets 5 to 3 with 32mm and 40mm pipe sections, as shown in case 2 in table 7.4 (steps 13 - 16). The tolerance is set to 5% and the 32mm section then gets through to outlet 2. Note that the curve for the 32mm pipe is sufficiently steep going through outlets 3 and 2 so as to obviate the need for the choking section. Step 18 shows that a shift up was indicated as the curve came into the kink at outlet 3. This is equivalent to *case c* in figure 4.3 of chapter 4. This is followed by a shift down at the end of the kink in step 19 (*case b* in figure 4.3). In both these cases however, the envelope was so narrow that any shift would have taken the curve outside of the envelope, and consequently no actual shift was affected.

Thus the 10m minimum length stipulation has actually provided a simpler design, that is also marginally cheaper, than the design for no minimum length. Also, the second case design moves out of the envelope only fractionally at outlets 3 (above the envelope) and 7 (below the envelope), whereas the first case design is considerably above the envelope at outlet 5.

7.3 Evaluation

7.3.1 Uniformity analysis

The results of the uniformity evaluation of the design, using the second case manifold (10m minimum length), are shown in table 7.6 below :

Table 7.6 Results of uniformity evaluation of Grabouw apple orchard (20% allowed pressure variation)

Inlet Pressure (kPa)	130	162	178	200
Inlet Flow (m3/h)	13.04	14.30	14.75	15.75
Uniformity (%)	98.1	97.8	97.6	97.2
Ave q (*l*ph)	44.2	48.6	50.2	53.4
Ave q/q nom	0.90	0.99	1.02	1.09
q min (*l*ph)	42.2	46.4	47.8	50.6
q max (*l*ph)	47.3	51.5	53.4	57.3
q variation (%)	11.1	10.5	11.2	12.5

The second column of table 7.6 shows the results for the required inlet pressure determined by the design. It can be seen that the 20% allowed pressure variation resulted in a 10.5% variation in discharge throughout the block. This is as expected from the emitter, which has an x exponent value in the pressure/discharge relationship of 0.54 (table 7.2). At 162 kPa the uniformity coefficient for the designed field is 97.8. This value increases with decreasing input pressure. However, at 130kPa the average emitter discharge is 44.2*l*ph, which is well below the nominal value.

Evaluation of the first case manifold produced very similar results. However the discharge variation was higher than for the second case manifold (11.6% at the design pressure) due to the high pressures occurring over the section between outlets 4 to 6.

7.3.2 Economic analysis

The input data used for the economic evaluation are shown in table 7.7 :

* These data were taken from detailed records, maintained collectively by a group of farmers in the area, and are considered to be accurate.

* The capital costs of the overall irrigation system are R 3162.54/Ha, of which about 2/3 are actual in-field system material costs (including an estimated additional 10% for "other materials" e.g. fittings). Note that all manifold costs are apportioned on a per hectare basis.

Table 7.7 Summary of the input parameters for the economic evaluation of Grabouw apple farm

Design Data

1) Capacity/Operating regime :

Average emitter discharge	*(48.6 lph)	Gross application rate	(6.9mm/h)
Emitter spacing	(2.0 x 3.5m)	Gross system efficiency	96%
Total available water	800mm	Nett application rate	(6.7mm/h)
		Maximum irrigating time/set	6hrs
		Maximum application/cycle	(40mm)

2) Area :

Area of block	(0.2Ha)	Area of scheme	5Ha

3) Agronomic data :

Maximum crop yield 53tons/Ha

Period	Nov/Dec	Dec/Jan	Feb	March	Total
Crop water requirement (mm/cycle)	25	40	30	20	(590)
Duration of period (cycles)	6	6	4	4	(20)
ky value for each period	0.8	0.8	0.8	0.8	

4) Application fractions for operation point optimization : 0.90 1.00 1.05 1.10 1.15

Economic Data

1) Capital costs :

Material costs	
-block piping and emitters	(R 343.07)
-other block materials	R 34.30
-estimated mainline cost	R 2500.00
Installation costs	
-block	R 100.00
-mainline	R 500.00
Other capital items (fees,etc)	R 50.00
Total capital costs	(R 3162.54/Ha)

2) Maintenance costs :

Block	5% (R 91.68/Ha/yr)
Mainline	3% (R 15.00/Ha/yr)

3) Fixed production costs : R 2500/Ha/yr

4) Yield related production costs : R 75.00/ton

5) Operating costs :

Energy cost	5c/kWh
Estimated headloss in mainline	200kPa
Pump delivery pressure	(362kPa)
Overall energy cost	(0.66c/m^3)
Water base cost	10c/m^3
Water opportunity cost	0c/m^3

6) Discounted cash flow analysis :

Period	15years
Discount rate	13%
Inflation rates	
-energy	18%
-production costs	18%
-earnings	14%
-general	19%

7) Producer price to farmer : R 253.00/ton

*All values within ( ) are calculated by the computer from the design and other input data.

* Water for the farm is drawn from a storage scheme developed jointly by the farmers of the region, for which they are paying interest and redemption on capital loans. Hence the relatively high cost of the water which completely over shadows the energy costs.
* Over the past 4-5 years, the farmers have experienced a rapid escalation in production costs, that has not been matched by equivalent rises in earnings. Their predicament is reflected in the rates used for the discounted cash flow analysis.
* In the absence of any detailed information about the yield/water relationship for apples over individual periods in the growing season, a constant value of the crop response factor (*ky*) has been assumed throughout the season.

The results of this analysis are shown in figure 7.3 below.

GRABOUW APPLES 28/01/87
EVALUATION OF BLOCK (3 :) PAGE 6 ANALYSIS FOR CU = 98% : AV q = 48 l/hr

DEPTH (mm)	530	590	604	625	640
VOL ('000 m3)	1.06	1.18	1.21	1.25	1.28
YIELD					
Tons/ha	48.6	52.5	52.8	52.9	52.9
kg/m3	9.2	8.9	8.7	8.5	8.3
EFFICIENCY					
Applic - Ea (%)	99	98	96	93	91
Reqmnt - Er (%)	89	98	99	99	99
EQUIV.ANNUAL WORTH					
EAW (R/ha)	7348	8607	8672	8604	8549
EAW (R/m3)	1.39	1.46	1.44	1.38	1.34

OPTIMAL OPERATING POLICY DURING SEASON

PERIOD	1	2	3	4	5	6	7	8	TOTAL
Reqmnt (mm/cycle)	25	40	30	20					590
Applic (mm/cycle)	26	40	31	21					604

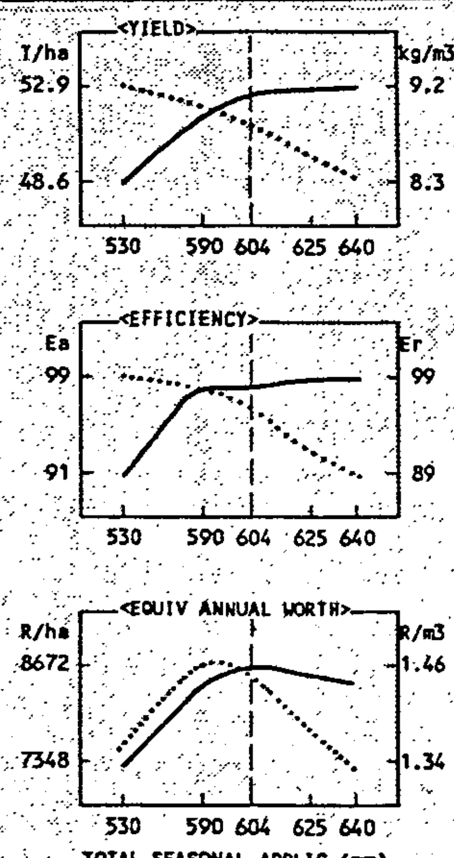


Figure 7.3 : Results of economic analysis of Grabouw apple farm

The following conclusions can be drawn from these results :

* The main table of results shows five different seasonal applications. The column with the depth equal to the optimal depth, shows results for an irrigating regime based on the optimal policy determined from the the solution of the dynamic programming exercise and shown in the second table, below the main table. The other four columns show results for irrigating at a fixed application ratio (*AR*) throughout the season, unless this ratio entails exceeding the maximum system application. Thus for example, the 604mm seasonal application shown in column three is derived from applications of 26, 40, 31 and 21 mm/cycle during each of the four periods respectively, whereas the 530mm application is derived from a constant *AR* of 0.9 throughout the season. The 625mm result is derived from *AR*=1.10 for periods 1, 3 and 4, and *AR*=1.00 (i.e. the maximum possible) in period 2.

* The yield per unit of land increases as the application is increased, up to a maximum of 52.9 tons/Ha. This is reached when the requirement efficiency is 100% in periods 1, 3 and 4 (at *AR*=1.10 for these periods). The maximum possible yield of 53 tons/Ha cannot be attained with this system in a moderately dry year since the peak requirement of 40mm/week during December/January cannot be exceeded by the system. The yield per unit of water decreases exponentially as the seasonal application is increased.

* The return on investment is maximal at a seasonal application of 604mm. However the maximum return per unit of water occurs at an operating point of 590mm/season. It is interesting to note that the maximum return is not derived from operating the system to produce maximum yield. The marginal costs of increasing the seasonal application from 604mm to 625mm outweigh the extra income from the increased yield.

7.4 Sensitivity Analysis

A number of alternative economic analyses were generated by varying selected parameters. The results of these analyses are summarized in table 7.8, and can be grouped as follows :

a) specific variations of the **data-input** parameters, and

b) variations of the **design** parameters.

In each case, only the specified parameters were changed, and all other parameters were restored to their original base case values.

7.4.1 Alternative Data-Input Parameters

* **Operating and production costs.** These affect the irrigation system in two ways. **Firstly,** the overall profitability of the scheme is dependent on the ratio between yield related production costs (e.g. packaging) and income, and the rate at which this ratio is either increasing or decreasing. **Secondly,** the optimal operating depth is dependent on the marginal costs of increasing the yield through increased water application. These two aspects were examined in alternatives *1* and *2* in table 7.8 :
  1) The producer price was reduced by 10% to R228/ton; the yield related production costs were increased by 10% to R82.50/ton; and the difference between the rate of inflation of production costs versus the rate of inflation of earnings was increased from 4 to 6 percentage points. This resulted in a negative *EAW* at all depths, the minimum loss being R89/Ha at a seasonal depth of 604mm. This depth also coincided with the depth at which the loss per cubic meter of water was minimized.
  2) The most sensitive of the operating costs was found to be the water cost. An analysis was therefore run considering the situation if, for example, the farmer had an alternative use for the water which paid him a premium of 15c/m^3 over and above the purchase price. This was input into the model by specifying the 15c/m^3 as a water opportunity cost. In this case the operating point giving a maximum *EAW/Ha* came down to 596mm, thus narrowing the gap between operating for maximum *EAW/Ha* and operating for maximum EAW/m^3. The result implies that for any application greater than 596mm, the marginal return would be less than could be realised through using the required water for the alternative purpose.

* **System capacity.**
  3) This analysis considered the question of what the optimum depth would be if the system, as designed, could apply more than 40mm/week. This was done by increasing the time available per irrigation shift to 8 hours, thereby increasing the capacity of the system to 53mm/week. The results in table 7.8 show that under these circumstances, the maximum return per unit of land is reached when the maximum yield is attained (requirement efficiency is 100%). This is achieved through a constant *AR* of 1.05 throughout the season. The increase in *EAW* from the base case is R18/Ha, which is equivalent to a nett present value of R120/Ha. This represents the amount that can profitably be spent on the irrigation system in order to increase its capacity to meet the required 42mm/week.

* **The water/yield relationship.** As discussed in chapter 5, the yield response factor (*ky*) varies during the growing season. Four distinct periods during the season can be

identified, *viz.* **vegetative growth, flowering, yield formation and ripening.** Typical *ky* values for each of these periods would be 0.6, 1.1, 0.7 and 0.2 respectively. In other words, the plant is typically more sensitive to water deficits during flowering and yield formation than during the other two periods.

4) The model was rerun using the abovementioned *ky* values, on the assumption that the four periods defined for the Grabouw region correspond to the four growing periods. The results for this alternative have the optimum seasonal application reduced to 600mm with only a small reduction in the *EAW* compared with the base case. This is due to the low *ky* value during the last period. It implies that the loss of yield due to under-irrigation in this period is small, and consequently the 5% excess irrigation during this period, suggested in the base model, is no longer economically advantageous.

* **Total available water.**

5) The model was then rerun using the new *ky* values, but this time with only 550mm total available water. In other words, the system was forced to operate under deficit (93%) irrigation conditions. This situation can occur in the area since the total water allocation from the storage scheme is limited and most of the farmers have their own supplementary storage, for which they rely on good winter rains. This condition provides the truest test of the **dynamic programming model** as an allocation optimization process. The results show that a yield of 51.0 tons/Ha can be achieved, giving an *EAW* of R8171/Ha. The optimal operating policy maintained a full irrigation in period 2 and only a slight reduction in period 3. As expected the greatest reduction in the application was allocated to period 4.

For comparison, the model was run with a forced constant $AR = 0.93$. This resulted in a yield of 50.6 tons/Ha and an *EAW* of R8030/Ha. Thus the regime proposed by the optimization model does give an improvement over the fixed *AR* regime.

* **Irrigation requirement.** The final analysis in this group was done for an assumption of the system, as designed, operating in an average rainfall season. The total seasonal requirement in this case is 490mm, made up from peak requirements of 20, 35, 25 and 15mm for each of the four periods respectively.

6) The results for this case are similar to those of case *3*. The optimum operation under these circumstances is with a constant *AR* of 1.05, in order to produce the full 53 tons/Ha. This results in a maximum *EAW* of R9061/Ha, which is considerably higher than the maximum in the base case, due to the reduced water requirement. The reduced requirement also has the affect that the application for maximum EAW/m^3 coincides with the application for maximum *EAW/Ha*.

Table 7.8 Summary of results of sensitivity analyses on the evaluation

	Unif. Coef (%)	Captl. Cost (R/Ha)	Opt. Opertng. Pt. Depth (mm)	Yield (t/Ha)	EAW (R/Ha)	Opt. Opertng. Policy mm/week/period 1	2	3	4	Depth for Max EAW/m³
BASE CASE RESULTS :	97.8	3163	604	52.8	8672	26	40	31	21	590
ALTERNATIVE DATA-INPUT PARAMETERS :										
1. Higher cost/earning ratio			604	52.8	-89	26	40	31	21	604
2. 15c/m³ water opport.cost			596	52.7	5566	26	40	30	20	590
3. 53mm/week capacity			616	53.0	8690	26	42	31	21	590
4. Changed ky values			600	52.8	8659	26	40	31	20	590
5. Limited available water			550	51.0	8171	23	40	28	15	550
6. Average requirement			516	53.0	9061	21	37	26	16	516
ALTERNATIVE DESIGN PARAMETERS :										
7. Smaller system, dry year	97.4	3092	582	37.1	2789	26	35	33	21	582
8. Smaller system, ave. year	97.4	3092	504	51.3	8478	21	35	26	16	504
Analysis over 15 years :										
9. 40% pressure variation	96.1	3083	604	52.7	8633	26	40	31	21	590
10. 30% pressure variation	96.8	3113	604	52.7	8652	26	40	31	21	590
11. 10% pressure variation	98.6	3242	604	52.9	8673	26	40	31	21	590
Analysis over 5 years :										
12. 40% pressure variation					6871					
13. 30% pressure variation					6879					
14. 20% variation(base case)					6885					
15. 10% pressure variation					6872					

7.4.1 Alternative Design Parameters

* **System capacity**: The possibility of designing the system for a lower capacity was considered, the rationale being that the resultant loss of production might be offset by the reduced capital cost of the system. This was done by specifying a sprayer that delivers 43l/ph at 150kPa (also with full wetting), giving a maximum nett application in 6 hours of 35mm.

  7) The new system was designed and then evaluated for the base set of input parameters. The results show that the saving in the capital cost of the system was only 2%; and that in a moderately dry year, the loss in production and subsequent earnings will be substantial.

  8) An alternative evaluation of the designed system was carried out using the irrigation requirements in an average year (as specified for case 6). Under these circumstances the system gives reasonably good results, but the *EAW/Ha* is still less than that achieved from the larger system.

* **Allowable pressure variation.** The question of the most appropriate allowable pressure variation to be used for design (the *20% rule* debate), was analysed by designing and evaluating a number of systems using different values of this parameter.

  9) 40% total allowable pressure variation;

  10) 30% total allowable pressure variation;

  11) 10% total allowable pressure variation.

  Note that the base case was designed using 20% allowable variation.

  The results show that as the allowable pressure variation was decreased, the uniformity and the capital cost of each system increased accordingly, as expected. The maximum *EAW/Ha* obtained from the system also increased in each case, implying that the *10%-system* is the optimal one, notwithstanding its higher capital cost. However, the additional return from the *10%-system* over the *20%-system* is small and considerations of cash-flow might mitigate in favour of the cheaper system.

  The four designs (40%, 30%, 20% and 10% allowable pressure variation respectively) were re-evaluated using a 5 year write off period, rather than the 15 year period used in the first set of evaluations. The results are shown in cases *12 to 15.* It can be seen that in this case, the maximum return from the *10%-system* was **less** than that from the *20%-system*, so that the *20% rule* is in fact an optimal one under these circumstances.

7.5 Accuracy of the Hydraulic Calculations

The widely used poly-plot graphical design procedure is based on an assumed constant emitter discharge, regardless of any variation in pressure. The extent of the error in this assumption was investigated using the computer programs in the following way :

* A fictitious emitter was defined in the data-base, giving a constant 4l/ph at all pressures (x exponent in the pressure/discharge relationship = 0).
* This emitter was then used to design the system, with all the other design parameters set to the same value that was specified for the base case described above.
* The original, real emitter was then respecified, before carrying out the evaluation of the designed system. This is necessary since running the evaluation using the fictitious emitter gives an erroneous uniformity of 100%.

The results of this analysis are shown in table 7.9

Table 7.9 Accuracy of the hydraulic calculations

	Design Lengths 12mm	Design Lengths 15mm	Inlet Press. (kPa)	Inlet Flow (lph)	Unif. Coef. (%)	Ave. Disch. (lph)	Cost	Max. EAW (R/Ha)
Single lateral								
1. Base case	14	29	157	1045			R26.10	
2. Error case, with fictitious emitter	12	31	157	1078			R26.40	
3. Error case, with true emitter	12	31	157	1046			R26.40	
Full system								
1. Base case	417	153	162	14 300	97.8	48.6	R3163/Ha	8672
2. Error case, with fictitious emitter	405	165	164	14 406	100	49.0	R3172/Ha	8783
3. Error case, with true emitter	405	165	164	14 330	97.8	48.6	R3172/Ha	8671

As can be seen from the results, the error in making the constant discharge assumption is very small, and has no significant effect on this design. In designing the full system with the fictitious emitter, only 5 of the 18 laterals were slightly changed from the base case design. The largest error occurs in the calculated value of the system discharge. The extent of this error depends on the extent to which the system average emitter discharge varies from the nominal value.

The **evaluation** of the error case with the fictitious emitter is interesting, in that the maximum *EAW/Ha* for this case represents the potential return from a "perfect" system. Thus the difference between this value and the value obtained for the base case is a measure of the amount that can profitably be spent on the system to install pressure compensating emitters. The difference is R111/Ha, which translates into a nett present value of R740/Ha. The 2 x 3.5m spacing gives 1 428 emitters/Ha, which means that the premium that can be afforded for a pressure compensating emitter, over the cost of the base case emitter, is R740/1428 = 51c. It should be noted however, that this result is for a design based on an allowable pressure variation of 20%. Using fully compensating emitters, the allowable variation can be increased considerably. Also, the example is on a steep slope where the topography is used to counteract the effects of friction losses. A more rigorous evaluation of the cost effectiveness of pressure compensating emitters should be done for a block on a flatter topography.

7.6 Conclusions

* The **design algorithms** have been shown to perform well in a range of design cases. The routines for :
  - changing down to a smaller diameter,
  - shifting the hydraulic grade line up and down within the allowable envelope, and

- widening the allowable envelope in order to move out of a cycling condition

all operated appropriately and effectively where required. In particular, the often encountered problem of steep slope design (>20%), was successfully overcome.

* The **evaluation** program successfully met the stated objective of providing a basis for formulating and evaluating alternative designs. In this respect it was shown to be highly flexible in accommodating changes to the input data. It was effectively used to :
  - assess the expected **performance** resulting from **a given design**, under varying economic and farm related conditions; and
  - evaluate **different designs**, generated by changing specific design parameters. An example of this was the investigation undertaken of the most appropriate value of the **allowable pressure variation** parameter.

* In the investigation of the most appropriate **allowable pressure variation**, the **20% rule** was shown to be optimal for the given case.

* All the computer design models require an **explicit statement of all values used for the design parameters.** This requirement is an important benefit of the models, because it enables the designer to consider aspects of the design problem that have been addressed up till now through unproven rules of thumb. For instance, the question of the optimal split of the allowable pressure variation between the manifold and the laterals is a "grey area" that has never been tested. It is believed that in time, as experience is gained from the use of the programs, tested values for such parameters will be developed.

* Important components of the success of the programs are their **ease of use** and **quick run times** :
  - the design parameters can be changed almost instantaneously while designing, by calling the *parameters table* as a pop-up window onto the screen;
  - entering the layout data for the Grabouw orchard, from the plan, took approximately 5 minutes;
  - once the layout had been fully specified, each complete redesign of all the laterals and the manifold took 4-5 minutes of computer time;
  - design of a single lateral or manifold took 10-15 seconds computer time;
  - the initial uniformity evaluation was the longest of the computer processes, taking 10 minutes for the Grabouw orchard;
  - each run of the economic evaluation took 20-30 seconds.

8. APPLICATIONS OF THE MAINLINE DESIGN MODEL

Four different farm irrigation mainline networks have been analysed, in order to test both the efficiency of the design routines and the range of different types of applications for which the models can be used.

8.1 The Sequencing algorithm

The efficiency of the sequencing algorithm was tested on a network of the Ciskei Agricultural Corporation at Ncera. The network supplies a 22Ha. vegetable farm from 24 valves, as shown in figure 8.1.

The network was designed to be operated in 12 irrigation shifts. Both the valve operation schedules that were proposed as part of the original design, and the schedules that were produced by the computer model, are shown in table 8.1.

Table 8.1 Valve operation schedules for the Ncera Network (fig 8.1)

	Original Schedules		Computer Schedules	
Shift No.	Open Valves	Total Flow (m3/h)	Open valves	Total Flow (m3/h)
1	3 & 4	60	6 & 20	89
2	1 & 2	51	5, 7 & 19	99
3	13 & 20	64	2,11 & 13	104
4	9 & 24	108	4 & 9	87
5	15 & 21	105	1 & 17	83
6	10 & 22	100	3 & 16	78
7	11 & 23	102	8 & 15	98
8	14 & 16	107	12 & 14	91
9	18 & 19	95	10 & 23	99
10	12 & 17	94	18 & 21	103
11	5 & 7	53	22	53
12	6 & 8	102	24	56

The two schedules can be compared in terms of three sets of criteria, as follows:

Flow dispersion in the network. Inspection of the two sets of schedules shows that the computer generated schedules appear to be more widely distributed than the original schedules, thereby resulting in a more *dispersed* flow through the network during its operation.

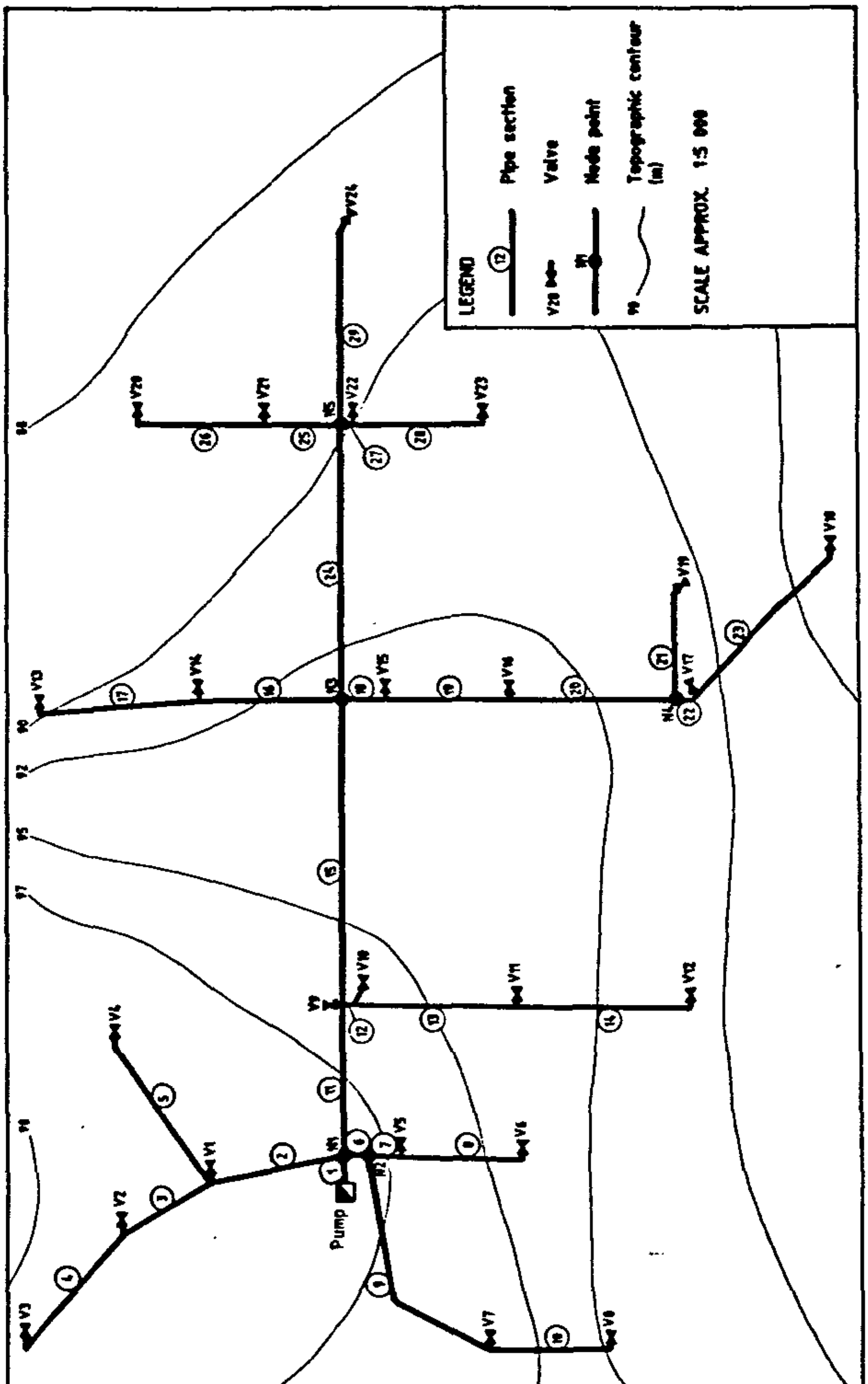


Figure 8.1 : Ncera irrigation project, mainline network

The original design has valves clustered together in the following shifts :

(1) : valves 3 and 4;

(2) : valves 1 and 2;

(3) : valves 13 and 20;

(5) : valves 21 and 15;

(8) : valves 14 and 16; and

(9) : valves 18 and 19.

Whereas the only clustering in the computer schedules occurs in shift (10) : valves 18 and 21.

This is borne out further by considering the maximum flows in each section, as generated in the output of the quick evaluation procedure. Table 8.2 shows these flows. It can be seen that the flows are reduced in the computer generated schedules in 8 out of the 29 pipe sections, constituting 463m out of a total of 1764m of piping. The flows in the remaining 21 sections are the same for both sets of schedules.

Variation in flow demand at the pump. The discharge required from the pump, in each schedule, is shown in table 8.1. The average flow in both cases is 86.7m^3/h. The average deviation from this mean is 19.9m^3/h for the original design and 12.8m^3/h for the computer design. Thus the flow requirements for the computer design are more evenly re.g.ulated than those for the original design.

Affect on optimum design. The pipeline was designed using the optimizing procedure, with both the original and the computer generated schedules. The results, using PVC/class 4 piping, are shown in table 8.2.

It can be seen that the computer generated schedules yielded a design with a net saving on total costs, of R1 273.29 over that for the original schedules.

General notes

In general, the scheduling algorithm has been found to give good results. In some instances, when a schedule has been allocated up to a flow level which is close to the current W% tolerance level and the remaining unallocated valves all have relatively large discharges, then the algorithm tends to make illogical selections. However the backward seeking mechanism described in Chapter 6 tends to be self correcting in this regard.

In addition to the computer scheduling algorithm, the program provides the user with the facility to alter the schedules manually once they have been displayed on the screen. An updated display of the total flow requirement in each schedule is maintained on the screen during this process. This facility has proved to be most useful in incorporating the designer's preferences, to correct or modify computer generated schedules. It should be noted that the computer algorithm does not incorporate cognizance of any practical, non hydraulic, factors that may affect the schedules. These may include, for example, the need to schedule together blocks with similar precipitation rates or similar soil types. Or, alternatively, the need to keep the physical distances between valve operations (opening and closing) in each schedule within reasonable limits. The manual adjustment facility enables the designer to incorporate these considerations.

Table 8.2 Optimized design of the Ncera network (fig 8.1)

	Original Schedules		Computer Schedules	
1) Pipes:				
Pipe Section	Max Flow (m^3/h)	Diameter (mm)	Max Flow (m^3/h)	Diameter (mm)
1	108	200	104	200
2	60	160	35	140
3	25	125	25	125
4	25	125	25	125
5	35	90/75	35	90/75
6	102	140	57	125
7	57	110/90	57	90
8	57	90	57	110/90
9	45	110/90	45	110/90
10	45	90	45	90
11	108	160	103	140
12	49	110	49	90
13	49	110/90	49	110/90
14	39	90/75	39	90/75
15	107	160/140	103	140
16	52	110/90	52	90/75
17	32	75/63	32	75/63
18	95	140	55	110
19	95	140	55	110
20	95	140	55	110
21	44	90	44	90/75
22	55	110/90	55	90
23	50	90	50	110/90
24	56	125/110	56	125/110
25	52	90/75	52	90
26	32	75/63	32	75/63
27	53	90	53	110
28	52	90	52	110/90
29	56	110/90	56	90/75
Total piping costs:		R15 373.95		R14 539.20
2) Energy:				
Operating pressure (m)		24.8		24.6
Seasonal energy reqmnt (kWh)		18 759		18 608
Present value cost		R41 882.52		R41 443.98
Total network costs		R57 256.47		R55 983.18

8.2 The Design Optimization Procedure

The Ncera network was used further, to compare the design results obtained using the linear programming optimization procedure with those obtained from conventional manual design procedures.

It was noted that the optimization procedure tends to select larger pipe diameters, for given flows, than is accepted in general practice. This results in pipelines with higher capital costs than would normally be expected. However these capital costs are then offset against the present value of the pumping costs. The fact that the optimal pipelines are larger, and therefore more *capitally* expensive, than those being currently designed, implies that designers are not taking cognizance of the extent of the energy portion of the total system costs.

This is illustrated using the Ncera network. Three different design cases are summarized in table 8.3. The first shows the network as it was designed manually, on the basis of prevailing rules of thumb concerning commonly selected diameters for given flows. It can be seen that whilst the piping costs are lower than those obtained for either of the schedules in table 8.2, the pumping costs, and hence the overall total costs, are significantly higher than those obtained in table 8.2.

The second design summarized in table 8.3 is the result of allowing the linear programming procedure to optimize the network for a fixed pump pressure of 30.3m, which is the same as that required for the manually designed network. Under these circumstances, the algorithm ignores the cost of energy, and simply minimizes the pipe capital costs whilst preserving the pressure requirements at the valves. It is interesting to note that the optimization procedure produces a cheaper network than the manual design procedure, due mainly to the tapering of pipe sections between node or valve points.

The designs discussed thus far have all been based on the following energy cost factors:

* Power capital cost (R/kW) : 330
* Pumping hours/season : 2 400
* Energy cost (c/kWh) : 8.00
* Load Cost (R/KVA/month) : 0.00
* Fixed (installation) costs (R/month) : 20.00
* Analysis period (years) : 20
* Bank interest rate (%) : 17.0
* Energy cost inflation (%) : 18.0

These values of the energy cost parameters resulted in a present value cost of R1 690 per meter pumping head.

Table 8.3 : Further design of the Ncera network

	Original Design	Optimized for Fixed Pressure Head	Optimized for 10% Inflation
1) Pipes:			
Pipe Section No.	Diameter (mm)	Diameter (mm)	Diameter (mm)
1	160	160	160
2	110	110	110
3	90	75	110
4	75	90/75	110
5	75	75/63	90/75
6	160	140	110
7	90	110	110
8	90	75	90/75
9	90	90	110/90
10	90	75	90
11	160	140	140
12	90	125	125
13	90	110	110
14	75	75/63	75/63
15	160	140	140
16	90	90	90
17	75	63	75/63
18	110	140	125
19	110	140	110
20	110	140	110
21	90	75	75
22	90	110	110
23	90	90/75	110/90
24	90	110	110
25	90	90	90
26	75	63	75/63
27	90	125	125
28	90	90/75	110/90
29	90	90/75	90/75
Total piping costs	R13 515.68	R13 230.55	R13 990.00
2) Energy:			
Operating pressure (m)	30.3	30.3	25.2
Seasonal energy reqmnt (kWh)	22 919	22 919	19 061
Present value cost	R51 207.00	R51 207.00	R23 307.68
Total network costs	R64 722.68	R64 437.55	R37 297.68

The third design shown in table 8.3 summarizes the results of a full optimization using the computer generated schedules, for which the expected rate of inflation was reduced from 18% to 10% per annum. This change resulted in the energy cost being reduced from R1 690 down to R916 per meter of pumping head. As a result, the optimum design used some smaller pipes, bringing the capital cost down from R14 539.20 to R13 990.00; and the pressure head required at the pump was increased from 24.6 to 25.2 meters. The pipe costs were still greater than those for the manual design. However the overall pumping costs, and hence the total system costs, were significantly reduced.

It can be seen that the energy costs constitute the greater part of the overall system costs. When the expected annual inflation rate is set to 18%, the energy costs are, on average, 76% of the total system costs; and even when the expected inflation rate is reduced to 10% per annum, the energy costs are still as much as 62% of the total system costs. They are therefore an important aspect of the overall set of design considerations, which cannot be overlooked.

It is interesting to compare the head loss gradients in the networks, for the different solutions. The original design had an average head loss of 3.4% of the length, for the critical valves (i.e. those at the ends of each branch). This increased to 4.3% for the design that was optimized with a fixed pressure head at the pump. As soon as the energy costs were incorporated into the optimization procedure, then the average head loss gradient dropped to 2.3% for the design using the original schedules and 2.5% for the design using the computer generated schedules.

8.3 Example of an Application on New Design

The model was used for the design of a new system on 50Ha of Citrus orchards on the Theron-Treurnicht farm in Lydenburg (South Africa). The water source is a reservoir situated more or less in the centre of the lands, splitting the orchards into two sections above and below the reservoir, as shown in figure 8.2.

The question arises as to whether to construct a single network feeding both sections, in which case the required pumping for the upper lands will result in excess pressures on the lower lands, which can be throttled using small diameter pipes for these sections; or alternatively to construct separate networks for each section, thereby reducing the total pumping requirements. The model was therefore run for three different layouts *viz :*

* the complete single network;
* the upper network alone; and
* the lower network alone.

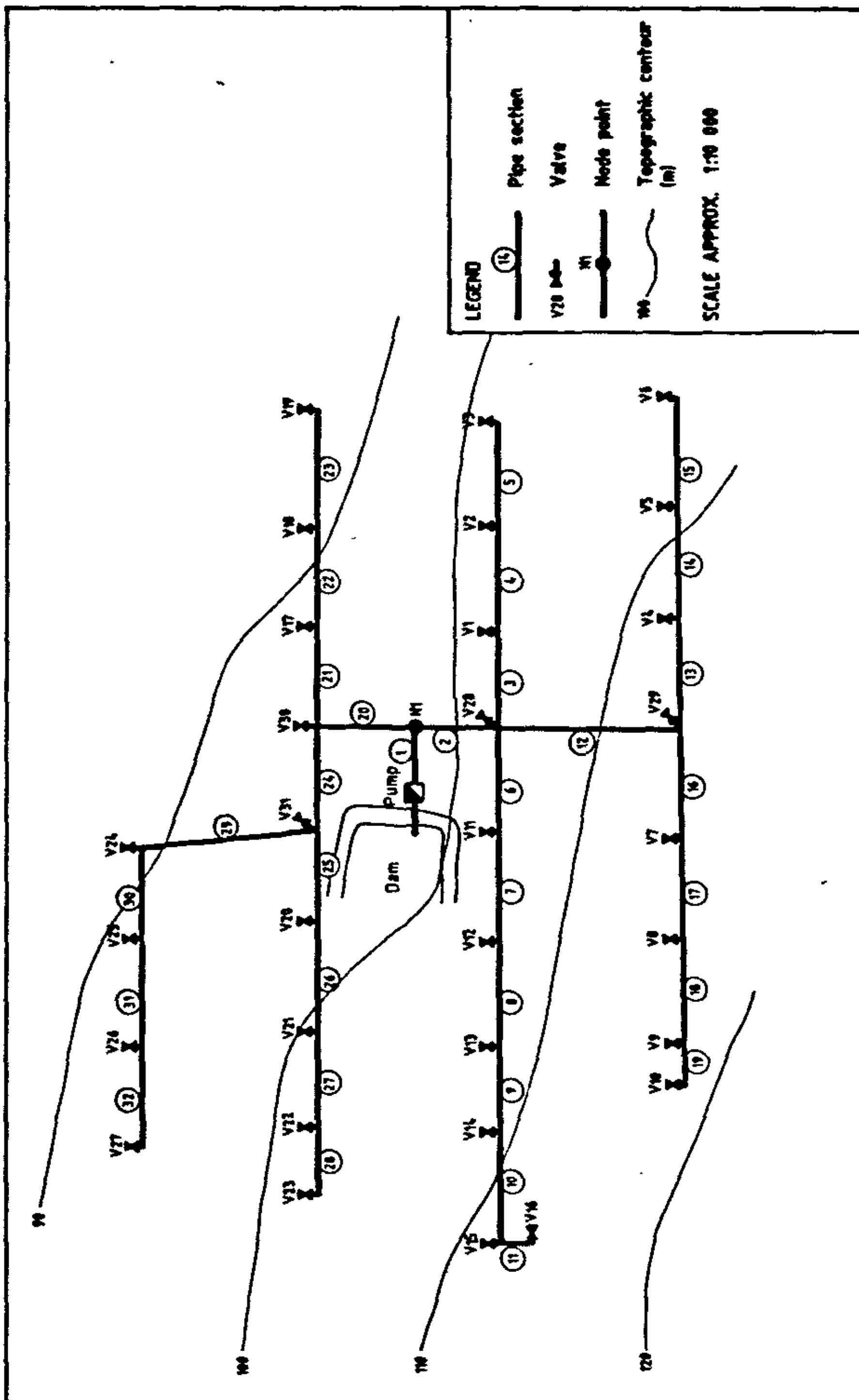


Figure 8.2 : Theron-Treurnicht farms, irrigation mainline network

The energy cost parameter values for each of these cases were as follows :

Parameter	Complete Network	Upper Network Alone	Lower Network Alone
Power cost (R/kW)	120	120	120
Pumping hours per season	2016	1368	662
Energy cost (R/kWh)	2.40	2.40	2.40
Load cost (R/KVA/month)	13.00	13.00	13.00
Fixed costs(R/month)	250.00	170.00	80.00
Analysis period (years)	20	20	20
Interest rate (%)	14.0	14.0	14.0
Inflation rate (%)	10.0	10.0	10.0

In each case, computer generated schedules were used. The results are summarized in table 8.4.

Thus, although the capital costs are lower for the single network than they are for the two separate networks, the energy costs are considerably higher; and hence the option of two separate networks is economically better.

It should be noted that the significant difference in the energy requirements between the two options is masked by the high fixed costs that were specified in the energy cost parameters. Although the separate-networks option requires only 67% as much energy as the single network option, this only yields a saving in energy costs of 8.9%. This is because the fixed costs, which are made up principally of an ESCOM (power authority) installation fee, are constant irrespective of the energy being used and constitute over 75% of the total energy cost. The full R250 per month has been apportioned to the two separate networks in the ratio of their respective pumping hours per season.

Table 8.4 : Optimized designs for the Theron-Treurnicht network (fig 8.2)

	Complete Network	Upper network Alone	Lower network Alone
1) Pipes:			
Pipe Section No.	Diameter (mm)	Diameter (mm)	Diameter (mm)
1	160	160	140
2	160	160	
3	63	63	
4	75/63	75/63	
5	75	75	
6	125	125	
7	125/90	25/90	
8	110	110	
9	110	110	
10	110	110	
11	50	50	
12	160	160	
13	90	90	
14	90	90	
15	110/90	110/90	
16	140	140	
17	140	140	
18	140	140	
19	140/125	110/90	
20	90/75		140
21	63		90/75
22	75/63		90
23	75		75
24	75		140
25	75		140
26	75		140
27	75		140
28	50		90/75
29	75		90
30	75/63		90/75
31	63		75
32	63/50		75/63
Total pipe costs	R25 449.05	R17 832.57	R11 064.49
		(R28 897.06)	
2) Energy:			
Operating pressure (m)	36.3	36.3	16.4
Seasonal energy reqmnt (kWh)	18 208	11 080	1 180
		(12 260)	
Present value cost	R71 785.10	R41 059.80	R24 321.05
		(R65 380.85)	
Total network costs	R97 234.15	R58 892.37	R35 385.54
		(R94 277.91)	

8.4 The Use of Booster Pumps

The design model was used to optimize the possible incorporation of booster pumps in a network for a proposed 1500Ha coffee and pepper plantation on the Bushbuckridge Trust Farms in the Eastern Transvaal (South Africa). The network is in fact a mainline supplying 17 different regions, each with their own sub-mainlines. Thus each valve operates together in one schedule. A sketch of the network is shown in figure 8.3. The energy cost parameters

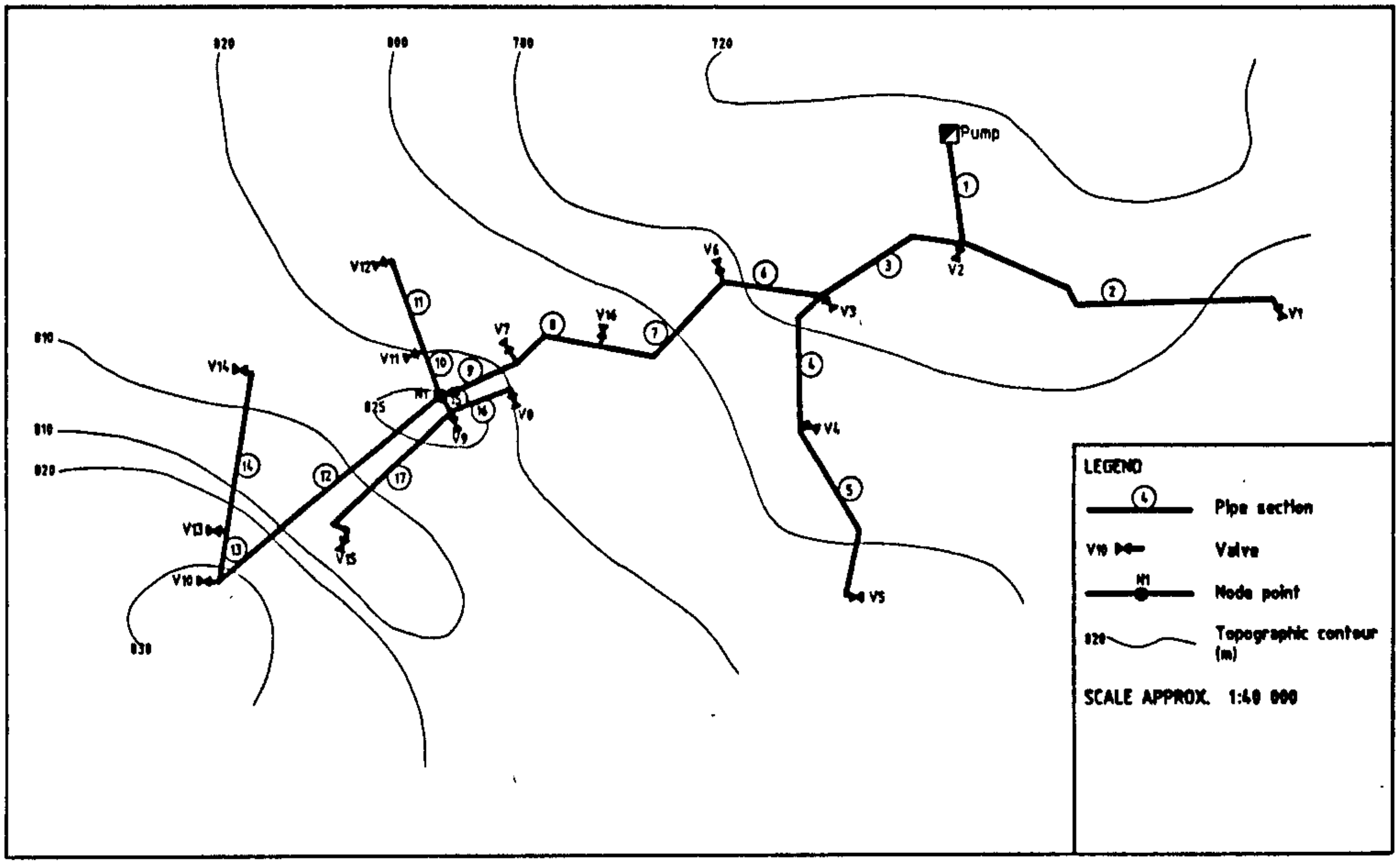


Figure 8.3 : Bushbuckridge Trust farms, irrigation mainline network

were the same as those used for the Theron-Treurnicht network described in section 8.3 above.

The network was optimized twice, once with no booster pumps specified, and once with booster pumps in sections 2,4,6,8,12 and 13 respectively. The results are summarized in table 8.5.

Table 8.5 : Optimized design of the Bushbuckridge network (fig 8.3)

	Without Boosters	With Boosters
Pipe costs	R389 062.60	R303 444.20
Required pump pressures (m) :		
At source	172.3	84.1
Booster (2)		17.4
Booster (4)		51.5
Booster (6)		45.1
Booster (8)		14.5
Booster (12)		26.8
Booster (13)		0.0
Total seasonal energy requirements (kWh)	1 236 729	954 740
Present value of energy costs	R847 640.40	R654 196.80
Total system costs	R1 236 703.00	R957 641.00

The pipe diameters for the two networks were principally the same, however the booster pumps resulted in considerably lower pressures in the pipes and hence enabled the use of lower class pipes, which yielded the savings shown in table 8.5.

The booster pumps resulted in a significant saving in the total energy requirement; and in this case this saving is reflected equally significantly in the total costs, since unlike the case described in section 8.3, the fixed costs are now overshadowed by the direct energy costs.

The list of pressures required at the booster pumps indicates that the booster specified in section 13 is redundant, but that each of the other boosters contributes to reducing the total system costs.

8.5 The Design Model as an Aid to Network Operation

The final application of the mainline design model to be described in this chapter relates to the use of the model to assist in the operation of an existing network. The network is on the Beyers Trust Farms at Riviersonderend in the Western Cape. A dam situated on a hill above the irrigated lands supplies 31 separate orchards covering a total of 140Ha, as shown in figure 8.4.

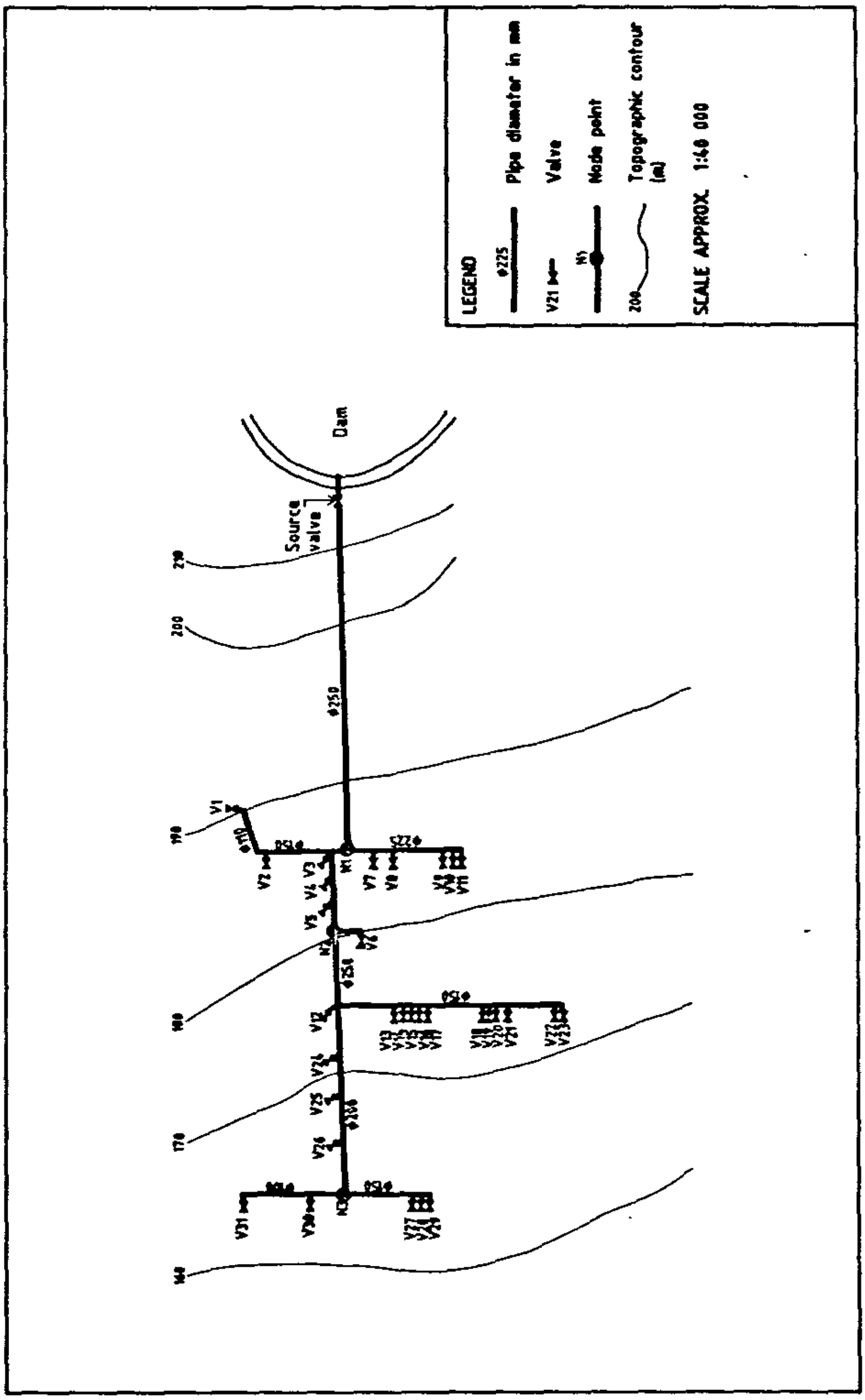


Figure 8.4 : Beyers Trust farms, Irrigation mainline network

The existing pipeline was installed over several years, without ever having been fully designed with the whole system taken into consideration. Until some recent expansions to the system, the network operated adequately under gravity. No pumping was needed. However, the current system is heavily loaded and the farmer has experienced difficulty in obtaining adequate pressures at the valves. The possibility of adding a pump to the system was considered.

The design model was used to establish operating schedules that would require the minimum amount of pumping pressure, if any pumping was needed at all. This was done as follows :

The layout was specified with the existing pipes. The scheduling algorithm was then run to establish operating schedules for 9 irrigation shifts. This was the maximum possible number of schedules, taking into account the required time for each schedule, the total available time and the time required for an alternative use of the network to irrigate large pastures. The optimization procedure was then run to find the minimum pumping requirement for these schedules.

As discussed in chapter 6, the design procedure allows optimization on the basis of two different specifications of the required pumping pressure. The first is a single overall pressure which will be the same in each schedule. The rationale for this being that the pressure operating point of a centrifugal pump can not normally be varied over a very large range, and it is preferable to have a constant operating point for all schedules. The second possible specification is for a separate optimum pressure in each schedule, which enables the designer to establish the extent to which the requirements in each schedule are unbalanced; and sometimes to design a bank of pumps that will meet the varying network requirements. For the Beyers network, the latter case was specified, so that if only some of the schedules required pumping, then they could be identified.

The resulting valve operation schedules are shown in table 8.6.

The results of the optimization showed that the required pressures at the valves could be met in all schedules with zero pressure head at the pump. In other words, there was no need for any pumping. In fact, the pressures at all valves, except for valve number 1, were at least 4m greater than the required minimum of 15m. The available pressure at valve 1 was exactly 15m, which means that any undue pressure losses in the pipeline will cause valve 1 to operate below the required pressure.

Table 8.6 : Valve operation schedules for the Beyers Network (fig 8.4)

Shift No.	Operating valves	Total flow (m^3/h)
1	7 & 28	215
2	9,23,25, & 30	218
3	10,17,24 &29	218
4	1,11,22 & 26	207
5	6,20 & 21	203
6	2,31 & 19	198
7	8,15,16 & 27	221
8	3,12 & 14	185
9	4, 5,13 & 18	233

8.6 Conclusions

The mainline design model has been shown to provide good results in a number of different applications. Specifically, it has been used to :

* provide efficient valve operating schedules;
* optimize the pipe diameters and pumping requirements of complete networks;
* enable rational decision making regarding alternative network layouts and structures;
* investigate and optimize the incorporation of booster pumps into a mainline design; and finally to,
* assist in planning the operation of an existing network.

The experience gained in using the model showed it to be flexible in the formulation of alternative designs, and relatively fast in generating solutions to each alternative. Once the basic data had been fed into the computer, then the approximate run times required to complete each alternative design, for each of the examples described above, were as follows :

1. The Ncera network (24 valves) :

   Valve sequencing - 4 minutes;

   Optimized design - 9 minutes.

2. The Theron-Treurnicht network (31 valves) :

   Valve sequencing - 6 minutes;

   Optimized design of the full network - 14 minutes.

3. The Bushbuckridge network (16 valves) :

   Optimized design with 6 boosters - 8 minutes.

4. The Beyers network (31 valves) :

   Valve sequencing - 5 minutes;

   Pump optimization - 2 minutes.

These "*turn around*" times enable a designer to investigate numerous alternatives relatively painlessly in the same time that it would take him to complete a single design by conventional manual methods.

Experience showed further that it is important for the user to have a good understanding of both the structure of the computer programs and the solution algorithms they employ. The programs **do not provide a "black box"** for use by unqualified designers.

The results obtained from a number of applications indicate that current design practice tends to underestimate the effect of energy costs on the total system costs, in favour of lower capital costs. Several designers, when spoken to about this, felt that the reason for it was that their clients (the farmers) were more concerned about the immediate cashflow effects of any development, than the longer term minimization of expenditure. It would be useful to use the design model to build up a series of recommended head loss gradients to be applied in the design of mainlines under varying circumstances.

In general, the tendency with existing manual techniques has been to design mainline networks with only a single diameter pipe in each section. One of the strengths of the LP procedure is that it can determine the optimal lengths for up to two diameters in a given section. In some cases, if a particular section is extremely long, it may be advantageous to specify a *dummy* node at some suitable point (e.g. at a sudden change in topography, or at the mid point) along the length of the section, so that more than two diameters can be calculated for the section.

A number of areas for future improvement of the programs have been identified :

* The data input routines should be streamlined. The whole layout, including valves, nodes and pipe sections could be specified in one table.
* The results of the optimization routine should include a fuller specification of the resulting energy and hydraulic characteristics of the network.
* The candidate diameter constraint currently has to be investigated by the user, and rectified manually in the layout evaluation table, prior to re-optimization. A routine to automate this process should be developed.
* The diameter selection criterion for the layout evaluation appears to work well, since few cases requiring candidate diameter adjustment were experienced. Nevertheless, it would be useful if the limiting velocities used for establishing the first estimate diameter were user specified (with the currently set values as defaults). This would enable the user to develop a "*feel*" for the most appropriate values under different circumstances.

9. CONCLUSIONS

9.1 Principal Results

The main aim of the work reported in this thesis has been to develop a functional set of computer programs for the design of irrigation systems. It is felt that in achieving this goal, the principal results of the research have been as follows :

9.1.1 Structuring of the design process

The research has resulted in a complete and formal specification of the irrigation systems design process, *viz* :

* Classification of the requirements of the design process into **system and hardware characteristics**, and identification of the individual components within these two groups;
* Formulation of **three main design modules** (preliminary, block and mainline) incorporating all of the required components;
* Identification of the **specific design routines** within each module;
* Specification of the **input, output, design parameters and design objectives** involved in the execution of each routine; and
* Identification of the **links** between various routines and the **inter-dependence** between various design components.

An overview of this structure is provided in table 2.1 and figures 2.1 and 2.2 in chapter 2, together with the synoptic maps of the computer models in Appendix 1. Documenting this structure provides a reference wherein each element of the design process is placed in its full context; as such it formed an essential first step in the development of the complete design model.

9.1.2 Comprehensive evaluation models

In developing the computer based design models, a principal objective has been to incorporate on-line evaluation procedures that enable the designer to focus his attention on the expected performance of the designed system and the extent to which it meets the prevailing requirements. Both the mainline design and the block design modules contain comprehensive evaluation processes which enable the designer :

* to assess the extent to which the design satisfies his objectives;
* to generate alternative designs; and then
* to compare these alternative designs.

Appropriate use of these evaluation models will provide the designer with an insight into the performance of the irrigation system that has not been practically attainable using current manual design procedures.

The evaluation procedures are based on a thorough study of the state of the art in measuring irrigation performance, leading to the formulation of a set of **performance parameters** which have been incorporated into the models. These parameters are discussed in detail in Chapters 3, 5 and 6.

The main evaluation model at the end of the block design process provides the designer with information on :

- the **uniformity of distribution** that will result from the designed system;
- the **expected yield** per unit area and per unit of water applied, that will be attained using the system;
- the **requirement and application efficiencies** that will be achieved by the system;
- the estimated **cost of the system** and the **financial return on investment** per unit area and per unit of water applied, that will be achieved; and
- information on the **optimal application depths** during the irrigation season.

It is important to note further that the principle of evaluation has been incorporated into the *whole* design process. Thus, apart from the *main* evaluation model described above, more *general* evaluation is carried out continuously throughout the design process. The models have been structured so that whenever the designer wants to investigate the effects of possible adjustments to the design, he is able to consider these effects in terms of immediately updated evaluation parameters. For example :

* In the **lateral and manifold design routines** of the block design module, the designer has the choice of (a) letting the computer carry out the design, either by the *poly-plot* routine or by extrapolation of results obtained from previously designed pipes, or (b) he may specify the design himself, by inputting the required lengths and diameters. Once the design is complete, the table in which the design details are listed (figure 4.10) is updated with statistics on :
  - the cost of the pipe being designed;
  - the total cost of the whole block;
  - the total length of the pipe being designed; and
  - the total lengths, for the whole block, of each diameter of pipe being used.

If the designer now wishes to change the design, for example by altering the length of one of the pipe sections used for a specific lateral, he will immediately be able to see the effect of this change on the cost of the lateral and on the overall block costs. Furthermore, having made the change, he can then obtain a graphical plot or a numeric listing of the pressure envelope, and thereby investigate the hydraulic effects of this change.

* In the **valve sequencing routine** of the mainline design module, once again the designer has the choice of either letting the computer determine the schedules or of specifying his own schedules. Once the valves have been fully allocated, the resulting schedules are displayed in a table which also lists a summary of the flow required in each shift. If the designer decides to alter any of the schedules, the table is updated as soon as any change is made and the designer is therefore able to monitor the extent to which the flow requirements in each shift are balanced.

* In the **quick evaluation of layouts** routine of the mainline design module, the results of this evaluation are presented in two tables. The first lists the estimated *capital* costs of the mainline, based on "first shot" pipe diameter selections (figure 6.4). The second lists the estimated *operating* costs, based on prevailing energy cost tariffs, the expected efficiency of the pumping system and the rates used for the present value analysis (figure 6.5).

  Both of these tables can be edited by the designer and the effects of any changes will be updated immediately. Thus for example, he can investigate the sensitivity of the overall network cost to changes in any of the selected pipe diameters. Alternatively, he can test the sensitivity of the operating costs to changes in the energy tariffs or interest/inflation rates.

* Similarly, the **linear programming** routine of the mainline module presents the results of the optimization in two tables; one showing the optimum pipe diameters and their associated lengths for each section of the network, and the second showing the resulting pressures at each valve and at the pump.

  The designer may change any of the selected pipe diameters or their lengths, and the effects of such changes will be shown in the cost summaries and in the pressures table.

9.1.3 Design algorithms.

The research did not attempt specifically to develop new design procedures, but rather wherever possible to adapt well established methods into a workable format for the type of interactive computing that was being strived for. This was achieved in the following algorithms :

* the routines for lateral and manifold design. The mathematical specification of the graphical *poly-plot* procedure was a continuation of the work of Perold (1979 - ref. 7 in chapter 4). Several modifications were developed in order to render the process efficient and stable for common design situations; as well as to incorporate accurate point-to-point calculation of the pipe and emitter hydraulics, for both laterals and manifolds.
* the linear programming routine for optimization of the mainline pipe diameters and pumping requirements. Modifications to established algorithms include the generation of the candidate diameters, the calculation of the pumping costs and the allowed option for generating either an overall optimized pumping head or an optimum for each shift.

In addition, a number of new procedures for the design of various parts of the overall process have been proposed. These include :

- the valve sequencing algorithm in the mainline design ;
- the layout evaluation process in the mainline design ;
- the dynamic programming algorithm for optimizing the system operating point which forms part of the block design evaluation process.

9.1.4 Rationalization of the design process

Although the design model does not provide a completely rationalized solution to the design problem at this stage (as discussed in Chapter 1), it does enable the designer to carry out a quantitative study of the various trade-offs involved in the design process. For example :

* the suitability of the 20% allowable head loss criterion can be investigated on a case by case basis ;
* the *maximum yield* criterion can be modified to a *maximum profit* criterion, which allows for rational consideration of the possibility of deficit irrigation ; and
* the ratios of cost vs performance, system vs operating costs and block network vs mainline network costs can also be investigated.

9.1.5 Computer aided design (CAD)

The computer programs have been designed to exploit, to the fullest possible extent, the impact of computer aided *design* (as opposed to computer aided *draughting*) on the general "Engineering Design" process. Experience gained during the course of the research showed that both the on-line availability of the various evaluation facilities, and the user friendly and interactive man/computer dialogue, encouraged the designer to undertake a more broadly based and investigative approach to the design problem than was practically feasible in the past. As such, the Author believes that the programs constitute a significant contribution to the state of the art of CAD in the field of Irrigation Engineering.

In particular, a specific objective of the work was that the programs should be applicable to real on-farm situations. To this end they were developed for use in a design office environment on personal computers with locally developed databases, rather than on any remote main-frame facility with hypothetical data.

9.2 Applications of the design models

The computer based design models have been tested on a number of different design problems. The results of these tests, on both block design and mainline design cases, are reported in chapters 7 and 8 respectively.

In general, the models were found to perform well, providing rapid calculation times and good operating flexibility. More specific conclusions regarding the performance of the computer models in specific design circumstances are drawn at the end of each of the abovementioned chapters.

9.3 Development of the computer programs

The structuring and design of the computer programs played an important part in the successful development of the design models. The programs evolved in three distinct phases, as experience was gained in the use of computer aided design techniques. These phases can be characterized by the nature of the man/machine interface in each case :

9.3.1 Phase 1 : Initial individual design routines

These programs employed a *consecutive input* dialogue, followed by a *unidirectional* operating procedure. This meant that the required data was input on a line by line basis, and the designer then sat back while the computer carried out the calculations. Once the design was complete the designer could study the results in order to decide on any changes he might want to investigate further. These changes would then be incorporated into a new set of input data and the programs would be rerun from the start, using the new data.

The rigidity of this structure precluded the type of continuous evaluation throughout the design process described in section 9.1.2 above.

9.3.2 Phase 2 : The BASIC programs for mainline design, written on the IBM-PC

These programs employ a *tabular input* dialogue and a *multidirectional* operating procedure. This implies that the data are input through fully editable tables. The designer can make any additions or corrections he wants to the tables, before continuing with the various calculation routines. The results of the calculation procedures are also presented in tables on the computer screen, and the designer can once again edit these result tables for use in any on going calculations. Furthermore, the designer does not have to run the program in a fixed order of steps, but can rather move between the various routines at will.

Thus the use of the programs is no longer based on individual "runs" of the design routines, but rather on extended "work sessions" in which the designer and the computer work together to investigate numerous alternatives on the way to optimizing the final results. These programs incorporated the continuous evaluation principles discussed above.

Another aspect of the programs that was developed during this phase was the incorporation of on-line routines for immediate validation of the data input. This means that the data is checked as far as possible for its general feasibility, **while it is being entered into the computer**, rather than allowing the program to fail while performing the calculations.

9.3.3 Phase 3 : The PASCAL programs for block design, written on the IBM-PC

These programs are operated by similar processes to those of the phase 2 programs. However, they are considerably more flexible in their ability to move between different routines. The prevailing philosophy is one of letting the user determine the path followed through the program, rather than the program directing the user. All procedures are accessed through displayed menu functions.

Also, greater accessibility to various reference facilities (e.g. the databases and the maximum length calculator) is achieved through the use of the *"pop-up windows"*. Attention has been given to details such as a uniform screen layout, constant use of prompts which are always displayed in the same place on the screen, and the use of graphics to illustrate results. It has been found that these aspects greatly enhance the operation of the programs.

9.4 Indications for further work

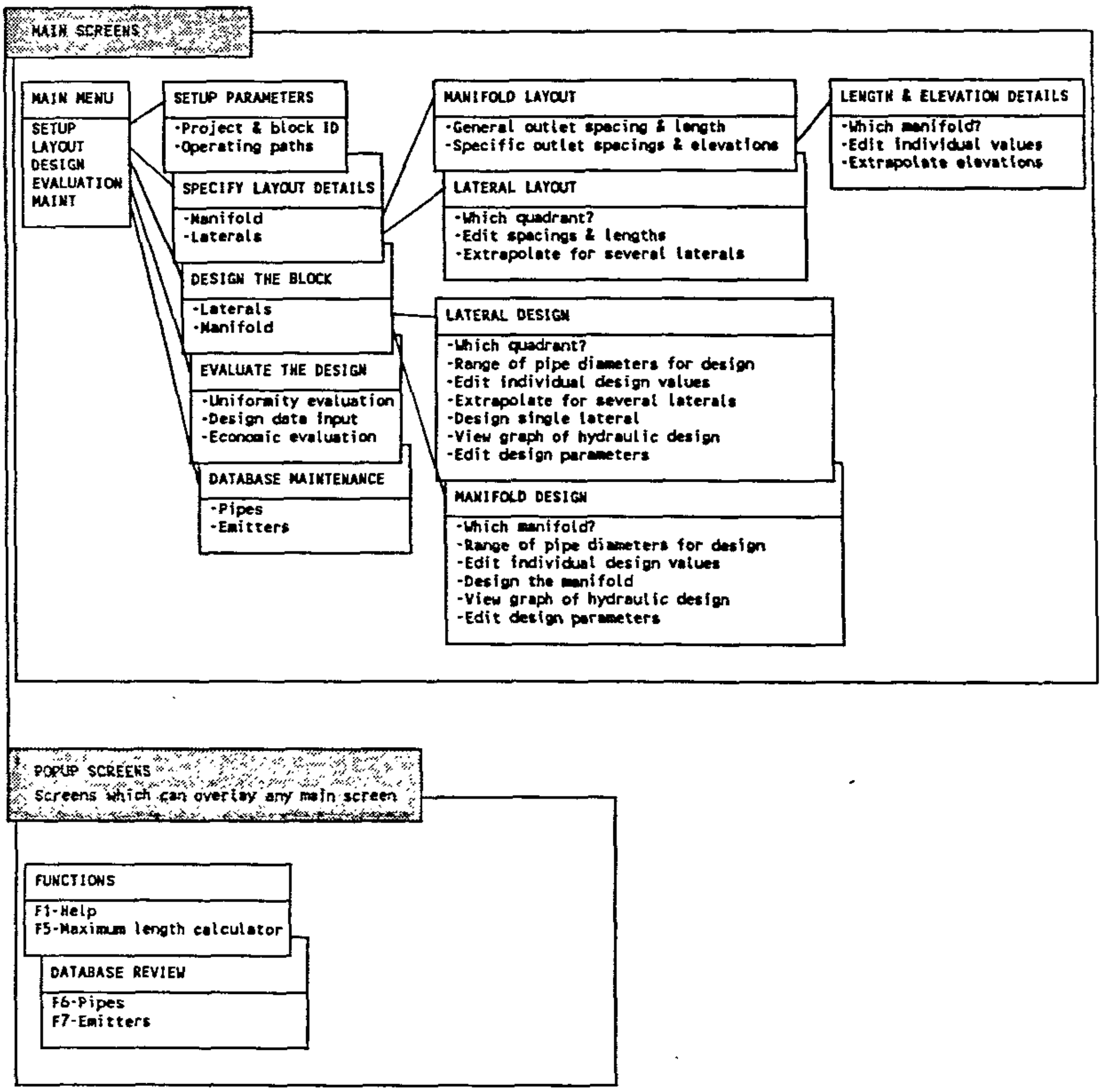
It is felt that continued research on the subject of this thesis could follow two main directions :

* Once the results of more applications have been acquired, they can be used to develop a set of design norms that could form the basis of a **code of practice**. For example, recommendations could be formulated on the following :
  - The most appropriate overall system *coefficients of uniformity*, under different conditions;
  - Expected *application and requirement efficiencies* under different operating conditions.
  - Allowable pressure variation within a system;
  - Appropriate ratios of block versus mainline and hardware versus operating costs;
  - The most economical pipe sizes to be used in mainlines, with due consideration to the cost of energy in the overall cost of the system.
  - One of the most interesting parameters in this category, is an investigation of the potential for *deficit irrigation* under varying conditions.

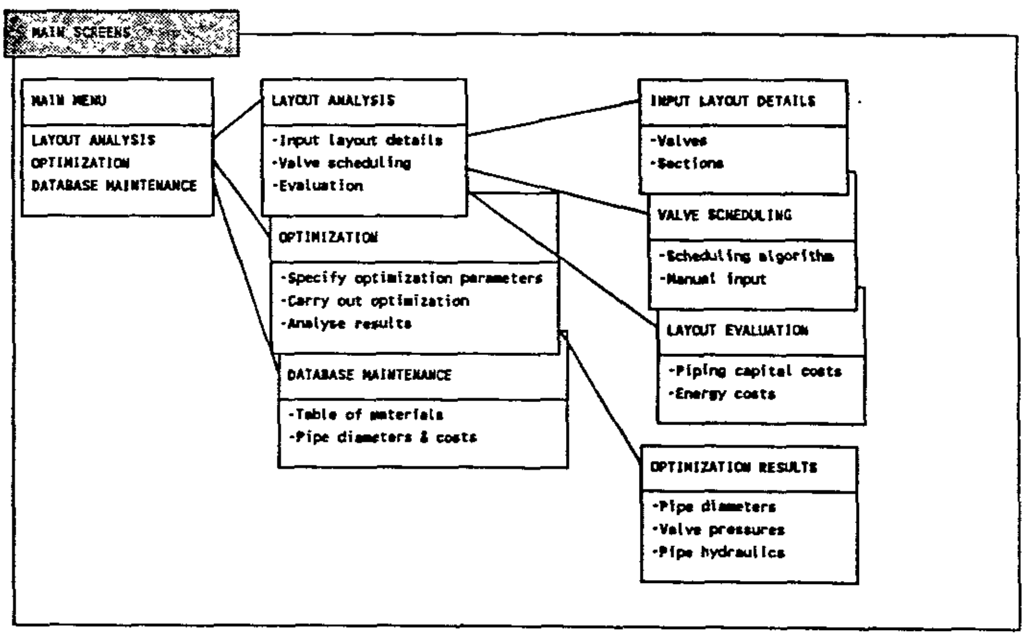
* Continued development of the models themselves. This would entail :
  - Development of a preliminary design module, and integration of all three modules (preliminary, block and mainline) into a single package utilizing shared data input and output routines and similar operating procedures ;
  - Interfacing the *design* package with one of the commercially available computer aided *draughting* packages in order to enable graphical data input and results output ;
  - The development of a complete irrigation equipment database to be used as a master bill for interfacing with the design process and subsequent detail specification of bills of quantities. This specification would be linked to the output of the design model.

APPENDICES

APPENDIX 1a : Synoptic Map of the Block Design Process



APPENDIX 1b : Synoptic Map of the Mainline Design Process



APPENDIX 2a: Pascal Listings of Design Routines (Manifold case)

PROCEDURE hydro_design;

```
  {-design manifold(half)-}
TYPE
  __mode=(_up,_down);
VAR
  cs,                    {-current section}
  I,                     {-current lat #}
  loop_cnt,tol_tag,
  m,shift_cnt,
  fail_start:INTEGER;
  start_pt,end_pt,       {-of each section}
  tag,
  dia_posn:_10INTARRAY;  {-index of each sect dia in the m_dia array}
  cd:_10REALARRAY;       {-current diameter}
  P,q:ARRAY[1..10,1..Max_outlet] OF REAL;
  del_hl,                {-friction head-loss}
  del_hg,                {-topographic head gain}
  shift_amt,tolerance,start_dia:REAL;
  _mode,new_dia:__mode;
  minlen_check,
  no_shift,_stuck:BOOLEAN;
  t_synop:TEXT;
  man:STRING[3];
```

PROCEDURE WriteSynop(S:Anystr);

```
  BEGIN
    WriteLn(t_synop,cs:1,'      ',
      MDes[Half].m_dia[dia_posn[cs]]:3,'       ',start_pt[cs]:3,'  ',
      P[cs,start_pt[cs]]:5:1,'     ',I:3,'  ',P[cs,I]:5:1,'    ',I-1:3,
      '  ',P[cs,I-1]:5:1,'  ',S,' (',tolerance:4:2,')');
  END;                   {-endof proc WriteSynop}
```

PROCEDURE calc_fwd;

```
    {-Calculates the pressure one lateral forward, i.e. towards the inlet}
  VAR
    LatIn_q,t_dist:Real;
  BEGIN
    {*}               {- establish the flow}
    IF Lk1[I]=0 THEN LatIn_q:=0
    ELSE LatIn_q:=Lk1[I]+Lx1[I]*P[cs,I];
    IF Lk2[I]<>0 THEN LatIn_q:=LatIn_q+(Lk2[I]+Lx2[I]*P[cs,I]);
    IF I=last_lat THEN q[cs,I]:=LatIn_q
    ELSE q[cs,I]:=q[cs,I+1]+LatIn_q;

    {*}               {- establish the section length}
    t_dist:=Manif_out.outlet_spac[Half,I];
```

```
  {*}              {- headloss & elevation change calcs}
  del_hl:=(Exp(Ln(Hlp[1])+Hlp[2]*Ln(q[cs,I]/(1000*Hlp[4]))
   +Hlp[3]*Ln(cd[cs]))*t_dist);
  del_hg:=Manif_out.outlet_elev[Half,I-1]-Manif_out.outlet_elev[Half,I];
  P[cs,I-1]:=P[cs,I]+del_hl-del_hg;

  (*  -trace used for debugging purposes, now commented out:
    gotoxy(42,8);write('latinq, P, del_hl, del_hg, lk1, lx1, lk2, lx2');
    gotoxy(42,9);write(latin_q:8:3,P[cs,i]:8:2,del_hl:8:2,del_hg:8:2);
    gotoxy(42,10);write(lk1[i]:8:2,lx1[i]:8:2,lk2[i]:8:2,lx2[i]:8:2);
    pause;        *)
 END;              {-of proc. Calc_fwd}
```

PROCEDURE calc_back;

```
  {-Calculates the pressure one lateral back, i.e. away from the inlet}
 VAR
  LatIn_q,t_dist,t_hl,t_hg:Real;
 BEGIN
  {*}              {- establish the flow}
  IF Lk1[I]=0 THEN LatIn_q:=0
  ELSE LatIn_q:=Lk1[I]+Lx1[I]*P[cs,I];
  IF Lk2[I]<>0 THEN LatIn_q:=LatIn_q+(Lk2[I]+Lx2[I]*P[cs,I]);
  q[cs,I+1]:=q[cs,I]-LatIn_q;

  {*}              {- establish the section length}
  t_dist:=Manif_out.outlet_spac[Half,I+1];

  {*}              {- headloss & elevation change calcs}
  t_hl:=(Exp(Ln(Hlp[1])+Hlp[2]*Ln(q[cs,I+1]/(1000*Hlp[4]))
    +Hlp[3]*Ln(cd[cs]))*t_dist);
  t_hg:=Manif_out.outlet_elev[Half,I]-Manif_out.outlet_elev[Half,I+1];
  P[cs,I+1]:=P[cs,I]-t_hl+t_hg;
 END;              {-of proc Calc_back}
```

PROCEDURE stuck(X:Integer);

```
  {-Handles the various stuck situations}
 VAR
  Reason:STRING[30];
  J,jj:Integer;
  MsgString:Anystr;
  smallStr:Anystr;
 BEGIN
  _Stuck:=True;
  WriteSynop('              ');

  {*}              {- establish message }
  IF X=1 THEN Reason:=' (in an shift loop)';
  IF X=2 THEN Reason:=' (need a larger dia)';
  IF X=3 THEN Reason:=' (need a smaller dia)';
  IF X=4 THEN Reason:=' (min dia>all dias in list)';
  IF X=5 THEN Reason:=' (in a change diams loop)';

  {-beep & write a msg}
  Beep;
  MsgString:='';
  MsgString:='Design stuck at dia # ';
  Str(MDes[Half].m_dia[dia_posn[cs]]:3,smallStr);
```

```
    MsgString:=MsgString+smallStr;
    MsgString:=MsgString+', emitter # ';
    Str(I:3,smallStr);
    MsgString:=MsgString+smallStr;
    MsgString:=MsgString+Reason;
    Instr(MsgString+'..hit any key');
    WriteLn(t_synop,MsgString);
    Pause;

    (*)              {- establish the plot_pts *}
    WITH MDes[Half] DO
      BEGIN
        FOR J:=1 TO cs DO
          FOR jj:=start_pt[J] DOWNTO end_pt[J] DO
            m_plot[jj]:=P[J,jj];
        m_designed:=False;
      END;
    WritePlotFile(Half);

    {- update col totals & exit }
    Fill_manif_descrip;
  END;               {-of proc. stuck}
```

PROCEDURE change_dia(nn:__mode);

```
    {-Change diameter when pressure profile reaches envelope boundaries}
  VAR
    aa:Integer;
  BEGIN
    WITH MDes[Half] DO
      BEGIN
        no_shift:=false;
        cs:=cs+1;
        start_pt[cs]:=I;end_pt[cs-1]:=I+1;
        P[cs,I]:=P[cs-1,I];
        q[cs,I+1]:=q[cs-1,I+1];
        IF _mode=_up THEN _mode:=_down
        ELSE _mode:=_up;
        minlen_check:=True;
        IF end_pt[cs-1]>start_pt[cs-1] THEN end_pt[cs-1]:=start_pt[cs-1];
        tag[cs]:=I;
        dia_posn[cs]:=dia_posn[cs-1]+1;

        (*)           {-change to a larger diameter }
        IF nn=_up THEN
          BEGIN
            IF dia_posn[cs]>10 THEN dia_posn[cs]:=10;
            WHILE (m_dia[dia_posn[cs]]<=m_dia[dia_posn[cs-1]])
              AND (dia_posn[cs]<10) DO dia_posn[cs]:=dia_posn[cs]+1;
            IF m_dia[dia_posn[cs]]<=m_dia[dia_posn[cs-1]] THEN stuck(2);
            IF NOT _Stuck THEN
              BEGIN
                new_dia:=_up;
                cd[cs]:=
                  Mat_Detail.int_dia[_classIndex,m_dia_index[dia_posn[cs]]];
              END;
          END
        ELSE
```

```
      (*)        {-change to a smaller diameter }
      IF nn=_down THEN
        BEGIN
          IF dia_posn[cs]>10 THEN dia_posn[cs]:=10;
         WHILE (m_dia[dia_posn[cs]]>=m_dia[dia_posn[cs-1]])
            AND (dia_posn[cs]>1) DO dia_posn[cs]:=dia_posn[cs]-1;
          IF (m_dia[dia_posn[cs]]<d_param.dp_manif_dia)
            OR (m_dia[dia_posn[cs]]>=m_dia[dia_posn[cs-1]]) THEN stuck(3);
          IF NOT _Stuck THEN
            BEGIN
              new_dia:=_down;
              cd[cs]:=
                Mat_Detail.int_dia[_classIndex,m_dia_index[dia_posn[cs]]];
            END;
        END;
    END;
  END;                {-of proc Change_dia}
```

PROCEDURE Shift_up;

```
    {-Shifts the current pressure curve up to meet the top of the envelope}
  BEGIN
    no_shift:=false;
    tag[cs]:=I;
    shift_cnt:=shift_cnt+1;
    minlen_check:=True;
    shift_amt:=d_param.dp_pMax-P[cs,I];
    _mode:=_down;
    IF shift_amt>0 THEN
      BEGIN
        I:=start_pt[cs];
        P[cs,I]:=P[cs,I]+((1-shift_amt/d_param.dp_pMin)*shift_amt);
      END
    ELSE BEGIN
        no_shift:=true;
        minlen_check:=false;
      END;

    IF (cs>1) AND (NOT no_shift) THEN
      BEGIN
        IF new_dia=_up THEN
          BEGIN
            q[cs,I+1]:=q[cs,I+1]*(1+shift_amt/d_param.dp_pMin);
            WHILE (P[cs,I]>=P[cs-1,I]) AND (P[cs-1,I]<>0)
              AND (P[cs-1,I]<d_param.dp_pMax) DO
              BEGIN
                calc_fwd;
                I:=I-1;
              END;    {-of while}
            IF (P[cs,I]<P[cs-1,I]) OR (P[cs-1,I]=0) OR
              (P[cs-1,I]>d_param.dp_pMax) THEN I:=I+1;
          END
        ELSE
          IF new_dia=_down THEN
            BEGIN
              q[cs,I]:=q[cs,I]*(1+shift_amt/d_param.dp_pMin);
              WHILE (P[cs,I]>=P[cs-1,I]) AND (I<=start_pt[cs-1])
                AND (P[cs-1,I]>P[cs-1,I+1]) DO
```

```
              BEGIN
                calc_back;
                I:=I+1;
              END;  {-of while}
            IF (P[cs,I]<P[cs-1,I]) OR (I>start_pt[cs-1])
              THEN I:=I-1;
          END;
      IF I=start_pt[cs] THEN
        BEGIN
          no_shift:=true;
          minlen_check:=false;
        END
      ELSE
        BEGIN
          start_pt[cs]:=I;
          end_pt[cs-1]:=I+1;
          IF end_pt[cs-1]>start_pt[cs-1] THEN end_pt[cs-1]:=start_pt[cs-1];
        END;
      P[cs,I]:=P[cs-1,I];
      q[cs,I+1]:=q[cs-1,I+1];
    END;
  END;               {-of proc Shift_up}
```

PROCEDURE Shift_down;

```
    {-Shifts the current pressure curve down to meet the bottom of the envelope}
  VAR
    backcnt : INTEGER;
  BEGIN
    no_shift:=false;
    tag[cs]:=I;
    backcnt:=0;
    shift_cnt:=shift_cnt+1;
    minlen_check:=True;
    shift_amt:=P[cs,I]-d_param.dp_max_pinlet;
    _mode:=_up;
    IF shift_amt>0 THEN
      BEGIN
        I:=start_pt[cs];
        P[cs,I]:=P[cs,I]-((1-shift_amt/d_param.dp_pMin)*shift_amt);
      END
    ELSE BEGIN
        no_shift:=true;
        minlen_check:=false;
      END;

    IF (cs>1) AND (NOT no_shift) THEN
      BEGIN
        IF new_dia=_up THEN
          BEGIN
            q[cs,I]:=q[cs,I]*(1-shift_amt/d_param.dp_pMin);
            WHILE (P[cs,I]<=P[cs-1,I]) AND (I<=start_pt[cs-1])
              AND (P[cs-1,I]>P[cs-1,I+1]) AND (q[cs,I]>0)
              AND (backcnt<tag[cs]+2) DO
            {-the last condition ensures that we dont go back too far
              if we might get to the end }
              BEGIN
                calc_back;
```

```
          I:=I+1;
          backcnt:=succ(backcnt);
        END;    {-of while}
      IF (P[cs,I]>P[cs-1,I]) OR (I>start_pt[cs-1]) OR
        (q[cs,I]<=0) THEN I:=I-1;
    END
  ELSE IF new_dia=_down THEN
    BEGIN
      q[cs,I+1]:=q[cs,I+1]*(1-shift_amt/d_param.dp_pMin);
      WHILE (P[cs,I]<=P[cs-1,I]) AND (P[cs-1,I]<>0) DO
        BEGIN
          calc_fwd;
          I:=I-1;
        END;    {-of while}
      IF (P[cs,I]>P[cs-1,I]) OR (P[cs-1,I]=0) THEN I:=I+1;
    END;
  IF I=start_pt[cs] THEN
    BEGIN
      no_shift:=true;
      minlen_check:=false;
    END
  ELSE
    BEGIN
      start_pt[cs]:=I;
      end_pt[cs-1]:=I+1;
      IF end_pt[cs-1]>start_pt[cs-1] THEN end_pt[cs-1]:=start_pt[cs-1];
    END;
  P[cs,I]:=P[cs-1,I];
  q[cs,I+1]:=q[cs-1,I+1];
 END;
END;              {-of proc. Shift_down}
```

PROCEDURE adjust;

```
  {-Adust the start_pt of the current dia,
  if after a shift the pressure moves outside the envelope}
 BEGIN
  minlen_check:=True;
  IF tag[cs]>I THEN tag[cs]:=I;
  IF _mode=_down THEN
   BEGIN
    IF cs=1 THEN
     IF P[cs,start_pt[cs]]=d_param.dp_max_pInlet THEN
      BEGIN
        I:=start_pt[cs];
        WriteSynop(' Adj Chng Up ');
        P[cs,I]:=d_param.dp_pMax;
        _mode:=_up;
        change_dia(_up);
        Exit;
      END
     ELSE
      BEGIN
        I:=start_pt[cs];
        P[cs,I]:=0.995*P[cs,I];
        IF P[cs,I]<d_param.dp_max_pInlet THEN
          P[cs,I]:=d_param.dp_max_pInlet;
      END;
```

```
          IF (cs>1) AND (new_dia=_up) THEN
            IF P[cs-1,start_pt[cs]+1]<P[cs-1,start_pt[cs]] THEN
              BEGIN
                start_pt[cs]:=start_pt[cs]+1;
                I:=start_pt[cs];
                end_pt[cs-1]:=I+1;
                P[cs,I]:=P[cs-1,I];
                q[cs,I+1]:=q[cs-1,I+1];
              END
            ELSE
              BEGIN
                WriteSynop(' Adj Chng Up ');
                _mode:=_up;
                Change_dia(_up);
                Exit;
              END;
          IF (cs>1) AND (new_dia=_down) THEN
            IF P[cs-1,start_pt[cs]-1]>d_param.dp_max_pInlet THEN
              BEGIN
                start_pt[cs]:=start_pt[cs]-1;
                I:=start_pt[cs];
                end_pt[cs-1]:=I+1;
                P[cs,I]:=P[cs-1,I];
                q[cs,I+1]:=q[cs-1,I+1];
              END
            ELSE
              BEGIN
                WriteSynop(' Adj Chng Up ');
                _mode:=_up;
                Change_dia(_up);
                Exit;
              END;
        END;
      IF _mode=_up THEN
        BEGIN
          IF cs=1 THEN
            IF P[cs,start_pt[cs]]=d_param.dp_pMax THEN
              BEGIN
                I:=start_pt[cs];
                WriteSynop(' Adj Chng Dn ');
                P[cs,I]:=d_param.dp_pMin;
                _mode:=_down;
                change_dia(_down);
                Exit;
              END
            ELSE BEGIN
              I:=start_pt[cs];
              P[cs,I]:=1.005*P[cs,I];
              IF P[cs,I]>d_param.dp_pMax THEN
                P[cs,I]:=d_param.dp_pMax;
            END;
          IF (cs>1) AND (new_dia=_up) THEN
            IF P[cs-1,start_pt[cs]-1]>P[cs-1,start_pt[cs]] THEN
              BEGIN
                start_pt[cs]:=start_pt[cs]-1;
                I:=start_pt[cs];
                end_pt[cs-1]:=I+1;
```

```
            P[cs,I]:=P[cs-1,I];
            q[cs,I+1]:=q[cs-1,I+1];
          END
        ELSE
          BEGIN
            WriteSynop(' Adj Chng Dn ');
            _mode:=_down;
            Change_dia(_down);
            Exit;
          END;
      IF (cs>1) AND (new_dia=_down) THEN
        IF P[cs-1,start_pt[cs]+1]>d_param.dp_max_pInlet THEN
          BEGIN
            start_pt[cs]:=start_pt[cs]+1;
            I:=start_pt[cs];
            end_pt[cs-1]:=I+1;
            P[cs,I]:=P[cs-1,I];
            q[cs,I+1]:=q[cs-1,I+1];
          END
        ELSE
          BEGIN
            WriteSynop(' Adj Chng Dn ');
            _mode:=_down;
            Change_dia(_down);
            Exit;
          END;
    END;
  END;                {-of proc. adjust}
```

FUNCTION Init_hydro_des:Boolean;

```
    {-Initiate variables at the start of the design}
  BEGIN
    Init_hydro_des:=True;
    _Stuck:=False;
    WITH MDes[Half] DO
      BEGIN
        FOR cs:=1 TO 10 DO
          BEGIN
            start_pt[cs]:=0;end_pt[cs]:=0;
            tag[cs]:=0;
            cd[cs]:=0;
            m_len[cs]:=0;m_noof[cs]:=0;
            FOR I:=1 TO Max_outlet DO
              BEGIN
                P[cs,I]:=0;q[cs,I]:=0;
              END;
          END;
        _mode:=_down;
        cs:=1;
        dia_posn[cs]:=1;
        WHILE m_dia[dia_posn[cs]]<d_param.dp_manif_dia DO
          BEGIN
            dia_posn[cs]:=dia_posn[cs]+1;
            IF dia_posn[cs]=11 THEN
              BEGIN
                stuck(4);
                Init_hydro_des:=False;
```

```
          END;
        END;        {-of while}
      IF NOT _Stuck THEN
        BEGIN
          no_shift:=false;
          I:=last_lat;
          start_pt[cs]:=I;
          tag[cs]:=I;
          tol_tag:=I;
          fail_start:=I;
          loop_cnt:=0;
          tolerance:=0;
          new_dia:=_up;
          shift_cnt:=0;
          P[cs,I]:=d_param.dp_pMax;
          minlen_check:=False;
          cd[cs]:=
            Mat_Detail.int_dia[_classIndex,m_dia_index[dia_posn[cs]]];
          start_dia:=cd[cs];
        END;
    END;            {-of with}
  END;              {-of proc. Init_hydro_des}
```

PROCEDURE Check_min_length;

```
    {-Check that length of previous diameter is not less than stipulated minimum}
  VAR
    ii:Integer;
    len:Real;
  BEGIN
    WriteSynop(' ChkMinLen   ');
    len:=0;
    minlen_check:=False;
    IF end_pt[cs-1]>start_pt[cs] THEN
      BEGIN
        FOR ii:=start_pt[cs-1] DOWNTO end_pt[cs-1] DO
          len:=len+Manif_out.outlet_spec[Half,ii];
      END;
    IF (len<d_param.dp_manif_minLen) OR (len=0) THEN
      BEGIN
        cs:=cs-1;
        I:=start_pt[cs];
        tag[cs]:=I;
        start_pt[cs+1]:=0;
        IF cs=1 THEN P[cs,I]:=d_param.dp_pMax;
        cd[cs]:=cd[cs+1];
        cd[cs+1]:=0;
        shift_cnt:=0;
        dia_posn[cs]:=dia_posn[cs+1];
        dia_posn[cs+1]:=0;
        _mode:=_down;
        new_dia:=_up;
        WriteLn(t_synop,'Minlen failed: length =',len:5:1);

        {*}         {Now check for looping - three conditions: }
        IF (cs>1) AND ((cd[cs]-cd[cs-1])<=0.001) THEN
          BEGIN
            cs:=cs-1;
```

```
          I:=start_pt[cs];
          tag[cs]:=I;
          start_pt[cs+1]:=0;
          IF cs=1 THEN P[cs,I]:=d_param.dp_pMax;
          cd[cs+1]:=0;
          tolerance:=tolerance+0.05;
          loop_cnt:=succ(loop_cnt);
        END
      ELSE IF ((cs=1) AND (cd[cs]<start_dia))
        OR ((cs>1) AND (I=fail_start)) THEN
        BEGIN
          tolerance:=tolerance+0.05;
          loop_cnt:=succ(loop_cnt);
        END;
      start_dia:=cd[cs];
      fail_start:=I;
    END;
  END;                {-of proc. Check_min_length}
```

PROCEDURE Open_synopsis;

```
  VAR
    it_cnt,t_cnt:Integer;
    line:ARRAY[1..100] OF STRING[80];
  BEGIN
    t_cnt:=0;
    IF Exist(DATAPATH+SYNOP) THEN
      BEGIN
        Assign(t_synop,DATAPATH+SYNOP);
        Reset(t_synop);
        WHILE NOT SeekEof(t_synop) DO
          BEGIN
            IF t_cnt<100 THEN
              BEGIN
                t_cnt:=Succ(t_cnt);
                ReadLn(t_synop,line[t_cnt]);
              END
            ELSE
              BEGIN
                FOR it_cnt:=1 TO 99 DO line[it_cnt]:=line[it_cnt+1];
                ReadLn(t_synop,line[100]);
              END;
          END;
        Close(t_synop);
        Erase(t_synop);
      END;
    Assign(t_synop,DATAPATH+SYNOP);
    Rewrite(t_synop);
    IF t_cnt>0 THEN FOR it_cnt:=1 TO t_cnt DO
      WriteLn(t_synop,line[it_cnt]);
  END;                {-of Open_synopsis}
```

PROCEDURE Phase1;

```
  BEGIN               {-main procedure}
    Open_synopsis;
    WriteLn(t_synop);
    IF Half=1 THEN man:='a/b' ELSE man:='c/d';
    WriteLn(t_synop,'DESIGN OF MANIF ',man,'      Date : ',DateString);
```

```
WriteLn(t_synop,'SECT  DIA(mm)  Start Pt & P  Current Pt & P  NEXT Pt & P  ACTION    TOLERANCE');
WriteLn(t_synop,'=====================================================================================');

IF Init_hydro_des THEN
  BEGIN
    {-*}          {--main loop until designed}
    REPEAT
      IF shift_cnt>10 THEN
        BEGIN
          stuck(1);
          Exit;
        END;

      IF loop_cnt>5 THEN
        BEGIN
          stuck(5);
          Exit;
        END;
      IF I<tol_tag THEN tol_tag:=I;
      calc_fwd;

      {-*}         {--set up cases}
      IF _mode=_down THEN
        IF (P[cs,I-1]>(1+tolerance)*d_param.dp_pMax)
          AND (I<start_pt[cs]) THEN
          BEGIN
            IF P[cs,I+1]<P[cs,I] THEN m:=1
            ELSE m:=2;
          END
        ELSE IF P[cs,I-1]>=(1-tolerance)*d_param.dp_max_pInlet THEN
          m:=2
        ELSE IF P[cs,I-1]<(1-tolerance)*d_param.dp_max_pInlet THEN
          m:=3;

      IF _mode=_up THEN
        IF (P[cs,I-1]<(1-tolerance)*d_param.dp_max_pInlet)
          AND (I<start_pt[cs]) THEN m:=1
        ELSE IF P[cs,I-1]<=(1+tolerance)*d_param.dp_pMax THEN m:=4
        ELSE IF P[cs,I-1]>(1+tolerance)*d_param.dp_pMax THEN m:=5;

      {*}          {-now implement the cases }
      CASE m OF

        1:IF no_shift THEN I:=I-1
          ELSE
            BEGIN
              WriteSynop(' Adjust      ');
              adjust;
            END;

        2:IF (I>=tag[cs]) OR (del_hl<=del_hg) THEN I:=I-1
          ELSE
            BEGIN
              WriteSynop(' Shft Dn     ');
              IF (I<tag[cs]) AND (del_hl>del_hg) THEN Shift_down;
            END;
```

```
          3:IF (no_shift) AND (I>tag[cs]) THEN I:=I-1
            ELSE
              BEGIN
                no_shift:=false;
                WriteSynop(' Chng Dn      ');
                change_dia(_down);
              END;

          4:IF (I>=tag[cs]) OR (del_hl>=del_hg) THEN I:=I-1
            ELSE
              BEGIN
                WriteSynop(' Shft Up      ');
                IF (I<tag[cs]) AND (del_hl<del_hg) THEN Shift_up;
              END;

          5:IF (no_shift) AND (I>tag[cs]) THEN I:=I-1
            ELSE
              BEGIN
                WriteSynop(' Chng Up      ');
                change_dia(_up);
              END;
        END;         {-of case}

        IF _Stuck THEN Exit;
        IF (minlen_check) AND (cs>1) THEN Check_min_length;

        IF (tolerance>0) AND (I<tol_tag) THEN tolerance:=0;

      UNTIL I=0;
    END;
  END;                {-endof Phase1}
```

PROCEDURE Phase2;

```
  BEGIN
    IF _Stuck THEN Exit;
    {-*}               {--now shift last dia}
    IF (P[cs,I]<0.99*d_param.dp_pMax) AND (new_dia=_up)
    AND (_mode=_up) THEN
      BEGIN
        WriteSynop(' End Shft Up ');
        Shift_up;
        WHILE I<>0 DO
          BEGIN
            calc_fwd;
            I:=Pred(I);
          END;
        WriteSynop(' Design Done ');
      END
    ELSE WriteSynop(' Design Done ');
    end_pt[cs]:=1;
    {-*}               {--now set up lengths & no-of emitterss}
    WITH MDes[Half] DO
      BEGIN
        FOR m:=1 TO cs DO
          BEGIN
            IF (m>1) AND (dia_posn[m]<dia_posn[m-1]) THEN
              BEGIN
```

```
            FOR I:=10 DOWNTO dia_posn[m-1]+2 DO
              m_dia[I]:=m_dia[I-1];
            m_dia[dia_posn[m-1]+1]:=m_dia[dia_posn[m]];
            dia_posn[m]:=dia_posn[m-1]+1;
            IF m<cs THEN
              FOR I:=m+1 TO cs DO
                BEGIN
                  IF dia_posn[I]>=dia_posn[m] THEN
                    dia_posn[I]:=dia_posn[I]+1;
                  IF dia_posn[I]>5 THEN
                    BEGIN
                      cs:=I-1;
                      stuck(2);
                      Exit;
                    END;
                END;{-of For I}
            Update_dia_cost;
          END;
        m_noof[dia_posn[m]]:=start_pt[m]-end_pt[m]+1;
        FOR I:=start_pt[m] DOWNTO end_pt[m] DO
          m_len[dia_posn[m]]:=m_len[dia_posn[m]]+
            Manif_out.outlet_spac[Half,I];
      END;        {-of for m}

    {-update design counter}
    m_designed:=True;
  END;              {-of with Des}
END;                {-phase2}


BEGIN    {-main procedure }
  Instr('Designing the manifold.....');
  Phase1;
  Phase2;
  Close(t_synop);
END;
```

APPENDIX 2b: Pascal Listings of Uniformity Calculation Routines

PROCEDURE Uniformity_calc(J:Integer);

```
  { Calculate the uniformity parameters for Unif[j].InletP }
 VAR
  Line:Anystr;
  a,b,Lat,e_cnt,quad,pass,n_emits,
  lnoof_cnt,l_cnt,mnoof_cnt,m_cnt:Integer;
  dia,t_dist,emit_q,HL,Slope,
  Q_tot,Q_curr,P_curr,EndP,InP:Real;
  InQ:ARRAY[1..2] OF Real;
  InP_adjust,InQ_adjust,early_adjust,lat_flag:Boolean;
  hlfstr:STRING[3];
  qdstr:STRING[1];
```

PROCEDURE SetVals;

```
  BEGIN
    WITH Unif[J] DO
      BEGIN
        IF pass=1 THEN
          BEGIN
            Av_q:=Av_q/n_emits;
            q_variance:=100*(q_max-q_min)/Av_q;
            Avq_nomq_ratio:=Av_q/d_param.dp_qNom;
            IF noof_highlow=0 THEN
              BEGIN
                noof_highlow:=1;
                highlow_laterals[1]:='none';
              END;
          END
        ELSE IF pass=2 THEN
          BEGIN
            Unif_coef:=Unif_coef/(n_emits*Av_q);
            Unif_coef:=(1-Unif_coef)*100;
          END;
      END;
  END;                 { endof SetVals }
```

PROCEDURE CalcStats;

```
  {-Calculate the statistics, once the discharges are established }
  VAR
    t_val:Real;
  BEGIN
    WITH Unif[J] DO
      BEGIN
        FOR quad:=1 TO 4 DO
          BEGIN
            IF Eval_gen.noof_pquad[quad]<>0 THEN
              BEGIN
                IF quad<=2 THEN Half:=1 ELSE Half:=2;
                FOR Lat:=1 TO Eval_gen.last_lat[Half] DO
                  BEGIN
                    lat_flag:=False;
```

```
e_cnt:=1;
CASE quad OF
  1:t_val:=qVal1^[Lat,e_cnt];
  2:t_val:=qVal2^[Lat,e_cnt];
  3:t_val:=qVal3^[Lat,e_cnt];
  4:t_val:=qVal4^[Lat,e_cnt];
END;
WHILE (t_val<>0) AND (e_cnt<=Max_outlet) DO
  BEGIN
    IF pass=1 THEN
      BEGIN
        n_emits:=Succ(n_emits);
        IF t_val<q_min THEN q_min:=t_val;
        IF t_val>q_max THEN q_max:=t_val;
        IF (t_val>
        Exp(Ln(d_param.dp_k)+d_param.dp_x*Ln(d_param.dp_pMax)))
        OR (t_val<
        Exp(Ln(d_param.dp_k)+d_param.dp_x*Ln(d_param.dp_pMin)))
        THEN
          IF NOT lat_flag THEN
            BEGIN
              lat_flag:=True;
              noof_highlow:=Succ(noof_highlow);
              { * }{ make a string of lat no & quad }
              Str(Lat:3,highlow_laterals[noof_highlow]);
              CASE quad OF
                1:highlow_laterals[noof_highlow]:=
                  highlow_laterals[noof_highlow]+'a';
                2:highlow_laterals[noof_highlow]:=
                  highlow_laterals[noof_highlow]+'b';
                3:highlow_laterals[noof_highlow]:=
                  highlow_laterals[noof_highlow]+'c';
                4:highlow_laterals[noof_highlow]:=
                  highlow_laterals[noof_highlow]+'d';
              END;
            END;
        Av_q:=Av_q+t_val;
      END
    ELSE IF pass=2 THEN
      Unif_coef:=Unif_coef+Abs(t_val-Av_q);
    e_cnt:=Succ(e_cnt);
    CASE quad OF
      1:t_val:=qVal1^[Lat,e_cnt];
      2:t_val:=qVal2^[Lat,e_cnt];
      3:t_val:=qVal3^[Lat,e_cnt];
      4:t_val:=qVal4^[Lat,e_cnt];
    END;
  END;{ endof WHILE t_val }

GoToXY(3,21);
IF Half=1 THEN hlfstr:='a/b' ELSE hlfstr:='c/d';
IF quad=1 THEN qdstr:='a';
IF quad=2 THEN qdstr:='b';
IF quad=3 THEN qdstr:='c';
IF quad=4 THEN qdstr:='d';
Write('<Half ',hlfstr,', Quad ',qdstr,', Pass ',pass:1,
', Lateral ',Lat:2,'>                    ');
```

```
                END;{ endof FOR lat cnt }
            END;    { endof IF tot_noof_emit<>0 }
        END;        { endof FOR quad cnt }
    END;            { endof WITH }
END;                { endof CalcStats }
```

PROCEDURE Check_InP_Adjust;

```
  { check for adjustment of InP against InletP }
 BEGIN
  IF Abs(InP-Unif[J].InletP)>0.02*Unif[J].InletP THEN
    BEGIN
      Lat:=Eval_gen.last_lat[Half];
      GetLateral(Half,Lat);
      EndP:=EndP-((InP-Unif[J].InletP)
      *(1-(Abs(InP-Unif[J].InletP))/Unif[J].InletP));
    END
  ELSE
    BEGIN
      Unif[J].InletQ:=Unif[J].InletQ+Q_tot;
      InP_adjust:=False;
    END;
 END;               { endof Check_InP_Adjust }
```

PROCEDURE SetNextLat;

```
  { establish inP & inQ for next lateral }
 BEGIN
  DialPipe(d_param.dp_manif_code);
  { * }             { get the head loss params }
  FOR I:=1 TO 4 DO
    Hlp[I]:=Mat_table[d_param.dp_manif_code].headloss[I];

  Q_tot:=Q_tot+InQ[a]+InQ[b];

  t_dist:=Manif_out.outlet_spec[Half,Lat];
  Slope:=Manif_out.outlet_elev[Half,Lat-1]
    -Manif_out.outlet_elev[Half,Lat];
  dia:=Mat_Detail.int_dia
    [MDes[1].m_class_index,MDes[Half].m_dia_index[m_cnt]];

  HL:=((Exp(Ln(Hlp[1])+Hlp[2]*Ln(Q_tot/(1000*Hlp[4]))
    +Hlp[3]*Ln(dia)))*t_dist)-(Slope);

  InP:=InP+HL;

  GoToXY(3,21);
  IF Half=1 THEN hlfstr:='a/b' ELSE hlfstr:='c/d';
  IF quad=1 THEN qdstr:='a';
  IF quad=2 THEN qdstr:='b';
  IF quad=3 THEN qdstr:='c';
  IF quad=4 THEN qdstr:='d';
  Write('<Half ',hlfstr,', Quad ',qdstr,', Lateral ',Lat-1:2,
    ', Inlet Pressure ',InP:6:2,'>');

  IF Lat<>1 THEN
    BEGIN
      Lat:=Lat-1;
      GetLateral(Half,Lat);
```

```
      InQ[a]:=LDes[a].l_coef[1]+LDes[a].l_coef[2]*InP;
      InQ[b]:=LDes[b].l_coef[1]+LDes[b].l_coef[2]*InP;
      mnoof_cnt:=mnoof_cnt+1;
      IF mnoof_cnt>MDes[Half].m_noof[m_cnt] THEN
        BEGIN
          mnoof_cnt:=1;
          m_cnt:=Succ(m_cnt);
          WHILE MDes[Half].m_noof[m_cnt]=0 DO m_cnt:=Succ(m_cnt);
        END;
    END
  ELSE Lat:=Lat-1;
END;                { endof SetNextLat }
```

PROCEDURE LateralCalc;

```
{-Calculate the discharges in the current lateral }
VAR
  emitq:Real;
BEGIN
  InQ_adjust:=True;
  REPEAT            { until InQ_adjust=false }
    P_curr:=InP;
    Q_curr:=InQ[quad];
    l_cnt:=5;
    WHILE LDes[quad].l_noof[l_cnt]=0 DO
      l_cnt:=Pred(l_cnt);
    lnoof_cnt:=0;
    early_adjust:=False;
    DialPipe(d_param.dp_lat_code);

    { * }           {  get the head loss params }
    FOR I:=1 TO 4 DO
      Hlp[I]:=Mat_table[d_param.dp_lat_code].headloss[I];

    { * }            { start the in-lateral loop }
    FOR e_cnt:=1 TO Lat_gen.noof_emit[quad] DO
      BEGIN
        IF Q_curr>0 THEN
          BEGIN
            lnoof_cnt:=lnoof_cnt+1;
            IF lnoof_cnt>LDes[quad].l_noof[l_cnt] THEN
              BEGIN
                lnoof_cnt:=1;
                l_cnt:=Pred(l_cnt);
                WHILE LDes[quad].l_noof[l_cnt]=0 DO l_cnt:=Pred(l_cnt);
              END;
            dia:=Mat_Detail.int_dia
            [sdia[quad]._class,sdia[quad]._sindex[l_cnt]];

            { * }     { t_dist }
            IF (e_cnt<>1) AND (Lat_gen.pr_emit_spec[quad]>0) THEN
              BEGIN
                IF NOT Odd(e_cnt) THEN t_dist:=Lat_gen.pr_emit_spec[quad]
                ELSE
                  t_dist:=Lat_gen.emit_spec[quad]-Lat_gen.pr_emit_spec[quad];
              END
            ELSE IF e_cnt<>1 THEN t_dist:=Lat_gen.emit_spec[quad];
            IF e_cnt=1 THEN t_dist:=Lat_gen.First_emit[quad];
```

```
          {*}     { slope }
          IF e_cnt<>1 THEN
            Slope:=el[quad,e_cnt-1]-el[quad,e_cnt]
          ELSE IF e_cnt=1 THEN
            Slope:=Manif_out.outlet_elev[Half,Lat]-el[quad,e_cnt];

          {*}     { head loss }
          HL:=((Exp(Ln(Hlp[1])+Hlp[2]*Ln(Q_curr/(1000*Hlp[4]))
            +Hlp[3]*Ln(dia)))*t_dist)-(Slope);
          P_curr:=P_curr-HL;
          emitq:=Exp(Ln(d_param.dp_k)+d_param.dp_x*Ln(P_curr));
          CASE quad OF
            1:qVal1^[Lat,e_cnt]:=emitq;
            2:qVal2^[Lat,e_cnt]:=emitq;
            3:qVal3^[Lat,e_cnt]:=emitq;
            4:qVal4^[Lat,e_cnt]:=emitq;
          END;
          Q_curr:=Q_curr-emitq;

        END
      ELSE early_adjust:=True;
    END;          { endof e_cnt FOR loop }

    {*}           {-test for satisfactory zero Q remaining in pipe }
    IF (Abs(Q_curr)<0.1*(emitq))
    AND (NOT early_adjust) THEN InQ_adjust:=False
    ELSE
      BEGIN
        IF early_adjust THEN InQ[quad]:=InQ[quad]*1.05
        ELSE InQ[quad]:=InQ[quad]-(Q_curr*0.95);
      END;

  UNTIL InQ_adjust=False;
END;              { endof LateralCalc }
```

PROCEDURE StartUp;

```
BEGIN
  Instr('Starting up...                              ');
  m_cnt:=1;
  WHILE MDes[Half].m_noof[m_cnt]=0 DO m_cnt:=Succ(m_cnt);
  mnoof_cnt:=1;
  Q_tot:=0;
  InP:=EndP;
  InQ[a]:=LDes[a].l_coef[1]+LDes[a].l_coef[2]*InP;
  InQ[b]:=LDes[b].l_coef[1]+LDes[b].l_coef[2]*InP;
END;              { endof StartUp }
```

PROCEDURE FindLastLat;

```
  { establish posn of last lateral on manif & initial endP }
BEGIN
  Instr('Finding last lateral.....                    ');
  Lat:=Des_hist.outlets[half];
  Eval_gen.last_lat[Half]:=0;
  REPEAT
    GetLateral(Half,Lat);
    IF (Lat_gen.noof_emit[a]<>0) OR (Lat_gen.noof_emit[b]<>0) THEN
      Eval_gen.last_lat[Half]:=Lat
```

```
      ELSE Lat:=Pred(Lat);
    UNTIL Eval_gen.last_lat[Half]<>0;
    EndP:=MDes[Half].m_EndP+(Unif[J].InletP-MDes[Half].m_InP);
    InP_adjust:=True;
  END;                  { endof FindLastLat }
```

PROCEDURE FlushqVals;

```
  {-flush the memory on the heap }
  VAR
    quad,L_t,Emit:Integer;
    t_str:STRING[3];
    m_str:STRING[10];
  BEGIN
    FOR quad:=1 TO 4 DO
      FOR L_t:=1 TO Max_outlet DO
        BEGIN
          str(quad:3,t_str);
          m_str:='('+t_str+',';
          str(L_t:3,t_str);
          m_str:=m_str+t_str+')';
          Instr('Flushing qVals on the heap for lateral '+m_str);
          FOR Emit:=1 TO Max_outlet DO
            BEGIN
              IF quad=1 THEN qVal1^[L_t,Emit]:=0;
              IF quad=2 THEN qVal2^[L_t,Emit]:=0;
              IF quad=3 THEN qVal3^[L_t,Emit]:=0;
              IF quad=4 THEN qVal4^[L_t,Emit]:=0;
            END;
        END;
    Unif[J].InletQ:=0;
  END;                  { endof FlushqVals }
```

BEGIN { main Uniformity Calc procedure }

```
  IF _page<>1 THEN
    BEGIN
      _page:=1;
      LoadScreen(eval1screen);
      FOR t_psn:=1 TO 4 DO fill_page(1,t_psn);
    END;
  __COffScn(True);

  WITH Unif[J] DO
    BEGIN
      Instr('Uniformity evaluation in progress.....             ');
      _Inverse;
      TextColor(0+Blink);
      TextBackground(7);
      GoToXY(x_p[_page,J],y_p[_page]);
      Write(InletP:4:1);
      _Inverse;

      { * }             { now begin the main calc }
      FlushqVals;
      FOR Half:=1 TO 2 DO
        BEGIN
          IF Half=1 THEN
            BEGIN
```

```
          a:=1;b:=2;
        END
      ELSE
        BEGIN
          a:=3;b:=4;
        END;
      IF (Eval_gen.noof_pquad[a]<>0)
      OR (Eval_gen.noof_pquad[b]<>0) THEN
        BEGIN
          FindLastLat;
          REPEAT                    { loop until inletP matches Unif[j].InletP }
            StartUp;
            Instr('Calculating the emitter discharges....        ');
            REPEAT{ loop with each lat until manif inlet reached }
              FOR quad:=a TO b DO{ do current lateral in each quad }
                IF Lat_gen.noof_emit[quad]<>0 THEN LateralCalc;
              SetNextLat;
            UNTIL Lat=0;            { all laterals done }

            Check_InP_Adjust;
          UNTIL InP_adjust=False;   { now InP matches Unif[j].InletP }
        END;      { endof IF half exists }
    END;          { endof FOR half }

    { * }          { now calc the uniformity statistics }
    Instr('Calculating the uniformity parameters....             ');
    q_min:=9999.0;
    q_max:=0.0;
    n_emits:=0;
    Av_q:=0;
    FOR pass:=1 TO 2 DO
      BEGIN
        CalcStats;
        SetVals;
      END;

    { * }          { restore the screen & save the discharge values = qVals }
    IF _page=1 THEN
      Line:='                                                        '
    ELSE
      IF _page=2 THEN
        Line:='Laterals with Pressure Profiles Beyond Allowable Limits';
    GoToXY(3,5);Write(Line);
    GoToXY(3,20);Write('':60);
    GoToXY(3,21);Write('':60);
    SaveqFile(J);
  END;            { endof With }
  __COffScn(False);
END;              { endof Uniformity calc }
```

APPENDIX 2c: Pascal Listings of Economic Evaluation Routines

PROCEDURE get_econ_eval;

```
{carry out the economic evaluation & display the results on page 6}
VAR
  EconFile:FILE OF econ_eval_type;
  Te:Char;
  Period,Frac,t_frac,quad,Lat,e_cnt,Half,applic_rem,
  I,state_cnt,applic_cnt,prev_state:Integer;
  CRF,PVF_prod,PVF_energy,PVF_earning,PVF_general,
  k_fixed,k_yield,k_operat,nett_earn,yield_ratio,yield_loss,
  income,max_yield:Real;
  NPV_cost,
  req_eff:ARRAY[1..8,1..5] OF Real;
  applic:ARRAY[1..8,1..5] OF Integer;
  cum_cost:ARRAY[1..5] OF Real;
  feasib_state:ARRAY[1..8,1..2] OF Integer;
  min_cum_cost:ARRAY[1..8,0..300] OF Real;
  opt_applic:ARRAY[1..8,0..300] OF Integer;{- put on heap??? -}
  opt_flag:Boolean;
  ee:Char;
```

PROCEDURE Setup_Costs;

```
    {-calculate CRF, PVF's & cost factors}

   FUNCTION PVF_calc(inflat:Real):Real;
     VAR
       rate,power_rate:Real;
     BEGIN
       IF inflat=0 THEN rate:=1+(page5_vars.PV_interest/100)
       ELSE rate:=(1+(page5_vars.PV_interest/100))/(1+(inflat/100));
       power_rate:=Exp(page5_vars.PV_period*Ln(rate));
       IF rate=1 THEN PVF_calc:=page5_vars.PV_period
       ELSE PVF_calc:=(power_rate-1)/((rate-1)*power_rate);
     END;

   BEGIN
     CRF:=1/PVF_calc(0);
     PVF_prod:=PVF_calc(page5_vars.PV_inflat_prod);
     PVF_energy:=PVF_calc(page5_vars.PV_inflat_engy);
     PVF_earning:=PVF_calc(page5_vars.PV_inflat_earn);
     PVF_general:=PVF_calc(page5_vars.PV_inflat_gnrl);

     k_fixed:=page4_vars.total_fixed_costs
       +((page4_vars.total_maint_costs
       +page4_vars.fixed_prod_costs)*PVF_prod);    {R/Ha}

     k_yield:=PVF_prod*page4_vars.yield_prod_costs;    {R/ton}

     k_operat:=((page5_vars.tot_energy_cost*PVF_energy)
       +((page5_vars.water_base_cost
       +page5_vars.water_opport_cost)*PVF_prod))
       *(10*page3_vars.block_area/100)*(100/page3_vars.gross_effic);    {R/mm}
```

```
    nett_earn:=(page5_vars.earnings*PVF_earning)-k_yield;   {R/ton}

    GoToXY(5,7);Write('PVF_production=',PVF_prod:5:2,'  PVF_energy =',PVF_energy:5:2);
    GoToXY(5,8);Write('PVF_earnings  =',PVF_earning:5:2,'  PVF_general=',PVF_general:5:2);
    GoToXY(5,9);Write('CRF=',CRF:5:2);
    GoToXY(5,10);Write('PV of fixed costs           = R',k_fixed:7:2,'/Ha');
    GoToXY(5,11);Write('PV of yield-production costs = R',k_yield:7:2,'/ton');
    GoToXY(5,12);Write('PV of operating costs       = R',k_operat:7:2,'/mm');

  END;               {-endof Setup_Costs}
```

PROCEDURE CalcDP_matrix;

```
    {-calc application (in units of 5mm), yield loss
    & loss_cost matrix}
  VAR
    t_val:Real;
    t_max:Integer;
  BEGIN
    FOR Period:=1 TO page3_vars.no_periods DO
      BEGIN
        FOR Frac:=1 TO 5 DO
          BEGIN
            req_eff[Period,Frac]:=0;

            {-Calc totl application in each period for each frac,
              to nearest 5mm. Cannot exceed max system capacity -}

            applic[Period,Frac]:=Trunc(0.5+(Eval_gen.applic_frac[Frac]
              *page3_vars.Reqmnt[Period]*page3_vars.cycles[Period]));

            t_max:=page3_vars.max_applic*page3_vars.cycles[Period];
            IF applic[period,frac]>t_max THEN applic[period,frac]:=t_max;

            applic_rem:=applic[Period,Frac] MOD 5;
            IF applic_rem<3 THEN
              applic[Period,Frac]:=applic[Period,Frac]-applic_rem
            ELSE
              applic[Period,Frac]:=applic[Period,Frac]+(5-applic_rem);

            {-establish yield loss & associated cost matrix-}
            yield_loss:=0;
            FOR quad:=1 TO 4 DO
              BEGIN
                IF quad<=2 THEN Half:=1 ELSE Half:=2;
                IF Eval_gen.noof_pquad[quad]<>0 THEN
                  BEGIN
                    FOR Lat:=1 TO Eval_gen.last_lat[Half] DO
                      BEGIN
                        e_cnt:=1;
                        CASE quad OF
                          1:t_val:=qVal1^[Lat,e_cnt];
                          2:t_val:=qVal2^[Lat,e_cnt];
                          3:t_val:=qVal3^[Lat,e_cnt];
                          4:t_val:=qVal4^[Lat,e_cnt];
                        END;
                        WHILE (t_val<>0) AND (e_cnt<=Max_outlet) DO
                          BEGIN
```

```
                    yield_ratio:=
                      (t_val/Eval_gen.Avq_select)
                      *(applic[Period,Frac]
                      /(page3_vars.Reqmnt[Period]*page3_vars.cycles[Period]));
                    IF yield_ratio>1 THEN yield_ratio:=1;

                    req_eff[Period,Frac]:=
                      req_eff[Period,Frac]+yield_ratio;

                    yield_loss:=yield_loss
                      +((1-yield_ratio)*page3_vars.Ky[Period]
                      *(page3_vars.cycles[Period]
                      /page3_vars.total_cycles));

                    e_cnt:=Succ(e_cnt);
                    CASE quad OF
                      1:t_val:=qVal1^[Lat,e_cnt];
                      2:t_val:=qVal2^[Lat,e_cnt];
                      3:t_val:=qVal3^[Lat,e_cnt];
                      4:t_val:=qVal4^[Lat,e_cnt];
                    END;
                  END;{-WHILE t_val}
              END;{-endof FOR lat cnt}
          END;{-endof IF noof_pquad<>0}
      END;    {-endof FOR quad cnt}

    yield_loss:=yield_loss/page3_vars.total_noof;
    req_eff[Period,Frac]:=
      req_eff[Period,Frac]/page3_vars.total_noof;

    NPV_cost[Period,Frac]:=(nett_earn*yield_loss
      *page3_vars.max_crop_yield*page3_vars.block_area)
      +(applic[Period,Frac]*k_operat);

    GoToXY(5,13+Frac);Write('Period ',Period:1,'; ',100*Eval_gen.applic_frac[Frac]:4:0,
      '% application; Yield loss=',100*yield_loss:5:2,'%');

  END;          {-endof FOR frac cnt}
 END;           {-endof FOR period cnt}
END;            {-endof CalcDP_matrix}
```

PROCEDURE PerformDP;

```
  {-dynamic programming algorithm}
 BEGIN
  {-establish range of possible avail water levels at each period}
  FOR I:=1 TO 8 DO
   BEGIN
    IF I=1 THEN
     BEGIN
      feasib_state[I,1]:=page3_vars.avail_water_mm;
      feasib_state[I,2]:=page3_vars.avail_water_mm;
     END;
    IF (I>1) AND (I<page3_vars.no_periods+1) THEN
     BEGIN
      {-max state}
      feasib_state[I,1]:=feasib_state[I-1,1]-applic[I-1,1];
```

```
          {-min state}
          feasib_state[I,2]:=feasib_state[I-1,2]-applic[I-1,5];
        END;
      IF I>page3_vars.no_periods THEN
        BEGIN
          feasib_state[I,1]:=0;
          feasib_state[I,2]:=0;
        END;
      IF feasib_state[I,1]<0 THEN feasib_state[I,1]:=0;
      IF feasib_state[I,2]<0 THEN feasib_state[I,2]:=0;
    END;

  {-now do cost calcs for each possible avail water state
    in each period}

  FOR Period:=page3_vars.no_periods DOWNTO 1 DO
    BEGIN
      {-establish cum. cost for each frac of each state
        Note: 1. only cum costs for period 1 will be saved
              2. state_cnt in single integers -}

      FOR state_cnt:=feasib_state[Period,2] DIV 5 TO
        feasib_state[Period,1] DIV 5 DO
        BEGIN
          FOR Frac:=1 TO 5 DO
            BEGIN
              IF Period<page3_vars.no_periods THEN
                BEGIN
                  prev_state:=state_cnt-(applic[Period,Frac] DIV 5);
                  IF applic[Period,Frac]>state_cnt*5 THEN
                    cum_cost[Frac]:=10000*NPV_cost[Period,Frac]
                  ELSE
                    cum_cost[Frac]:=NPV_cost[Period,Frac]
                      +min_cum_cost[Period+1,prev_state];
                END;
              IF Period=page3_vars.no_periods THEN
                BEGIN
                  IF applic[Period,Frac]>state_cnt*5 THEN
                    cum_cost[Frac]:=10000*NPV_cost[Period,Frac]
                  ELSE cum_cost[Frac]:=NPV_cost[Period,Frac];
                END;
              {-establish min cum cost & associated frac for each
                state -}
              IF Frac=1 THEN
                BEGIN
                  min_cum_cost[Period,state_cnt]:=cum_cost[Frac];
                  opt_applic[Period,state_cnt]:=Frac;
                END;
              IF (cum_cost[Frac]<min_cum_cost[Period,state_cnt]) THEN
                BEGIN
                  min_cum_cost[Period,state_cnt]:=cum_cost[Frac];
                  opt_applic[Period,state_cnt]:=Frac;
                END;
            END;    {-endof FOR frac cnt}
        END;        {-endof FOR state_cnt}
    END;            {-endof FOR period cnt}
END;                {-endof PerformDP}
```

PROCEDURE SortDepth;

```
{-sort the five depths & associated results into increasing order }
var
i,j:Integer;

PROCEDURE _Swap;
  {-swap 2 sets using t_vars}
  var
    t_depth:Integer;
    t_vol,t_yield_pHa,t_yield_pm3:Real;
    t_applic_effic,t_reqmnt_effic:Integer;
    t_eaw_pHa,t_eaw_pm3:Real;
  BEGIN
    WITH Econ_vars DO
      begin
        {-depth}
          t_depth:=depth[i];
          depth[i]:=depth[j];
          depth[j]:=t_depth;
        {-vol}
          t_vol:=vol[i];
          vol[i]:=vol[j];
          vol[j]:=t_vol;
        {-yield_pHa}
          t_yield_pHa:=yield_pHa[i];
          yield_pHa[i]:=yield_pHa[j];
          yield_pHa[j]:=t_yield_pHa;
        {-yield_pm3}
          t_yield_pm3:=yield_pm3[i];
          yield_pm3[i]:=yield_pm3[j];
          yield_pm3[j]:=t_yield_pm3;
        {-applic_effic}
          t_applic_effic:=applic_effic[i];
          applic_effic[i]:=applic_effic[j];
          applic_effic[j]:=t_applic_effic;
        {-reqmnt_effic}
          t_reqmnt_effic:=reqmnt_effic[i];
          reqmnt_effic[i]:=reqmnt_effic[j];
          reqmnt_effic[j]:=t_reqmnt_effic;
        {-eaw_pha}
          t_eaw_pha:=eaw_pha[i];
          eaw_pha[i]:=eaw_pha[j];
          eaw_pha[j]:=t_eaw_pha;
        {-eaw_pm3}
          t_eaw_pm3:=eaw_pm3[i];
          eaw_pm3[i]:=eaw_pm3[j];
          eaw_pm3[j]:=t_eaw_pm3;
      end;
  END;{-endof _Swap}
{-sort econ_vars.depth and friends on key=depth}
BEGIN
  for i:=1 to 4 do
    for j:=i+1 to 5 do
      if econ_vars.depth[i]>econ_vars.depth[j] then _swap;
END;{-endof SortDepth}
```

```
{- Now GetDP_results produces:
    a)the results for each yield fraction, assuming applic
      fraction to be constant in each period, except for the
      optimal frac in period 1 which then follows the optimal
      path through the DP matrices.
    b)the optimal operating depths in each period.-}
```

PROCEDURE GetDP_results;

```
  {-calculate results of DP}
VAR
  yldloss_cost,t_reqeff:Real;
BEGIN
  WITH Econ_vars DO
    BEGIN
      FOR I:=1 TO 8 DO opt_applic_pCycle[I]:=0;
      opt_applic_total:=0;

      FOR Frac:=1 TO 5 DO
        BEGIN
          {-1st period-}
          state_cnt:=page3_vars.avail_water_mm DIV 5;
          opt_flag:=(Frac=opt_applic[1,state_cnt]);
          depth[Frac]:=applic[1,Frac];
          IF opt_flag THEN
            BEGIN
              opt_applic_pCycle[1]:=
                trunc(0.5+(applic[1,Frac]/page3_vars.cycles[1]));
              opt_applic_total:=
                (opt_applic_pCycle[1]*page3_vars.cycles[1]);
            END;
          t_frac:=Frac;
          t_reqeff:=req_eff[1,Frac]*page3_vars.cycles[1];
          yldloss_cost:=NPV_cost[1,Frac];

          {-each other period-}
          FOR Period:=2 TO page3_vars.no_periods DO
            BEGIN
              state_cnt:=state_cnt-(applic[Period-1,t_frac] DIV 5);
              t_frac:=opt_applic[Period,state_cnt];
              IF opt_flag THEN
                BEGIN
                  opt_applic_pCycle[Period]:=
                    trunc(0.5+(applic[Period,t_frac]/page3_vars.cycles[period]));
                  opt_applic_total:=opt_applic_total+
                    (opt_applic_pCycle[period]*page3_vars.cycles[period]);
                  t_reqeff:=t_reqeff+(req_eff[Period,t_frac]*
                    page3_vars.cycles[Period]);
                END
              ELSE
                BEGIN
                  depth[Frac]:=depth[Frac]+applic[Period,Frac];
                  t_reqeff:=t_reqeff+(req_eff[Period,Frac]*
                    page3_vars.cycles[Period]);
                  yldloss_cost:=yldloss_cost+NPV_cost[Period,Frac];
                END;
            END;
          IF opt_flag THEN
```

```
          BEGIN
            depth[frac]:=opt_applic_total;
            yldloss_cost:=cum_cost[frac];
          END;


        {-now establish the values-}
        vol[Frac]:=depth[Frac]*page3_vars.block_area*10;
        state_cnt:=page3_vars.avail_water_mm DIV 5;

        income:=(nett_earn*page3_vars.max_crop_yield
          *page3_vars.block_area)
          -(yldloss_cost)-(k_fixed*page3_vars.block_area);

        EAW_pHa[Frac]:=income*CRF/page3_vars.block_area;
        EAW_pm3[Frac]:=income*CRF/vol[Frac];

        max_yield:=page3_vars.max_crop_yield*page3_vars.block_area;
        yield_loss:=
          (yldloss_cost-(depth[Frac]*k_operat))/nett_earn;
        yield_pHa[Frac]:=
          (max_yield-yield_loss)/page3_vars.block_area;
        yield_pm3[Frac]:=1000*(max_yield-yield_loss)/vol[Frac];

        reqmnt_effic[Frac]:=
          Trunc(100*t_reqeff/page3_vars.total_cycles);
        applic_effic[Frac]:=
          Trunc((page3_vars.total_reqmnt/depth[Frac])
            *reqmnt_effic[Frac]);
        IF applic_effic[Frac]>100 THEN applic_effic[Frac]:=100;

      END;          {-endof FOR frac}
    SortDepth;
  END;              {-endof WITH}
END;                {-endof GetDP_results}

BEGIN          { main procedure }
  IF (NOT ReadPage6) OR (NOT Eval_gen.econ_eval_exists) THEN
    BEGIN
      LoadScreen(Eval6Screen);
      Setup_Costs;
      CalcDP_matrix;
      PerformDP;
      GetDP_results;
      SavePage6;
      Eval_gen.econ_eval_exists:=True;
    END;
END;           { of proc }
```

Appendix 3 : Glossary and Notation

Application rate. The rate of application of water to the plant from an operating irrigation system. Normally expressed in mm/hr.

Block. An area of land commanded from a single point in the irrigation system and irrigated together in one irrigation set. The command point is normally a valve and the block or *in-field* irrigation system consists of a manifold pipe from the valve, distributing into several lateral pipes along the manifold.

Depth of irrigation. Represents the amount of water applied to the plant. Irrigation quantities are normally expressed in terms of volume per unit area or "*depth*". Usually expressed in mm.

Design process. The series of procedures whereby the irrigation system components are established.

> **Design criteria.** Values of the various design parameters by which the adequacy of the design is measured.
>
> **Design objectives.** A predefined set of values used as the basic aims of the design process.
>
> **Design parameters.** Factors which are used as pivots in the design process in order to establish the various system components.
>
> **Evaluation.** The process of determining the expected performance of the designed system. See chapter 3.
>
> **Irrigation quality.** A generic term representing the extent to which an irrigation satisfies the plant requirements. Measured in terms of several parameters such the coefficient of uniformity and the various efficiency values. See chapter 3 for a full discussion.
>
> **Performance.** A generic term representing the extent to which the designed system meets the design objectives. See chapter 3.

Efficiency. A measure of the extent to which water applied by an irrigation system is effectively used by the plant :

> **Application efficiency.** The fraction of the total volume of water applied during an irrigation that becomes available to the plant rather than being lost through evaporation or deep percolation in the soil. Normally expressed as a percentage.

Requirement efficiency. The fraction of the total plant water requirement that is met by the water that becomes available to the plant after an irrigation.

Emitters. The devices through which water is discharged from the irrigation system, to the plant environment. They may be sprinklers, micro-sprayers or drippers, and are located on the lateral pipes.

Emitter spacing. Two dimensions representing the spacing of emitters along the lateral and between laterals respectively.

Evaluation. See under "*design process*".

Evapotranspiration. Represents the combined rate of transpiration and evaporation of water from the plant environment. It can be considered as an expression of the crop water requirements needed for growth. Normally expressed in mm/day.

Actual evapotranspiration. The actual rate of transpiration of a specified crop grown under specified agronomic conditions which may or may not include adequate water.

Maximum evapotranspiration. The maximum rate of transpiration of a healthy crop grown in a large field under ideal agronomic conditions and in particular with adequate water.

Irrigation.

Cycle. The period of time between irrigations on a particular block. Thus each irrigation is intended to apply enough water to last for the duration of the cycle period.

Set. The length of time of a single irrigation on a particular block, i.e. the duration of a single water application.

Shift. The same as a "*set*". There are normally several shifts within a cycle, and different blocks are irrigated in each shift.

Laterals. The pipes which run parallel to each other at regular intervals in a field and onto which the emitters are attached, also at regular intervals. The laterals are connected to the "*manifold*" or "*branchline*".

Mainline. The network of pipes, usually buried, which carry water from the source to all the block command points. These command points are usually valves which control delivery to the manifolds.

Manifold or Branchline. The pipe running from the block command point (usually a valve) and distributing water to the laterals, which are attached to the manifold at regular intervals along its length.

Objectives. See under "design process".

Performance. See under "design process".

Quality of irrigation. See under "design process".

Soil.

Moisture holding capacity. The amount of water that can be stored by the soil and made available to the plant.

Infiltration rate. The rate at which the soil can absorb water from its surface. Normally expressed in mm/hr.

Soil/plant/water relationships. A generic term which refers to the various relationships governing the interactions between water, the soil, the plant and the atmosphere respectively. These relationships determine firstly the plant water requirements and secondly the availability of the water to the plant.

System components.

Hardware. The physical elements of an irrigation system, such as the pipes, the emitters and the accessories (pumps, valves, fittings, etc.)

System characteristics. All of the non-physical attributes of an irrigation system, such as the operating regime, the pumping requirements and the capacity of the system.

See chapter 2 for a full discussion.

Valve.

Sequencing. The order in which the irrigation block valves are operated within an irrigation cycle. The cycle is divided into a number of "*shifts*", and the valves which are to be grouped together to operate in each shift must be determined.

Schedules. The list of valves in each shift are refered to as "*operating schedules*", and the sequencing operation is sometimes termed "*valve scheduling*".

Notation

In general :

Q	=	flow in a pipe section (m^3/h)
q	=	discharge from an emitter ($l$ph)
J	=	friction headloss gradient (headloss per unit length)
Hl	=	actual friction headloss
Hg	=	actual head gain (or loss) due to topographic slope
RAW	=	readily available water which can be extracted by a plant from a soil that is wet to field capacity (mm)
TI_{max}	=	maximum irrigation interval (days)
Ea_g	=	gross application efficiency (%)
IA_{max}	=	maximum peak application per irrigation, due to agro-climatic factors (mm)
Nb	=	number of blocks to be irrigated in a field
Td	=	time per day available for irrigation (hrs)
AR_{max}	=	maximum system application rate (mm/h)
q_{nom}	=	nominal emitter discharge
AR_{act}	=	actual system application rate (mm/h)
AT	=	total area of system (ha)
AI_s	=	area irrigated per set (ha)
Ne	=	number of emitters per set
Q_{cap}	=	total discharge capacity of the system (m^3/h)

In the Linear Programming model :

$QP(l)$	=	pump discharge in shift or loading l (m^3/h)
$XP(l)$	=	pump pressure in shift or loading l (m^3/h)
XPM	=	maximum pump pressure (kpa or m)
X_{ijm}	=	the length of candidate diameter m in link ij (m)
L_{ij}	=	length of link ij (m)

In the Evaluation model :

Ya	=	actual yield/unit area (tons/ha)
Ym	=	maximum potential yield/unit area (tons/ha)
Eta	=	actual evapotranspiration (mm)
Etm	=	maximum potential evapotranspiration (mm)
ky	=	yield response factor
IA	=	average irrigation application depth (mm)
IR	=	irrigation requirement for maximum yield (mm)
Af_i	=	application factor for emitter i
YL_i	=	yield loss for emitter i
YL_b	=	yield loss for block
ar	=	application ratio