UNIVERSITY OF SOUTHAMPTON

A METHODOLOGY FOR EXPENDITURE PLANNING OF IRRIGATION INFRASTRUCTURE USING HYDRAULIC MODELLING TECHNIQUES

By

Khaled Mohamed Samir El-Askari

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Department of Civil and Environmental Engineering

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ABSTRACT

FACULTY OF ENGINEERING AND APPLIED SCIENCE

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Poor performance of existing irrigation schemes has been well documented. Increased world population and competition with other uses of water require that irrigation schemes be better managed and more productive. One of the main causes of poor performance is the low expenditure on irrigation infrastructure due to the limited financial resources of developing countries, where the majority of irrigated agriculture exist. Consequently, irrigation systems are failing well within their design lifetime, wasting the large capital investments made in their construction.

In addition there is an increasing interest in the long-term performance of irrigation schemes, and in expenditure and asset management planning to ensure sustained levels of service. At present, no complete methodology or procedure exists for linking the expenditure on irrigation infrastructure to improvement in system performance. Such methodology is paramount for planning efficient expenditures which, when made, should sustain the level of performance/service of irrigation systems as expected by their beneficiaries, at the least cost possible.

A methodology for linking irrigation system performance to structure and conveyance system condition has been developed in this research. It enables different expenditure options to be considered and assessed, which is an essential element of any asset management planning tool. The methodology uses hydraulic modelling techniques as its main analytical tool besides performance assessment and cost-benefit analysis. Approaches of multi-criteria analysis are used to aggregate the various hydraulic performance and other criteria used to evaluate expenditure alternatives into overall performance scores.

The development of the methodology was achieved through the investigation of some common and important infrastructure-related problems. Two main problems related to canal networks and regulator structures were investigated on a real-life case study using hydraulic modelling. Procedures for quantifying the impacts of each problem and analysing the possible alternatives for curing them are presented. The research shows that the developed methodology has been successful as a planning and decision aid tool in analysing the expenditure alternatives of the cases studied. Nevertheless, the methodology is not limited to these two cases only. General procedures for analysing any infrastructure-related problem which affects hydraulic performance are also outlined.

In addition, the methodology is also applicable to the problem of evaluating and planning long-term investments on infrastructure upgrading/modernisation.

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Abbreviations

AHP	=	Analytic Hierarchy Process
AMP	=	Asset Management Planning
ASCE	=	American Society of Civil Engineers
DCR	=	Discharge Capacity Ratio
DPR	=	Delivery Performance Ratio
FAO	=	Food and Agriculture Organization of the United Nations
ICID	=	International Commission on Irrigation and Drainage
IQR	=	Interquartile Ratio
IWMI	=	International Water Management Institute
LFb	=	Lost Freeboard
MEA	=	Modern Equivalent Asset
USD ($)	=	United States Dollars

1. Introduction

1.1 Background

The role irrigation plays in increasing agricultural production is well recognised. About 200 million hectares or 17% of the world's croplands are irrigated, consuming 72% of the total withdrawals for the world at large (IWMI, 1998). This land produces one third of the world food production (Oi, 1997). In arid regions, irrigated agriculture is almost entirely responsible for producing the entire food production of those regions. The importance of irrigation to the economics of many countries, especially developing ones, is reflected in the large expenditures made in irrigation projects, which are funded by governments and international loans and grants (Skutsch, 1998). At present, almost three-quarters of the world's irrigated area is in developing countries.

It is widely recognized in many countries that increased agricultural production to match ever increasing population growth will have to come from the irrigated agriculture sector (Hennessy, 1993). The Food and Agriculture Organization of the United Nations (FAO, 1996) has estimated that 60% of the extra food needed to meet population growth throughout the first half of the 21^{st} century will need to be provided by irrigated agriculture. However, many irrigation systems in several parts of the world are performing well below their potential. Governments and donor agencies are becoming more interested in improving the performance of existing irrigation schemes due to the high costs of constructing new projects compared with the much lower costs of rehabilitating existing ones (about 15% of the cost of new schemes) (Clyma & Lowdermilk, 1988). Around 66% of recent loans from the World Bank to the irrigation sector has been spent on rehabilitating systems which have suffered premature technical failure due to neglect of maintenance (World Bank, 1994).

International lending banks and agencies provide loans for the construction and rehabilitation of irrigation systems, but have traditionally been reluctant to fund recurrent expenditure, seeing operation and maintenance as the responsibility of the beneficiaries. Normal lending agreements require increased spending on operation and maintenance in order to protect the capital investment. Such spending should be funded by higher water and other service charges. There is no evidence, however, of better cost recovery or adequate expenditure by developing governments on system maintenance (Jones, 1995). The result of which is billions of dollars invested in the original infrastructure of irrigation systems are being written off because recurrent expenditure is inadequate.

1.2 Problem Outline

The above brief background regarding the current status of expenditure on irrigation infrastructure and achieved levels of performance can be outlined as follows:

- Presently, there is a significant pressure on the agriculture sector for increased food production to provide for the growing world population. In addition, there is an increasing interest in the long-term performance of irrigation schemes, and in expenditure planning to ensure sustained levels of performance (World Bank, 1994).

- Not enough money is being spent on the maintenance of irrigation infrastructure to sustain their performance due to pressures on governments in developing countries to continuously extend their rural development programmes to provide for growing populations and/or failure to recover the actual cost from the beneficiaries of the irrigation systems.

- Irrigation systems are failing well within their design lifetimes, wasting the large capital investments made in their construction.

- No methodology or procedure currently exists for linking expenditure on irrigation infrastructure to return and performance improvement (Burton et al., 1999[1]).

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[1] The first draft of the paper was submitted in 1997.

1.3 Research Hypothesis

The hypothesis of this research is that hydraulic modelling techniques can be used for analysing and linking changes in the condition of irrigation infrastructure to performance improvement and return to expenditure/investment. This hypothesis is based on the fact that in hydraulically linked systems such as irrigation networks, changes in one part of the system have consequences/impacts on other parts. The use of hydraulic modelling as an analytical tool for assessing such impacts is possible, if not essential, for replacing the subjective methods which are currently used.

1.4 Research Objectives

This research sets out to develop a methodology for assessing and planning the expenditure on irrigation hydraulic structures using hydraulic modelling techniques. In particular, the research focuses on the following objectives:

- to develop a methodology for assessing irrigation system performance based on structure and conveyance system condition;

- to investigate the potential and the ways in which hydraulic modelling techniques can be utilised in developing the methodology;

- to link the expenditure/investment on irrigation infrastructure to the potential for performance enhancement;

- to demonstrate the application of the methodology in analysing the costs and benefits of expenditure options in order to test their financial viabilities; and,

- to investigate the application of the methodology in evaluating and prioritising different expenditure strategies, especially when resources are scarce.

It must be noted that the objective of the research is to study the impact of physical infrastructure improvement on the performance of irrigation systems. It is **not** the intention to study the impact of **both** operation and physical structure improvement on performance. Whilst it is true that the role of better management and operation in improving scheme performance is now widely recognised and that improvement in physical infrastructure must be twined with improved management, both time and resources do not permit studying the interaction between these two factors in this research. It is, therefore, assumed in this work that no other factors limit scheme performance except the condition of its infrastructure and consequently improvements in performance due to infrastructure improvements will not be impeded by other factors.

The research will focus on open-channel irrigation systems since they form the majority of irrigation schemes worldwide. However, the research aims at developing a generic expenditure planning methodology which is not limited to certain situations.

1.5 Approach and Methodology

The approach and methodology to achieving the goals of the research can be summarised as follows:

1) Obtain the following data from selected scheme(s):
   - the layouts of the irrigation systems and their physical components;
   - typical cropping patterns and potential crop yields;
   - available water resources, the climate and crop water requirements;
   - procedures explaining the operation and maintenance of the systems; and,
   - the expenditures on maintenance, rehabilitation and modernisation (if applicable) including the processes, activities and resources used.

2) Set up hydraulic models for the selected irrigation system(s). ISIS Flow was selected as the hydraulic simulation software to be used in this research (an overview of the software is available in Appendix I). Ensure that under design conditions, the output from the hydraulic model is sufficiently close to that which the design indicates (model calibration).

3) Develop and test performance assessment measures and indicators which can be used with the output from hydraulic modelling to assess the hydraulic performance of the scenarios simulated.

4) Introduce some of the problems which are common to the hydraulic infrastructure (e.g. malfunctioning of regulator gates) in the hydraulic models and simulate the operation.

5) Assess the performance of the scenarios and estimate the benefits gained/foregone.

6) Simulate the systems again after introducing some/full repairs to the problematic structures to ascertain the consequences and impacts on performance. Use both hydraulic and financial indicators to test the viabilities of the expenditures.

7) Report the outcomes and findings of the simulations and develop the targeted methodology for assessing and planning the expenditure on irrigation infrastructure.

1.6 Structure of the Thesis

Chapter 2 reviews the issues of expenditure on irrigation infrastructure, covering the different types of expenditure and the linkage between those types. Expenditure and asset management planning for irrigation infrastructure are also reviewed with a discussion of the currently available planning methodologies and their current shortcomings which are to be overcome by this research.

Chapter 3 then moves on to highlight and discuss performance assessment as one of the tools which will be used in the methodology of the research. First, a general framework for performance assessment of irrigation schemes is presented, and then its application to the current research is outlined. The performance measures and indicators which are applicable to the research are listed and the formulae which will be used to quantify them from the output of hydraulic modelling are given. Finally, the chapter outlines some of the approaches of multi-criterion analysis and how they will be adopted in this research for evaluating overall performance.

Hydraulic modelling techniques and their potential applications in irrigation engineering are discussed briefly in Chapter 4. A set of criteria for evaluating hydraulic modelling software based on their ability to model irrigation systems is presented and the justification for selecting the ISIS model for this research is made. The chapter also lists the general data required for modelling irrigation systems. It then focuses on the main role hydraulic modelling plays in the research methodology and identifies the opportunities and constraints to using hydraulic modelling for studying common irrigation infrastructure-related problems.

The development of the target methodology of the research is covered in Chapters 5 to 7. Chapters 5 and 6 deal with the use of hydraulic modelling techniques for linking infrastructure condition and interventions to hydraulic performance.

In Chapter 5 the target methodology of the research is developed through the investigation of the problems of sedimentation and vegetation in irrigation canals. The sedimentation problem is analysed in detail with emphasis being given to planning the expenditure on canal desilting. Hydraulic modelling techniques for ascertaining the impacts of the problem are described and then several alternatives for tackling the problem are examined. The methodology is applied to those alternatives in order to select the most appropriate one.

Further development and application of the methodology which is introduced in Chapter 5 are presented in Chapter 6 which investigates the problem of malfunctioning of gated canal regulators. Several scenarios for analysing the problem and planning the expenditure on curing it are presented.

The final part of the methodology is presented in Chapter 7 which deals with the financial aspects of expenditure and asset management planning. The costs and benefits of the scenarios investigated in Chapters 5 and 6 and how they can be included in a multi-criterion decision analysis system are discussed. The implementation of cost-benefit analysis in the research methodology is demonstrated on the scenarios modelled in Chapters 5 and 6.

Chapter 7 also presents the application of the research methodology in long-term expenditure planning and the evaluation of the viability of infrastructure upgrading/modernisation.

An outline of the methodology which has been developed in the research is given in Chapter 8 followed by the general steps describing how it can be used in asset management planning. Finally, the general conclusions and recommendations of the research are stated in Chapter 9.

2. Expenditure on Irrigation Infrastructure

2.1 Irrigation System Deterioration

The irrigation network is perhaps the most costly element of any irrigation scheme and is usually designed to last for a long time (Sagardoy et al., 1982). However, all too often one finds that irrigation schemes not long constructed bear little resemblance to the original construction and design. System deterioration due to silt deposition, weed and other vegetation growth, malfunctioning of structures and other undesirable situations make it practically difficult to control the optimum flow distribution and delivery in the canals. As a result, the system becomes unable to deliver the necessary service to the farmers and other beneficiaries and achieves poor performance.

Infrastructure deterioration is the wear and tear and malfunctioning that occur to the assets due to natural forces (wind, rainfall, heat, floods, etc.), ageing, misuse, and insufficient maintenance (ASCE, 1991). While some of these causes of asset deterioration, such as misuse and insufficient maintenance, can be overcome, others, such as the natural forces and ageing, cannot. Irrigation assets cannot therefore be expected to live forever and a time will come when they will have to be replaced (useful life of the asset) if system performance is to be sustained.

Whether the assets of an irrigation system are maintained regularly or left to deteriorate rapidly and then replaced, some money will be spent on those assets. The different types of expenditure on irrigation assets and their interactions are described briefly in the following sections.

2.2 Types of Expenditure

Expenditure on irrigation infrastructure can be in one of three forms:

(1) Expenditure required for running the system which is normally known as operation

cost (referred to as OPEX for operation expenditure in the UK water industry). This typically covers the recurrent costs of running the system and carrying out light routine maintenance activities such as painting the metallic parts of the gates and filling the small animal burrows in canal banks.

(2) Expenditure on periodic maintenance of the system in order to reduce system deterioration due to weathering factors, ageing, etc. In irrigation systems, periodic maintenance is usually carried out once a year with some certain activities being carried out at shorter or longer intervals. For example, removing the vegetation from irrigation canals is often carried out more than once a year (Sagardoy et al., 1982).

(3) Capital expenditure on rehabilitation or investment on modernisation[2] of the irrigation system in which assets are usually reconstructed/replaced with modern ones. Such expenditure is referred to as CAPEX, for capital expenditure, in the UK water industry (IIS, 1995).

The first two types of expenditures are characterised as being short-term, i.e. expenditures made will usually have significant impact on performance for a short period of time (usually a year or slightly longer) after which another expenditure may have to be made. Capital expenditures, on the other hand, are long-term expenditures which have impact on performance for a much longer time.

2.3 Interaction Between Expenditures

Short- and long-term expenditures are highly interrelated: short-term expenditures are required to protect long-term expenditures by sustaining agricultural production and other services delivered by the irrigation system (such as flood protection, rural water supply,

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[2] In economic terminology operation, maintenance, and rehabilitation are expenditures; while modernisation and construction of new assets are investments (pers. comm.; Perry, 1998).

etc.). While capital expenditures have direct impacts on short-term expenditures, as for example when an irrigation system is rehabilitated, its operation and maintenance costs in the years following the rehabilitation are usually reduced.

Expenditure on system operation and light routine maintenance is more or less static, i.e. does not change much from a year to another, and is mainly dependent on the capital expenditure (type of irrigation structures, e.g. whether they are manually operated or automatic) and hence the establishment cost, and the level of service that is required from the system. The expenditure on system operation and routine maintenance will therefore need to be revised only in one of the following cases:

(1) When establishment costs are to be increased due to inflation or other reasons.

(2) When the level of service required from the system is to be changed because of changes in the demands and beneficiaries needs as resources become scarce.

(3) After making a large capital investment in system modernisation and consequently changing irrigation infrastructure and the resources required to operate the system.

Expenditure on periodic system maintenance and capital expenditure, on the other hand, are more dynamic and require reassessment more often. It is clear therefore that there is not much decision making to be made in the case of expenditure on operation and routine maintenance - unlike the cases of the other two types of expenditure. Consequently, the methodology of this research will be more focused on expenditure on periodic maintenance and system rehabilitation. These types of expenditure will be discussed in more detail in the following sections.

2.4 Maintenance of Irrigation Systems

2.4.1 Maintenance Objectives

Various definitions for the objectives of the maintenance of irrigation and drainage systems can be found in the literature. Examples of which are the definition by Malano (1998):

'Maintenance of irrigation and drainage systems is the process of keeping the infrastructure assets in good repair and working order to fulfil the functions for which they were created. When system functions are no longer being fulfilled, the cause of this should be identified and corrected. Maintenance must ensure that the components (channels, structures, and roads) that make up an irrigation system can fulfil their individual functions and operate together to deliver an acceptable level of service to the farmers and other beneficiaries.'

It is interesting to note that while the previous definition of the objectives of maintenance concentrates on the up keeping of the functions of the system only (the technical aspect of system performance), the following two definitions consider another important aspect in the process: costs and benefits. The first definition is given by the ASCE (1991) as:

'Maintenance of an irrigation and drainage system may be concerned with acquiring, storing, conveying, delivering, removing water, or with all of these activities. The objectives of maintenance should be: to keep the system in top operating condition at all times through proper maintenance; to obtain the longest life and greatest use of system facilities by providing adequate maintenance and replacement; to achieve the foregoing two at the lowest possible cost; and, to avoid interruption in water deliveries, particularly at times when crop damage would occur.'

While Jurriens and Pinkers (1993) confirm the importance of the economic aspects in planning and implementing system maintenance as they state:

'Maintenance should only be done to remedy the causes of not fulfilling the required functions or to avoid such causes to develop. The need for maintenance as such may be evident: if things are not being maintained, they lose their function and value, investments are not properly being used and all this costs money. However, maintenance as well costs money and further decisions on what, how, and when to maintain, therefore should actually depend on a more quantitative analysis of the costs and benefits.'

2.4.2 Causes of Neglect of Maintenance

It is generally recognised that one of the greatest problems affecting the performance of public sector projects in developing countries is the inadequacy of the operation and maintenance standards. In many instances performance falls far short of the technical potential and the targets set because projects are not maintained properly. It is by no means rare for expensive rehabilitation programmes to have to be undertaken only a few years after a scheme has been built, mainly because insufficient maintenance work has been done which leads to rapid system deterioration (Finney, 1984). The causes of neglect of maintenance of irrigation systems can be outlined as follows:

1) Poor construction quality of irrigation infrastructure leading to high rates of asset deterioration and failure. The poor quality of construction not only affects the structural condition of the assets but can also affect their hydraulic performance (Murray-Rust & Halsema, 1998). Such structures will not only require more expensive maintenance, but may also need complete rehabilitation/replacement earlier than properly constructed structures.

2) Maintenance of irrigation systems in many parts of the developing world is seriously under-funded and often poorly carried out. Most governments assign low priority to maintenance to provide for basic amenities like water supply and sanitation, electricity, health, and education for inflating populations. Consequently, the budgets allocated to operation and maintenance are inadequate to cover the cost of the maintenance necessary to prevent system deterioration. The World Bank's report on India notes that deterioration of systems for lack of maintenance was one of the biggest problems facing irrigation in India (World Bank, 1991). The usual diagnosis of the problem was that resources devoted to maintenance were not sufficient.

3) Grouping the budget of maintenance with that for operation. Available money is usually allocated to establishment costs, operations, and then maintenance in that order. When an increase in the operation and maintenance costs is not met by a

similar increase in the budget allocated to them, most of the money is consumed in large establishment costs covering staff activities. Singh and Jain (1993) reported on the changes in the operation and maintenance financial allocations in India in the period from 1986 to 1991. They found that the average percentage of the money allocated to establishment costs to the total operation and maintenance costs increased from 45% in 1986 to 55% in 1991. Nowadays, the money left in the budget after deducting establishment costs, typically between 20 to 40%, is usually not enough to cover the cost of maintenance works (Skutsch, 1998).

4) The money available for maintenance is usually allocated on the basis of a constant rate per unit of area. The nature and requirements of the maintenance may, however, vary widely within the same project. Maintenance needs of a facility depend on such variable factors as topography, geology, size, construction quality, and purpose served. Some maintenance items may be critically important for one structure in a given system but less important for another similar structure in the same system (ASCE, 1991). **No procedure for prioritising maintenance works and linking maintenance with system performance so as to utilize the available funds in an optimum way is available to date** (Thoreson et al., 1997).

5) Realistic irrigation service fees which cover all or most of the real cost of operation and maintenance are not levied in many large schemes or not properly collected. The budget of operation and maintenance is therefore effectively paid by governments and is seen as a major drain on national revenues.

6) Collected water charges are not retained in the schemes where they are collected, but are generally sent to the central treasury for re-allocation. There is thus no direct link between fees collected and budgets allocated to operation and maintenance. Operation staff will in many cases favour the farmers in their schemes and not collect all the fees due.

7) Even when irrigation systems are turned-over, governments still have to spend on the maintenance of the main system, a major component of the overall maintenance

cost, which is not usually turned-over (Vermillion, 1997).

8) Shortage of skilled and competent maintenance staff is widespread and severely reduces the efficiency of maintenance. The problem is compounded by the fact that in determining the allocation of staff, in terms of both number and quality, operation and maintenance often receive low priority. The salaries and incentives of those staff are also not high enough to attract skilled personnel who can be employed in industry by the private sector for higher salaries.

9) Lack of staff training in identifying, reporting, and processing of maintenance requirements. Consequently, staff are not able to carry out or supervise and monitor maintenance work properly.

10) Inadequate planning and design is undoubtedly a significant factor in many instances. Examples include the use of too high a level of technology, with excessive dependence on imported equipment requiring sophisticated maintenance and foreign exchange which is in most cases a scarce resource for many developing countries.

2.4.3 Effects of Poor Maintenance

Carruthers and Morrison (1993) summarise the effects of poor operation and maintenance as follows:

1) Reduced irrigation system capacity and/or erratic water supplies which lower the area cultivated and depress yields. Research has shown that a discharge capacity can be reduced by more than 25% within one season due to neglected maintenance (Hebbink, 1993).

2) Water logging and salinity problems due to improper water control or damage to drainage systems. It is estimated that at present 15% or more of the world's irrigated area, mainly in arid regions, is to some extent degraded or at risk because

of waterlogging or salinisation, partly due to shortcomings in the irrigation systems (ICID, 1993; Oi, 1997).

3) As a consequence, farmers tend to shift to lower value crops, reduce the cropping area leaving the rest of their lands to fallow and/or reduce on-farm expenditure by using less or lower-quality inputs.

4) All the above lead to reduced food production which could require scarce foreign exchange to be spent on importing food and food products.

5) Quick system deterioration leading to a premature rehabilitation which consumes large local funds and most probably requires international loans.

6) Negative environmental impacts such as waterlogging and salinity caused by impeded drainage and stimulation of water-related diseases.

7) Adverse socio-economic impacts such as the inequity between the incomes of farmers according to their location within the system. Farmers in least affected areas (usually at the top ends) may continue to receive an adequate supply whilst those in affected areas (usually at the tail) may not receive any supply. The situation can be severe to the extent which forces affected farmers to sell their land and lose their business.

2.4.4 Maintenance Activities

The maintenance activities that will usually be required in irrigation and drainage systems can include:

1) Maintenance and repair of the canal or drain profile, which includes removal of sediment and trash from the open channels and culverts and repair of collapsed slopes and eroded banks.

2) Removal of aquatic weeds, both from the bottom and the slopes of the canals or drains (may need to be carried out several times per year).

3) Checking and repairing bank erosion and potential breaches caused by burrowing animals or rotten plants and roots which were not removed from the canal bank during construction or previous maintenance.

4) Repairs to canal lining.

5) Maintenance of irrigation and drainage structures on main and branch canals.

6) Restoration and remodelling of outlets on distributaries and quaternaries.

7) Maintenance of buried pipe systems, if present.

8) Maintenance of buildings.

2.5 Rehabilitation and Modernisation of Irrigation Systems

The severe shortage of maintenance funds has been reported as the main cause for neglect of maintenance of irrigation and drainage systems. When funds are not enough to cover all scheduled maintenance work, some of it have to be deferred to be carried out later when enough funds are available. In most cases, however, major reconstruction (rehabilitation) will be required because the accumulation of deferred maintenance needs becomes so great that the operation of the irrigation system is significantly hampered (Skogerboe & Merkley, 1996).

Rehabilitation is the process of renovating an existing system (or asset) whose performance is failing to meet its original objective to its *original* design specifications.

Modernisation is the process of technical and managerial upgrading (replacing) of an existing scheme (or asset) combined with institutional reforms, if required, in order to meet *enhanced* technical, level of service or social objectives (FAO, 1997).

It is reasonable to assume that in most cases the costs of routine maintenance, rehabilitation, and modernisation increase rapidly in this order. For instance, over the same period of time, the accumulated cost of proper routine maintenance will usually be lower than the accumulated cost of no/poor maintenance and early rehabilitation. Figure 2.1 is constructed from the data of a case study by Skutsch (1998) in which it is shown that the accumulated cost of poor maintenance and rehabilitation is about 2.5 times the accumulated cost of good (satisfactory) maintenance over 30 years.

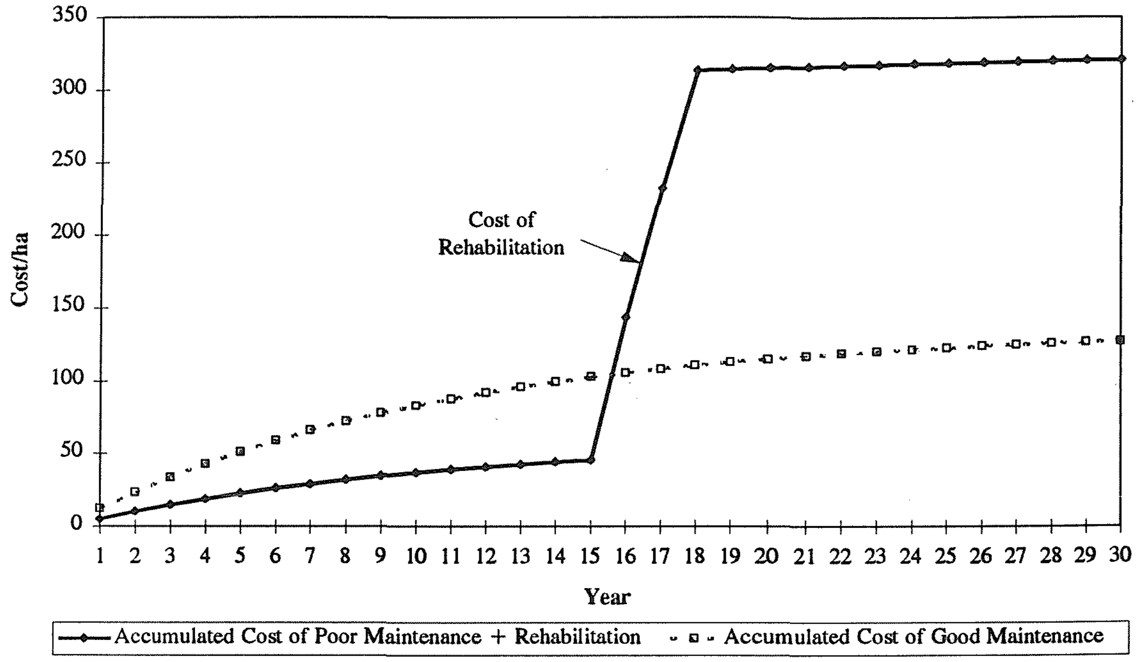


Figure 2.1 Comparison between the accumulated costs of good maintenance and poor maintenance and rehabilitation (after Skutsch, 1998)

Since the cost of an activity is usually a determining factor in making the decision whether it should be undertaken or not, the decision to rehabilitate or modernise an irrigation scheme is more difficult to make, i.e. requires more investigation and analysis, than the decision to just maintain it. Furthermore, modernisation seems to be the most difficult in all of them since it is not triggered by the physical condition of the infrastructure only, but by changes in the resources, socio-economic factors, and the level of service as well. Unlike rehabilitation, one cannot directly predict when an investment in system modernisation may need to be made in the future based on the level of recurrent expenditure on periodic maintenance and hence the rate of infrastructure deterioration.

2.6 Expenditure Planning and Optimisation

Planning the future activities, and hence expenditure, is particularly important in countries where government allocations for operation and maintenance are made on the basis of planned expenditure. A good justification of the work to be done and the consequences if it is not undertaken is of foremost importance to obtain financing for maintenance. Even where this is not the case, planning the activities that can be executed within the limited resources available is a useful exercise (Sagardoy et al., 1982).

The research which has been carried out in this area to date resulted in the development of many methodologies, some of which were simply trying to link maintenance to performance, while others went a bit further to develop expenditure planning models. A review of these methodologies is presented in the following sections.

2.6.1 Models for Expenditure Planning

a. Operation and Maintenance Expenditure and Performance

Chaudhry and Ali (1989) described the problem of rapid deterioration of public irrigation infrastructures in Pakistan because of continuous deferred maintenance. They stated that the main sources of the problem were financial constraints which appeared more binding because the revenues generated by the system had not kept pace with rising operation and maintenance costs. They developed an economic model to determine the returns to the expenditure on operation and maintenance on different types of irrigation schemes in Pakistan (canal gravity systems, government tubewells, and private tubewells). The hypothesis was that an increase in operation and maintenance spending would increase the agricultural production through increase in both irrigated area and yield per unit of land. The study concluded that the marginal benefits to past prospective operation and maintenance expenditures on canals and tubewells were substantially better than unity, providing a basis for increased water charges.

It is worth noting, however, that the model included averaged actual expenditure on operations and maintenance as a primary variable. The role of maintenance was not separately examined. In fact, the proportion of the total operation and maintenance sum actually spent on maintenance could be expected to vary considerably, in particular between canal systems and tubewells (for which the energy costs of pumping will dominate expenditure). In addition, the linkage between increased expenditure on operation and maintenance and improvement in performance was not based on field observations and appears somewhat speculative (Skutsch, 1998).

b. Determining Maintenance Needs By Hydraulic Modelling

Nawazbhutta et al. (1996) studied the usability of hydraulic modelling in assisting canal system managers in planning and targeting maintenance activities on secondary canals. The work focused on Lagar distributary canal in Punjab, Pakistan. The canal is about 19 km long and distributes a design discharge of 1.08 m^3/s to a culturable command area (CCA) of 6619 ha. The study investigated the problem of canal desilting since it was one of the main maintenance activities of the canals in Punjab. The usual approach followed by the irrigation department was to concentrate the desilting work in the lower half of the canal. However, hydraulic modelling predicted that a scenario that focused desilting at selected locations in the upper two-thirds of the canal produced a marked improvement in the proportion of water distributed to the tail reach offtakes than the usual practice. These findings were then confirmed in the field when the proposed scenario was implemented.

In a similar effort, Van Waijjen et al. (1997) studied the impact of maintenance on the water distribution equity in a secondary canal in Pakistan. A hydro-dynamic model was used to evaluate the effect of alternative desilting strategies and structural modifications to the outlets on the secondary canal on the equity of water distribution. Although the study focused on certain maintenance activities and one performance measure only, it showed the strength and usefulness of the methodology. It was concluded, however, that the methodology was not appropriate for routine use by irrigation managers, because of its high resource requirements, but could be used for strategic studies by researchers.

c. Maintaining the Value of Irrigation and Drainage Projects

Skutsch (1998) examined how maintenance affects project economic outcome. The study was based on averaged crop outputs and maintenance expenditures on two schemes which had been rehabilitated under international funding. Because almost no data linking maintenance and scheme performance were available, informed assumptions about scheme performance over time, which were derived from background experience, were used instead.

Two maintenance regimes were analysed in the study, namely 'poor' and 'satisfactory/good' maintenance. 'Satisfactory' maintenance was assumed to sustain system operations for its design life (30 years), whilst 'poor' maintenance was assumed to lead to premature system rehabilitation (after 15 years). The outputs over time under the two maintenance regimes were assumed to follow a non-dimensional production profile (Figure 2.2). Typical high value and low value cropping patterns were used under each maintenance regime on each of the two studied schemes, giving a total of eight cases to analyse. Maintenance expenditure for 'satisfactory' and 'poor' maintenance, crop/input prices, farm budgets, were based on published information on internationally-funded rehabilitation projects in Asia.

The work concluded that in all the cases studied, 'satisfactory' maintenance, whilst safeguarding output and infrastructure, costs less over the life time of the project (30 years) than the combined expenditures involved in 'poor' maintenance and early rehabilitation. **It was still recommended that better methods of identifying maintenance works and putting priorities on those with principal impact on performance were needed to enable scheme managers to have a rational method for deciding where maintenance expenditure is most needed.**

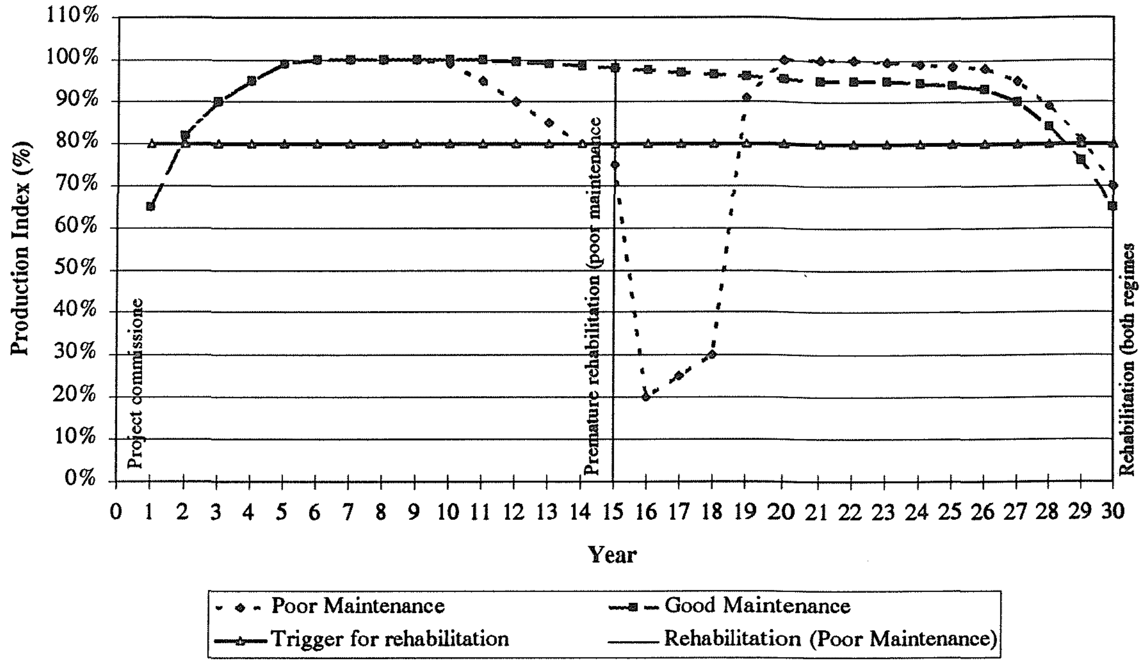


Figure 2.2 Relative output over project life — two maintenance regimes (Skutsch, 1998)

d. Asset Management Planning

Asset management planning (AMP) was developed for the UK water industry where a device was needed to quantify the extent, nature, condition and value of the infrastructure of the system prior to privatisation in 1989. As much as 70% of the assets of the system were underground and there was much speculation about their true condition. Asset management planning was developed for this purpose and has since evolved to become a comprehensive strategic business plan. Currently asset management plans are prepared on a five year cycle and have a twenty year strategic time horizon (IIS, 1995).

Burton et al. (1996) examined the potential application of asset management planning to irrigation in developing countries. The steps involved in the process of asset management planning can be summarised as follows:

(1) defining systems and functions;

(2) stratified random sampling;

(3) establishing the environmental, legal and development context;

(4) assessing system performance - achieved levels of service, how these fit with present and future requirements and what infrastructure adjustments are needed;

(5) studying management and operations - a parallel review of the organisation and its procedures;

(6) doing an asset survey - their extent, value, and the liabilities they represent;

(7) building the cost model - analysis of historical capital expenditure and operational expenditure as a basis for future projections.

In step (6) the *condition*, *serviceability*, and *importance* of every asset surveyed are to be assessed. The asset condition is measured on a four-grade system (Good, Fair, Poor, Bad) which gives a description of the general condition of the asset. The serviceability of the asset is defined as its ability to perform its function and can be one of the following four grades: 'Fully Functional', 'Minor Functional Shortcomings', 'Seriously Reduced Functionality', and 'Ceased to Function'. The asset importance is defined as the potential influence of an individual asset on system performance. It was measured by recording the positional importance of the assets (the downstream area directly served by the asset as a proportion of the total irrigated area).

The study recommended that asset management procedures for irrigation schemes were feasible and that the methodology provided a framework for strategic management in the sector.

It is important to note that the asset management procedures outlined above allow the identification of total investment needs and timing of expenditure over the plan period (5 or 20 years). They do not identify the specific assets within any system that require maintenance, repair or replacement. They are suitable therefore for planning general investment strategies, but not for preparing detailed asset maintenance/modernisation schedules.

e. A Procedure for Planning Irrigation Scheme Rehabilitation

Cornish and Skutsch (1997) developed a procedure for planning irrigation scheme rehabilitation. The procedure is based on three principal elements: checklist of performance constraints, questionnaire for farmers, and function-based condition indexing.

The checklist is intended to detect the nature and approximate scale of constraints, both technical and non-technical, on the performance of a system. By answering five groups of questions covering agriculture and economics, system design and operation, deterioration of system infrastructure, land degradation, and supply at headworks, one should be able to identify the problematic areas in a scheme. The answers to the questions in the checklist should indicate the significance (Major/Minor/None) of each of the factors in the list on the performance. It is suggested that a factor will have a major significance on performance when the farmers from around 15% or more of the command area report it as a problem which regularly limits crop yields. When a factor limits crop yields in less than 15% of the command area its significance should be considered minor. Finally, a factor which does not appear to limit performance is said to have no significance.

The questionnaire should provide views from the field level about the functioning of the system, the needs for technical improvements, general problems faced by farmers, and the relative importance of technical and non-technical issues. In conjunction with the checklist, the questionnaire should provide a crosscheck on initial findings.

The third principal element of the procedure, condition assessment, is to be used at feasibility stage if the checklist and questionnaire indicate that there are physical constraints to improved system performance. The condition-indexing system used in the procedure uses several of the concepts included in the repair, evaluation, maintenance, and rehabilitation (REMR) technology developed by the US Army Corps of Engineers. The fitness of an asset to perform its function is assessed by field inspection. The inspection is to be carried out on two stages: first an 'overseers inspection' is carried out by relatively unskilled staff by answering a set of questions to every type of structure/component and then assessing its condition (Good, Fair, Poor, Very Poor) based on the answers to the questions. Components which are rated 'poor' or 'very poor' are then inspected by engineering staff to confirm their condition (Engineers Inspection).

Once an inventory of asset condition has been prepared, the priority of works can be established by combining the assessed condition of a component with a measure of its strategic importance and the area served, in an overall score (Priority Index). The priority index (PI) is defined as:

$$PI = CI * \sqrt{\frac{a}{A}} * Is \tag{2.1}$$

where

CI	=	condition index
a	=	the area served by, or dependent on, the asset
A	=	total command area of the scheme
Is	=	importance score (1 to 4)

The strategic importance of a given type of asset to the overall functioning of the scheme is based on consideration of the following three components:

- Function (Essential/Important/Minor): The significance of the asset to the proper functioning of the system. Considers the effect of removing that type of asset from a system.

- Hazard (High/Medium/Low): The potential impact on the integrity of the system should the asset fail. This does not consider the risk to life and limb. It anticipates the most likely type of a failure — a slow deterioration, which has a low hazard, or sudden, catastrophic failure and high hazard.

- Worth (High/Medium/Low): An approximate measure of the relative cost of repairing or replacing the asset.

Assets were grouped into four classes of importance, with importance 4 being the highest, as follows:

Importance 4	Diversion weirs, embankment dams, intake works, and barrages.
Importance 3	Scour sluices, cross drainage culverts, aqueducts, syphons, and sediment traps.
Importance 2	Canals, drains, head regulators, cross regulators, drops/chutes, inspection roads, side weirs, and bridges.
Importance 1	Measurement structures.

The description of the procedure given above shows that it can be used as a tool for identifying the needs of irrigation system rehabilitation. It is to be used when the performance of a scheme is perceived to be unsatisfactory and that a rehabilitation is thought to be the answer to improve the performance. The procedure will in this case identify the potential problematic areas and suggest priorities for the work that needs to be done.

However, the use of the procedure on the long-term planning side is not so clear. It was not described in the procedure how it can be used to plan long-term expenditures (for example, how to use it to forecast future rehabilitations/modernisation needs).

The procedure highlights the importance of identifying the condition and performance of the asset which can have an effect on other parts of the irrigation systems. However, the procedure is qualitative rather than quantitative as there is no mechanism for linking the impact of the condition/performance of one asset on another or on the rest of the system.

2.6.2 Condition Indexing

The previous review of the currently existing expenditure planning procedures shows that many of them use a form of asset condition indexing or another (IIS, 1995; Cornish & Skutsch, 1997). In this respect, condition indexing is an important part of these procedures. Andersen and Torrey (1995) define condition indexing as '*a set of rules (methodology) that defines the current physical state of a facility in terms of a numerical* *value (condition index). The resulting condition index is a number between 0 and 100, with 0 representing the worst condition and 100 representing the best. The condition-index scale is a series of qualitative condition descriptions that establishes the link between the physical state and the numerical condition index.'*

Due to its importance several condition-indexing systems have been developed which formalise the decision-making process. While some of the systems are specialised for certain types of infrastructure such as road pavements, others are more generic.

a. US Army Corps of Engineers REMR Condition Indexing Scale

The condition indexing scale developed by the US Army Corps of Engineers (called REMR for repair, evaluation, maintenance and rehabilitation) is composed of seven general levels of condition ranging from 'Excellent' to 'Failed'. Broad descriptions are given for each level in terms of state of deterioration or loss of functionality. These levels and their descriptions are reproduced in Table 2.1.

It should be born in mind that it is not always necessary to adopt this seven-level scale of condition indexing. Depending on the importance of the assets analysed and the required precision of their condition assessment, it is possible to reduce these levels of condition indexing to five or less (IIS, 1995; Cornish & Skutsch, 1997).

b. Function-based Condition Indexing

Assessing the condition of most civil engineering structures, including irrigation structures, is not an easy task due to their complexity. Andersen and Torrey (1995) presented a methodology which formalizes the logical process necessary to develop a condition-indexing system for aging civil engineering facilities. The methodology is based on the total-systems approach. A facility is first analysed as individual components, the condition index for each component is then rated and finally those rates are combined into the overall condition index for the facility. The process is to be carried out in the following systematic seven steps:

Table 2.1 US Army Corps of Engineers REMR condition indexing scale

Condition Index	Condition Description	Recommended Action
100-85	*Excellent*: No noticeable defects. Some aging or wear may be visible.	Immediate action not required.
84-70	*Very Good*: Only minor deterioration or defects are evident.	
69-55	*Good*: Some deterioration or defects are evident, but function is not significantly affected.	Economic analysis of repair alternatives is recommended to determine appropriate action.
54-40	*Fair*: Moderate deterioration. Function is still adequate.	
30-25	*Poor*: Serious deterioration in at least some portions of the structure. Function is inadequate.	Detailed evaluation is required to determine the need for repair, rehabilitation, or reconstruction. Safety evaluation is recommended.
24-10	*Very Poor*: Extensive deterioration; Barely functional.	
9-0	*Failed*: No longer functions. General failure or complete failure of a major structural component.	

1. Identify specific objectives for the condition-indexing activities.
2. For each of the objectives identified in step 1, identify the functional system that meets the objective, define relevant interactions between the components, and place these into an interaction matrix. Therefore, there will be an interaction matrix for each of the objectives.
3. Code each interaction matrix to represent the strength of each interaction it contains.
4. Define ranges between ideal and failed conditions for each functional system

component that are tied to a condition-indexing scale.

5. Develop weighting functions for the condition of each functional-system component to combine them into the overall index of the facility.
6. The repetition of steps 2 - 5 for each objective will form the condition-index vector whose elements represent the condition of the facility for each of the objectives.
7. Prioritise the individual objectives to develop weighting functions that can be used to form a condition-index scalar from the condition-index vector.

The methodology highlights two important points when dealing with condition-indexing of civil engineering facilities:

(1) because of the complexity of those facilities, due to the large number of components in each of them and the strong interactions between those components, when assessing the overall condition of a facility it is necessary to divide it into individual components, assess the condition of each component separately, and then combine them into an overall condition index; and,

(2) function-based condition indexing means that a facility may have more than one condition index depending on the number of identified objectives/functions of the facility. For example, when assessing the condition of a gated cross-regulator in an irrigation network, two different objectives for this assessment can be identified: assessing the physical condition of the structure which reflects its safety against failure, and assessing its hydraulic performance. Consequently, the structure will have two condition indexes which define each of these objectives.

2.7 Discussion

The various models for planning the expenditure on irrigation infrastructure reviewed in this chapter highlight the importance and significance of this aspect to irrigated agriculture. The review shows that among these models only two are advanced enough to be seriously considered for practical application. Unfortunately, these two models also suffer from shortcomings which must be overcome before they can be widely accepted.

The first of these two models is the procedure developed by Cornish and Skutsch (1997), in which two points of concern are important to highlight. These are the condition indexing and the importance scores.

Condition indexes are assigned to irrigation structures in the procedure by means of answering a set of standard questions for each asset type. Although quite simple and transparent, it can be argued that this system cannot be generalised for all the assets of the same type as suggested. For example, the guidelines suggest that the condition index of a gated intake/head regulator structure in which any of the gates is difficult to fully open or close should be 45% (very poor). Another example is that the condition index of a canal reach where there is serious siltation or weed growth *at any location* should be 55% (poor). It is clear that the conditions of the assets in these examples under the given circumstances may not necessarily always be as suggested in the procedure, and that further study of the impact of such problems on the overall system performance is required. Serious siltation, for example, will cause different negative impacts depending on the location where it takes place within the one canal and within the whole network. In addition, the procedure suffers from the typical problem of not linking structure conditions to performance. For instance, no guidelines are given in the procedure on how to assess or estimate the impact of a structure whose condition is rated 'very poor' on the overall performance.

The principle of structure importance is already recognised in the practice of many irrigation managers. Varshney (1993) and Cornish (1998) observe from their study of some systems in India and Sri Lanka that the maintenance of the headworks usually takes the highest priority due to the importance of these structures in controlling the flow diverted to whole schemes.

The importance of an asset to the proper functioning of a system was worked out in the procedure based on the ratings of the asset function, hazard and worth. It was not clearly explained how this task was exactly done in the procedure but the given final importance scores can sometimes be arguable. For example, bridges are given importance score 2 while measurement structures are given importance score 1 (i.e. considered less important than bridges). It can however be envisaged that flow measurement structures are more important to the efficient management and operation of a system, and hence to its performance, than bridges. Accordingly they should be given a higher importance score. Such argument is supported by the asset importance guidelines suggested by Davies (1993). Although working out asset importance based on the experience of practitioners is a good starting point for tackling this problem, a more scientific approach is required in order to eliminate the discrepancies in the resulting importance scores as demonstrated above.

The second model is asset management planning as currently being used in the UK water industry. Although Burton et al. (1996) found that it is possible to adopt the procedure for managing the assets in large irrigation schemes in developing countries, more development to the procedure needs to be done to overcome its shortcoming of not being able to identify the specific assets which should be invested in, in addition to developing the linkage between the condition of irrigation assets and the performance of irrigation systems as is currently available to water supply and sewage networks.

2.8 Summary and Conclusions

With limited and shrinking funds for operation and maintenance of irrigation systems, the need to plan and prioritise such expenditure is increasingly difficult. Recent research and development efforts are focusing on the development of tools to forecast the economic benefits of, and evaluate the cost effectiveness of, operation and maintenance expenditures (McKay et al., 1999). Such tools are necessary to justify the use of money in an environment where necessary expenditures are deferred because of the lack of sufficient funds.

In spite of the current advancement in such tools, most of them still suffer from shortcomings which endanger their usefulness and effectiveness. Among the common shortcomings are the diversification of the criteria used to rate the conditions and importance of assets and the lack of methodologies for linking asset condition to overall system performance. It is the objective of this research to develop a more solid methodology and to overcome existing shortcomings.

3. Performance Assessment

3.1 Introduction

Performance assessment is inherent to any management activity, including the management of irrigation systems. The utilization of water and other resources for irrigation requires that the efficiency of their use is evaluated periodically (Bos & Wolters, 1990).

Molden et al. (1998) list the main reasons for performance analysis as: to improve system operations, to assess progress against strategic goals, as an integral part of performance-oriented management, *to assess impacts of interventions*, to diagnose constraints, and to compare the performance of a system with others or to monitor the variation in its performance with time. Performance assessment is used in this research for the purpose of assessing the impacts of infrastructure interventions on hydraulic performance and hence agricultural production.

This chapter is divided into two main sections: in Section 3.2 a general framework for assessing the performance of irrigation schemes is outlined and then the application of this framework to the current research is presented in Section 3.3.

3.2 Framework for Performance Assessment

In the context of irrigation scheme performance assessment it is necessary to define such matters as the purpose and objectives, the boundaries of the analysis, and the performance measures (criteria) and indicators. The proposed framework, which is generally based on that by Small and Svendsen (1992), is presented in Figure 3.1 and is discussed in the following sections. The main components of the framework are outlined in Table 3.1.

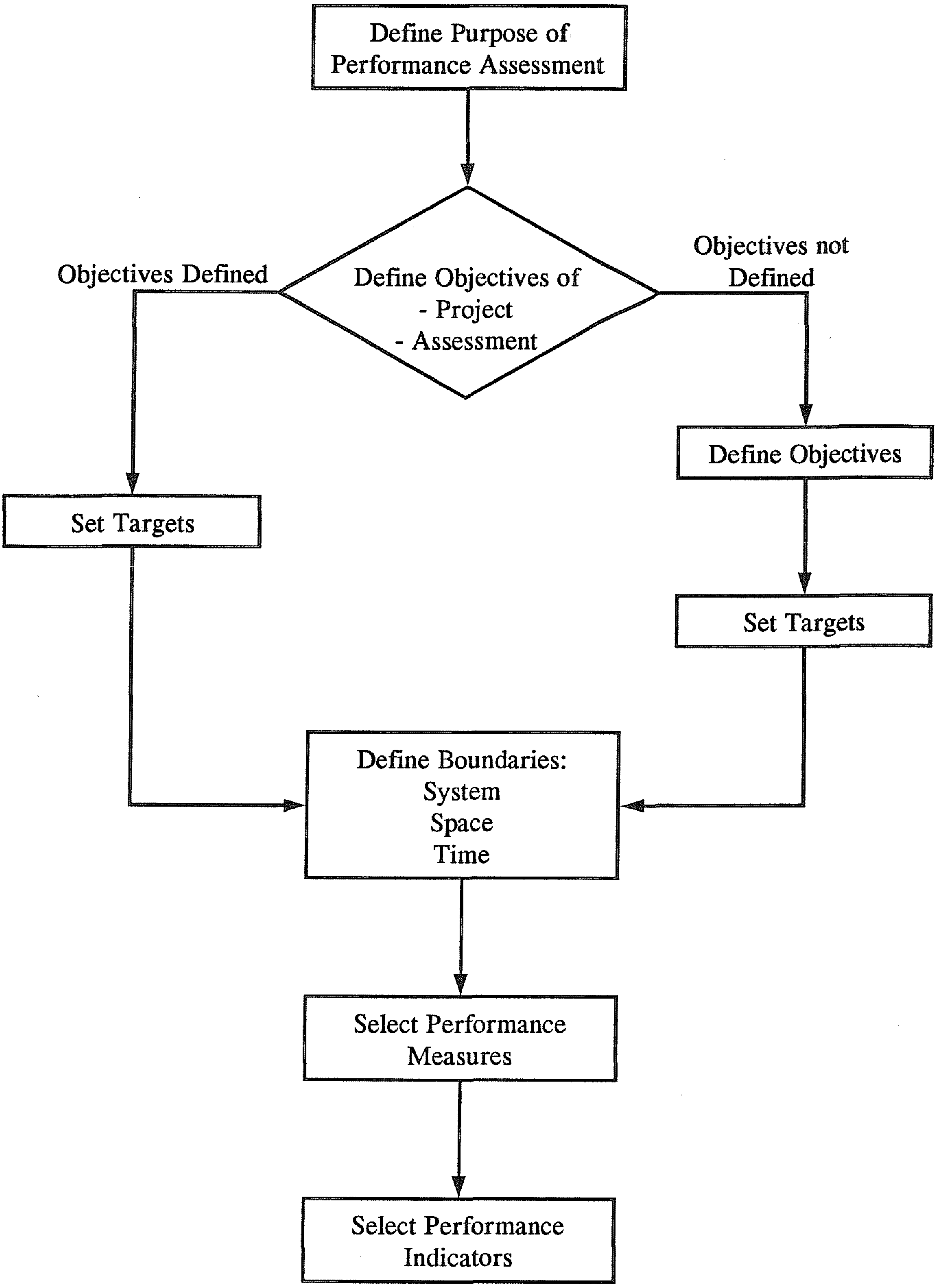


Figure 3.1 Performance assessment framework

Table 3.1 Performance assessment for irrigation schemes — main components

<table>
<tr><th>Framework Category</th><th colspan="3">Components</th></tr>
<tr><td>Purpose (Rationale)</td><td colspan="3">- Operational
- Accountability
- Intervention
- Sustainability</td></tr>
<tr><td>Objectives</td><td>
- Levels/Groups
- Emphases</td><td colspan="2"><u>Targets/Standards</u>
- Internal
- External
- Relative</td></tr>
<tr><td>Boundaries</td><td><u>System</u>
- Irrigation
- Irrigated Agr.
- Agr. Economic</td><td><u>Space</u>
- Geographic
- Social</td><td><u>Time</u>
- Single/Multiple</td></tr>
<tr><td>Performance Measures (Criteria)</td><td colspan="3">- Adequacy
- Equity
- Reliability
- Variability
- Efficiency
- Accuracy
- Water level/Freeboard
- Productivity
- Sustainability</td></tr>
<tr><td>Performance Indicators</td><td><u>Attributes</u>
- Scientific
- Quantifiable
- Without bias
- Ease of use
- Reference targets</td><td colspan="2"><u>Nature</u>
- Ratio
- Quantitative
- Qualitative</td></tr>
</table>

3.2.1 Purpose/Rationale

Before an assessment of the performance of an irrigation scheme can be carried out, the purpose of the performance assessment must be established. Small and Svendsen (1992) classified the purposes of performance assessment into the following broad categories: *operational*, *accountability*, *intervention* or *sustainability*.

- *Operational* assessment provides scheme managers with information to enable them to make correct decisions regarding the management and operation of their systems.

- *Accountability* assessment provides information to assess the performance of those responsible for scheme's performance.

- *Intervention* assessment is undertaken to determine how to improve some aspects of scheme's performance through physical, operational and/or socio-economic interventions. Small and Svendsen (1992) also note that this type of assessment is useful in applied research to understand and predict the level of performance likely to result from particular combinations of system configuration and environment.

- *Sustainability* assessment enables planners to assess the long term viability of a scheme. It is sometimes regarded as a variant of the *intervention* assessment.

3.2.2 Objectives

a. Group Objectives

Performance cannot be assessed unless there are objectives against which assessment may be made. The relevant objectives must be defined either by using existing objectives or by defining new ones.

Because irrigated agriculture involves different groups of people and interested parties, various objectives with different emphases can exist at different levels. There can be many different objectives and these may be complementary or conflicting (Rao, 1993). Jurriëns (1991) provides some useful examples of objectives at different levels:

- National
- Regional
- Scheme
- Water User Association / Village Water Management
- Farmer

The emphases of the objectives include:

- Technical
- Political
- Economic
- Social
- Environmental

Determining the level or the group of people from whose point of view performance will be assessed is therefore important such that the appropriate objectives are used.

b. Setting Targets/Standards

In order for most objectives to be assessed it will be necessary to set specific targets against which performance can be measured. The sources of these targets may be classified as *internal*, *external* or *relative*.

- *Internal* standards are set within a scheme/organization. In schemes where a government agency runs the system it is likely that the managers of the agency will set those standards.

- *External* standards are derived from various sources including technical, political, economic and ethical sources. They are based on an irrigation agency's accountability to outside organisations.

- *Relative* standards are derived from the performance of other similar schemes or systems. A normal standard can be set using data from all comparable schemes or systems against which performance is measured.

3.2.3 Boundaries

The boundaries of a performance assessment exercise can be defined in terms of the following dimensions: *the system*, *space*, and *time*.

a. The System

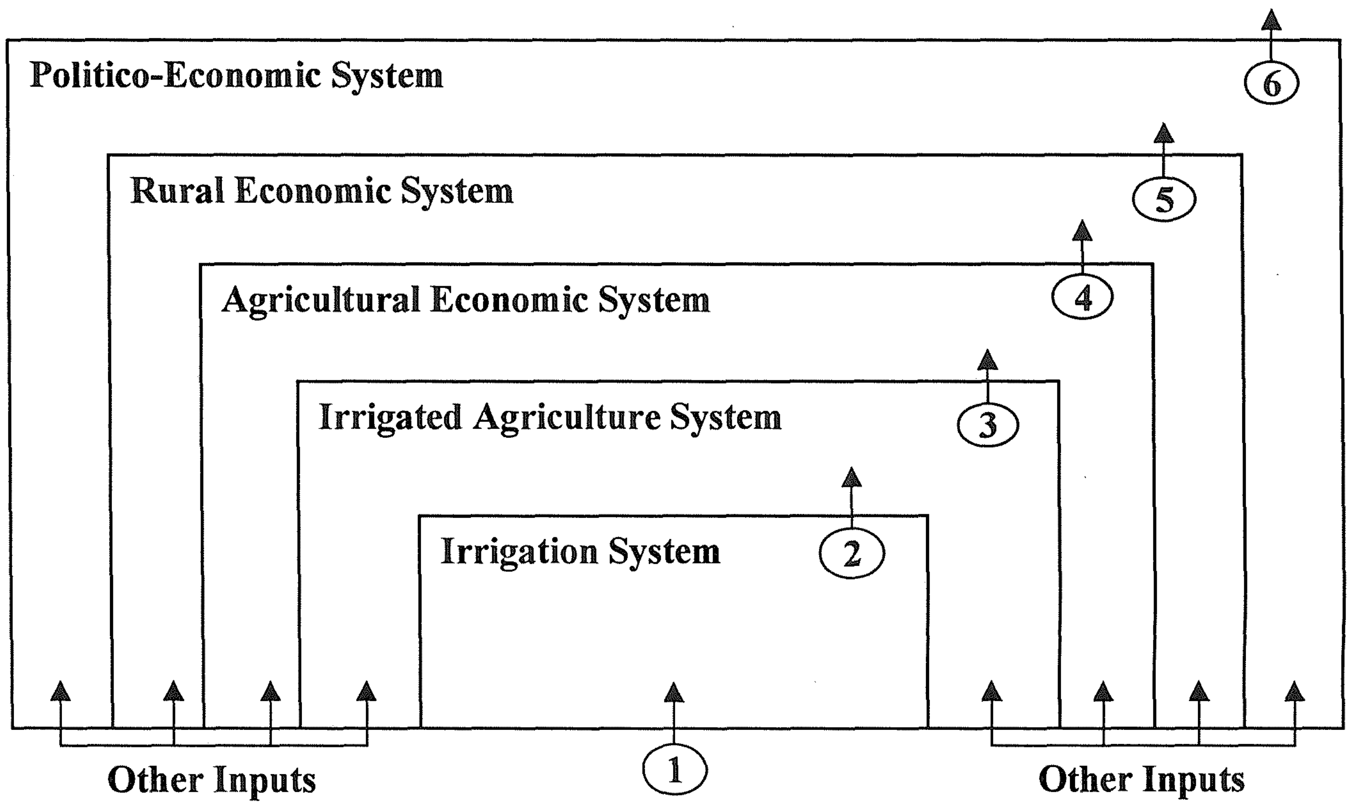
The system under consideration and its relation to other systems need to be identified and defined. Small and Svendsen (1992) define irrigation within the context of nested systems with the outputs from one system forming the inputs to the next (Figure 3.2). Each of these systems can be divided further into subsystems as required. For example, the irrigation system may be divided into three subsystems, namely the acquisition, the distribution and the application subsystem based on the function of each.

Once the system under consideration has been defined the spatial and dynamic boundaries can be defined in relation to the inputs and outputs of that system. Consideration should also be taken of the *processes* within a system that convert *inputs* to *outputs*.

b. Space

The spatial boundaries include geographic and social boundaries which are partly defined by the geographic extent of the physical components which comprise the system under consideration. However, social boundaries may not always coincide with geographic boundaries.

Figure 3.2 System boundaries showing inputs and outputs (Small and Svendsen, 1992)



1 Operation of irrigation facilities

2 Supply of water to crops

3 Agricultural production

4 Incomes in rural sector

5 Rural economic development

6 National development

c. Time

Dynamic boundaries can be short term, within the cropping cycle, or longer term, relating to the lifetime of the project.

3.2.4 Performance Measures/Criteria

Once all the limits of the assessment have been established the appropriate performance measures must be chosen. Commonly used performance measures for irrigation schemes are: *adequacy*, *equity*, *reliability*, *variability*, *efficiency*, *accuracy*, *water level/freeboard*, *productivity* and *sustainability* (Molden and Gates, 1990; Jurriens, 1996).

- *Adequacy* provides a measure of the ability of the system to meet the demand either for water or for other resources. The assessment of performance will come from measurements of how well demand is satisfied at different locations in the system.

- *Equity* compares performance at different points in a system. When applied to water-delivery systems, equity can be defined as the delivery of a fair share of water to users throughout a system (Molden and Gates, 1990). Although the definition of a "fair share" is not so simple, the most common definition is the spatial uniformity of the ratio of the delivered amount of water to the required or scheduled amount. Abernethy (1986) emphasised that equity should be one of the principal aims of the managers of irrigation systems that supply multiple users.

- *Reliability* is a measure of how closely actual performance matches expected performance. This expectation can be real or perceived. Real (technical) reliability measures focus on the frequency of achieving target levels, while perceived reliability measures focus on people perceptions, and are thus difficult to quantify (Murray-Rust & Snellen, 1993).

- *Variability* can be used as a measure of reliability although it measures deviations from a mean rather than from a target value. For example, if the supply is constant at 85% of the target then the variability is considered to be very low, regardless to the fact that actual supply was less than the target. The latter issue should be addresses by other measures such as *adequacy*.

- *Efficiency* measures are used to compare the actual performance of a system to its potential performance and as a measure of the efficiency of resource use. Measures can be taken of the whole system or of parts of it.

- *Accuracy* measures help assess the extent to which supply is able to respond to demand, for example, how fast and accurate the supply can be reduced in the case of unexpected heavy rain.

- *Water level/Freeboard* measures can be used for comparison of design with actual water levels within a system for the purpose of monitoring command and system safety. These measures are only applicable to gravity systems (mostly open-channel systems).

- *Productivity* measures are used to assess the absolute performance of a scheme. Some productivity measures can be compared to resources used to give efficiencies.

- *Sustainability* measures monitor the stability of some objectives over long periods of time (usually five years and longer).

3.2.5 Performance Indicators

Performance indicators are variables for which data can be collected to enable quantification of performance. They are often quoted as ratios. Different indicators may be required to quantify in detail one performance measure. Conversely, one indicator may be useful for two or more measures. The spatial distribution of the performance indicators can also provide information on system performance such as equity.

A performance indicator should have certain attributes that make it practical and reliable for measuring performance. Bos (1997) defines these attributes as having scientific basis, being quantifiable, being without bias, being easy to use and referring to a target value.

The nature of performance indicators can also be classified as:

- *Ratio* indicators usually relate an actual measurement to a reference/target value. They are particularly useful as they relate achievement to targets set, and are readily understood.

- *Quantitative* indicators are absolute measures of performance which can be used when comparing the performance of a scheme with external standards.

- *Qualitative* indicators are usually subjective indicators related to perceptions rather than to numerical values.

3.3 Application to this Research

The framework for performance assessment presented in Section 3.2 is general and applies to all the boundaries of irrigation schemes. Since this research deals with irrigation infrastructure, only performance assessment of irrigation systems is applicable. The objectives of the following sections are:

- to apply the framework to the assessment of the performance of irrigation delivery and distribution systems according to the needs of this research;

- to outline the performance measures and indicators which have been selected for this work and when necessary to justify the selection of certain indicators; and,

- to describe the assessment of performance indicators from the output of hydraulic modelling.

3.3.1 Purpose/Rationale

Since this research deals with expenditure on irrigation infrastructure and its impact on their functionality, *intervention* and *sustainability* performance assessments are required for quantifying the impacts of infrastructure conditions on hydraulic performance. Intervention assessment here is however limited to physical/technical interventions only. Other forms of intervention, such as management and social interventions, are not dealt with in this research.

3.3.2 Objectives

Although a real-life irrigation system (system *A*) has been used as a case study in this research, the exact objectives of this particular scheme were not taken into consideration because the research set out to develop a generic methodology that is not restricted to a certain environment. Consequently, the following set of *typical* scheme objectives has been adopted based on examples given by Burt (1987) and Jurriëns (1991):

- Achieving target outputs or better maximising outputs.
- Increasing irrigated area.
- Using resources efficiently, especially scarce resources such as water.
- Allocating water equitably.
- Providing a reliable supply.
- Sustaining agriculture by minimising negative environmental impacts.
- Keeping the system in good condition in order to be able to deliver targeted levels of service.

The list is neither exhaustive nor does it list the objectives in priority order, but it covers the main objectives. Additionally, it is true that some objectives may be conflicting. For example, increasing the irrigated area does not necessarily achieve efficient use of water resources and vice versa. Nevertheless, such conflicts will again be situation specific and therefore were not considered in the generic case.

3.3.3 Boundaries

When defining the boundaries of the performance assessment as implemented in this research according to Small and Svendsen's (1992) nested systems (Figure 3.2), it might be thought that most of the application will be within the irrigation system boundary. In order to clarify why this was not the case, the exact role which performance assessment played in this research will be discussed.

One of the objectives of the research has been to develop a methodology for linking expenditure/investment on irrigation infrastructure to the potential improvement in performance (see Section 1.4). Since the traditional and most transparent method of testing the viability of expenditure/investment is to compare them to the returns, it is important that performance enhancements due to physical structural interventions be translated into monetary benefits. According to Figure 3.2 this can only be possible if the boundary of the assessment was widened to include the agricultural economic system as the most inner system which allows the monetary values of the outputs to be easily quantified. In this respect the *processes* which take place in the irrigation system (i.e. the water delivery) are simulated using hydraulic modelling and then the *output* of the system is evaluated using the appropriate performance measures (e.g. *adequacy*, *equity*, etc.). The *output* of the irrigation system as water deliveries is then used as an *input* to the irrigated agriculture system. The agricultural processes which take place in the latter system for using water to produce crops are not modelled in this research and are considered to be ideal. Consequently, crop production is directly linked to water deliveries using functions such as that developed by Doorenbos and Kassam (1979). Finally, the revenue of the scheme from agricultural production is worked out using information on production costs and crop prices from the agricultural economic system.

3.3.4 Performance Measures and Indicators

The exercise of assessing the performance of irrigation delivery and distribution systems has been often carried out and reported in the literature. In his review on this subject, Rao (1993) concluded that irrigation water delivery should be evaluated using *adequacy*, *timeliness* and *equity*. Murray-Rust and Snellen (1993) used the *adequacy*, *equity*, and *reliability* as the main performance measures in the evaluation of the performance of 15 irrigation systems. Vander Velde (1990) reported on the performance of the distributary level of large irrigation systems in Pakistan. The main performance measures he used in the evaluation were the *equity* and *variability*. Clemmens and Dedrick (1984) studied the performance of irrigation water delivery by assessing the *variability* in the flow rate.

The performance measures which have been adopted in this research and the indicators used to quantify them are listed in Table 3.2. As has been explained before, some of the measures and indicators (e.g. equity, adequacy and water level) directly assess the *output* of the *irrigation system* while others (e.g. crop production) indirectly do this by linking that output to the *processes* and *output* from larger systems such as the *irrigated agriculture system*. This will be demonstrated further when these indicators are used for assessing the performance of some scenarios in later chapters.

Table 3.2 The performance measures and indicators adopted in the research

Performance Measure	Performance Indicators
Equity	Delivery Performance Ratio (DPR) and Interquartile Ratio (IQR)
Adequacy	Delivery Performance Ratio (DPR) Crop Yield
Water level/Freeboard	Ratio/Percentage of Lost Freeboard (LFb)
Productivity	Crop Yield

It should be noted that some of the performance measures listed in Section 3.2.4 were not used in this work. For example, the *reliability*, *variability* and *efficiency* were all not included. This is not due to shortcomings in the research but because these measures were actually insignificant in the simulation scenarios which have been investigated (as will be presented in later chapters). For instance, both the *reliability* and *variability* relate more or less to the operation side of irrigation system management. Although the condition and type of irrigation infrastructure can affect the reliability of the supply (e.g. disruption of supply if a structure fails in the middle of a season) the management effect is likely to be more predominant. Since this research investigates the linkage between irrigation infrastructure interventions and performance, and assumes that the operation side is not a constraint (see Section 1.4), the performance measures which are highly dependent on operation were not taken into consideration.

The following sections give a description of the indicators listed in Table 3.2 and how they can be determined from the output of hydraulic modelling.

a. Water-Delivery Indicators

$$\text{Delivery Performance Ratio (DPR)} = \frac{\text{Actual Discharge}}{\text{Target Discharge}} \tag{3.1}$$

Measures[3]: Adequacy, equity and variability

Range and performance:

0.0 . . . . 1.0 . . . . >

Poor | Ideal | Oversupply (waste)

$$\text{Interquartile Ratio (IQR)} = \frac{\text{Water Received by Best Supplied Quartile}}{\text{Water Received by Worst Supplied Quartile}} \tag{3.2}$$

Measures: Equity

Range and performance:

1.0 . . . . >

Equitable | Inequitable

Note The interquartile ratio (IQR) is a special indicator. The inputs to this indicator should be the values of other indicators such as the Delivery Performance Ratio

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3 Refers to the potential use(s) of the indicator, not what it has been used for in this research.

(DPR) in order to assess the equity of water distribution for example.

b. Water-Level Indicators

One of the variables which are often controlled in open-channel irrigation systems are the water levels in the irrigation network. Among the reasons for maintaining certain water levels in irrigation canals is to allow for water diversion to branching canals and to provide sufficient command to enable farmers to irrigate by gravity as in the case of upstream-water-level-control systems. Another objective for controlling water levels in almost any open-channel irrigation network is to prevent encroachment on the freeboard, which can cause canal overtopping leading to flooding and water wastage. It is important, therefore, to monitor and assess the performance of irrigation systems with regard to this criterion.

Although measuring the water levels in a canal system may be one of the easiest tasks to do, assessing the performance with respect to water level variation is not as easy. The two objectives mentioned above for controlling water levels (commanding land and maintaining safe freeboard) require contradicting control. In order to maintain sufficient command for the irrigated land the water levels should be controlled not to be *lower* than target levels. On the other hand, maintaining a minimum safe freeboard requires the water levels not to be *higher* than target levels. Consequently, it is not easy to use the same indicator(s) to assess the performance with respect to both criteria. Instead, separate indicators are used to assess each criterion. The following sections present some of the indicators which are suggested in the literature and outline the shortfalls in some of them.

i. Monitoring Command

Two performance indicators can be used for assessing the impact of water level variation on command. These are the *Water Level Ratio (WLR)* (Bos et al., 1993)[4] and the *Water Depth Ratio (WDR)*.

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[4] They call the indicator *Water Surface Elevation Ratio*

$$Water\ Level\ Ratio\ (WLR) = \frac{Actual\ Water\ Level}{Target\ Water\ Level} \qquad \textbf{(3.3)}$$

Although this indicator is very easy to use, it has a serious drawback because its numeric value is dependent on the numerical values of the water levels. Consider the case of comparing between the performance of two schemes in which the target water levels are 10 m and 100 m at some locations. If the water levels at those locations rise by 0.5 m above the targets, the water level ratios will work out as:

$$\text{Water Level Ratio 1 (WLR1)} = \frac{10.5}{10} = 1.05$$

and

$$\text{Water Level Ratio 2 (WLR2)} = \frac{100.5}{100} = 1.005$$

These results clearly show that although the rise in the water levels was the same in magnitude in both schemes, the values of the Water Level Ratios were not the same and were distorted by the numerical values of the water levels in each scheme.

$$Water\ Depth\ Ratio\ (WDR) = \frac{Actual\ Water\ Depth}{Target\ Water\ Depth} \qquad \textbf{(3.4)}$$

Unlike the Water Level Ratio, the Water Depth Ratio is independent of the numeric values of the water levels. However, it has another drawback which is inherent in it. The Water Depth Ratio allows for larger variations in the water levels for large target water depths. For example, if a $\pm 10\%$ allowance in the Water Depth Ratio is acceptable, the actual water level will be allowed to vary by up to ± 0.15 m if the target water depth is 1.5 m and ± 0.25 m if the target water depth is 2.5 m. So despite the fact that the Water Depth Ratio will indicate that the previous two situations are equivalent in performance, the impact on command in these situations may vary, depending on the design and characteristics of the control structures at the affected locations.

ii. Monitoring the Encroachment on the Freeboard

Neither the Water Level Ratio nor the Water Depth Ratio can be used as indicators for assessing the encroachment on canal freeboard because none of them uses the design/target freeboard as the indicator's reference/target. If for example the Water Level Ratio indicates that the actual water level has risen by 10% over the target, this figure cannot be used to know how much of the freeboard has been lost in this case. It is imperative in such cases to use performance indicators which directly link the variations in the water levels to changes in the freeboard. The *Ratio/Percentage of Lost Freeboard (LFb)* was developed in this research for this purpose. The indicator is defined as follows:

$$Ratio\ of\ Lost\ Freeboard\ (LFb) = \frac{Rise\ in\ Water\ Level\ Above\ Design}{Design\ Freeboard} = \frac{Actual\ Water\ Level - Max.\ Design\ Water\ Level}{Design\ Freeboard} \quad \textbf{(3.5)}$$

Range and performance:

< .... 0.0 .... >

Extra freeboard (low water level) — Ideal — Encroachment on freeboard (high water level)

Alternatively, the Ratio of Lost Freeboard (LFb) may be evaluated as:

$$Ratio\ of\ Lost\ Freeboard\ (LFb) = \frac{Actual\ Water\ Level - Max.\ Design\ Water\ Level}{Bank\ Level - Max.\ Design\ Water\ Level} \quad \textbf{(3.6)}$$

It should be noted, however, that Equ. 3.6 should be used with caution since the value of the indicator may be affected by local conditions. Often the banks of canal sections which are built in cut are formed from the natural land in the canal path, provided that the natural land levels achieve the minimum design freeboard. Consequently, the actual freeboard of a canal can vary from a section to another. Figure 3.3 shows an example of such a canal where the actual freeboard at section 2 is greater than that at section 1. A uniform rise in the water levels in the shown canal reach will not result in the same LFb at sections 1 and 2 if Equ. 3.6 is used to work them out, which can be misleading sometimes. The application of Equ. 3.5, on the other hand, will yield the same LFb for both sections 1 and

2. Consequently, Equ. 3.5 was used in this work.

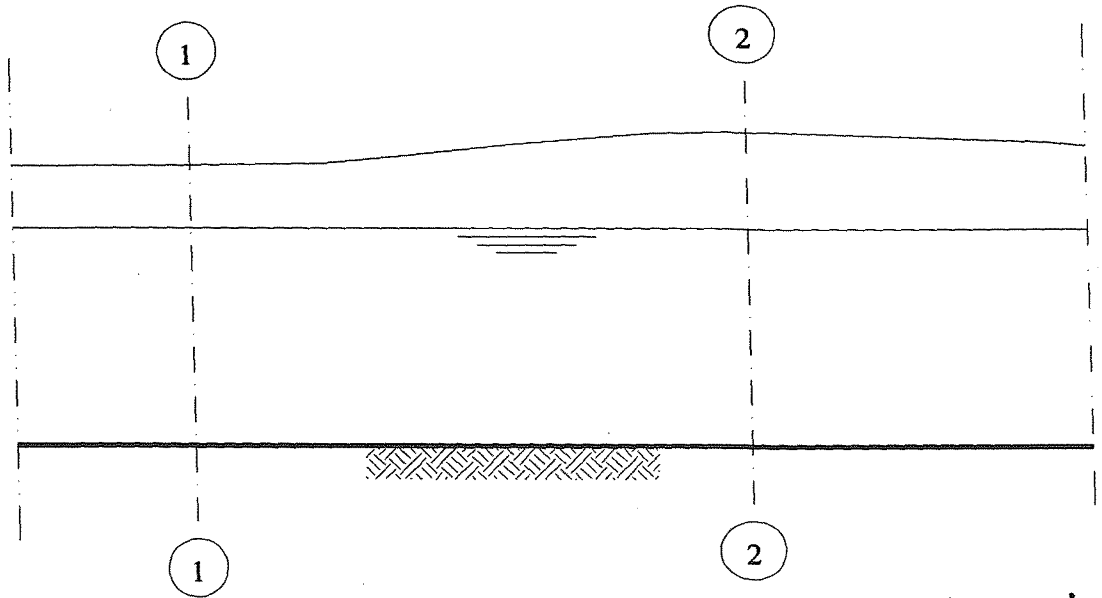

Figure 3.3 A typical bank profile for a canal in cut showing varying freeboard

3.3.5 Evaluating Performance Indicators from the Output of Hydraulic Modelling

A hydraulic model of an irrigation system is defined by a group of 'nodes'. A node is a location in the modelled system where the hydraulic parameters such as the flow and water level are of interest (e.g. at control structures, changes in open channel dimensions, etc.). Every node in a hydraulic model is given a unique identification (ID). When a hydraulic simulation is run, a simulation start time (T_o), end time (T_e), and time step (ΔT) have to be entered into the simulation software. Usually, the output from the hydraulic simulation model includes the following hydraulic parameters for all the nodes at the different simulation time steps:

$Q_{i,t}$, $Stage_{i,t}$, $v_{i,t}$, etc.

where $Q_{i,t}$ = flow at node i at simulation time t

$Stage_{i,t}$ = stage (water level) at node i at simulation time t

$v_{i,t}$ = average velocity at node i at simulation time t

Thus:

- The Delivery Performance Ratio (DPR) at any node i and simulation time t can be evaluated as:

$$DPR_{i,t} = \frac{Q_{i,t}}{Q_{T_{i,t}}} \tag{3.7}$$

where $Q_{i,t}$ = the actual flow at node i at time t

$Q_{Ti,t}$ = the target flow at node i at time t

- The actual volume of water (V_a) delivered to node i during the time period T_o to T_e is:

$$V_{a_i} = \sum_{t=T_o}^{T_e-\Delta T} \frac{Q_{i,t} + Q_{i,t+\Delta T}}{2} \tag{3.8}$$

Note It is possible that some nodes might have negative flows during some simulation time steps. A typical example will be the node of a bifurcation point where a negative flow indicates that the water was flowing opposite to its normal direction (i.e. from a lower level canal to a higher level one). This situation may occur when the water in the parent canal is being depleted at a higher rate than the depletion in the lower-level canal. Attention should be given to such situations as negative flows will affect the result of Equ. 3.8.

- And the target volume (V_T) to be delivered to the node is:

$$V_{T_i} = Q_{T_i}\,(T_e - T_o) \tag{3.9}$$

provided that Q_T is constant from T_o to T_e.

Note The calculations of the actual and target volumes of water at some nodes will be required for calculations such as the estimation of crop yields.

3.3.6 Overall Performance

For the purpose of making final decisions regarding the performance of a scenario or a case study, it is often desirable to use one overall performance indicator or score that aggregates the values of the different performance indicators which were used to assess the performance of the scenario. Using an overall performance score not only simplifies the decision making in that case but also makes it easier to compare between the performance of different alternatives.

Calculating an overall hydraulic performance score based on the values of different indicators is analogous to the general well established multi-criteria decision making (MCDM) process. Among the variety of approaches which are currently available for dealing with such problems (Stewart, 1992; Snell, 1997), two are commonly used. These are *additive value functions* and the *Analytic Hierarch Process (AHP)*.

a. Weighted-Additive Value Functions

The weighted-additive value function (weighted-average) is a conceptually well-validated approach to multiple criteria analysis (Belton et al., 1997). The underlying model is in the form:

$$V = \sum_{i=1}^{n} v_i \, w_i \tag{3.10}$$

where V = the overall evaluation score of an alternative or option

v_i = the score of the alternative on criterion *i*

w_i = the weight assigned to criterion *i*

n = the number of criteria

When applied to hydraulic performance, the function can be rewritten as:

$$OPS = \sum_{i=1}^{n} PI_i \, w_i \tag{3.11}$$

where OPS = the overall performance score

PI_i = the value of performance indicator i

w_i = the weight assigned to performance indicator i

n = the number of performance indicators used in the assessment.

The application of this approach to some of the simulation scenarios investigated in this research is presented in Chapter 5.

b. The Analytic Hierarch Process (AHP)

At its heart, the AHP is a form of an additive value function. Differences do exist however between the AHP and other additive value functions such as the Weighted-Additive Value Function described above. For instance, while a Weighted-Additive Value Function can be used to work out an overall evaluation of one alternative only, the AHP can only be used to rank a group of alternatives based on a set of criteria.

The first step in the AHP calculation procedure is the elicitation of priorities (scores) for the set of alternatives under consideration with reference to each evaluation criterion. This is done by constructing a pairwise comparison $m \times m$ matrix, where m is the number of alternatives, for each criterion. The scores (priorities) in the matrix should reflect the relative importance of the alternatives with respect to one of the criterion. If alternative i is preferred to alternative j then element (i, j) of the matrix is the strength of preference for i over j and element (j, i) the reciprocal of that number. In theory, any positive numbers can be used for the scores, however, Saaty (1990) suggests the following 1 to 9 scale:

Score 1	The two alternatives being compared have equal importance.
Score 3	Moderate importance of one alternative over another.
Score 5	Strong importance of one alternative over another.
Score 7	Very strong importance of one alternative over another.
Score 9	Extreme importance of one alternative over another.

(Intermediate scores may be used as appropriate).

The next calculation step is the normalisation of each pairwise comparison matrix. Two competing methods exist for the normalisation (Harker, 1989): eigenvector and least squares method. Both methods require iterative calculations on the matrices which makes them difficult to be done by hand.

Finally, weights are given to the different criteria to designate their relative importance. The overall priority of each alternative is then computed by summing its priority under each criterion times the weight of that priority (which is similar to the calculation of weighted-additive values).

In order to compare between the Weighted-Additive Value Function approach and the Analytic Hierarchy Process they were used in this research to rank the same group of simulation scenarios based on the same set of performance indicators (criteria). The results obtained from both methods were identical. Consequently, the weighted-additive value approach was chosen for the multi-criteria analysis of the scenarios investigated in the research due to its simplicity and transparency. In addition, no advantages would have been gained if the slightly more complicated AHP was used.

3.4 Summary

A general framework for performance assessment of irrigation schemes has been presented in this chapter. The main components of the framework are the purpose of performance assessment, the objectives of the scheme to undergo the assessment and those of the assessment activity itself, the boundaries of the system to be assessed, and the performance measures and indicators.

In the context of this research, with focus on planning the expenditure on irrigation infrastructure, only performance assessment of irrigation conveyance and distribution systems is applicable. The main performance measures which have been used in this research are *equity*, *adequacy*, *water level/command* and *productivity*. These exclude the performance measures which are highly affected by operation rather than control structures such as *reliability*. The indicators used to assess these measures have been outlined and the formulae defining their evaluation from the output of hydraulic modelling have been given. The application of the selected performance indicators in the methodology of the research and the role performance assessment plays in it are included in the following chapters.

4. Application of Hydraulic Modelling for Irrigation Systems

4.1 Introduction

Rapid developments have been made in computer applications in many fields of engineering in recent years. In the field of irrigation and drainage engineering, computer software are now available for a number of applications including planning, design, management and project operation (Lenselink & Jurriens, 1993). With the swift advancement in personal computers, many computer applications were ported from the mainframe computers, for which they were originally developed, to personal computers, thus making them available to a much wider audience.

4.2 Hydraulic Modelling and Irrigation

4.2.1 Potential Applications

Early applications of hydraulic modelling techniques were primarily for simulating the propagation of waves and floods in rivers and natural plains. As similar interest in the simulation of steady and unsteady flow in irrigation networks grew, river modelling software were adapted to deal with irrigation networks and man-made channels, in addition to the development of new specialised software. Due to the increase in the applications of hydraulic modelling in irrigation, the American Society of Civil Engineers dedicated a whole issue of the Journal of Irrigation and Drainage Engineering to cover canal system hydraulic modelling (ASCE, 1993). The Journal reviewed some of the potential applications of hydraulic modelling in irrigation:

- To test the effectiveness and efficiency of different operational procedures and to correct those procedures if the resulting performance is not satisfactory or needs improvement.

- To evaluate the characteristics of existing or planned irrigation systems such as the lag times, in-storage capacity, physical constraints, incompatible and interfering structures, storage reservoirs, and others. Knowing these characteristics and thus taking them into consideration can greatly improve the operation and performance of irrigation systems.

- To analyse the impact of floods which may enter irrigation systems and test the effectiveness of the available alternatives to route the flood waves through the system in order to prevent or minimize the damage.

- To develop and test canal control algorithms (examples of which are CARDD, BIVAL, and EL-FLO). This application for hydraulic modelling is indispensable since testing canal control algorithms on real systems is in practice not possible.

- To assist in system rehabilitation and modernization studies by assessing the improvement in system performance due to modified canal sections and control structures.

- To train design engineers and system operators on the basic principles of unsteady flow in open channels and the consequences of changes made in system design and operation on the flow and water levels in the system. The better understanding of such issues by design and operation engineers should help them make better designs and plan more effective and achievable operational procedures.

4.2.2 Existing Limitations

Regardless to the wide range of hydraulic modelling applications in irrigation engineering, the current status and capabilities of available software impose difficulties on their use for:

- *Real-time Operation:* Simulating unsteady flow in large and complicated irrigation networks is not easy and cannot be done very accurately. Almost all simulation models have built-in assumptions to simplify some modelling constraints.

Investigating the possible causes of simulation failures, which frequently occur, requires inputs from experienced modellers with good knowledge of the software employed and can be time consuming. Such problems can therefore be hazardous in systems which rely on modelling unsteady flow for real-time operation.

- *Simulating Manual Operation:* Another reason for the difficulty of simulating real-time operation using hydraulic modelling techniques is the inability of precisely modelling the operational procedure of manually-operated structures. Most hydraulic models *virtually* simulate the manual operation of control structures by allowing the modeller to predefine sets of structure settings against simulation times. Unlike what happens in the *real* manual operation in the field where structure settings are changed based on the hydraulic conditions (e.g. water levels or flows) in the irrigation system, these settings have to be made before the start of the simulations and cannot be changed during the runs. Consequently, simulating what exactly happens in manually-operated systems using hydraulic modelling may not be a straight forward task.

- *Studying Water Losses from Irrigation Canals and Reservoirs:* The review of currently available hydraulic modelling software (see Appendix II) showed that only one model can account for seepage losses from irrigation canals and none takes evaporation losses into consideration. There are approximate solutions for these shortcomings. Those approximations may be sufficient to generally account for the losses but will not be satisfactory if seepage losses are of major concern or the central issue of a study.

4.2.3 Data Requirements

The volume and level of detail of the data required for building hydraulic models for irrigation systems are often some of the concerns regarding the use of hydraulic modelling. The exact data required will vary according to the type of the software used and the purpose of the modelling. The following is, however, a list of the data which is likely to be required for almost any hydraulic modelling work:

- *Canal Cross Sections and Roughness:* Design canal cross sections can be used for modelling lined canals and newly constructed or rehabilitated earth canals. Surveyed cross sections will be required for modelling earth canals which have been in operation for some time in order to consider the deformations in the actual cross sections. Essentially, canal cross-section data will be required at the locations where changes in the cross sections exist such as changes in cross-section dimensions and drops in bed levels. However, additional cross-section data will be required at more or less regular spacing in order to improve the accuracy of the simulation results and to prevent or minimise model divergence. The spacing between the cross sections in a model depends on many factors such as the slope of the canals and the requirements of the specific hydraulic model used.

  The piece of information which is often not accurately available, mainly because it is more difficult to measure in the field, is the actual roughness of the canals. The common practice in this case is to assume the values of the roughness of the canals based on design values, experience and recommendations in standard texts (e.g. Chow, 1959 and Ilaco, 1985) and then refine them through model calibration. In this way, hydraulic modelling can be used to work out the actual roughness of irrigation canals.

  Finally, the design discharges and maximum water levels in each canal will also be required.

- *Control Structure Details:* The requirements of this type of data in particular may vary from one model to another but generally the following information will be required for each structure:

  - Structure type (weir, vertical gates, radial gates, etc.) and operation mode (manual, automatic, etc.);
  - Location (chainage);
  - Dimensions (e.g. width, number of bays, crest elevation, etc.);
  - Design discharge and water level;

- Discharge/friction coefficients;
- Design head loss; and,
- Operational procedure and schedule.

This information can usually be obtained from the design or as-built drawings, however, some data such as the friction coefficients may need to be updated from the field. Missing data may be reasonably estimated/assumed and then verified through model calibration.

- *Environment Data:* If hydraulic modelling will be used for simulating system operation during a whole growing season or a year climate data, cropping patterns and the areas served by each field outlet will be required. This data will be used for calculating the crop water requirements during the simulation period and hence adjusting the flows into the system accordingly or comparing actually released flows to the requirements. These calculations will then enable many performance indicators to be evaluated if required.

- *System Operational Procedure:* The design and hydraulic characteristics of the physical system are not the only data required for modelling irrigation systems. Information concerning the operational procedure of the system and structure operation schedules will also be required in order to simulate those procedures in the model. If such data is not collected the modelled scenarios might be different from the actual practice in the field, thus producing irrelevant results.

The output from hydraulic modelling can be reasonably accurate only if the input data is accurate. Since this is sometimes not the case, model calibration should be carried out by comparing the results from some runs which simulate already known situations to data collected from the field. The input data can then be refined until the output from hydraulic modelling closely matches the measured data.

4.3 Model Selection

The review of currently available hydraulic modelling software (see Appendix II) highlighted the general capabilities and limitations of each program reviewed and focused on their ability to model irrigation networks in particular (e.g. the number and types of control structures which can be modelled by each program, etc.). Most of the models reviewed had similar capabilities, although some had special features. Consequently, it can be concluded that none of the models could be regarded as the best.

4.3.1 Selection Criteria

A better approach for selecting a hydraulic model for a study is to define the exact requirements of the study and hence the output required from hydraulic modelling and then compare them to the capabilities of available models. The following checklist is proposed as a basic set of criteria which should be considered when selecting hydraulic models for simulating canal flows:

a. Data units supported

Most models support the International System of Units (SI) and some also support the Imperial system. It is important that the model selected be able to support the system of units which is used by the users since unit conversion can be a major source of errors.

b. Network size and layout

Some models have limitations on the maximum size of the network which can be modelled in one run. The limitations may be set as maximum number of reaches, nodes, turnouts, branches, etc. The capability of the model in simulating branched networks and the maximum number and levels of branches are also important.

c. Canal sections and roughnesses

Although most open-channel irrigation networks are man-made with prismatic *design* cross-sections and uniform roughness, as the canals get older the cross sections tend to lose their uniform shape and roughness. Some hydraulic models are capable of modelling certain shapes of canal cross sections only or have restrictions on the variations of the canal roughness which they can handle. Such restrictions may render some hydraulic models unsuitable for modelling some systems.

d. Structure library

Most irrigation networks utilise some sort of control structures for managing water distribution. For an accurate simulation of an irrigation system, not only should canal sections and roughnesses be accurately modelled, but control structures should also be accurately simulated. When a certain type of structures is not supported by a hydraulic model, the modeller may resort to modelling it using another similar type which is supported by the model. For example, if a hydraulic model cannot readily model broad-crested weirs, they may be modelled as short-crested weirs instead. However, the results must be carefully examined in those cases such that the implications of the differences between the actual system and its model on the results can be evaluated and understood. A hydraulic model which can readily model broad-crested weirs should be preferable in this example.

e. Structure automation

Most hydraulic models are able to handle changes in control structure settings (e.g. gate openings) during simulations runs. The changes in the settings need however to be predefined by the modeller as pairs of simulation times and structure settings before the run starts and cannot be changed during the run. Such control may be suitable for modelling manually operated systems but will be very difficult to implement for modelling automated systems where structure settings change based on flow/water level conditions. Some hydraulic models offer some means of simulating structure automation. It must be noted however that those means can vary in terms of ease of use from a model to another. It is recommended therefore that this criterion be given high consideration if the system to be modelled is automated.

f. Estimation of the initial conditions

After building the digital model of the actual system and before the hydraulic modelling software can run and produce any useful results, the modeller has to define what is called the *initial conditions* of the simulation. These include determining the flows and water levels in various parts of the modelled network at the first simulation time step. The exercise will not be so difficult for small networks, but may prove to be a point of concern for larger networks. Because the values of the initial conditions are used by the hydraulic modelling software in the calculations of the first few simulation time steps, the stability of the run is highly dependent on the quality of the initial conditions. Using the "reasonable estimations" approach for determining the initial conditions might not produce satisfactory results with large and complicated networks. Some hydraulic models have special methods which can help estimate the initial conditions. These can prove to be very useful especially with large networks.

g. Input data editing

Most hydraulic models operate by reading one or more input data files which 'describe' the problem to be modelled. Modern software is provided with graphical user interfaces (GUI) which facilitate data entry and the preparation of the necessary data files. Nevertheless, in some situations, e.g. when numerous systematic changes to the data files are to be made, it might be easier, especially for experienced modellers, to edit the input data files directly without using the model's interface. This will only be easy or possible if the data files are stored in plain text format. Consequently, when a large number of simulation scenarios which requires dynamic modification of the input data files is expected, it is recommended that only modelling software which accepts plain text data files be used.

h. Output extraction and presentation

For the output of a hydraulic model to be used efficiently in analysing and interpreting the scenarios being modelled it is helpful if the model can:

- present data graphically in various ways;
- export output data such that they can be read by spreadsheet-type applications for further processing; and,
- preferably, be able to work out some basic performance indicators.

i. Customer support

Hydraulic modelling software vary considerably in their prices and the support offered by their developers or distributors. A slightly more expensive software with good support can be a better choice than cheaper ones with less or no support. Many models are not near maturity yet, so users may encounter problems due to programming errors and the like. When a model has a good customer support, the users should be able to report the problems they encounter to the developers in order to fix them quickly leading to minimum disruption to the work.

4.3.2 Why ISIS?

The hydraulic model ISIS has been used for carrying out all the simulation runs investigated in this research. A description of the model is given in Appendix I. Nevertheless, it is important to highlight the facts which led to the selection of this model for the research:

- ISIS uses SI units which are used in this research.
- It can model networks with up to 2,000 nodes[5] which should be sufficient for modelling medium-size irrigation systems (the model of the case study, system *A*, has about 900 nodes).
- The model can handle any sensible branched and looped network (which is required for modelling the case study system *A*).

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[5] This is the capacity of the hydraulic module, ISIS Flow. Other modules, such as the Sediment Module, may have lower capacities.

- It accepts canal sections of any shape and variable roughness coefficients. This feature is particularly important for modelling sedimentation and vegetation.
- ISIS can simulate both steady and unsteady flow.
- ISIS has a large irrigation structure library (which included all the types of structures in system *A*).
- Simulating structure automation in ISIS is possible using two different methods. This was again very important in this research because all the scenarios investigated were simulated twice, once for testing them on manually operated systems and another for testing them on automated systems (Chapters 5 & 6).
- Simulation *initial conditions* can be estimated by the software using two different methods.
- ISIS has adequate data presentation capabilities, although the facility of exporting output data to spreadsheet applications was not available. (This was overcome by the author through the development of special software as described in Appendix I. This software has now been adopted by the developers of ISIS).

4.4 Application to Expenditure Planning

The role hydraulic modelling plays in the methodology of this research can be summarised as simulating the impacts of changes in irrigation infrastructure conditions on hydraulic performance in order to establish and quantify the linkage between the two. In this respect, two important points need some consideration:

(1) Not every change in infrastructure condition will have an impact on hydraulic performance.

(2) The use of hydraulic modelling techniques requires committing time and other resources which can be valuable and therefore should be optimised.

In order to clarify these two points, the following need to be established:

(1) The potential linkage between the various cases of change in infrastructure

conditions and hydraulic performance which determines the applicability of the research methodology to those cases.

(2) The significance of the impact of the changes on hydraulic performance and hence whether they are worth modelling or not.

There are a large number of ways in which the condition of infrastructure may change. For example, the condition of irrigation canals may deteriorate due to sediment deposition, and may improve when the sediment is removed. Table 4.1 outlines some typical cases for the main types of irrigation infrastructure. (Although the cases are listed in the table as problems, i.e. indicate deterioration in the condition of the infrastructure, the table is equally valid for the opposite situations when the condition of the infrastructure is improved.) More importantly, the table establishes the potential impact of each case on hydraulic performance and hence the applicability of the research methodology to that case. Finally, each case is classified in one of four groups according to the combination of its potential impact on hydraulic performance and modelling viability/worthiness. The four classification groups can be defined as follows:

Group 1	Cases which have no hydraulic impact, hence are not possible to model.
Group 2	Cases which have hydraulic impacts but are difficult to model sensibly.
Group 3	Cases which have hydraulic impacts but may not be worth modelling.
Group 4	Cases which have hydraulic impacts and are important to model.

Group 2 contains all the cases which cause loss of water as their main impact on hydraulic performance. As has been discussed earlier in Section 4.2.2, most of the currently available hydraulic modelling software cannot directly model water losses from canals or structures. The work-around solution for this limitation requires the modeller to estimate the quantities of the losses and feed them into the model. The solution, however, reduces the usefulness of hydraulic modelling in these cases and introduces uncertainty in the output of the model. Additionally, long-term expenditure planning will not be possible since it will be difficult to estimate the quantity of water which might be lost in the future as the condition of the structure deteriorates.

Table 4.1 The applicability of the research methodology to various infrastructure problems

Structure Type *	Potential Problems	Affects Hydraulic Performance?	Can be Modelled?	Group
Open channel	Sedimentation and/or erosion	Yes	Yes	4
	Vegetation	Yes	Roughly	4
	Lining damage (cracks, cavities behind the wall panels)	Yes	No	2
	Excessive seepage	Yes	No	2
	Breaches	Yes	No	2
	Bank erosion	No	No	1
	Deterioration of access roads	No	No	1
Typical concrete structure †	Deterioration of concrete (cracks, wear, etc.)	No	No	1
	Structure movement (settlement, displacement, etc.)	No	No	1
	Local scouring	No	No	1
	Damage to approach channels/aprons	No	No	1
	Damage to wing walls	No	No	1
	Piping under structure	No	No	1
Gated structures (sluices)	Rust and damage to gates	Yes	Yes	3
	Damage to gate-moving mechanism (jammed gate)	Yes	Yes	4
	Missing gate	Yes	Yes	4
	Damage to sill under gates (if present)	Yes	Yes	4
	Blockage of water way through structure	Yes	Yes	4
Fixed structures §	Damage to structure crest	Yes	Yes	3
Culvert/ Aqueduct/ Syphon	Leakage from joints	Yes	No	2
	Blockage due to sedimentation and debris	Yes	Yes	4
	Damage to piers/supporting structure	No	No	1

Notes:

* The list includes only structures which have hydraulic functions. Other structures which do not have hydraulic functions such as bridges and roads are not listed.

† Typical problems with most concrete or masonry structures.

§ Weirs, orifices, etc.

The above definitions clarify that the methodology of the research is applicable to the cases in Groups 3 and 4 only. Furthermore, only the cases in Group 4 are worth modelling. The distinction between Groups 3 and 4 is therefore necessary in order to optimise the time and resources committed to hydraulic modelling. However, deciding whether a case should be classified as Group 3 or Group 4 might not be straightforward sometimes and will need some experience with the system under consideration. If necessary, quick hydraulic simulations can be used to test the extent of the hydraulic impacts of the cases which are otherwise difficult to classify.

The classification of the cases listed in Table 4.1 may not necessarily be applicable to all irrigation systems. Some cases may be re-classified according to the exact situation in every system.

To explain the previous points, the case of deteriorating weir crests is used as an example. According to Table 4.1, this problem is classified in this research as Group 3 (i.e. not worth modelling). This was based on the fact that a deteriorated weir crest can lead to lower coefficient of discharge, thus higher upstream water levels. Such problem can be simulated in hydraulic modelling by reducing the coefficient of discharge and/or slightly raising the crest level of the structure in the model. However, in most cases the impact of the problem on the overall performance of the irrigation system is unlikely to be significant and therefore may not justify the time and effort required for the modelling work.

On the other hand, if the affected weir is a large structure (e.g. the main diversion weir of the system) the impact of damages to its crest may be significant and hence worth modelling (i.e. the case will be classified as Group 4 in such situations).

Most of the cases in Group 4 are related to irrigation open channels and gated structures. For the open channels the problems of sedimentation and vegetation are paramount. The problems with gated structures can be summarised as problems related to the malfunctioning of the gates and the water ways through the structures. These problems are dealt with in detail in Chapters 5 and 6.

4.5 Summary

The applications of hydraulic modelling in the field of irrigation engineering have been on the increase in the past few years. Hydraulic modelling is used in the design, operation, maintenance and rehabilitation of irrigation systems and in training operation staff. Various hydraulic modelling software are currently available. As their capabilities and features are more or less equal it is difficult to elect one of them as the best. Instead, some criteria for the selection of hydraulic modelling software have been suggested. The criteria are selected such that they test the ability of hydraulic modelling software to model irrigation systems in general and their capabilities on satisfying the requirements of individual studies. The hydraulic model ISIS has been selected for this research since it satisfies most of the research needs.

Because the research investigates the potential of using hydraulic modelling techniques in expenditure planning, its methodology is only applicable to structural interventions which affect hydraulic performance. A classification system for the typical infrastructure-related problems which should be dealt with in expenditure planning procedures has been presented. This system is used to screen the various problems under consideration such that only those that have potential significant impacts on performance are selected for investigation using hydraulic modelling. Among the significant identified problems are sedimentation in irrigation canals and malfunctioning of gated structures. These problems are dealt with in the following two chapters.

5. Expenditure on Irrigation Canals

5.1 General

The network of irrigation canals is the largest and probably the most expensive component in an irrigation system, especially manually operated ones. The expenditure on the maintenance of irrigation canals can therefore be expensive as well and may consume a large percentage of the budget allocated for maintenance. Verdier and Millo (1992) estimated that the typical rate of the annual maintenance of irrigation canals is around 2% of their capital cost, while the rate for irrigation structures is about 1%. Horner (1991) worked out the costs of periodic maintenance of a medium-scale scheme in Indonesia. In his calculations, almost 70% of the total cost of maintenance was required for the maintenance of the canal network (more than 50% for desilting the canals) and the remaining 30% for the structures. Skutsch (1998) gave similar figures from Bangladesh where maintenance funding is proportional to the capital cost of the system components so the rate for main canals was 3%, distributaries 2% and hydraulic structures 1%. The proportions of maintenance budgets allocated to maintenance tasks from different irrigation schemes worldwide were also provided. On average, 50% of the total budget was allocated to the maintenance of canals, while the percentage allocated to structures varied between 10% and 35%. When the expenditure on the maintenance and rehabilitation of an irrigation system needs to be reviewed, a good starting point will therefore be the expenditure on the canal network.

The expenditure on the annual or less frequent maintenance and rehabilitation of canals will usually cover four main tasks: (1) desilting the canals; (2) cleaning the vegetation and weeds; (3) repairing damaged and eroded banks; and (4) repairing the leaks and cracks in canal bodies. Actual distributions of funds between these tasks show that desilting usually consumes the largest proportion (25% to 45%), then bank and road repairs (10% to 20%) and finally cleaning of weeds (5% to 10%) (Cornish, 1998).

The objective of this chapter is to demonstrate the development of the targeted methodology for planning and prioritising the expenditure on irrigation infrastructure through application to the first and largest component of irrigation systems, namely the canal network. The application will focus on sedimentation and vegetation as the two most serious and costly problems directly affecting the hydraulic performance of irrigation canals and because they are highly interrelated. First, the effects of the problems on the hydraulics and other performance criteria will be described. This will be considered as a reference case against which the change in performance due to interventions will be measured (the 'before' situation). The potential ways for tackling the problems will then be investigated. Of more importance will be the application of the proposed methodology for prioritising the maintenance activities regarding these problems. In practical terms, the prioritisation of maintenance activities is important in either of the following cases:

(1) when the resources, especially the financial resources, allocated to maintenance are not sufficient and therefore not all the maintenance work necessary for keeping a system in good condition are possible to carry out; and

(2) when ranking of maintenance activities (importance) is required to be used in a procedure like asset management planning.

5.2 Sedimentation in Irrigation Canals

Excessive sedimentation is perhaps the most common problem affecting the performance of earth canals (Sagardoy et al., 1982). Many irrigation systems withdraw water from sediment carrying rivers. Some systems are provided with some control (sediment excluders/ejectors/extractors) at their headworks in order to minimise/eliminate the problem. However, fluvial, hydraulic and sediment regimes may have changed radically since scheme implementation, owing to upstream developments, catchment deterioration, and climate change. Sediment may also be entering the system below the headworks from subsidiary water sources and from bank erosion. Apart from the regime designs of the Indian sub-continent, most canal systems were not designed specifically to transport sediment: designers try to ensure a minimum velocity under design conditions (Cornish & Skutsch, 1997). Sediment deposition in irrigation canals can therefore sometimes be inevitable.

The main causes for canal siltation can be outlined as:

1) Excessive sediment entry through the main canal intake.
2) Disproportionate withdrawal of sediment by branch canals and field outlets due to malfunctioning or wrong settings of intake structures.
3) Inadequate sediment transport capacity of the canals.
4) Prolonged heading up at control structures.
5) Wrong channel regulation by allowing larger flows or higher velocities, thereby causing erosion to channel beds and side slopes.
6) Drifting of sand.
7) Haphazard desilting during maintenance.
8) Re-entry of excavated material by rain and wind action.
9) Excessive weed growth which chokes sections of the irrigation channels, thus reducing the flow velocity and causing the deposition of sediment.

5.3 Methodology of Investigation and Case Study

Several hydraulic simulations have been carried out in order to investigate the problem of sedimentation in irrigation open channels. The objectives of these simulations varied from studying the impact of sedimentation on system performance to investigating the effectiveness of different sediment cleaning alternatives in order to prioritise them. Each scenario investigated was simulated twice by modelling the same case study under:

(i) manual operation; and,

(ii) automatic operation.

Those pairs of simulations were used to study the difference the operation mode may make to the scenarios investigated. Although automated irrigation systems are becoming more and more widely used all over the world, the large majority of irrigation systems, especially in the Third World, are still manually operated (Clemmens, 1998). Studying the impact of sedimentation on the performance of those systems should therefore be of value, even in very recent work.

All the simulations were carried out on the same case study, irrigation system *A*, such that comparisons between the results can be made easily. A schematic layout of system *A*, showing the components which have been modelled in details, is depicted in Figure 5.1. The irrigation system comprises open channels and undershot gated regulators. The canal network consists of a main canal (MC), six distributaries and two tertiaries. All canals are made of trapezoidal earth cross-sections with 2:1 side slopes. The design Manning *n* of the main canal is 0.022 and for the other canal types is 0.03.

Apart from one gated weir cross-regulator on the main canal, the rest of the canals have composite structures for the head and cross regulators. Each composite regulator consists of one sluice-gate-type structure and pipe culverts. All structures are manually operated to achieve upstream water-level control.

Water is delivered to the fields through 76 outlets, each serving an average area of about 82 ha. The field outlets are provided with composite structures similar to those on the canals but of smaller sizes. The details of the parts of the irrigation system below the field outlets have not been built into the hydraulic models used in this research. The assessment of the performance of the system was carried out assuming that the flow diverted to any field outlet will be evenly applied to the whole area served by that outlet. Consequently, if the supply to a field outlet was less than the crop water requirements, the crop yields from the area served by the outlet were reduced instead of concentrating the limited supply to a smaller area in order to produce full potential yield from that area only.

The scheme has a total net irrigable area of about 6,400 ha. The climate of the scheme area is semi-arid so irrigation is essential for crop production. The primary crops grown are paddy rice, upland rice, maize and cotton. The growing season starts in April and ends in January. Figure 5.2 depicts the water demand of the scheme and the proposed canal supply for a typical year. Both the supply and demand are presented in the figure as percentages rather than water quantities or rates. These percentages, referred to in this work as **supply ratios/percentages**, are *the ratios of the supply/demand flows to the maximum design flow* (6.67 m^3/s). For example, a 60% supply percentage indicates that the water supply through the intake of the main canal of the system is about $6.67 * 0.6 = 4.0$ m^3/s.

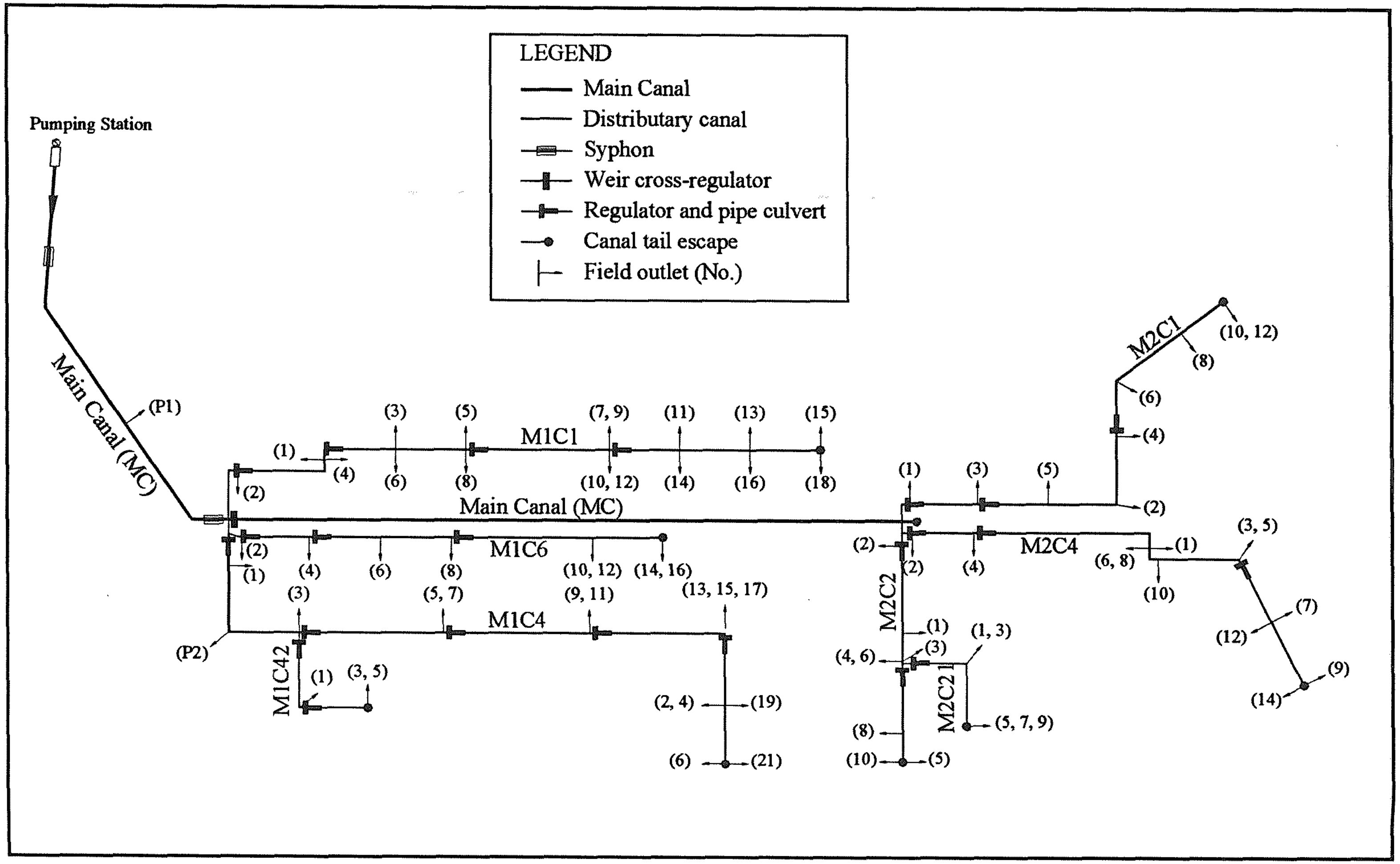

Figure 5.1 Schematic layout of irrigation system *A*

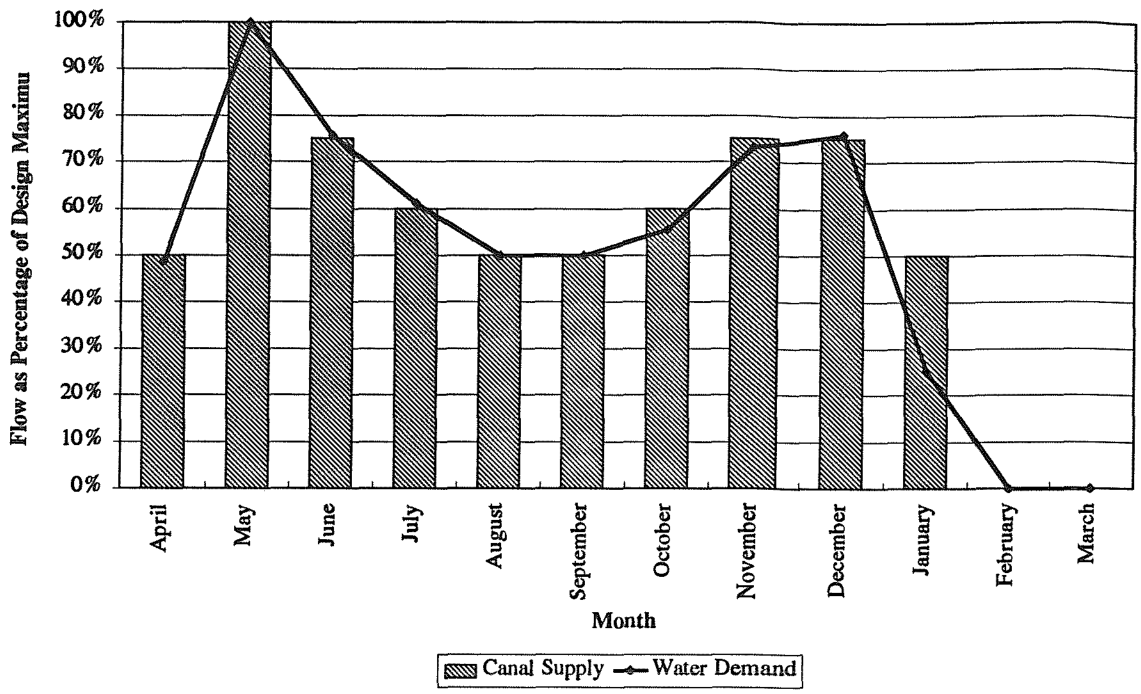

Figure 5.2 **The actual patterns of the water demand and canal supply for a typical year in irrigation scheme *A***

A more detailed description of system *A* is available in Appendix III.

5.4 Predicting the Likely Sediment Profile in a Canal Network

5.4.1 Methods Used

The first step in studying the problem of sedimentation in irrigation canals is to establish the problem itself in the case study. No real data regarding the actual sediment deposition in the case study, system *A*, was available. Consequently, the profile of sediment deposition in the system had to be predicted. The first method used was to assume that the depth of sediment at any canal section was linearly proportional to the water depth at that section. This method was then considered as too simplistic and hence not appropriate for the research. A second alternative was to use hydraulic and sediment modelling to predict the deposition of sediment in the canal network based on the concentration of the sediment entering the system and the actual hydraulic characteristics of the canals. The hydraulic model which was used to carry out the simulations (ISIS) has a special Sediment Module for modelling sediment transport in open channels (a brief description of the Sediment Module is available in Appendix II). The module was used to predict the likely profile of sediment deposition in the canal network of system *A*. Although the output of the sediment model could not be validated against actual data, the sediment profile was rationalised to be acceptable for the following reasons:

1) Generally, it is difficult to accurately predict the quantity of sediment that will be deposited in irrigation canals (Brabben, 1990). As an example, the concentration of sediment in the Blue Nile at Roseires, and hence the quantity of sediment which was likely to deposit in the Sudan's Gezira scheme which feeds from it, was found to vary by $\pm 400\%$ in three consecutive years (Mott MacDonald, 1990). The prediction of the quantity of sediment that actually deposited in this scheme in any of these years would have been very inaccurate.

2) Even if actual sediment deposition data was available, it would not have been possible to ensure that this data would represent a *typical* sediment profile. Based on the argument given above, the actual sediment deposition in a scheme can vary widely from a year to another. Hence, it is difficult to choose a particular sediment profile and study it as a typical case for a scheme.

3) The main objective of this research is to develop a generic methodology rather than to investigate the particular problems of a scheme. Studying the very actual sediment profile in a scheme may not help much in achieving this objective. Of more interest to the research was the *profile* of the sediment not the actual quantities (e.g. it was important to know the ratio of the quantity of sediment which deposits at the end of a canal to the quantity which deposits at the top).

5.4.2 Implementation Difficulties

ISIS Sediment Module has not been widely used by the current users of the model and therefore has not been thoroughly tested. Consequently, some problems were initially encountered while using the module in this research. These problems were identified and reported to the software developers who carried out the necessary corrections (see Appendix I for a list of some of these problems). One problem still remained related to the capacity of the Sediment Module. The capacity of the module is currently much lower than the capacity of the hydraulic module. This means that the hydraulic module can model larger models than what the Sediment Module can handle. Because the size of the full model of system *A* exceeded the capacity of the Sediment Module, the model had to be cut down into smaller ones, each contained one canal only, when modelling sediment transport. The models of the canals at the top of the system were run first and then their output was taken as input into the models of the canals at their downstream. A comparison between the hydraulic results obtained using this procedure and those obtained by running the whole model as one piece verified the correctness and accuracy of the procedure and hence the results.

5.4.3 Model Descriptions

The exact features of the simulations carried out to establish the likely profile of sediment deposition in system *A* can be outlined as follows:

- Simulate the operation of system *A* during a whole year as follows:

    - Change the flow entering system *A* in the simulation to follow the pattern of the canal supply depicted in Figure 5.2. The concentration of the sediment entering the system was similarly varied relative to the variation in the flow with a peak concentration of 4,000 ppm. The ISIS Sediment Module was set up to use the Engelund-Hansen equation for predicting sediment transport.

    - The gates of the canal regulators and field outlets were adjusted during the simulation to maintain either target flows (canal head regulators and field outlets) or the target water levels (canal cross regulators) as the flow entering the system changed.

- The dynamic capabilities of ISIS made it possible to instruct the model to update the shapes of the canal cross sections during the simulation according to the quantity of deposited sediment. An example of how ISIS updates the shape of a typical trapezoidal cross section is shown in Figure 5.3.

- It was not possible, however, to change the values of Manning's coefficient n during the simulation as with the shapes of the canal cross sections. Design n values were therefore used throughout the whole run.

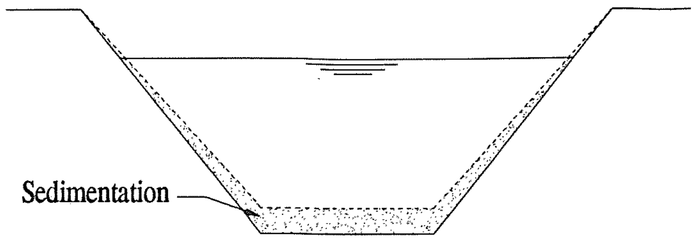

Figure 5.3 Updated shape of a typical canal section due to sedimentation

- In order to study the impact of the concentration of the sediment entering the system on the resultant sediment profile, another simulation was carried out which had exactly the same features listed above but with the maximum concentration of the sediment increased from 4,000 to 7,000 ppm.

5.4.4 Simulation Results

The resultant sediment profiles in the distributary canal M1C4 are depicted in Figure 5.4 as an example (the locations of the canal structures are shown on the schematic diagram in Figure 5.5). Although the results could not be verified by actual field data, they were considered acceptable because the sediment profile followed the pattern of the profiles produced by similar simulations such as those carried out by Mendez (1998). Some observations on the simulation results are however important to highlight:

- Figure 5.4 shows that there is a typical pattern for the sediment profile in each canal reach (between two canal regulators). At the top of each reach, the sediment load is larger than the sediment transport capacity of the reach so large quantities of sediment deposit. Due to these depositions the sediment load decreases towards the bottom of the reach and hence the quantities of sediment depositions decrease as well.

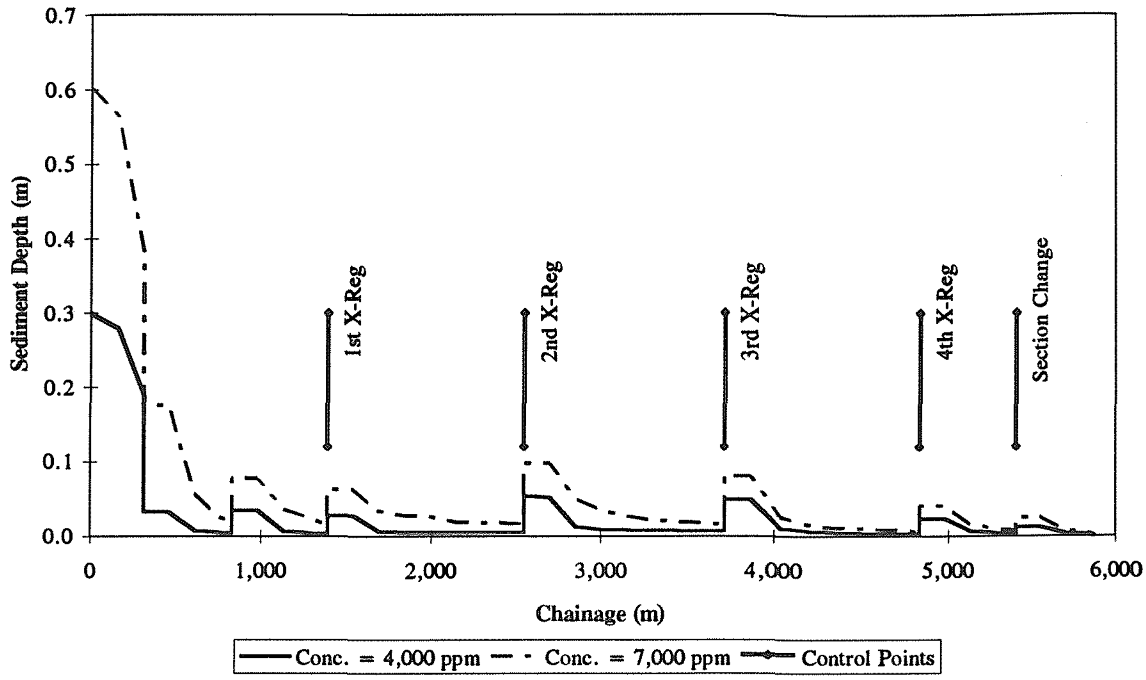

Figure 5.4 The relationship between the concentration of the sediment entering system *A* and the profile of sediment deposition in canal M1C4

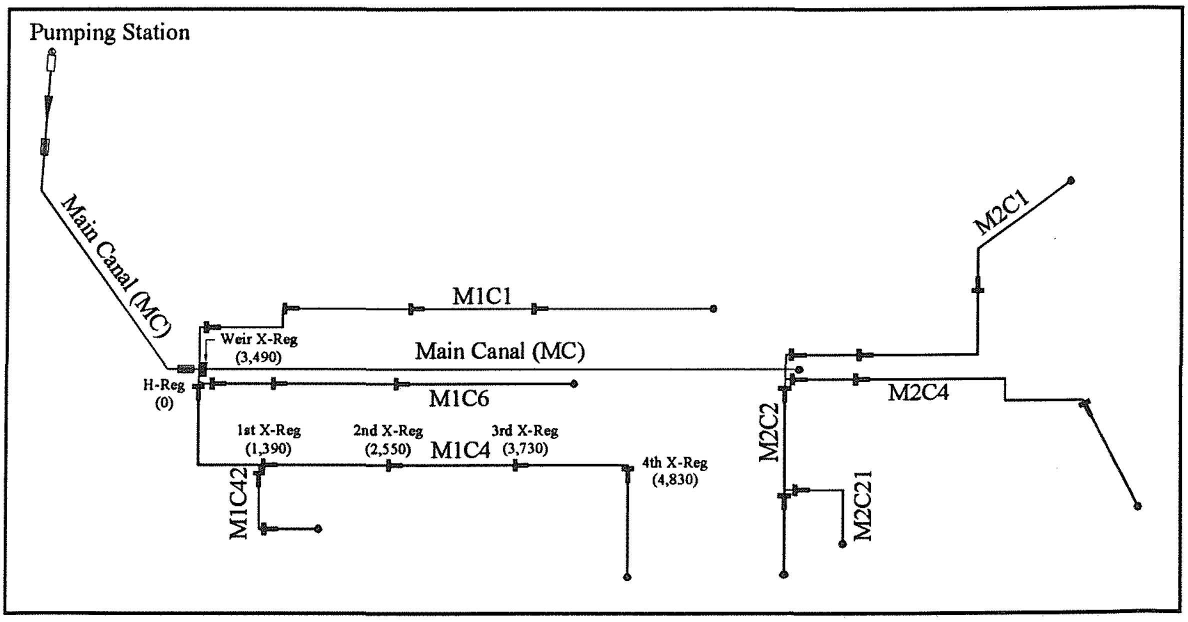

Figure 5.5 Schematic layout of system *A* showing the locations of the regulators of canals MC and M1C4

The above also explains why the depths of sediment immediately downstream from the cross regulators are larger than the depths immediately upstream from the regulators. All the cross regulators on canal M1C4 are located downstream from field intakes to serve them. Consequently, both the flow and the cross-sections of the canal become smaller downstream from each cross regulator. Since the capacity of a stream to transport sediment is affected by its geometry and hydraulic characteristics (Simons & Sentürk, 1992), each canal reach may exhibit the behaviour described above. Additionally, because the cross sections get smaller downstream from some cross regulators, the *depth* of deposited sediment may increase although the *quantity* of deposition might be smaller.

- With the exception of the very top sections of canal M1C4, the sediment profiles predicted by ISIS had an approximate linear relationship with the concentrations of the sediment entering the system (Figure 5.4). For example, the sediment profile at 7,000 ppm sediment concentration can be approximated by raising the sediment profile at 4,000 ppm by the ratio of the two concentrations (7,000/4,000).

5.5 The Percentage of Sedimentation

The sedimentation in a canal is often reported in terms of the average depth of sediment deposition. This method of describing sedimentation was considered not to be representative and informative enough in this research. If for example it is said that the average depth of sediment in a canal is 0.15 m, what indication will this figure give regarding the size of the problem and the possible impact on the canal. Additional information, such as the design water depth for example, will be required in order to give a good 'feeling' of the situation. The ***ratio/percentage of sedimentation*** is suggested as an alternative for describing the condition of sediment deposition in a canal system. It is defined as ***the ratio/percentage of the total volume of sediment to the total water volume in a canal network.*** The advantage of using this indicator is that it immediately gives an indication of one of the most important effects of sedimentation on the hydraulic characteristics of irrigation networks; that is the reduction in the system's carrying capacity.

What adds to the significance of the percentage of sedimentation as an indicator is its readiness to be used in making comparisons between different irrigation systems with respect to sedimentation. For instance, knowing that the average depth of sediment in canal "A" is 0.15 m and that the average depth in canal "B" is 0.25 m does not necessarily imply that the problem in canal "B" is more acute than that in canal "A" (depending on the design water depth of each canal). Comparing the percentages of sedimentation in the two systems will be more accurate and decisive.

Consequently, the percentage of sedimentation was used to define how much sediment existed in all the simulations which will be described in this chapter.

5.6 The Impact of Sedimentation on System Performance

In order to develop a better understanding of the effects of sedimentation on the functionality of irrigation systems a set of hydraulic simulations has been carried out using system *A* as a case study. As explained briefly before, the other purpose of assessing the performance of the system with sedimentation is to use this performance as a reference against which the performance of sediment removal interventions can be compared.

The principal idea was to introduce sediment profiles which correspond to different percentages of sedimentation in the canal network and then to evaluate the performance under full design discharge (the most critical case). Two sets of runs were formulated; one for the simulation of a fully automated system and another for the simulation of a manually-operated system. To enable a comparison between the performances of the two modes of operation to be easily made, the same case study (system *A*) was used in both sets of simulations. The difference was in the operational procedure of the control structures in each set. The details of the simulations carried out are given in the following sections.

5.6.1 Investigating the Impact of Sedimentation on Automated Systems

a. Methodology

Three scenarios have been modelled in order to study the impact of sedimentation on the performance of automated irrigation systems. The features of these hydraulic simulations can be outlined as follows:

- Assume a certain percentage of sedimentation in the canal network of system *A*. Three cases were simulated with 10%, 20% and 30% sedimentation[6] (see Table 5.1). It was shown earlier in Section 5.5 that there is a reasonable correlation between the sediment profiles in system *A* and the concentrations of the sediment entering the system (see Figure 5.4). Consequently, in order to predict the sediment profile which corresponds to any percentage of sedimentation, the sediment profile predicted by ISIS was shifted according to the ratio of the required percentage to the percentage of sedimentation of the profile predicted by ISIS. For example, about 10% of sedimentation deposited in system *A* when the concentration of the sediment entering the system was 4,000 ppm. The sediment profile corresponding to 30% sedimentation can then be worked out by increasing the depths of sediment predicted by ISIS by threefold (30/10).

- Increase the roughness coefficients (Manning *n*) of the canals to allow for the irregularities in the surface, which usually develop in sedimented canal beds, and the roughness of the sediment material itself. In all three cases simulated, Manning *n* was increased by 13% for the main canal and 10% for the distributary canals of system *A* based on general guidelines from the literature (Chow, 1959; Ilaco, 1985).

- Simulate the system under full design discharge, allowing the settings of the gates of the canal regulators and field outlets to be adjusted to maintain design discharges/water levels according to the function of each. These adjustments are

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6 Higher sedimentation percentages are not likely to exist in systems with reasonable canal maintenance programmes. In addition, the simulation of higher sedimentation percentages in hydraulic modelling was critically unstable which indicates the difficulty of the practical operation of irrigation systems which have such large quantities of sediment deposition.

required because of the changes in the shapes of the canal sections and hence their hydraulic characteristics due to sedimentation. This procedure allows for simulating how the gates will operate if they are automated.

- Assess the hydraulic performance of the irrigation system from the output of each simulation.

The characteristics of the hydraulic simulation scenarios which have been carried out are summarised in Table 5.1.

Table 5.1 Brief description of the simulation scenarios for investigating the impact of sedimentation on the performance of fully automated systems

Scenario	Description	Volume of Sediment (m^3)
Sed10-30 *	Sediment profile corresponding to 10% sedimentation in the whole network	19,398
Sed20-30	Sediment profile corresponding to 20% sedimentation in the whole network	38,788
Sed30-30	Sediment profile corresponding to 30% sedimentation in the whole network	58,181

* Throughout the rest of this chapter, the two digits (*nn*) which immediately follow the code name of a scenario (Sed*nn*-xx) indicate the percentage of sedimentation in that scenario.

b. Simulation Results

Due to the fact that the flow entering the system was maintained steady at full design discharge in all the scenarios it can be expected that neither the variability nor the reliability performance measures will be important to assess. In the current scenarios, the important performance measures to evaluate were the adequacy, equity and freeboard.

Table 5.2 summarises the results of evaluating the performance with respect to the adequacy and equity of the water diverted to the field outlets in system *A*. The Delivery Performance

Ratio (DPR) was used to measure both the adequacy and equity. The average values of the Delivery Performance Ratio in Table 5.2 reflect the adequacy of the supply, while the average of the highest 25%, average of the lowest 25% and interquartile ratios (IQR) reflect the equity of water distribution. It is clear from the results that both the adequacy and equity were fairly high in all the scenarios, i.e. **the performance of a fully automated irrigation system with respect to water distribution adequacy and equity is not likely to be seriously degraded due to sediment depositions of percentages up to 30%.**

Table 5.2 Evaluation of the delivery performance ratio (DPR) of the field outlets in system *A* — sedimented automated canals under full design discharge

Scenario	Average	Average of Highest 25%	Average of Lowest 25%	IQR*
Sed10-30	1.00	1.01	0.98	1.04
Sed20-30	1.00	1.01	0.97	1.04
Sed30-30	1.00	1.05	0.94	1.12

* Interquartile Ratio

Nevertheless, before a final conclusion regarding the overall performance of these scenarios can be reached, an important performance criterion, namely the freeboard, must be evaluated.

The percentage of Lost Freeboard (LFb) was used to evaluate the performance with respect to the encroachment on the freeboard. The steps of calculating this performance indicator can be outlined as follows:

1) Work out the percentages of Lost Freeboard of the canal sections for which results are available from hydraulic simulation.

2) Because these canal sections may not be spaced evenly (according to the way the hydraulic model was set up) it is necessary to interpolate the values of Lost Freeboard such that a value is available each, say, 10 m of each canal in the system. Accordingly, if the total length of all the canals in a system is, for example,

2500 m, 250 values of Lost Freeboard should be available.

3) Work out the average, mode[7] and maximum values for the group of Lost Freeboard figures obtained in the previous step.

An example of the results of these calculations is shown in Figure 5.6, which depicts the assessment of the ratio of Lost Freeboard (LFb) for the main canal (MC) of system *A* in scenario Sed30-30. The figure shows that the highest loss of freeboard occurred immediately downstream from the control structures (the head-regulator of the main canal at chainage 0 m and the gated-weir cross-regulator at chainage 3,500 m). The lowest loss of freeboard occurred immediately upstream from the weir cross-regulator because the gate of the weir was adjusted during the runs to maintain the design water level at the weir, thus minimised any encroachment on the freeboard.

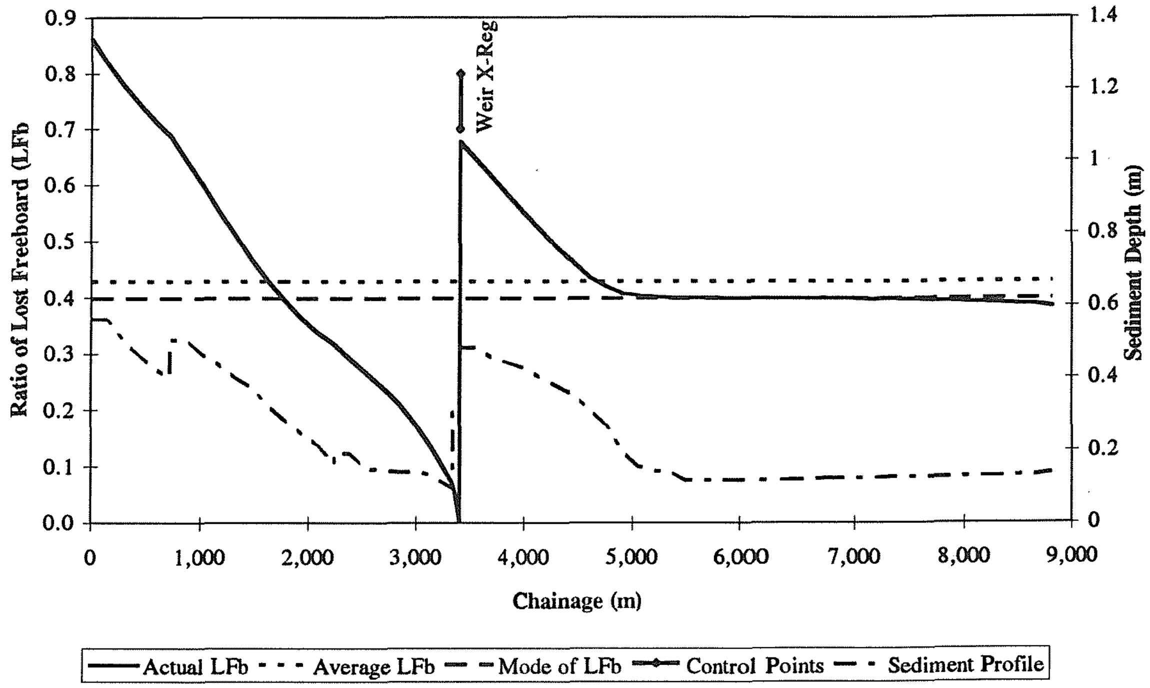

Figure 5.6 Evaluation of the Lost Freeboard of the main canal of system *A* — sedimented automated canals under full design discharge

[7] The *mode* of a range of values is the most frequently occurring value.

c. Categorising the Percentages of Lost Freeboard

Working out the overall values of average, mode and maximum percentage of Lost Freeboard (LFb) for an irrigation system as described above may not always be so helpful in making comparisons between different hydraulic simulation scenarios. The evaluation of the ratio of Lost Freeboard of the main canal in scenario Sed30-30 (Figure 5.6) shows that although the ratio of Lost Freeboard (LFb) varied between about 0.9 at the top of the canal and 0.4 at the tail, both the average and mode values worked out as about 0.4 only. Consequently using only the average and/or mode values of the ratios of Lost Freeboard in assessing the encroachment on the freeboard may be misleading.

A further analysis of the data by classifying the values of Lost Freeboard into categories can be useful in developing a better understanding of the situation. The basic idea is to work out the total length of canal reaches whose Lost Freeboard falls within a certain range (category). The categories of the percentages of Lost Freeboard which were adopted in this work are listed in Table 5.3. The first two categories can also be combined together and described as the *acceptable category/range*, while the last one can be described as the *unacceptable category/range*.

It should be noted that the boundaries defining the three categories adopted in the current work may not be suitable for other studies and therefore different boundaries may be adopted. For example, 25.0% may be considered as high or low as an upper boundary for the *acceptable category* and another figure may be used instead.

Applying this categorisation system to the results of the simulation scenarios investigated yields the output shown in Figure 5.7. It can be seen that the percentage of canal sections which fell within the *unacceptable range* of Lost Freeboard (LFb > 25.0%) increased as the percentage of sedimentation in the canals increased, while the percentage of canal sections which fell within the *acceptable range* decreased. The figure clearly makes the comparison between the performance of the different scenarios with respect to the loss of freeboard much easier and more accurate rather than simply using the overall average or mode of the ratios of Lost Freeboard in making the comparison.

Table 5.3 Categories of the percentage of Lost Freeboard (LFb)

Category of Percentage of Lost Freeboard (LFb)	Description
No Encroachment: **LFb <= -5.0%** (*Acceptable*)	Canal reaches whose percentages of Lost Freeboard (LFb) are less or equal to -5.0%, i.e. the actual water levels are below design levels. Although from the perspective of maintaining safe freeboard, this category is considered to be safe, it may still be desirable to minimise the number of canal reaches which fall in this category because there could be adverse impacts on maintaining sufficient command for irrigating the land and hence on other performance measures.
Target Range: **LFb > -5.0%** & **LFb <= 25.0%** (*Acceptable*)	Canal reaches whose percentages of Lost Freeboard (LFb) are more than -5.0% but do not exceed 25.0%, i.e. the water levels fluctuate slightly above and below design levels. This category is the optimum one because it is characterised by a medium loss of freeboard which does not endanger the safety of the canals and in the meanwhile maintains command.
High Encroachment: **LFb > 25.0%** (*Unacceptable*)	Canal reaches whose percentages of Lost Freeboard (LFb) exceed 25.0%, i.e. the water levels are higher than design levels. It is clear that this category will include canal reaches where a significant encroachment on the design freeboard takes place and consequently should be considered as unacceptable.

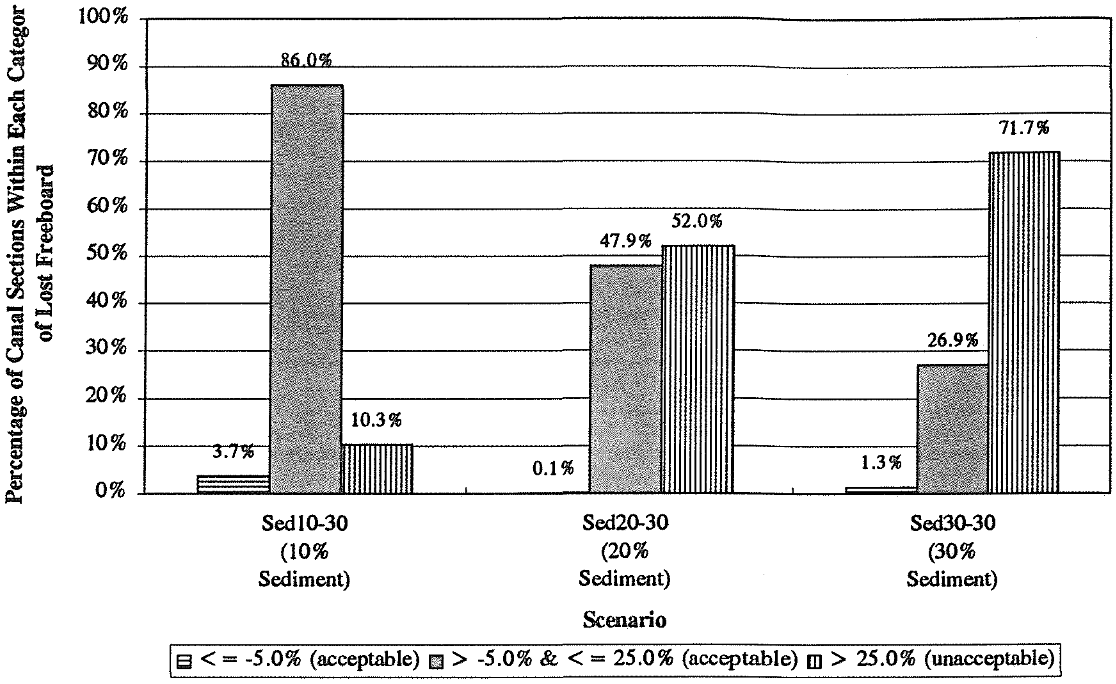

Figure 5.7 **Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — sedimented automated canals under full design discharge**

5.6.2 Investigating the Impact of Sedimentation on Manually-operated Systems

a. Methodology

Three simulation scenarios for investigating the impact of sedimentation on the performance of automated irrigation systems were presented in the previous section. In this section three similar scenarios are presented with the notable difference being the operational procedure of the irrigation control structures. The characteristics of these scenarios can be outlined as follows:

- Assume a certain percentage of sedimentation in the canal network of system *A*. Three cases were simulated with 10%, 20% and 30% sedimentation (see Table 5.4).

- Increase the roughness coefficients (Manning *n*) of the canals to allow for the irregularities in the surface, which usually develop in sedimented canal beds, and the roughness of the sediment material itself. In all three cases simulated, Manning

n was increased by 13% for the main canal and 10% for the distributary canals of system A.

- Simulate the system under full design discharge, allowing the settings of the gates of the canal regulators to be adjusted to maintain the design discharges/water levels according to the function of each. These adjustments are required because of the changes in the shapes of the canal sections and hence their hydraulic characteristics due to sedimentation. The gates of the field outlets were fixed throughout the whole run at the settings which should deliver the maximum design discharge to each field. This arrangement closely follows the operational procedure of most manually-operated irrigation systems, where operation staff usually operate the control structures on the canals while the gates of the field outlets are either left unattended or left to the farmers to operate them. The farmers will naturally leave those gates fully open to divert as much water as possible to their fields irrespective to the actual water available in the canals.

- Assess the hydraulic performance of the irrigation system from the output of the simulations.

The characteristics of these hydraulic simulations are summarised in Table 5.4.

Table 5.4 Brief description of the simulation scenarios for investigating the impact of sedimentation on the performance of manually-operated systems

Scenario	Description	Volume of Sediment (m^3)
Sed10-50	Sediment profile corresponding to 10% sedimentation in the whole network	19,398
Sed20-50	Sediment profile corresponding to 20% sedimentation in the whole network	38,788
Sed30-50	Sediment profile corresponding to 30% sedimentation in the whole network	58,181

b. Simulation Results

As in the case of the previous set of simulations, the evaluation of the system performance focused on the equity and adequacy only. Other measures such as the variability and reliability of the supply to the fields were not important to assess because the flow entering the system was maintained steady throughout all the simulations. The adequacy and equity of water distribution were assessed using the Delivery Performance Ratio (DPR) as outlined in Table 5.5. The average and mode values of the Delivery Performance Ratio reflect the adequacy of the supply, while the average DPR of the highest 25%, average DPR of the lowest 25% and interquartile ratios (IQR) reflect the equity of water distribution.

Table 5.5 Evaluation of the Delivery Performance Ratio (DPR) of the field outlets in system *A* — sedimented manually-operated canals under full design discharge

Scenario	Average	Mode	Average of Highest 25%	Average of Lowest 25%	IQR*
Sed10-50	1.01	1.08	1.17	0.82	1.42
Sed20-50	1.00	0.95	1.32	0.67	1.98
Sed30-50	1.01	0.92	1.52	0.51	2.97

* Interquartile Ratio

Comparing the results in Table 5.5 to those from the previous set of runs (Table 5.2) shows that the performance of the current simulations is poorer. Because the gates of the field outlets were not readjusted to overcome the changes in the hydraulics of the system due to sedimentation, the water distribution equity was too low; with the outlets at the top end withdrawing as much as three times as those at the tail end. The overall adequacy of the supply was therefore also affected (mode of DPR less than 1.0). The performance became poorer as the percentage of sedimentation increased (the performance level of scenario Sed30-50, with 30% sedimentation, was lower than that of scenario Sed10-50, with only 10% sedimentation). **The automation of the gates of the field outlets seems to have compensated for many of the problems of sedimentation leading to better hydraulic performance — a performance which was not achieved with manually-operated outlets.**

Such strong impact of sedimentation on the equity of water distribution in manually-operated irrigation systems has also been observed in the field. Vander Velde (1990) studied the equity of water distribution on Lagar distributary canal in Pakistan. The Delivery Performance Ratio (DPR) was used to assess the water delivery to the offtakes along the 20-km-long canal. The results of the assessments before and after desilting the canal are reproduced in Figure 5.8. Although in this case the equity of water distribution was not so ideal after desilting, it significantly improved. For instance, the Interquartile Ratio (IQR) of the Delivery Performance Ratios (DPR) changed from 3.68 before sediment removal to 1.84 after desilting. In other words, these field observations show that water distribution equity may be lowered by about twofold due to sedimentation; which is in general agreement with the simulation results in Table 5.5.

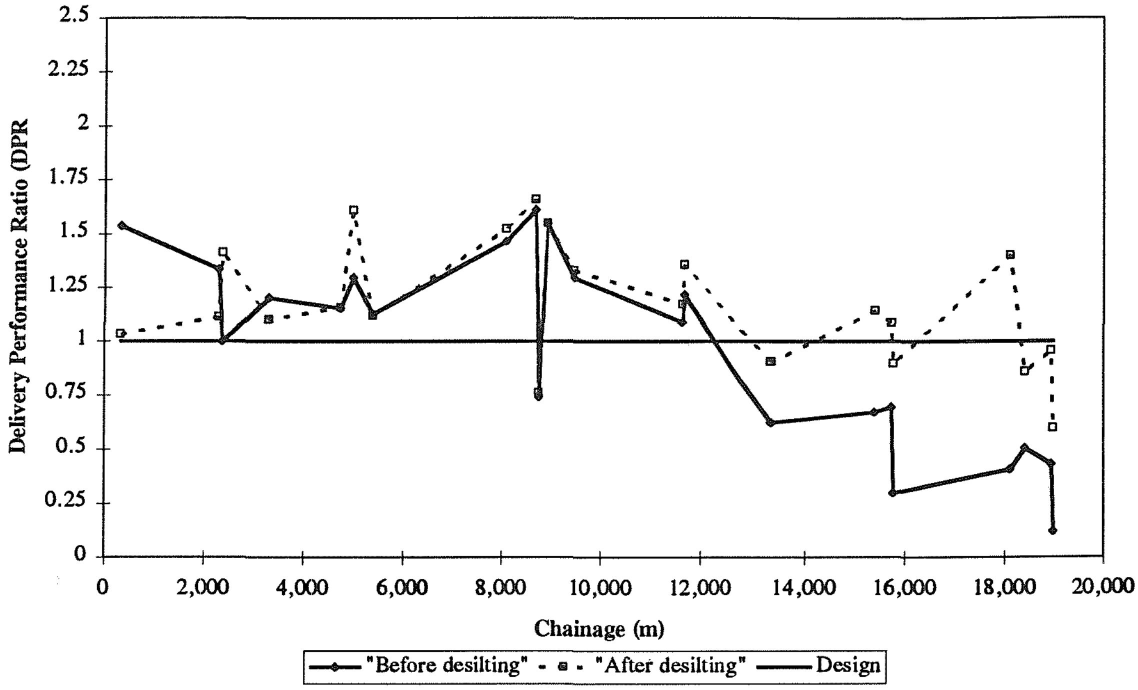

Figure 5.8 Water distribution equity on Lagar distributary in Pakistan before and after desilting the canal (after Vander Velde, 1990)

The assessment of the other important criterion, the freeboard, using the categorisation of the percentages of Lost Freeboard (LFb) is shown in Figure 5.9. On the contrary, the performance of the current set of scenarios has improved compared to the performance of the previous set (automated system). The percentages of canal sections which fell within the *unacceptable range* of Lost Freeboard (LFb > 25%) were smaller than these in the previous set (see Figure 5.7). Because the sediment deposition usually raised the water levels in the canals the field outlets at the top end of the system had a better opportunity to withdraw more water than they should. This did not happen in the previous set of simulations because the outlet gates were automated so they were continuously adjusted to maintain the design discharges. On the other hand, the gates of the outlets in the current simulations were not automated so the outlets at the top end withdrew more water than they should (which affected the equity of water distribution as shown above). This left the lower parts of the system with less flow in the canals and hence the water levels in those parts were also lowered, reducing the encroachment on the freeboard. In other words, the inadequacy of water distribution, although not desirable on its own, had a positive impact on other performance criteria such as the encroachment on freeboard.

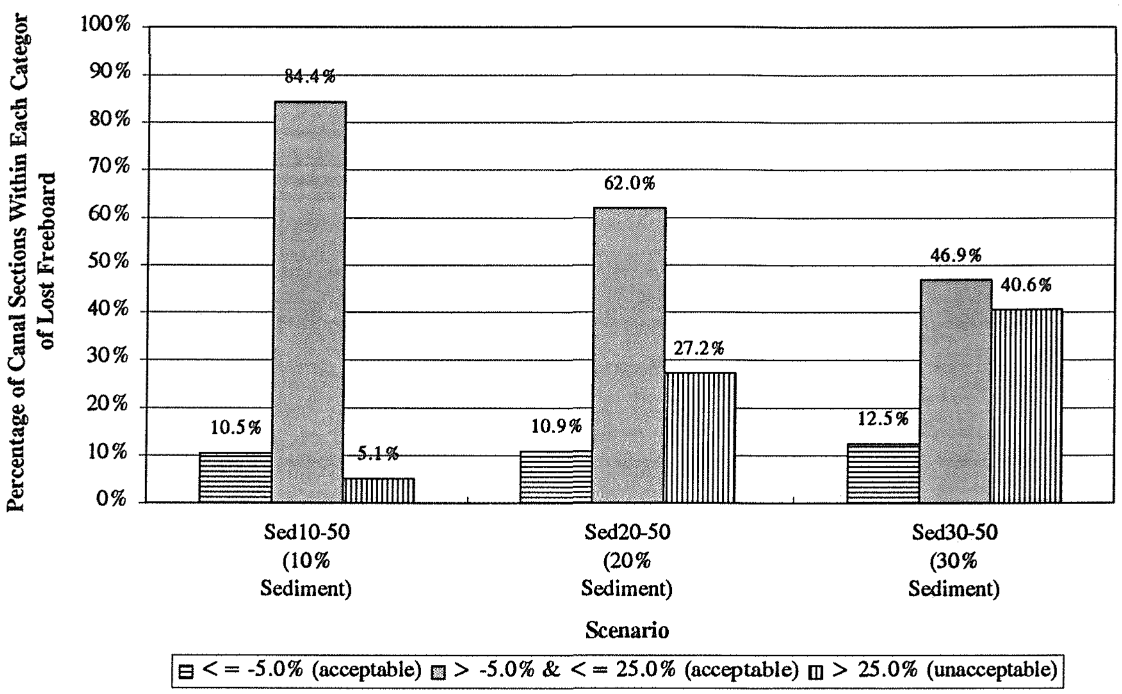

Figure 5.9 **Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — sedimented manually-operated canals under full design discharge**

5.7 Tackling the Problem of Sedimentation in Irrigation Canals

It has been demonstrated in the previous sections that sedimentation in irrigation canals can have adverse impacts on the performance and safety of both automated and manually-operated systems when the full design flow is delivered. It is therefore obvious that sedimentation should be either avoided/minimised by means of providing efficient sediment traps in the irrigation systems prone to sedimentation, or cured by implementing 'good' regular maintenance to remove any serious sedimentation from the canals. However, when such good maintenance cannot be carried out for reasons such as lack of funds, other temporary solutions should be sought. Two possible temporary solutions are to:

(1) reduce the maximum permissible flow in the system when excessive sedimentation takes place in order to maintain its safety (by ensuring sufficient freeboard) at the expenses of scheme production; or,

(2) carry out selective maintenance under the restrictions of available resources. This solution will need a system of prioritisation/optimisation which can aid in selecting the best alternative for implementation.

The following sections address these two solutions by examining their effectiveness and hydraulic efficiencies. The financial aspects of these solutions are dealt with in Chapter 7.

5.7.1 Maximum Permissible Flows in Sedimented Canals

When sufficient funds cannot be secured for carrying out 'good' maintenance to remove or reduce the sedimentation in irrigation canals, system operators may often be left with one of two options to choose from: (1) either to keep on operating the system normally allowing the full design discharge into the system and consequently encroaching on the design freeboard thus putting the system at the risk of canal failure and flooding, or (2) to reduce the maximum permissible flow in order to maintain safe freeboard. While the risks associated with the first option have some uncertainties, the second option seems to have more definitive effect of reducing potential crop production and hence the revenue of the scheme. This decision making should be guided by a financial analysis of the consequences of each option.

Studying the first option involves two main steps: (i) predicting the sections of the canals and other control structures where failures such as canal breaches might take place due to high water levels, and (ii) estimating the damage which those failures could cause. Hydraulic simulation can be used in the first step to locate the most hazardous sections in a canal network. For example, in the case depicted in Figure 5.6 the two most critical locations in the main canal were downstream from the two control structures. The second step will however be more difficult to carry out, with or without hydraulic modelling. Estimating the damage caused by a canal breach in terms of loss of property, lives, land and crops, etc. will be somehow speculative.

The second option, on the other hand, can be fully examined using hydraulic simulation techniques as will be presented in the following sections.

a. Methodology of Investigation

The methodology of investigating and ascertaining the consequences of tackling the sedimentation problem by reducing the maximum permissible flow in the canals using hydraulic simulation techniques can be outlined as follows:

- Assume a certain percentage of sedimentation in the canal network of system *A*. Three cases were simulated with 10%, 20% and 30% sedimentation (see Table 5.6).

- Increase the roughness coefficients (Manning *n*) of the canals to allow for the irregularities in the surface, which usually develop in sedimented canal beds, and the roughness of the sediment material itself. In all the cases simulated Manning *n* was increased by 13% for the main canal and 10% for the distributary canals of system *A*.

- Allow the settings of the gates of the canal regulators to be adjusted to maintain the design discharges/water levels according to the function of each. These adjustments are required because of the changes in the shapes of the canal sections, and hence their hydraulic characteristics, due to sedimentation. Such adjustments will take place if the structures are automated and are also very likely to take place if they are manually operated since operation staff will usually operate the main structures on the canals.

For the lower order field outlets two cases need to be simulated: (i) the case of automated structures where the outlets should also be adjusted during the simulations to maintain target discharges, and (ii) the case of manually operated structures where the outlets should be kept fixed at the fully-open position (not adjusted) throughout the simulations.

- Find out the maximum permissible discharge which can be allowed in the system without endangering its safety. The criterion adopted in this research defined the maximum permissible flow as **the discharge which when allowed in sedimented canals will cause a maximum of 10% of all canal reaches to fall in the *unacceptable category* of percentage of Lost Freeboard (LFb)**, i.e. only 10% or less of the total length of the canals in the system may have LFb > 25%.

- Assess the hydraulic performance of the irrigation system from the output of the simulations.

- Estimate the losses in the potential agricultural production of the scheme and compare them with the cost of removing the sediment from the canals if the budget was available, thus find out whether it is economically viable to secure the funds to clear the sediment or not. This step is carried out in Section 7.4.

This methodology was implemented in the hydraulic simulation scenarios summarised in Table 5.6.

b. Simulation Results

Because the hydraulic model ISIS did not have any built-in features for finding out the maximum permissible discharge according to the criterion set out above, this exercise had to be done by trial and error. A reasonable reduced inflow was assumed and then the model was run under constant flow to simulate the operation during a period of about four days to allow it to reach steady state. The output of the model was then evaluated against the criterion mentioned above. If the results complied with this criterion then the solution was found, otherwise the assumed reduced flow was altered in guidance of the results and then another trial was carried out.

Table 5.6 Brief description of the simulation scenarios for investigating the maximum permissible flow in sedimented canals

Scenario	Description	Volume of Sediment (m^3)
Sed10-31	- Sediment profile corresponding to 10% sedimentation in the whole network. - Fully automated irrigation system.	19,398
Sed20-31	- Sediment profile corresponding to 20% sedimentation in the whole network. - Fully automated irrigation system.	38,788
Sed30-31	- Sediment profile corresponding to 30% sedimentation in the whole network. - Fully automated irrigation system.	58,181
Sed10-50	- Sediment profile corresponding to 10% sedimentation in the whole network. - Manually-operated irrigation system.	19,398
Sed20-51	- Sediment profile corresponding to 20% sedimentation in the whole network. - Manually-operated irrigation system.	38,788
Sed30-51	- Sediment profile corresponding to 30% sedimentation in the whole network. - Manually-operated irrigation system.	58,181

The maximum permissible flows for the different scenarios are depicted in Figure 5.10 as ratios of the maximum design discharge (Discharge Capacity Ratio, DCR). Two separate relationships between the percentage of sedimentation and the discharge capacity ratios (DCR) in the case of automatic operation and the case of manual operation were established. Both relationships are almost linear for system A. For the same percentage of sedimentation, the discharge capacity ratio in the case of the manually-operated system is usually 5% larger than that in the case of the automated system. The explanation of the difference in the performance of the two modes of operation with respect to the loss of freeboard has been given in Section 5.6.2.

Although the process of finding out the maximum permissible flows using hydraulic modelling as described above was time consuming, it is more accurate than using uniform flow calculations (for example, Manning's uniform depth equation). Hydraulic modelling takes into consideration many hydraulic factors such as back water curves and other non-uniform flow situations which are difficult to work out manually. For instance, it would have been very difficult to estimate the difference in the discharge capacity ratios between the two modes of operation using manual calculations as was done using hydraulic simulation (Figure 5.10).

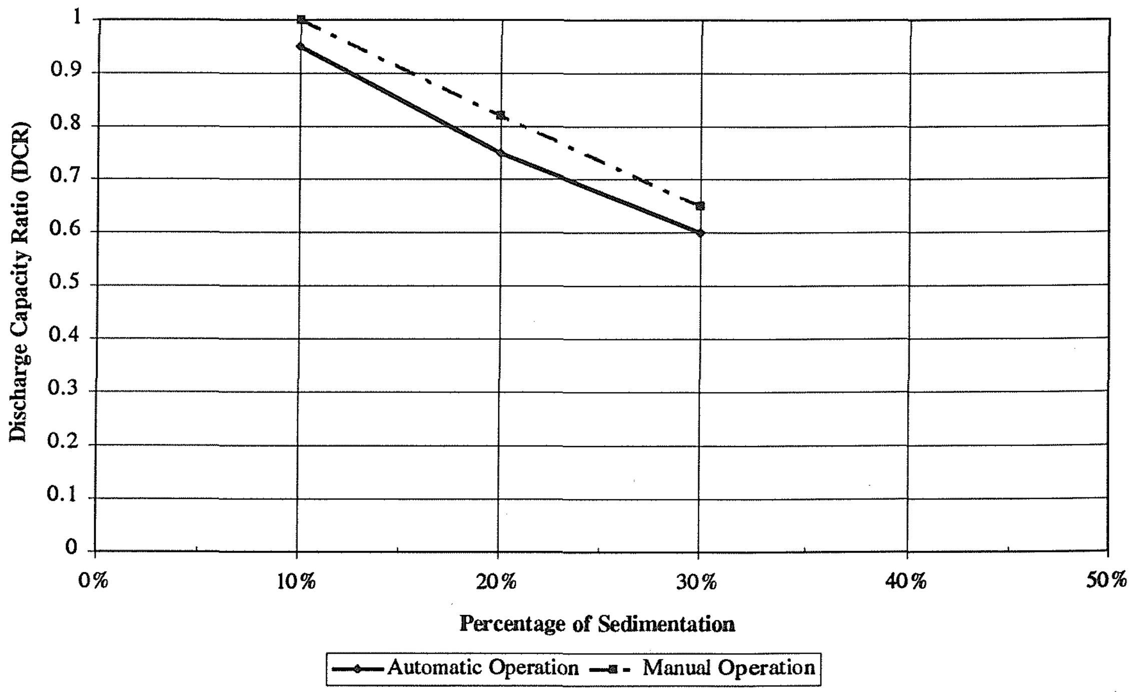

Figure 5.10 Maximum permissible flows in system A for different percentages of sedimentation under two operation modes

Figure 5.10 is one of the examples which proves the usefulness of the approach adopted in this research in using hydraulic modelling techniques for expenditure planning. For example, such a figure can be used for controlling the operation of a system within safe limits. It can also be used to evaluate the potential loss, as detailed in Section 7.4.

To ensure that the output of the simulations complied with the criterion of maximum encroachment on freeboard, the distributions of the canal reaches within each category of Lost Freeboard for each simulated scenario were worked out and depicted in Figure 5.11. The maximum percentage of canal reaches within the *unacceptable category* of Lost Freeboard (LFb > 25%) did not exceed the previously-set 10% limit in any of the simulations.

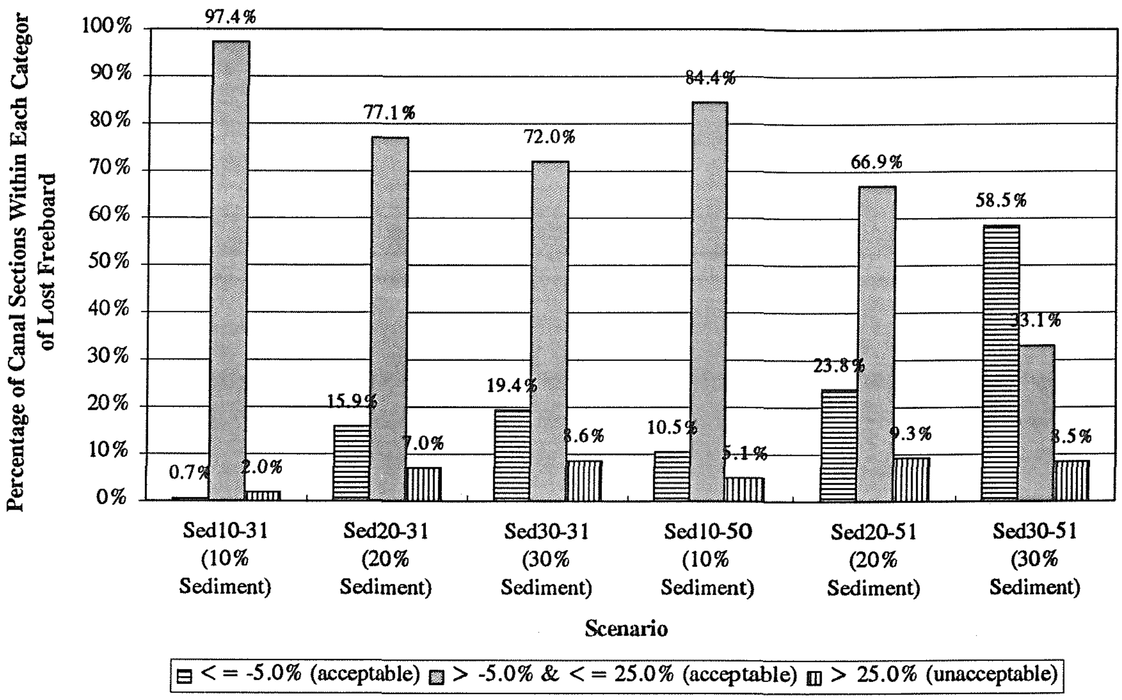

Figure 5.11 Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — sedimented canals under maximum permissible flows

The assessment of another important performance measure, the equity of flow distribution, is summarised in Table 5.7. The Delivery Performance Ratio (DPR) was calculated using the maximum permissible flow of each scenario as its target flow. Consequently, the theoretical optimum value for any of the entries in the table is 1.0. The results show that the equity of water distribution between the field outlets in irrigation system *A* was fairly high in all the scenarios simulating the automated system (the first three scenarios in the table). However, the equity was much poorer in the case of manual operation (the last three scenarios in Table 5.7). Additionally, for the automated system the equity was not much different from the full-supply case, but was significantly different in the manually operated system (compare the figures in the IQR and Reference IQR columns in the table).

Table 5.7 Evaluation of the Delivery Performance Ratio (DPR) of the field outlets in system *A* — sedimented canals under maximum permissible flow

Scenario	Average	Mode	Average of Highest 25%	Average of Lowest 25%	IQR*	Reference IQR
Sed10-31	1.00	1.00	1.01	0.98	1.04	1.04 §
Sed20-31	1.00	1.01	1.01	0.97	1.05	1.04 §
Sed30-31	1.00	1.01	1.06	0.91	1.16	1.12 §
Sed10-50	1.01	1.08	1.17	0.82	1.42	1.42 †
Sed20-51	1.00	1.18	1.38	0.47	2.96	1.98 †
Sed30-51	1.01	0.91	1.75	0.39	4.46	2.97 †

* Interquartile Ratio

§ Figure from Table 5.2

† Figure from Table 5.5

The adequacy in terms of supplying the maximum permissible flow to every field outlet was relatively high in all the scenarios. Accordingly, it can be concluded that reducing the maximum permissible flow in irrigation systems suffering from sedimentation problems can yield good results in terms of improved system safety if the systems are fully automated, but will not perform as well in the case of manually-operated systems. Yet, in all cases the revenue of the scheme might be reduced due to the reduction in the crop yield as a result of not supplying the maximum design discharge which might be needed in the period of maximum crop water requirements.

5.7.2 Prioritising Sediment Removal Activities

A second possibility for curing the sedimentation problem under the restrictions of a limited budget is to opt for selective maintenance such that only *important* tasks or parts of tasks are carried out. The important question which needs answering in this case will be how the tasks (desilting in this case) can be prioritised to know which ones to start with. The following sections attempt to answer this question through the development of a methodology for prioritising sediment removal activities.

a. Possible Alternatives for Partial Sediment Removal

A practical scenario of a sediment deposition problem was required in order to use it as a case study for the investigation. The scenario chosen assumed that the canal network of system *A* had 30% sedimentation. The funding available for sediment removal was sufficient for *only half of the sediment volume to be removed* (which equates to about 29,000 m^3 at 30% sedimentation in system *A*). Several alternatives for the partial removal of sediment were identified for investigation. These are listed in Table 5.8. A system for ranking these alternatives was therefore required.

Schematic illustrations of how alternatives 1 to 6 may be implemented in system *A* are given in Figures 5.12 to 5.17 respectively. Although the descriptions of alternatives 3 & 4 imply that the sediment in the upstream or downstream half of the canals respectively should be removed, Figures 5.14 and 5.15 show that the reaches which were actually chosen to be cleaned were always confined within canal control structures (regulators). In other words, the descriptions of alternatives 3 & 4 should not imply that exactly 'half' of the length of each canal should be cleaned. For practical reasons, it is more convenient to choose to clean or not clean a reach that is defined by control structures.

Table 5.8 Possible alternatives for the partial removal of sediment from the canals of system *A*

Alternative No.	Brief Description
1	Remove all the sediment in the main canal only
2	Remove all the sediment in the distributary canals only
3	Remove all the sediment in the upstream half of every canal only
4	Remove all the sediment in the downstream half of every canal only
5	Remove all the sediment in all the canals in the upstream half of the system
6	Remove all the sediment in all the canals in the downstream half of the system
7	Remove half of the sediment depth in the whole network

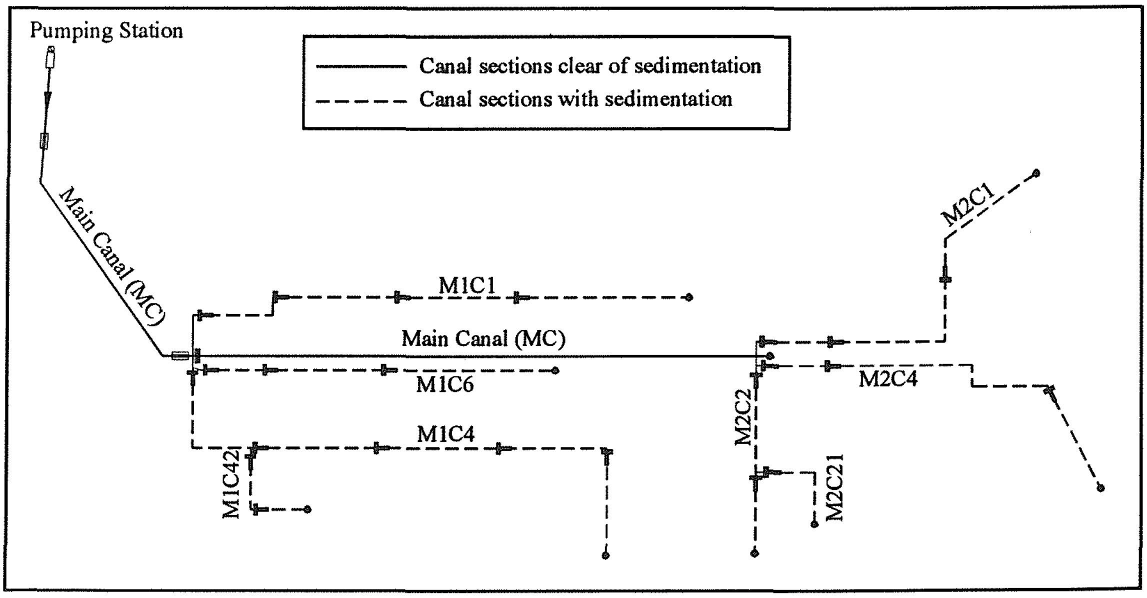

Figure 5.12 Schematic layout of system *A* showing the alternative of removing the sediment in the main canal only

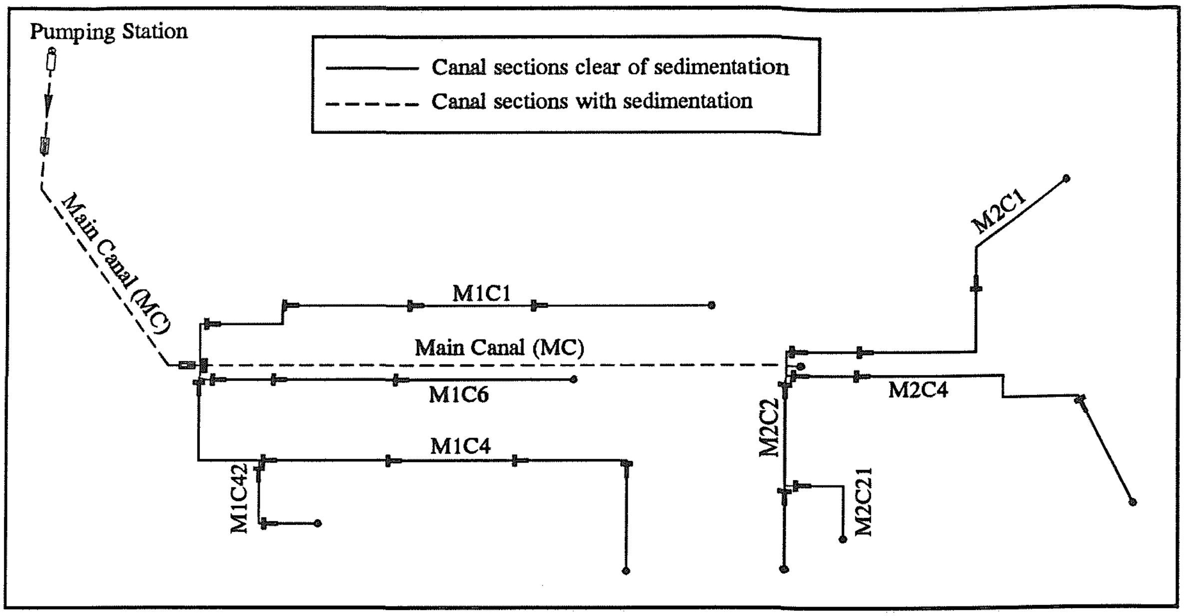

Figure 5.13 **Schematic layout of system *A* showing the alternative of removing the sediment in the distributary canals only**

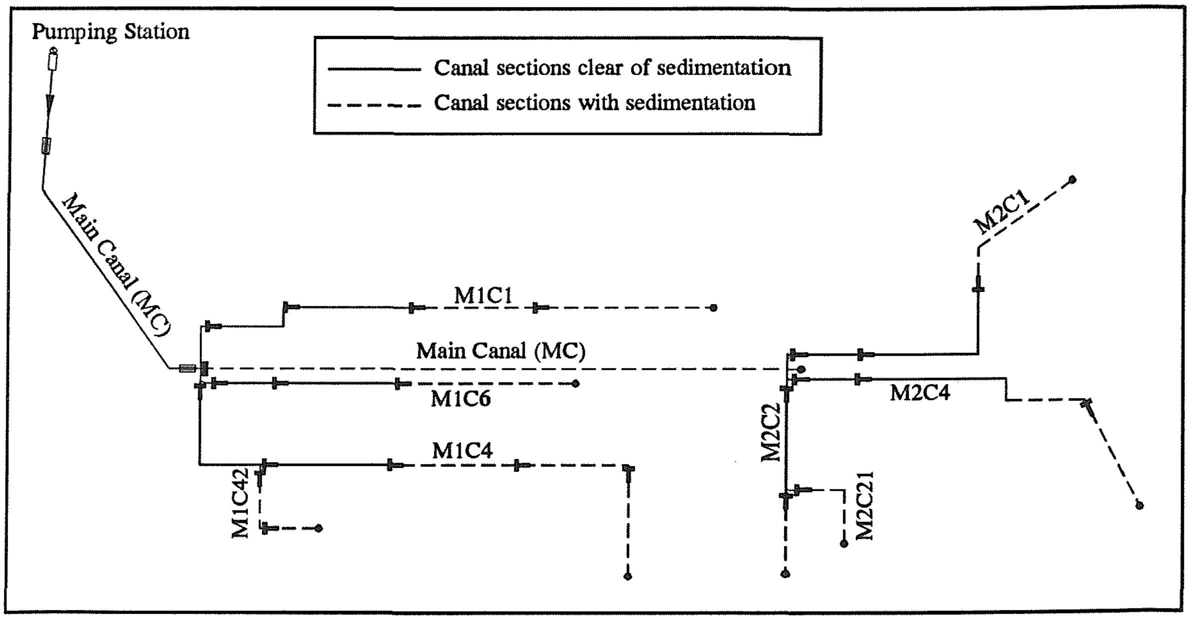

Figure 5.14 **Schematic layout of system *A* showing the alternative of removing the sediment in the upstream half of every canal only**

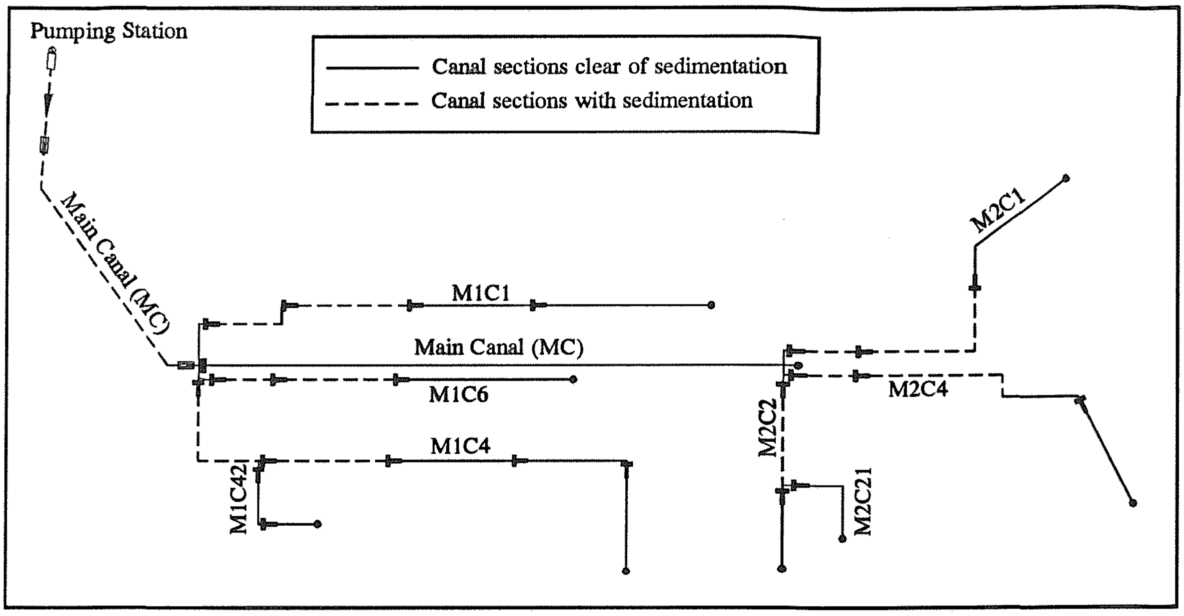

Figure 5.15 Schematic layout of system *A* showing alternative of removing the sediment in the downstream half of every canal only

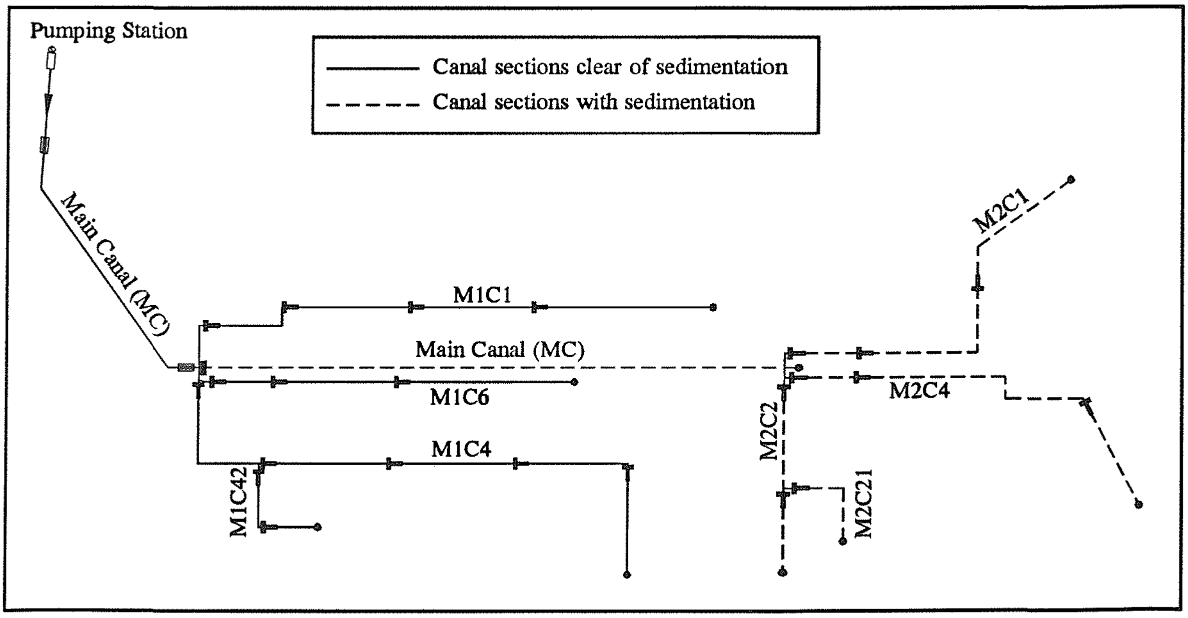

Figure 5.16 Schematic layout of system *A* showing the alternative of removing the sediment in all the canals in the upstream half of the system only

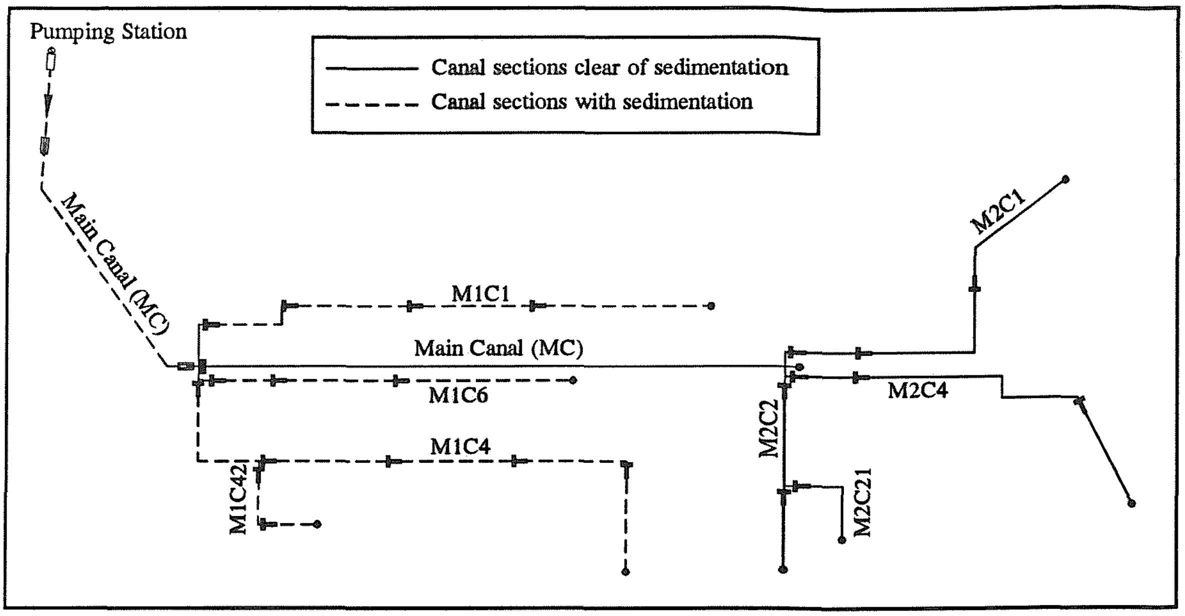

Figure 5.17 Schematic layout of system *A* showing the alternative of removing the sediment in all the canals in the downstream half of the system only

Besides these different alternatives for sediment removal another two factors need consideration. These are the mode of system operation (manual vs automatic) and the sensitivity of the change in the roughness coefficients (Manning n) before and after cleaning the sediment. The results of the simulations presented earlier in this chapter proved that the mode of system operation had important impacts on their performance so it was also important to consider this factor in the current case. Studying the sensitivity of the change in the roughness coefficients is similarly important to ensure that any conclusions drawn from the results of the investigation will rationally be valid for other situations.

Many sets of simulation scenarios were set up to cover all the possible alternatives which should be studied. In each set a certain mode of system operation and a change in the roughness coefficients were chosen according to the specific objectives of the set. With these two variables maintained constant in each set, the different alternatives for partial sediment removal were introduced one by one, thus creating the members of the set (individual simulations). Each set of scenarios and the characteristics of its individual simulations will be described in subsequent sections.

Examining the performance of irrigation systems with sedimentation problems under reduced maximum permissible flows (Section 5.7.1) showed that in the case of system *A* with 30% sedimentation it was possible to reduce some of the adverse impacts of sedimentation by not allowing discharges which exceed a discharge capacity ratio of about 0.65 (Figure 5.10). It is therefore logical to study the prioritisation of sediment removal under higher capacity ratios since one of the objectives of cleaning the sediment from the canals should be to restore their original carrying capacities for better water delivery. In this case the full design discharge was used as the target that should be achieved, hence all the simulations for testing the sediment removal alternatives were carried out under full design discharge.

b. Prioritising Sediment Removal from Automated Systems

i. The Base Case (scenarios Sed30-33 to Sed30-39)

This first set of scenarios for establishing a system of prioritisation for the alternatives of partial sediment removal from automated irrigation systems can be considered as a base case for subsequent ones. The features of the hydraulic simulations carried out are outlined as follows:

- Introduce the sediment profile corresponding to 30% sedimentation in the canals of the case study (system *A*).

- Remove the sediment from certain parts of the irrigation network of system *A* according to one of the possible alternatives described before. If the alternative indicated that all the sediment in a certain canal section should be removed, the original design dimensions of this section were assumed to be restored after sediment removal.

- Increase the roughness coefficients (Manning *n*) of the canal sections where there are sediment depositions to allow for the irregularities in the surface, which usually develop in sedimented canal beds, and the roughness of the sediment material itself.

Manning n was increased by 13% for the main canal and 10% for the distributary canals of system A. The original roughness coefficients assumed in the design of the system were used for canal sections from which the sedimentation was cleaned.

- Simulate the system under full design discharge, allowing the settings of the gates of the canal regulators and field outlets to be adjusted to maintain the design discharges/water levels according to the function of each. These adjustments are required because of the changes in the shapes of the canal sections and hence their hydraulic characteristics due to sedimentation. This procedure allows for the simulation of how the gates will operate if they are automated.

- Repeat the scenario by adopting a different sediment removal alternative and modify the canal sections and roughness coefficients accordingly.

- Assess the performance of the different simulation scenarios to compare between them.

A list of the simulations and the sediment removal alternative adopted in each one is given in Table 5.9.

Simulation Results

Table 5.10 summarises the results of evaluating the adequacy and equity of water distribution for the different simulation scenarios. The Delivery Performance Ratio (DPR) was used as an indicator for evaluating both performance measures. The average and mode values of the Delivery Performance Ratio reflect the adequacy of the supply, while the average DPR of the highest 25%, average DPR of the lowest 25% and interquartile ratios (IQR) reflect the equity of water distribution between the field outlets. It is clear from the results that both the adequacy and equity were fairly high in all the scenarios which means that in this respect all the alternatives of sediment removal were reasonably equal in performance.

Table 5.9 **Brief description of the scenarios for investigating the prioritisation of sediment cleaning activities for automated systems with medium to normal roughness**

Scenario	Description	Illustration	Volume of Removed Sediment (m^3)
Sed30-33	Clean all the sediment in the main canal only	Figure 5.12	24,428
Sed30-34	Clean all the sediment in the distributary canals only	Figure 5.13	33,753
Sed30-35	Clean all the sediment in the upstream half of every canal only	Figure 5.14	30,963
Sed30-36	Clean all the sediment in the downstream half of every canal only	Figure 5.15	27,218
Sed30-37	Clean all the sediment in all the canals in the upstream half of the system	Figure 5.16	35,229
Sed30-38	Clean all the sediment in all the canals in the downstream half of the system	Figure 5.17	22,952
Sed30-39	Clean half of the sediment depth in the whole network	--	29,091

* Total volume of sediment at 30% sedimentation ratio is 58,181 m^3.

The results of assessing the performance of these scenarios with respect to the loss of freeboard using the categorisation of the percentages of Lost Freeboard (LFb) are depicted in Figure 5.18. The figure shows that some scenarios performed better than others. The objective function in these scenarios is to *minimise* the percentages of canal sections which fall within the *unacceptable range* of Lost Freeboard (LFb), and therefore *maximise* the percentages of canal sections which fall in the *acceptable range*. In this respect, scenarios Sed30-39 (clean half of the sediment depth in the whole canal network) and Sed30-34 (clean all the sediment in the distributary canals) achieved the best performance respectively.

Scenarios Sed30-35 (clean all the sediment in the upstream half of every canal) and Sed30-33 (clean all the sediment in the main canal), on the other hand, were the worst.

Table 5.10 Evaluation of the Delivery Performance Ratio (DPR) of the field outlets in system *A* — automated canals with partial sediment removal and medium to normal roughness

Scenario	Average	Mode	Average of Highest 25%	Average of Lowest 25%	IQR*
Sed30-33	1.00	1.00	1.05	0.94	1.12
Sed30-34	1.00	0.99	1.01	0.99	1.02
Sed30-35	1.00	1.01	1.01	0.98	1.03
Sed30-36	0.99	0.99	1.05	0.95	1.10
Sed30-37	1.00	1.01	1.01	0.98	1.03
Sed30-38	0.99	0.99	1.05	0.93	1.12
Sed30-39	1.00	1.00	1.01	0.98	1.03

* Interquartile Ratio

Sediment Removal Effectiveness

When formulating the sediment deposition problem earlier in this section it was assumed that available resources were sufficient for cleaning only half of the sediment depth in the canal network (about 29,000 m^3 at 30% sedimentation in system *A*). However, Table 5.9 shows that the quantities of sediment which should be removed in each alternative are not exactly equal and differ slightly from this figure. These variations in the volumes of the sediment to be cleaned in each alternative were permitted for two reasons:

1) the alternatives had to be as realistic and practical as possible. Thus, to assume that a whole canal reach should be cleaned, even if this means that the volume of sediment to be cleaned is slightly larger than the maximum set, is more realistic than assuming that say only 85% of the reach is to be cleaned because this matches exactly with the maximum limit; and

2) there is usually some allowance in most maintenance budgets to cover any extra urgent work or to allow for the differences between the estimated quantity of work and the actual quantities.

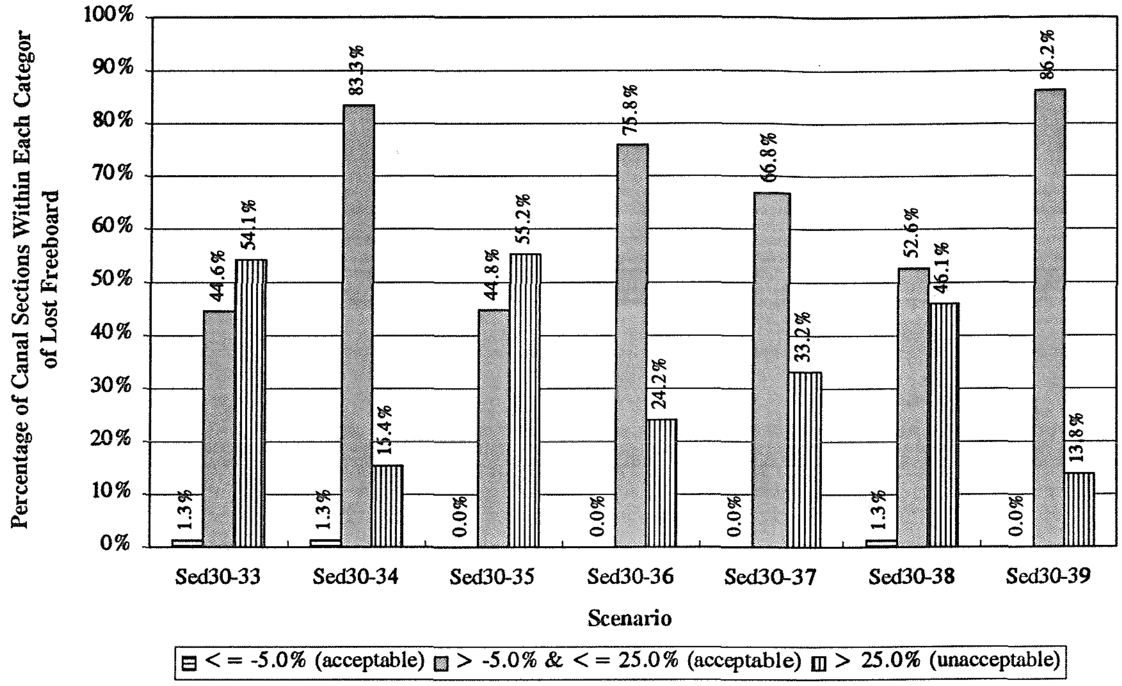

Figure 5.18 Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — automated canals with partial sediment removal and medium to normal roughness

Consequently, in order to consider the differences between the volumes of the sediment cleaned in each scenario when comparing their performance, another performance indicator was introduced. This indicator, the **Sediment Removal Effectiveness**, correlates the volume of removed sediment with the change in the percentage of canal sections which fall within the *unacceptable range* of Lost Freeboard. It is defined as **the reduction in the percentage of canal sections which fall within the unacceptable range of Lost Freeboard (LFb > 25%) due to cleaning 1 ha.m (10,000 m^3) of sediment.** Consequently, the larger the value of the Sediment Removal Effectiveness the better the scenario for sediment removal. The Sediment Removal Effectiveness can numerically be worked out as the difference between the percentages of canal sections within the *unacceptable range* of Lost Freeboard before and after removing the sediment divided by the volume of removed sediment.

The results of evaluating the Sediment Removal Effectiveness for the simulation scenarios under investigation are shown in Figure 5.19. It is interesting to observe the slight difference between the results in Figures 5.18 and 5.19. When the categorisation of the percentage of Lost Freeboard (LFb) was used as the basis for making comparisons between the scenarios, scenario Sed30-39 came first in performance, followed by scenarios Sed30-34 and Sed30-36 (Figure 5.18). This order changed slightly when the Sediment Removal Effectiveness was used as the basis for comparison; so it became Sed30-39, Sed30-36 and then Sed30-34; i.e. the orders of the latter two scenarios were swapped. The change is due to the observable difference between the volumes of sediment cleaned in the two scenarios (see Table 5.9) which the Sediment Removal Effectiveness considers, but which the categorisation of the percentages of Lost Freeboard does not take into account.

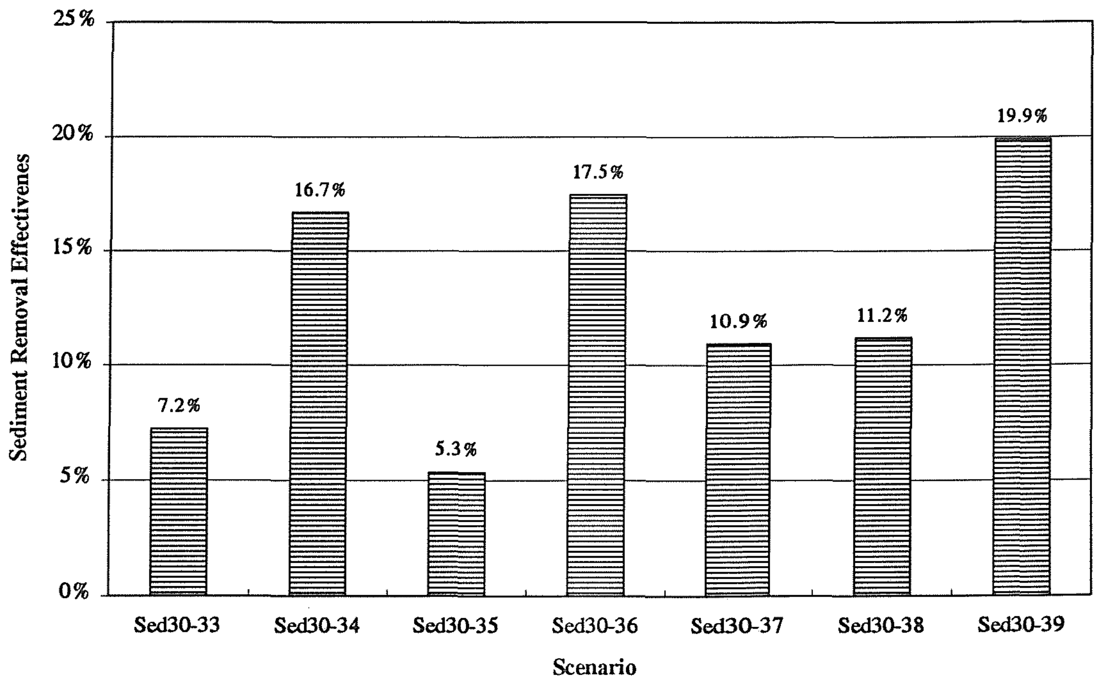

Figure 5.19 Effectiveness of the different scenarios for partial sediment removal from system *A* — automated canals with medium to normal roughness

ii. Sensitivity of the Impact of Sedimentation on the Change in the Roughness

Sediment deposition usually increases the roughness of irrigation canals due to the development of bed forms such as the ripples and dunes (ASCE, 1963), the roughness of the sediment material itself which may be brought in from distant catchments thus has different material composition, and the growth of vegetation which is encouraged by sedimentation. Predicting the exact increase in the roughness due to sedimentation is usually difficult because of the variations in each of the factors involved. The main objective of this section is to test the sensitivity of the results obtained from the previous set of simulations to the variations in the roughness coefficients due to sedimentation.

In all the hydraulic simulations presented so far in this chapter two assumptions regarding the roughness of the canals were made:

1) for the canal sections where sedimentation existed (was not cleaned), Manning n was increased by 10 to 13% above design figures based on general guidelines from the literature (Chow, 1959; Ilaco, 1985); and

2) for the canal sections from which sedimentation was cleaned, it was assumed that the quality of cleaning the sediment was high and therefore the roughness coefficients of those canal sections would drop again to design values.

Because there is a good chance that the actual situation in many irrigation systems might be different from these assumptions, the sensitivity of the two assumptions was tested. The sensitivity analysis was split into two cases: (1) to test the sensitivity of the variation in the increase in the roughness of the sedimented sections; and (2) to test the sensitivity of the assumption that cleaning the sediment reduces the roughness to the original figures chosen in the design of the system. A separate set of simulation scenarios was prepared for studying each case. The details of each of these sets are given below.

Roughness of Sedimented Sections (scenarios Sed30-43 to Sed30-48)

The specific objective of the current set of simulations is to investigate the sensitivity of the change in the roughness coefficients due to sedimentation on the results obtained from the previous set. To account for the increase in the roughness of the canal sections where sediment deposition took place, Manning n was increased above design figures by 13% for the main canal and 10% for the distributary canals of system A in the previous set of simulations (*The Base Case*). In the current set the roughness coefficients of the sedimented sections were increased above design figures by 30% for all the canals while the clean sections were assumed to have design roughness. The outline of the scenarios is as follows:

- Introduce the sediment profile corresponding to 30% sedimentation in the canals of the case study (system A).

- Remove the sediment from certain parts of the irrigation network of system A according to one of the possible alternatives described before. If the alternative indicated that all the sediment in a certain canal section should be removed, the original design dimensions of this section were assumed to be restored after sediment removal.

- Increase the roughness coefficients (Manning n) of the canal sections where there is sediment deposition to allow for the irregularities in the surface, which usually develop in sedimented canal beds, and the roughness of the sediment material itself. Manning n of all the sedimented sections was increased by 30% above design values. The original roughness coefficients assumed in the design of the system were used for canal sections from which the sedimentation was cleaned.

- Simulate the system under full design discharge, allowing the settings of the gates of the canal regulators and field outlets to be adjusted to maintain the design discharges/water levels according to the function of each. These adjustments are required because of the changes in the shapes of the canal sections and hence their hydraulic characteristics due to sedimentation. This procedure allows for the simulation of how the gates will operate if they are automated.

- Repeat the scenario by adopting a different sediment removal alternative (see Table 5.8) and modify the canal sections and roughness coefficients accordingly.

- Assess the performance of the different simulation scenarios to compare between them.

Table 5.11 Brief description of the scenarios for investigating the prioritisation of sediment cleaning activities for automated systems with high to normal roughness

Scenario	Description	Illustration	Volume of Removed Sediment (m^3)
Sed30-43	Clean all the sediment in the main canal only	Figure 5.12	24,428
Sed30-44	Clean all the sediment in the distributary canals only	Figure 5.13	33,753
Sed30-45	Clean all the sediment in the upstream half of every canal only	Figure 5.14	30,963
Sed30-46	Clean all the sediment in the downstream half of every canal only	Figure 5.15	27,218
Sed30-47	Clean all the sediment in all the canals in the upstream half of the system	Figure 5.16	35,229
Sed30-48	Clean all the sediment in all the canals in the downstream half of the system	Figure 5.17	22,952
Sed30-39	Clean half of the sediment depth in the whole network	--	29,091

* Total volume of sediment at 30% sedimentation ratio is 58,181 m^3.

A list of the simulations and the sediment removal alternative adopted in each one is given in Table 5.11.

Simulation Results

The Delivery Performance Ratio (DPR) was used as an indicator for evaluating both the adequacy and equity of water distribution between the fields. The results are summarised in Table 5.12 for the different simulation scenarios. The average and mode values of the Delivery Performance Ratio reflect the adequacy of the supply, while the average DPR of the highest 25%, average DPR of the lowest 25% and interquartile ratios (IQR) reflect the equity of water distribution between the field outlets. It is clear from the results that both the adequacy and equity were fairly high in all the scenarios which means that in this respect all the alternatives of sediment removal were reasonably equal in performance.

Table 5.12 Evaluation of the Delivery Performance Ratio (DPR) of the field outlets in system *A* — automated canals with partial sediment removal and high to normal roughness

Scenario	Average	Mode	Average of Highest 25%	Average of Lowest 25%	IQR*	Reference IQR §
Sed30-43	1.00	1.00	1.07	0.92	1.16	1.12
Sed30-44	1.00	1.00	1.01	0.99	1.02	1.02
Sed30-45	1.00	1.00	1.01	0.97	1.04	1.03
Sed30-46	0.99	1.00	1.06	0.93	1.14	1.10
Sed30-47	1.00	1.00	1.01	0.98	1.03	1.03
Sed30-48	0.99	0.99	1.06	0.92	1.16	1.12
Sed30-39	1.00	1.00	1.01	0.98	1.03	1.03

* Interquartile Ratio

§ Figures from Table 5.10

The second essential assessment, that is the evaluation of the loss of freeboard due to sedimentation, is carried out using the categorisation of the percentages of Lost Freeboard

(LFb) as shown in Figure 5.20. As in the case of the previous set of simulations, differences between the performance of the scenarios are observed in this case too. Scenarios Sed30-39 (clean half of the sediment depth in the whole canal network) and Sed30-44 (clean all the sediment in the distributary canals) still achieve the best performance with respect to the loss of freeboard respectively. Scenarios Sed30-45 (clean all the sediment in the upstream half of every canal) and Sed30-43 (clean all the sediment in the main canal) are also the worst. In fact a thorough inspection of Figures 5.18 and 5.20 shows that the performances of the two sets of scenarios are very similar.

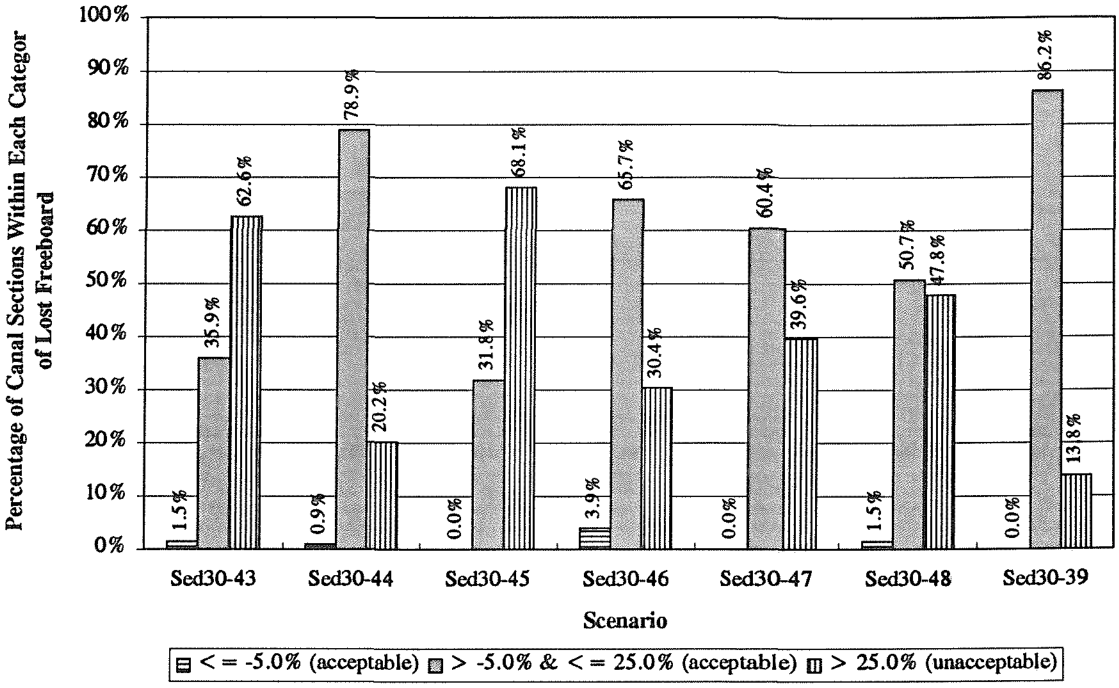

Figure 5.20 Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — automated canals with partial sediment removal and high to normal roughness

And finally, the evaluation of the Sediment Removal Effectiveness, which is necessary in order to take into consideration the differences in the volumes of the sediment removed in the different scenarios, is depicted in Figure 5.21. It shows that scenario Sed30-39 achieved the highest sediment removal effectiveness, followed by Sed30-46 and Sed30-44 which had very similar effectiveness.

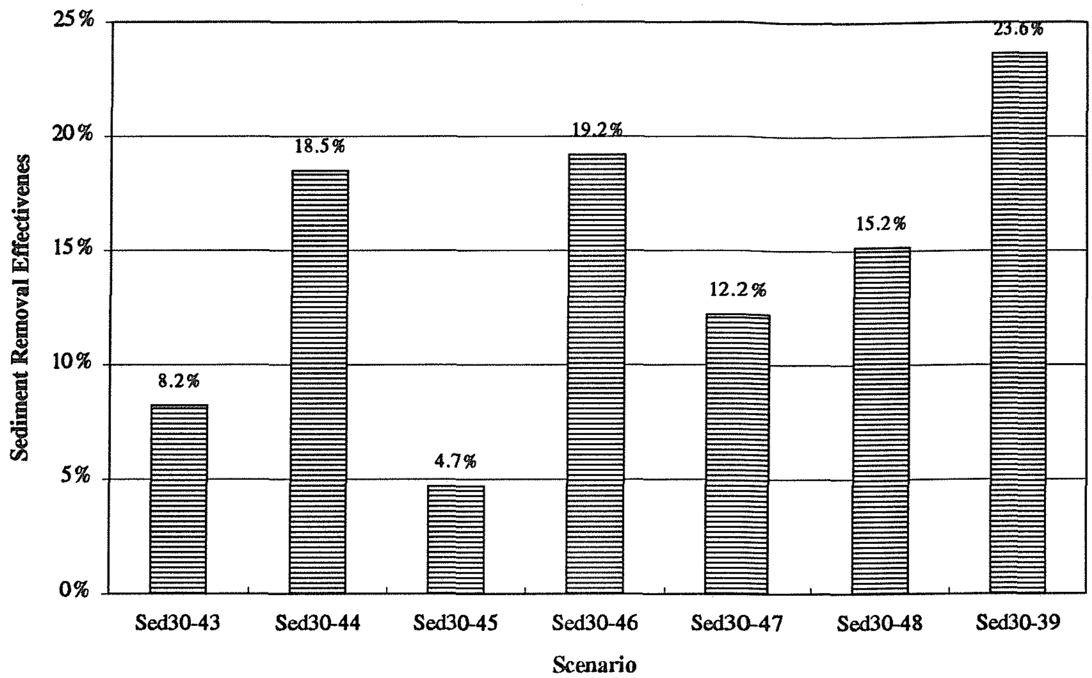

Figure 5.21 Effectiveness of the different scenarios for partial sediment removal from system *A* — automated canals with high to normal roughness

A comparison between all the results from the previous set of simulations (Table 5.10 and Figures 5.18 & 5.19) and these presented in this section (Table 5.12 and Figures 5.20 & 5.21) reveal that the performance of the equivalent scenarios in both sets (e.g. Sed30-33 and Sed30-43) is very similar. If the different scenarios in each set of simulations were ranked from best to worst based on their performance with respect to the adequacy and equity of the supply to the fields, loss of freeboard and sediment removal effectiveness the rankings obtained from both sets will match precisely. For example, scenario Sed30-36 in the previous set will be ranked second best with respect to the Sediment Removal Effectiveness criterion and so will scenario Sed30-46 in the current set based on the same criterion. (It should be noted that both scenarios adopt the same alternative for partial sediment removal, i.e. removing all the sediment in the downstream half of every canal.)

To summarise, the results presented in this section and in the previous one show that the relative performance of the different alternatives of partial sediment removal will be the same whether the roughness of the sedimented sections increases by 10% or 30%. This leads to a very important conclusion. It is apparent that the system of prioritisation of the different alternatives of partial sediment removal will not be affected by the magnitude of change in the roughness of the sedimented sections of the canals. In other words, the change in the roughness of the canals after sedimentation should not be considered as a key factor when prioritising sediment desilting activities.

Roughness of Cleaned Sections (scenarios Sed30-73 to Sed30-79)

The main objective of this set of scenarios is to further test the sensitivity of the roughness coefficients as a factor influencing the prioritisation of partial sediment removal. In the previous two sets of scenarios it was assumed that the removal of sediment from the canals was carried out with high/good standards and therefore the assumption that the roughness of the canal sections from which sediment depositions were removed would drop to design values was made. The high/good maintenance standards are not likely to be achieved in developing countries where many of the maintenance activities are usually carried out by manual labour and sometimes by the farmers themselves (De Veen, 1980). The case of relatively poor maintenance quality is investigated in the current set of simulations by not lowering the roughness coefficients of the canal sections from which sediment is cleaned to design values. The details of the simulations are outlined as follows:

- Introduce the sediment profile corresponding to 30% sedimentation in the canals of the case study (system *A*).

- Remove the sediment from certain parts of the irrigation network of system *A* according to one of the possible alternatives described before. If the alternative indicated that all the sediment in a certain canal section should be removed, the original design dimensions of this section were assumed to be restored after sediment removal.

- Increase the roughness coefficients (Manning *n*) as follows: (i) for canal sections where sediment depositions were not removed the roughness was increased by 30% above design figures, and (ii) for canal sections from which sediment depositions were removed the roughness was increased by 10% above design figures (i.e. the roughness decreased by 20% after sediment removal).

- Simulate the system under full design discharge, allowing the settings of the gates of the canal regulators and field outlets to be adjusted to maintain the design discharges/water levels according to the function of each. These adjustments are required because of the changes in the shapes of the canal sections and hence their hydraulic characteristics due to sedimentation. This procedure allows for the simulation of how the gates will operate if they are automated.

- Repeat the scenario by adopting different sediment removal alternatives (see Table 5.8) and modify the canal sections and roughness coefficients accordingly.

- Assess the performance of the different simulation scenarios to compare between them.

A list of the simulations and the sediment removal alternative adopted in each one is given in Table 5.13.

Simulation Results

The Delivery Performance Ratio (DPR) was used as an indicator for evaluating both the adequacy and equity of water distribution between the fields. The results are summarised in Table 5.14 for the different simulation scenarios. The average and mode values of the Delivery Performance Ratio reflect the adequacy of the supply, while the average DPR of the highest 25%, average DPR of the lowest 25% and interquartile ratios (IQR) reflect the equity of water distribution between the field outlets. The results show that both the adequacy and equity were fairly high in all the scenarios which means that in this respect all the alternatives of sediment removal were reasonably equal in performance.

The evaluation of the loss of freeboard due to sedimentation is carried out using the categorisation of the percentages of Lost Freeboard (LFb) as shown in Figure 5.22. The results in this case are slightly different from those obtained from the previous two sets.

Scenario Sed30-74 (clean all the sediment in the distributary canals) was the best one with respect to the loss of freeboard, followed by Sed30-79 (clean half of the sediment depth in the whole canal network). Scenarios Sed30-75 (clean all the sediment in the upstream half of every canal) and Sed30-73 (clean all the sediment in the main canal) still had the poorest performance.

Table 5.13 Brief description of the scenarios for testing the sensitivity of the roughness of the cleaned sections on the prioritisation of sediment cleaning activities for automated systems

Scenario	Description	Illustration	Volume of Removed Sediment (m^3)
Sed30-73	Clean all the sediment in the main canal only	Figure 5.12	24,428
Sed30-74	Clean all the sediment in the distributary canals only	Figure 5.13	33,753
Sed30-75	Clean all the sediment in the upstream half of every canal only	Figure 5.14	30,963
Sed30-76	Clean all the sediment in the downstream half of every canal only	Figure 5.15	27,218
Sed30-77	Clean all the sediment in all the canals in the upstream half of the system	Figure 5.16	35,229
Sed30-78	Clean all the sediment in all the canals in the downstream half of the system	Figure 5.17	22,952
Sed30-79	Clean half of the sediment depth in the whole network	--	29,091

* Total volume of sediment at 30% sedimentation ratio is 58,181 m^3.

Table 5.14 **Evaluation of the Delivery Performance Ratio (DPR) of the field outlets in system *A* — automated canals with partial sediment removal and high to medium roughness**

Scenario	Average	Mode	Average of Highest 25%	Average of Lowest 25%	IQR*	Reference IQR [§]
Sed30-73	1.00	1.00	1.07	0.92	1.16	1.12
Sed30-74	1.00	1.01	1.01	0.98	1.03	1.02
Sed30-75	1.00	1.00	1.01	0.97	1.04	1.03
Sed30-76	1.00	0.99	1.07	0.93	1.15	1.10
Sed30-77	1.00	1.01	1.01	0.98	1.03	1.03
Sed30-78	0.99	0.99	1.07	0.92	1.16	1.12
Sed30-79	1.00	1.01	1.01	0.98	1.03	1.03

* Interquartile Ratio

§ Figures from Table 5.10

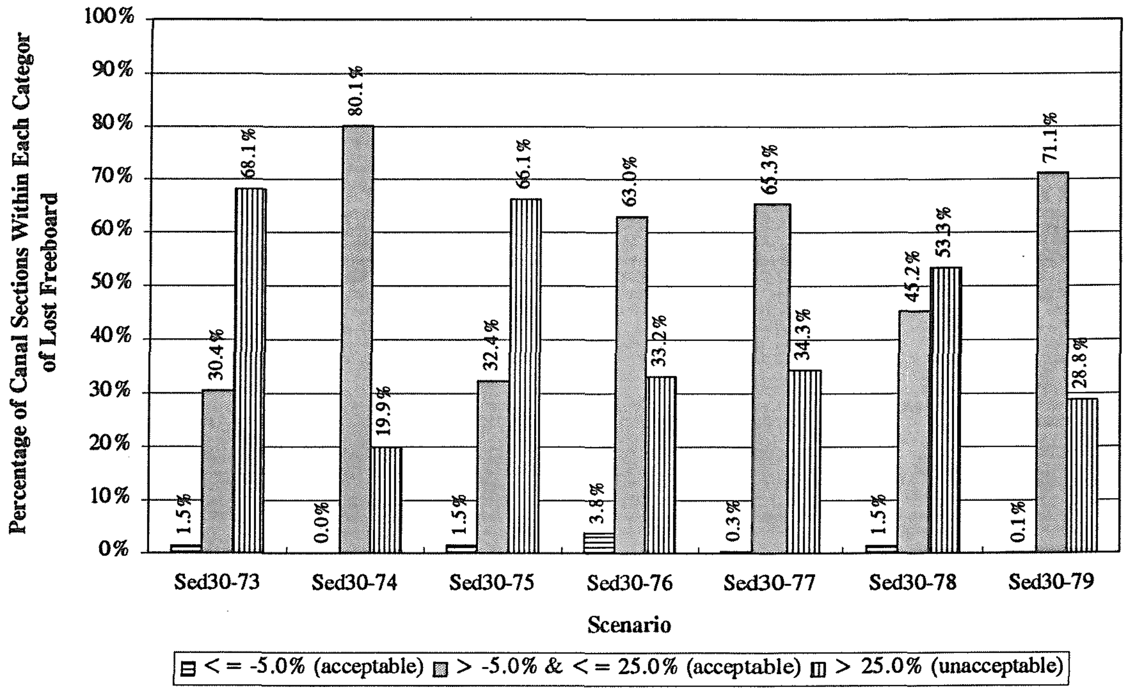

Figure 5.22 **Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — automated canals with partial sediment removal and high to medium roughness**

The relative performance of the different scenarios does not change when using the Sediment Removal Effectiveness as the basis for the evaluation. Figure 5.23 shows that scenario Sed30-74 achieved the highest sediment removal effectiveness, followed by scenario Sed30-79 as second best.

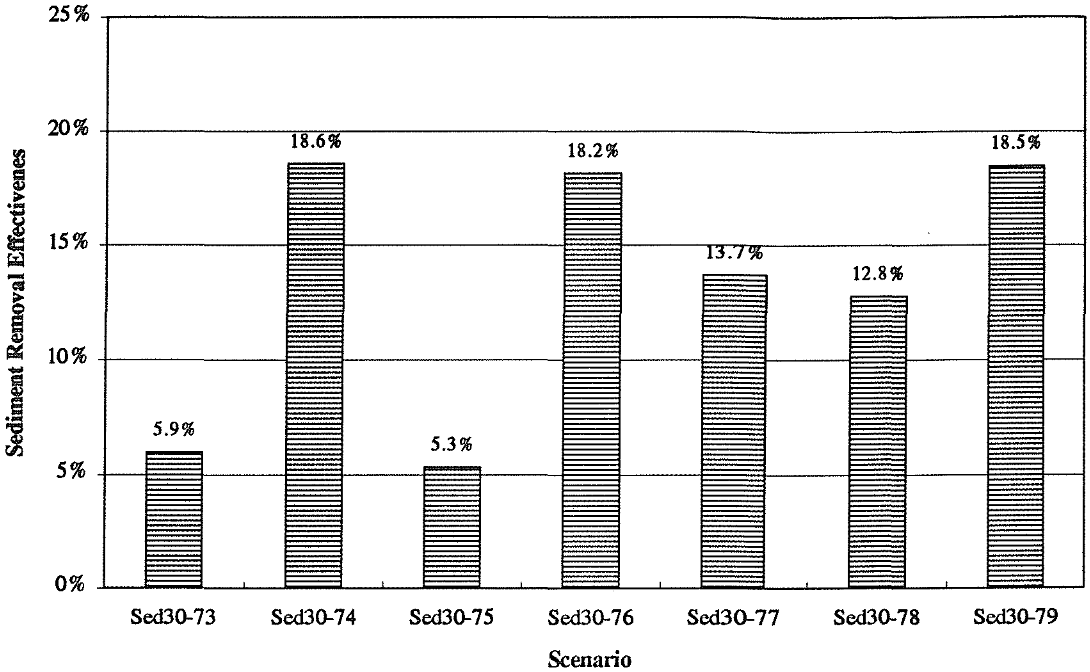

Figure 5.23 Effectiveness of the different scenarios for partial sediment removal from system *A* — automated canals with high to medium roughness

iii. Conclusions

Three sets of hydraulic simulation scenarios for investigating the prioritisation of the partial sediment removal from automated irrigation systems have been presented. The factors taken into consideration in the investigation were the alternatives of sediment removal and the changes in the roughness of the canals after sediment deposition and after removing the sediment. The following conclusions can be drawn from the results of the fore-mentioned scenarios:

- Partial sediment removal is a convenient intervention for tackling the problem of sedimentation in fully automated irrigation systems (down to the gates of the field outlets) where limited financial resources restrict the quantity of the work which can be carried out.

- All the alternatives of partial sediment removal (see Table 5.8) are equally good in achieving high performance with respect to the adequacy of the supply and the equity of water distribution between field outlets.

- Differences between the performance of the alternatives of partial sediment removal with respect to the loss of canal freeboard are, however, observable. Consequently, this performance measure is the key criterion for making comparisons between the efficiencies of the alternatives and choosing the best one.

- Although all the alternatives reduce the loss of design freeboard due to sedimentation, none of them reduces the loss to or below the acceptable limit (the percentage of canal sections which fall within the *unacceptable range* of Lost Freeboard, LFb > 25%, should not be more than 10%). Nevertheless, some scenarios (Sed30-34 and Sed30-30) came very close towards achieving this objective.

- Based on the results obtained from all the simulations the best two alternatives for the partial sediment removal from automated irrigation systems are alternative 7 (remove half of the sediment depth in all the canals) and alternative 2 (remove all the sediment in all the distributary canals only), Table 5.8. The two alternatives achieved more or less consistent performance in all the scenarios simulated. From the practical application point of view, alternative 2 is perhaps the ultimate best one because it is easier to implement cleaning all the sediment in the distributary canals only rather than cleaning half of the sediment depth in all the canals. Additionally, it may be desirable in some situations to minimise the disruption to the operation of main canals, as they might have other functions besides irrigation which require uninterrupted supply all year round. Such need can be met by alternative 2 because it does not require removing the sediment from main canals.

- If the best alternative for partial sediment removal is implemented it will be possible to supply the maximum design discharge without putting the irrigation scheme at significant risk, thus reducing any potential yield loss due to inadequacy of the supply if the maximum design discharge could not be supplied at the times of peak demands.

- Although it is more likely that sedimentation increases the roughness of the canals (ASCE, 1963; Chow, 1959; Ilaco, 1985), some field observations show that the opposite can also be true (Nawazbhutta et al., 1996). This has been dealt with in this research by investigating three sets of simulations with different changes to Manning n in the sedimented canal sections. These sets were presented to show how an *increase* in the roughness due to sedimentation may impact the performance of the partial sediment removal alternatives. However, the same three sets can also be used to investigate the other possibility, when the roughness *decreases*. For example, if the second set of simulations (Sed30-4x) is considered to be the base case instead of the first set, then the first set (Sed30-3x) will be investigating the impact of decreased canal roughness due to sedimentation. Nevertheless, the research has shown that in all cases, the change in the roughness due to sedimentation in system A does not affect the performance of the partial sediment removal alternatives.

- Hydraulic modelling enables quantification of outcomes from identified scenarios to facilitate better decision making.

c. Prioritising Sediment Removal from Manually-operated Systems

The following sections are a continuation of the examination of the impact of the mode of system operation on the prioritisation of sediment removal. In the previous sections the prioritisation of sediment removal from automated systems was investigated. In Section 5.6, a comparison between the impact of sedimentation on the performance of automated and manually-operated systems showed that the adverse impacts on the latter were greater than the impacts on the earlier. The prioritisation of sediment removal activities from manually-operated irrigation systems is therefore as much needed as it is for automated systems.

In order to isolate the impact of the mode of operation alone on the prioritisation of sediment removal the previous sets of simulations for automated systems were repeated with the only difference being a change in the mode of operation from automatic to manual. Each of the new sets is described in more details below.

i. The Base Case (scenarios Sed30-53 to Sed30-59)

The first set of scenarios is analogous to the set described in Table 5.9. It is a case of maintenance being carried out at high standards which restores the condition of cleaned canal sections to near design conditions. The main features of the simulations are as follows:

- Introduce the sediment profile corresponding to 30% sedimentation in the canals of the case study (system *A*).

- Remove the sediment from certain parts of the irrigation network of system *A* according to one of the possible alternatives described before. If the alternative indicated that all the sediment in a certain canal section should be removed, the original design dimensions of this section were assumed to be restored after sediment removal.

- Increase the roughness coefficients (Manning *n*) of the canal sections where there is sediment deposition to allow for the irregularities in the surface, which usually develop in sedimented canal beds, and the roughness of the sediment material itself. Manning *n* was increased by 13% for the main canal and 10% for the distributary canals of system *A*. The original roughness coefficients assumed in the design of the system were used with canal sections from which the sedimentation was cleaned.

- Simulate the system under full design discharge, allowing the settings of the gates of the canal regulators to be adjusted to maintain the design discharges/water levels according to the function of each. These adjustments are required because of the changes in the shapes of the canal sections and hence their hydraulic characteristics due to sedimentation. The gates of the field outlets were maintained fixed at the settings which should deliver the design discharge to each field throughout the whole run. This procedure allows for the simulation of how the system will be manually operated by field staff.

- Repeat the scenario by adopting different sediment removal alternatives (see Table 5.8) and modify the canal sections and roughness coefficients accordingly.

- Assess the performance of the different simulation scenarios to compare between them.

A list of the simulations and the sediment removal alternative adopted in each one is given in Table 5.15.

Simulation Results

Table 5.16 summarises the results of evaluating the adequacy and equity of water distribution for the different simulation scenarios. The Delivery Performance Ratio (DPR) was used as an indicator for evaluating both performance measures. The average and mode values of the Delivery Performance Ratio reflect the adequacy of the supply, while the average DPR of the highest 25%, average DPR of the lowest 25% and interquartile ratios (IQR) reflect the equity of water distribution between the field outlets. Unlike the case of the fully automated system (Table 5.10), neither the adequacy nor the equity were high in all the runs. Scenario Sed30-54 (clean all the sediment in the distributary canals) was ultimately the best with respect to both criteria. Scenario Sed30-56 (clean all the sediment in the downstream half of each canal) was the poorest in terms of the adequacy of the supply (mode of DPR) and scenario Sed30-53 (clean all the sediment in the main canal) was the poorest in terms of the equity of water distribution (IQR).

Table 5.15 Brief description of the scenarios for investigating the prioritisation of sediment cleaning activities for manually-operated systems with medium to normal roughness

Scenario	Description	Illustration	Volume of Removed Sediment (m^3)
Sed30-53	Clean all the sediment in the main canal only	Figure 5.12	24,428
Sed30-54	Clean all the sediment in the distributary canals only	Figure 5.13	33,753
Sed30-55	Clean all the sediment in the upstream half of every canal only	Figure 5.14	30,963
Sed30-56	Clean all the sediment in the downstream half of every canal only	Figure 5.15	27,218
Sed30-57	Clean all the sediment in all the canals in the upstream half of the system	Figure 5.16	35,229
Sed30-58	Clean all the sediment in all the canals in the downstream half of the system	Figure 5.17	22,952
Sed30-59	Clean half of the sediment depth in the whole network	--	29,091

* Total volume of sediment at 30% sedimentation ratio is 58,181 m^3.

The results of assessing the performance of the scenarios with respect to the loss of freeboard are depicted in Figure 5.24. As seen before, some scenarios performed better than others. Scenarios Sed30-59 (clean half of the sediment depth in the whole canal network) and Sed30-54 (clean all the sediment in the distributary canals) achieved the best performance with respect to minimising of the percentages of canal sections which fall within the *unacceptable range* of Lost Freeboard (LFb) respectively. Scenarios Sed30-58 (clean all the sediment in the canal in the downstream half of the system) and Sed30-53 (clean all the sediment in the main canal), on the other hand, were the worst.

Table 5.16 **Evaluation of the Delivery Performance Ratio (DPR) of the field outlets in system *A* — manually-operated canals with partial sediment removal and medium to normal roughness**

Scenario	Average	Mode	Average of Highest 25%	Average of Lowest 25%	IQR*
Sed30-53	1.01	0.92	1.52	0.51	2.98
Sed30-54	1.00	1.01	1.02	0.98	1.04
Sed30-55	1.01	1.06	1.34	0.59	2.27
Sed30-56	1.00	0.84	1.34	0.80	1.68
Sed30-57	1.01	1.00	1.22	0.79	1.54
Sed30-58	1.00	0.92	1.39	0.63	2.23
Sed30-59	1.01	0.98	1.18	0.81	1.47

* Interquartile Ratio

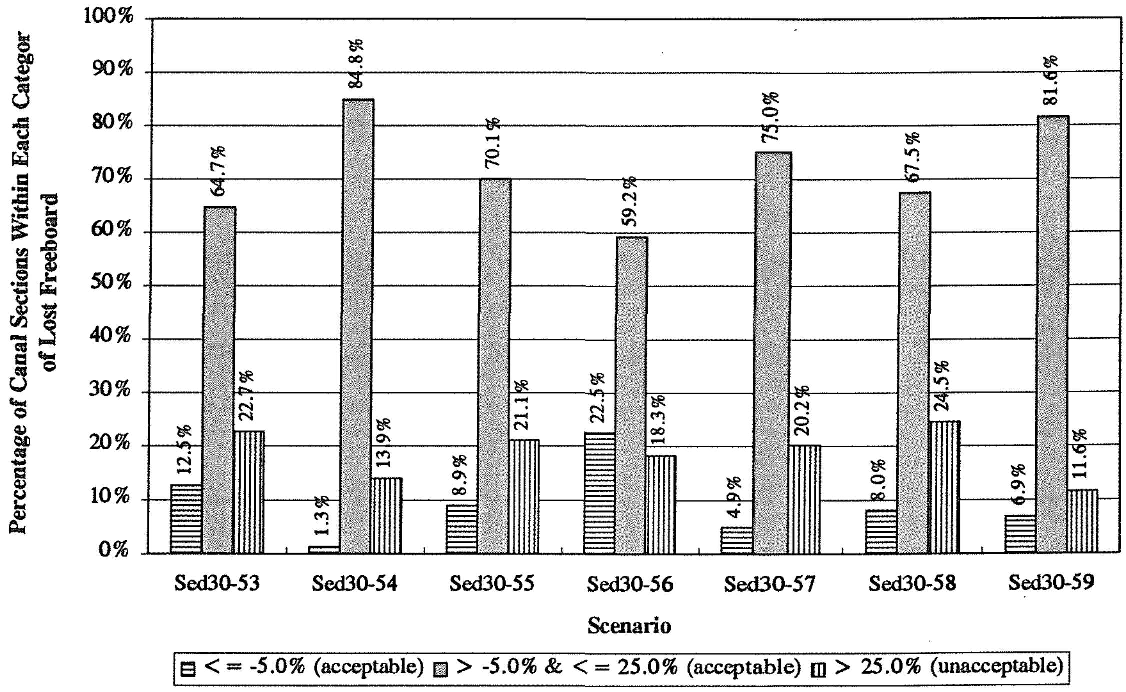

Figure 5.24 **Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — manually-operated canals with partial sediment removal and medium to normal roughness**

To take the differences in the volumes of the sediment removed in the different scenarios into consideration, the Sediment Removal Effectiveness is used as a second indicator for assessing the performance with reference to the loss of design freeboard. The assessment is shown in Figure 5.25 which indicates that when using this criterion scenario Sed30-59 was still the best (had the highest Sediment Removal Effectiveness), but scenario Sed30-56 (clean all the sediment from the downstream half of each canal) came second. On the other hand, scenario Sed30-57 (clean all the sediment from the canals in the upstream half of the system) was poorest. (It should be noticed that Figure 5.25 has the same vertical scale as the similar figures presented in earlier sections such that a visual comparison between the results of the different runs can be made easily.)

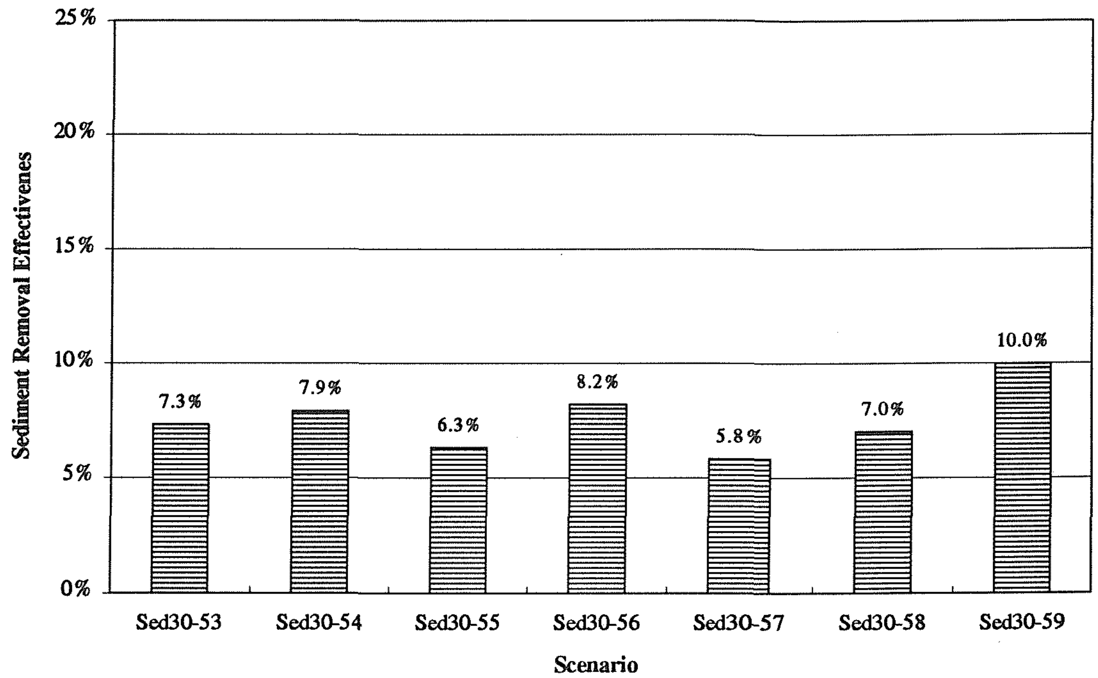

Figure 5.25 Effectiveness of the different scenarios for partial sediment removal from system *A* — manually-operated canals with medium to normal roughness

The Overall Performance

It has been shown earlier that in the case of automated irrigation systems the Lost Freeboard was the only significant criterion in the evaluation of the overall performance of the scenarios. This is because the performance of the scenarios with respect to the other criteria (e.g. the adequacy and equity) was very equal. This is not the case, however, in the current set of simulations with manual operation. Table 5.16 and Figure 5.24 show that the performance varies from a scenario to another with each performance criterion. For instance, scenario Sed30-54 achieved the best performance with respect to the equity of water distribution but was not the best with respect to the loss of freeboard. The performance of scenario Sed30-59, on the other hand, was the opposite. Consequently, the evaluation of the overall performance of the scenarios in the current set cannot be done using one performance criterion only — an overall performance measure is required.

The evaluation of the overall performance based on different criteria has been discussed in Section 3.3.6. The weighted-additive value approach has been used to evaluate the overall performance of the current set of scenarios. Equal weights were given to the adequacy (the mode of the Delivery Performance Ratio), equity (the Interquartile Ratio of DPR) and the Lost Freeboard (the percentage of canal sections within the *unacceptable range* of Lost Freeboard) to produce the results depicted in Figure 5.26 (the calculation details are available in Appendix IV). The overall performance is ranked from 0 to 1, with 1 being the best. The figure shows that scenario Sed30-54 achieved the highest overall performance followed by scenario Sed30-59.

Equal weights were given to all the performance criteria when working out the overall performance because there was no strong argument to emphasize the importance of one or more performance criteria over the others. Nevertheless, it may be argued that in the case of sedimentation the encroachment on the freeboard may be more important than other criteria such as the adequacy or the equity. It was decided, however, to use equal weights in this study to try to produce results that are as generic as possible. The sensitivity of the impact of the weights given to the different criteria on the overall performance of the scenarios should not be difficult to analyse though based on the results presented earlier.

The categorisation of the percentages of Lost Freeboard (LFb) was used as the measure for assessing the encroachment on the freeboard instead of the Sediment Removal Effectiveness in the evaluation of the overall performance because the Sediment Removal Effectiveness of all the scenarios were relatively comparable (see Figure 5.25) and therefore would not have been very effective in differentiating between the scenarios. In addition the categorisation of the percentages of Lost Freeboard is indicative of the actual (hydraulic) performance in the field while the Sediment Removal Effectiveness is biased towards the financial viability of the scenarios because it compares the volumes of removed sediment (cost) to the change in the loss of freeboard (benefits). Although the financial analysis of any alternative is always important, the focus in the current assessment is on the hydraulic performance.

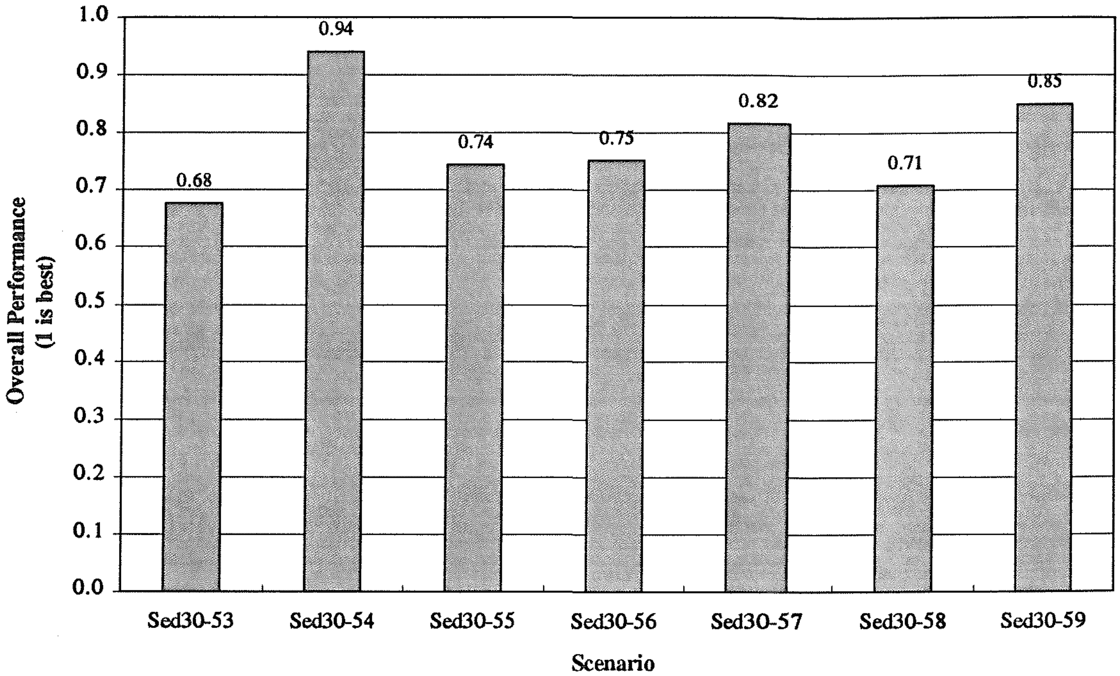

Figure 5.26 Overall performance of the different scenarios for partial sediment removal from system *A* — manually-operated canals with medium to normal roughness

ii. Roughness of Sedimented Sections (scenarios Sed30-63 to Sed30-68)

The specific objective of the current set of simulations is to investigate the sensitivity of the change in the roughness coefficients due to sedimentation on the results obtained from the previous set. A medium increase in Manning n (10 to 13% of design figures) of the canal sections where sediment deposition took place was used in the previous set of simulations. In the current set, Manning *n* of the sedimented canal sections was increased by 30% above design figures for all the canals. The clean canal sections, from which sedimentation was removed, were assumed to have design roughness as was the case in the previous set. The outline of the scenarios is as follows:

- Introduce the sediment profile corresponding to 30% sedimentation in the canals of the case study (system *A*).

- Remove the sediment from certain parts of the irrigation network of system *A* according to one of the possible alternatives described before. If the alternative indicated that all the sediment in a certain canal section should be removed, the original design dimensions of this section were assumed to be restored after sediment removal.

- Increase the roughness coefficients (Manning *n*) of the canal sections where there is sediment deposition to allow for the irregularities in the surface, which usually develop in sedimented canal beds, and the roughness of the sediment material itself. Manning *n* of all the sedimented sections was increased by 30% above design values. The original roughness coefficients defined in the design of the system were used for canal sections which are clean of sedimentation.

- Simulate the system under full design discharge, allowing the settings of the gates of the canal regulators to be adjusted to maintain the design discharges/water levels according to the function of each. These adjustments are required because of the changes in the shapes of the canal sections and hence their hydraulic characteristics due to sedimentation. The gates of the field outlets were maintained fixed at the settings which should deliver the design discharge to each field throughout the whole run. This procedure allows for the simulation of how the system will be manually operated by field staff.

- Repeat the scenario by adopting different sediment removal alternatives (see

Table 5.8) and modify the canal sections and roughness coefficients accordingly.

- Assess the performance of the different simulation scenarios to compare between them.

A list of the simulations and the sediment removal alternative adopted in each one is given in Table 5.17.

Table 5.17 Brief description of the scenarios for investigating the prioritisation of sediment cleaning activities for manually-operated systems with high to normal roughness

Scenario	Description	Illustration	Volume of Removed Sediment (m^3)
Sed30-63	Clean all the sediment in the main canal only	Figure 5.12	24,428
Sed30-64	Clean all the sediment in the distributary canals only	Figure 5.13	33,753
Sed30-65	Clean all the sediment in the upstream half of every canal only	Figure 5.14	30,963
Sed30-66	Clean all the sediment in the downstream half of every canal only	Figure 5.15	27,218
Sed30-67	Clean all the sediment in all the canals in the upstream half of the system	Figure 5.16	35,229
Sed30-68	Clean all the sediment in all the canals in the downstream half of the system	Figure 5.17	22,952
Sed30-59	Clean half of the sediment depth in the whole network	--	29,091

* Total volume of sediment at 30% sedimentation ratio is 58,181 m^3.

The Delivery Performance Ratio (DPR) was used as an indicator for evaluating both the adequacy and equity of water distribution between the fields. The results are summarised in Table 5.18 for the different simulation scenarios. The average and mode values of the Delivery Performance Ratio reflect the adequacy of the supply, while the average DPR of the highest 25%, average DPR of the lowest 25% and interquartile ratios (IQR) reflect the equity of water distribution between the field outlets. The assessment shows that neither the adequacy nor the equity were high in all the scenarios. In particular the performance with respect to the equity of water distribution was poorer than the performance with respect to the adequacy of the supply. Nevertheless, scenario Sed30-64 (clean all the sediment in the distributary canals) was the only scenario to achieve high performance in both criteria.

Table 5.18 Evaluation of the Delivery Performance Ratio (DPR) of the field outlets in system *A* — manually-operated canals with partial sediment removal and high to normal roughness

Scenario	Average	Mode	Average of Highest 25%	Average of Lowest 25%	IQR*	Reference IQR §
Sed30-63	1.00	0.88	1.62	0.42	3.85	2.98
Sed30-64	1.00	0.99	1.03	0.97	1.06	1.04
Sed30-65	1.01	1.05	1.44	0.49	2.91	2.27
Sed30-66	1.00	0.89	1.40	0.75	1.87	1.68
Sed30-67	1.01	1.00	1.28	0.75	1.71	1.54
Sed30-68	0.99	0.88	1.46	0.55	2.64	2.23
Sed30-59	1.01	0.98	1.18	0.81	1.47	1.47

* Interquartile Ratio

§ Figures from Table 5.16

The second essential assessment, that is the evaluation of the loss of freeboard due to sedimentation, is carried out using the categorisation of the percentages of Lost Freeboard (LFb) as shown in Figure 5.27. The relative performance of the scenarios is somehow similar to that from the previous set of simulations. Scenarios Sed30-59 (clean half of the sediment depth in the whole canal network) and Sed30-64 (clean all the sediment in the distributary canals) still achieved the best performance with respect to the loss of freeboard respectively. Scenarios Sed30-65 (clean all the sediment in the upstream half of every canal) and Sed30-63 (clean all the sediment in the main canal) were the worst.

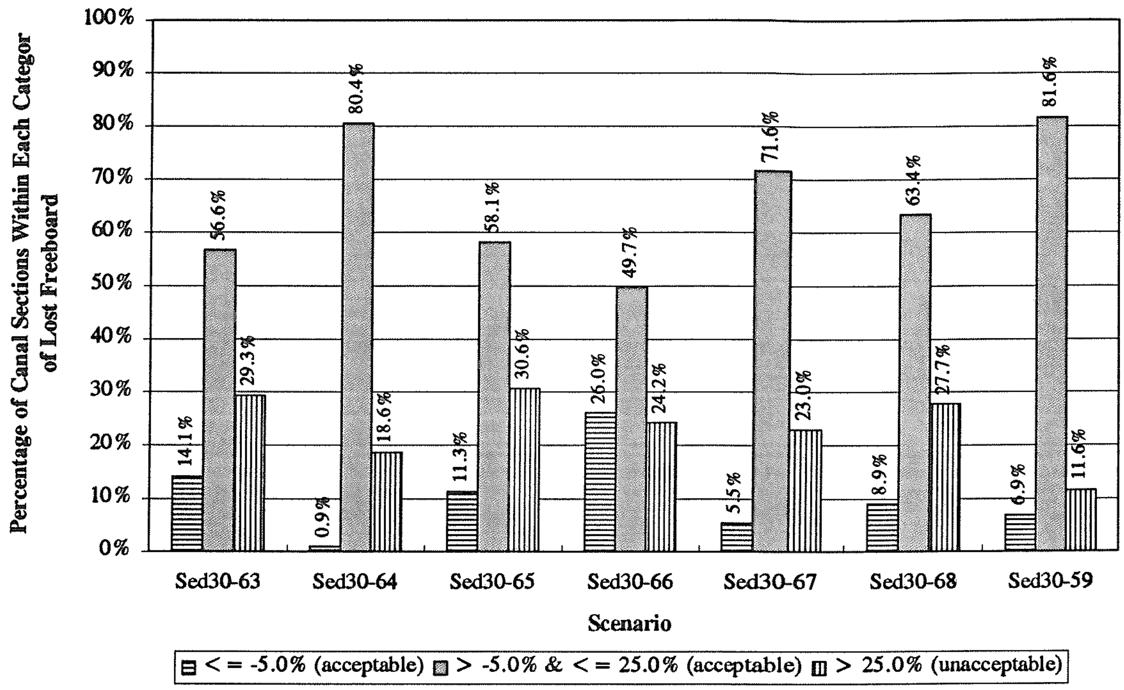

Figure 5.27 Categorisation of the percentages of Lost Freeboard (LFb) of the canals in system *A* — manually-operated canals with partial sediment removal and high to normal roughness

And finally, the evaluation of the Sediment Removal Effectiveness, which is necessary in order to take into consideration the differences in the volumes of the sediment removed in the different scenarios, is depicted in Figure 5.28. It shows that scenario Sed30-59 achieved the highest sediment removal effectiveness, followed by Sed30-64, Sed30-66 and Sed30-68 which had very similar effectiveness.

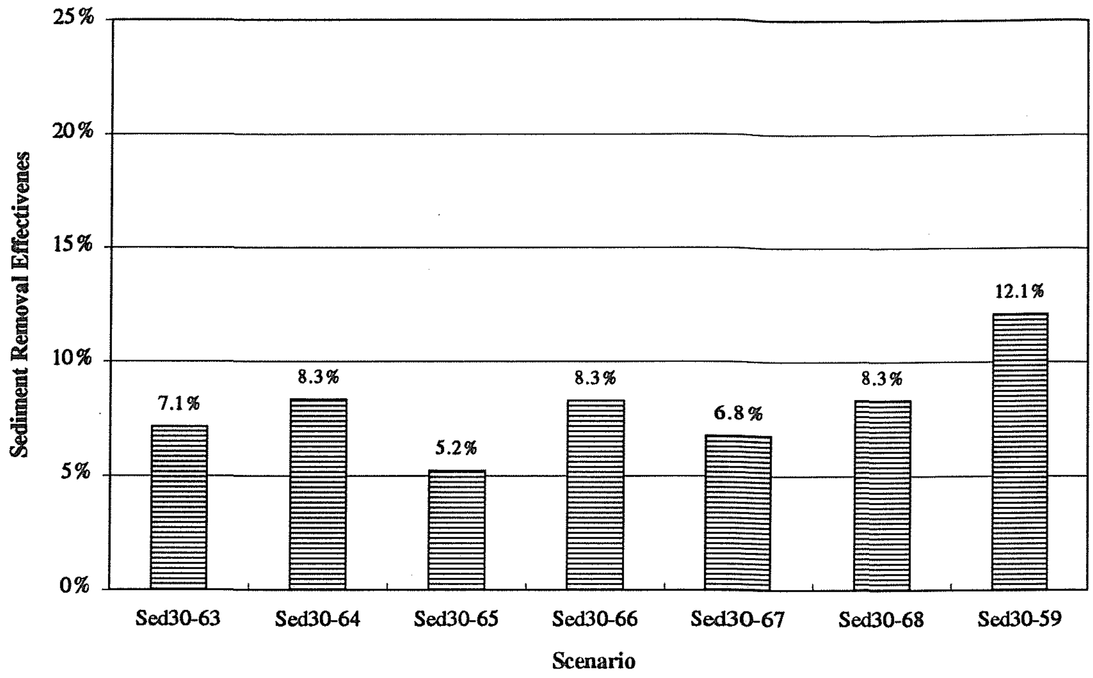

Figure 5.28 Effectiveness of the different scenarios for partial sediment removal from system *A* — manually-operated canals with high to normal roughness

The Overall Performance

Similar to the situation in the previous set of simulations, the overall performance of the scenarios in the current set needed to be worked out. The linear proportional weighting method with equal weights for the performance criteria was used to work out the overall performance. The performance criteria considered were the adequacy (the mode of the Delivery Performance Ratio), the equity (the Interquartile Ratio of DPR) and the Lost Freeboard (the percentage of canal sections within the *unacceptable range* of Lost Freeboard). The results are shown in Figure 5.29. Scenario Sed30-64 achieved the highest overall performance. In fact the ranking of all the scenarios with reference to their overall performance is similar to the ranking obtained from the previous set of simulations. In other words, the change in the roughness of the canals after sedimentation did not affect the prioritisation of the alternatives of partial sediment removal. These results therefore lead to the conclusion that the change in the roughness of the canals due to sedimentation is not a key factor which can affect system performance, regardless to whether the system is manually-operated or fully automated. Hence, it should not be important to study the sensitivity of this factor on most sediment-related analyses such as the prioritisation of some alternatives of partial sediment removal.

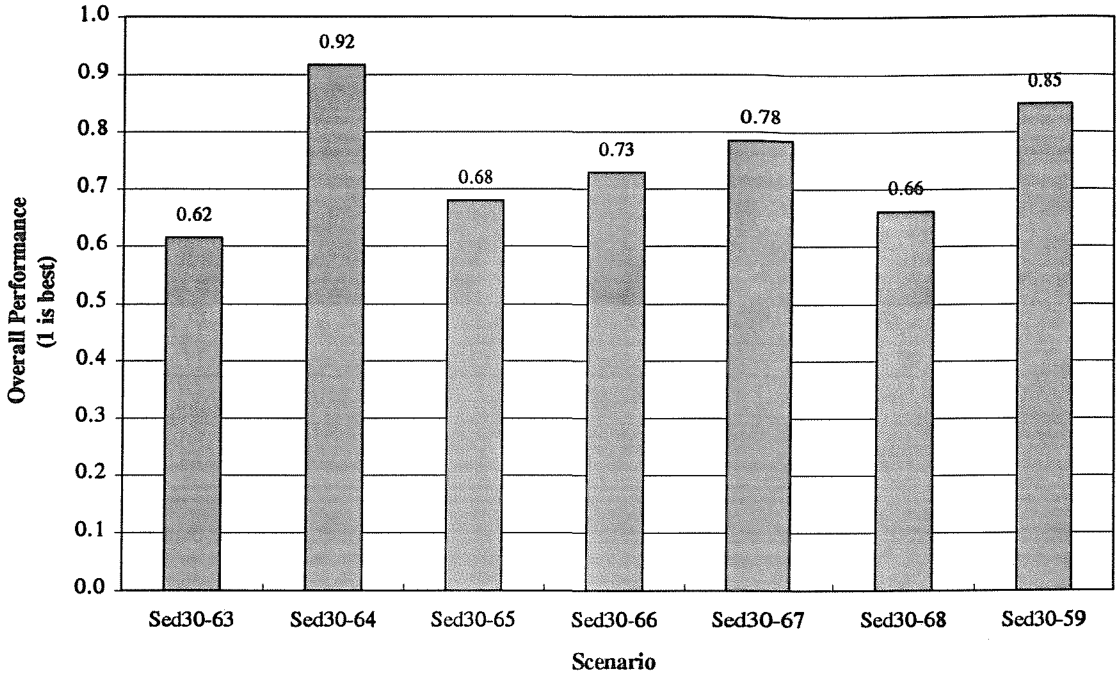

Figure 5.29 Overall performance of the different scenarios for partial sediment removal from system *A* — manually-operated canals with high to normal roughness

5.8 Vegetation in Open Channels

5.8.1 Definition of Weeds

The term weed refers to a plant which is not desired at its place of occurrence. The term is equally applied to land and aquatic plants. In the context of irrigation engineering, the term weeds is used to refer to any seed-bearing plant, fern, or moss which affects the performance of irrigation and drainage systems with respect to their water delivery function (Smout et al., 1997).

5.8.2 Effects of Vegetation in Irrigation Canals

Although the presence of low intensities of aquatic weeds in irrigation and drainage channels can be beneficial to the system because they help stabilise the banks and sides of the channels and provide habitat for aquatic fauna such as fish, the detrimental effects of weeds usually overweight their benefits. The detrimental effects of high intensities of weeds in irrigation and drainage channels are:

1) The discharge capacity of water through a vegetated open channel is to a large extent dependent on the density and height of the vegetation. The existence of vegetation in open channels reduces their capacity, often by 20 to 50%, because of the area occupied by the vegetation and the increase in the roughness of the channels infested (Bakry et al., 1992).

2) Weeds in irrigation channels reduce the flow velocity and thereby increase the deposition of suspended sediments. Consequently, siltation contributes to a reduction in the discharge capacity of channels and storage capacity of channels and reservoirs.

3) Weeds reduce the useful volume of water storage in reservoirs. The free-floating and submerged varieties displace a more-or-less fixed volume of water regardless to the depth of water in the reservoir. The emergent varieties, however, occupy a volume approximately proportional to the depth of water around them. The impact on loss of storage capacity is significant in the case of small reservoirs such as local night-storage reservoirs. In Zimbabwe, for example, the loss of storage capacity for a small reservoir with a mean depth of 0.5 m was found to be between 13 and 30 percent (Smout et al., 1997).

4) The loss of water from open channel irrigation systems due to evaporation is believed to be relatively small except evaporation losses from large surface reservoirs. However, evapotranspiration losses from certain weed varieties, particularly emergent weeds, have been found to exceed evaporative losses from

open water and water consumed by most field crops by 50 to 100% (Kraatz, 1977).

5) Operational problems arising from the interference of plants with pump intakes and gate mechanisms are common in vegetation-infested channels. The accumulation of debris at gates and intake structures can block them impeding flow passage. Fixed-root vegetation cause small tunnels through the lining of the channels damaging the lining material and increasing seepage losses.

   In addition, weeds change the hydraulic characteristics of channels and structures. Frequent calibration of the stage-discharge characteristics of open channels will be required if this method for flow measurement is used. Besides, measuring the flow in infested channels, using current metering for example, in order to establish the stage-discharge curves may prove difficult and inaccurate, depending on the density of the vegetation.

6) Crops irrigated from weeds-infested irrigation channels are very prone to be infested with the weeds as well. Weeds compete with crops for water and nutrients thus reducing crop yields and increasing the cost of crop production because of the added weed-resistance/removal cost.

7) Dense growth of aquatic weeds create or alter habitats which can favour pests and the development and spread of diseases such as schistosomiasis and malaria.

8) Weeds cause adverse environmental impacts when chemicals are used to kill or control them. When they die, they degrade water quality by reducing the dissolved oxygen content and adding odours.

9) The cost of maintenance of irrigation and drainage systems which are infested with vegetation are typically higher.

5.8.3 Interaction Between Sedimentation and Weed Growth

Silt loaded, turbid water hampers the development of aquatic weeds because turbidity reduces sunlight penetration through that water (necessary for weed photosynthesis). Once the silt is deposited it forms a raised, fertile stratum on which weeds flourish. Once established the weeds stimulate siltation as a result of reduced water velocity. The interaction between sedimentation and weed growth can be alternating, so in some situations sedimentation will be stimulating vegetation growth and in others the opposite will be true. In this respect, the interaction of sedimentation and weed growth is of interest. An integrated and well scheduled programme of silt and weed clearance should be developed if water wastage is to be avoided.

5.8.4 Modelling Vegetation in Irrigation Networks

Vegetation and weed problems in irrigation canals are as important as sedimentation problems. The detrimental effects of weeds have been listed in a previous section. They show that besides the strong interaction between sedimentation and weeds, both have the following common negative effects: (1) they reduce the cross-sectional area available to the flow; and (2) increase the roughness of the canals. These effects then in turn reduce the discharge carrying capacities of the canals and raise the water levels above design levels which endanger the safety of the system.

To simulate an irrigation system with sedimentation or vegetation problems using hydraulic modelling, both of the effects mentioned above should be accurately modelled. Nowadays hydraulic modelling packages can readily predict and simulate sediment deposition in irrigation networks and assess its impact on the performance. Those models are however far from being able to model vegetation. This is because although the features of sedimentation and vegetation which should be modelled are generally similar, the details of those features are quite different. In the case of sedimentation most of the changes in the geometry of the canal cross sections are much easier to simulate because they occur along the perimeters of the sections (see Figure 5.3 for an example of how this was simulated in this work). Additionally, if higher accuracy is required from the modelling, actual surveying of the sedimented cross sections can be carried out in the field and then modelled. The situation is much different in the case of vegetation where the changes in the geometry of the cross sections not only take place at the perimeters but also vegetation grows in the middle of the cross sections themselves. Modelling a cross section with vegetation in a hydraulic model will therefore be rather difficult and relatively inaccurate.

This problem of how to reasonably model the flow in a canal with vegetation has been the concern of some recent research. Querner (1997) measured the water velocity in a cross section of a water course with relative weed obstruction (area of weeds divided by the wetted area) of about 50%. The velocity within the area of the cross section obstructed by weeds was found to be less than 10% of the velocity in the unobstructed (weed-clear) area. Consequently, he concluded that the obstructed part has little or no effect on the discharge capacity of the water course and that the flow rate primarily depends on the unobstructed area, the hydraulic radius, and roughness on the edge of the unobstructed part of the water course. Accordingly, the flow rate through a water course with weed obstruction can be worked out as:

$$Q = \frac{1}{n_o} A_o R_o^{2/3} S^{1/2} \tag{5.1}$$

where
- Q = flow rate (m^3/s)
- n_o = Manning roughness coefficient for the unobstructed part ($s/m^{1/3}$)
- A_o = area of the unobstructed part (m^2)
- R_o = hydraulic radius of the unobstructed part (m)
- S = slope of the energy line (m/m)

The assumption made in this solution was that weed growth starts at the sides of the water course and then progresses towards the middle of the course as the infestation increases. It is possible therefore to define the obstructed and the unobstructed parts of the water section. Practically, such distinction in not so clear and is difficult to define. Submerged weeds, which normally grow in the water course itself, can be found in many water courses with shallow water depths. Defining the unobstructed parts of the water sections of such water courses will be relatively difficult.

Based on field measurements in two areas in the Netherlands, Querner (1997) showed that a relationship between the relative hydraulic radius ($R_l = R_o/R$) and the relative weed obstruction exists. Given a canal cross section and a relative weed obstruction, the hydraulic radius R_o for the unobstructed part can be estimated. However, no relationship between the relative roughness (n_o/n) and the relative weed obstruction was shown to exist. Finding the proper numeric value for the roughness coefficient n_o still remains a problem in this solution.

Based on the above discussion, the best approximation that can be made to model a canal with vegetation is to transform the problem to an equivalent sedimentation problem, model it as has been shown earlier in this chapter and then use the results as good estimates for the results of the actual problem. The relative weed obstruction (area of weeds divided by the original wetted area) is analogous to the percentage of sedimentation defined in Section 5.5, so if the relative weed obstruction of the system under investigation can be estimated the system can be modelled with equivalent percentage of sedimentation instead of the vegetation. However, the alternative solution will only take into account the loss of the water area due to the vegetation and will not consider the change in the geometry of the water area. The accuracy of the results cannot therefore be assured.

Because of these difficulties in modelling vegetated irrigation canals and the high likelihood that the results will not be accurate enough, modelling of vegetation in irrigation canals has not been carried out in this work. If a vegetation problem can be approximated by an equivalent sedimentation problem then the results presented in this chapter should be applicable to the vegetation problem as well.

5.9 Summary and Conclusions

- The expenditure on the maintenance of canal networks usually constitutes the largest proportion of the total allocated budget. It is very important that this expenditure be thoroughly rationalised if the budget is to be optimally utilised. When available resources are limited prioritising the expenditure on the maintenance of the canal network, especially desilting of the canals, should take a high priority in the process.

- Excessive sedimentation is perhaps the most common and serious problem affecting the performance of irrigation canals. The main adverse impacts of sedimentation are the reduction of the carrying capacities of the canals and the stimulation of the growth of aquatic weeds on the fertile stratum which develops as sediment deposits on the bed of the canals. Each of these problems then leads to other problems which finally affect the production of irrigation schemes. A methodology for assessing the impact of sedimentation on the hydraulic performance of irrigation systems using hydraulic simulation techniques has been presented in this chapter. The assessment of the financial implications of sedimentation is addressed in Chapter 7.

- Two alternatives for tackling the problem of sedimentation under the restrictions of limited resources are possible:
  (1) to reduce the maximum flow allowed in the canal network in order to maintain safe freeboard; or,
  (2) to remove some of the sediment from some selected canals (selective maintenance).

Methodologies for investigating the hydraulic efficiency of each alternative and making comparisons between them have been presented in this chapter.

The first alternative has no direct cost to the agency running and maintaining the irrigation system because no expenditure is required for any maintenance work. The actual cost of this alternative comes indirectly from the possible reduction in the level of service given to the users (e.g. the agricultural production of the scheme may be reduced if the maximum supply cannot meet the peak demands). The results of the research show that, contrary to expectations, manually-operated systems will be able to operate under slightly higher reduced flows than automated systems.

The second alternative, on the other hand, has a direct cost to the agency managing the system. The alternative tries to maintain the level of service expected by the users. However, in some cases, especially with manually-operated systems, it might cause some reduction to the level of service as good equity of water distribution might not be achieved, thus reductions in the production of the scheme can be anticipated.

This alternative is also analogous to, for example, changing the condition of a structure from condition/performance level 4 to level 2 instead of level 1 as is possible in some asset management planning procedures. Additionally, there is evidence from some real case studies that similar solutions have been investigated in some irrigation schemes (Vander Velde, 1990). Selective maintenance is an intervention that has practical applications and should not be viewed as a paper exercise only.

A financial analysis is required before a final conclusion regarding these two alternatives can be made (see Chapter 7).

- Among the different alternatives for partial sediment removal from irrigation canals which have been investigated in this research, Alternative 2 (remove all the sediment in all the distributary canals only) and Alternative 7 (remove half of the sediment depth in all the canals), Table 5.8, achieved more or less consistent performance in all the scenarios simulated. From the practical-application point of view, Alternative 2 is perhaps the ultimate best one due to the following reasons:

1) It is easier in practice to implement cleaning all the sediment from the distributary canals only rather than cleaning half of the sediment depth in all the canals.

2) It may be desirable in some situations to minimise the disruption to the operation of main canals, as they might have other functions besides irrigation which require uninterrupted supply all year round. Such requirement can be met if Alternative 2 is chosen because it does not require

removing the sediment from main canals.

3) According to sediment transport theories, more sediment tends to deposit in the high-level canals of a system as the concentrations of the sediment in those locations are usually higher because they are closer to the system headworks which is the usual main source of sediment. If only the sediment in high-level canals is to be removed, those canals tend to act like settling basins thus accelerate sediment deposition in those parts. Consequently, more frequent sediment cleaning from high-level canals will be required.

Nevertheless, all the alternatives will only reduce the risk of the loss of freeboard due to sedimentation. Some remaining risk will still have to be taken in such situations when the total required expenditure cannot be secured.

- The results of the simulations presented in this chapter clearly show that removing the sediment from downstream canals (distributaries) should take a higher priority than removing the sediment from upstream canals (main canals) because the former is more effective. Although this recommendation is against the practice that is currently being followed by some system managers and indeed some of the existing maintenance planning tools such as MARLIN (Cornish, 1998), it is possible to simplify the reasoning for this recommendation based on the output from hydraulic modelling.

Irrigation canals are usually flat and long. This makes the head loss in a system one of its key design concerns if the system is to be designed for free irrigation. Control structures, especially gated ones, are therefore usually designed to operate under the smallest head loss possible. This necessitates that the structures operate in a non-modular flow conditions. Consequently any change in the water levels downstream from a structure is likely to be reflected on the upstream water levels. With this is mind, it is easy to explain what happens in the case of sedimentation.

Figure 5.30 shows an example of a case where the sedimentation in the main canal has been removed while that in the secondary canal has not. Because of the sedimentation in the secondary canal, the water levels in the canal will rise. According to the design and hydraulics of the secondary canal head regulator, the regulator may not be able to accommodate the rise in the downstream water levels without raising the upstream water levels in the main canal as well, if the flow diverted to the secondary canal is to be maintained. Raising the water levels in the main canal in this way will probably cause the same encroachment on the freeboard which may also occur if the sediment in the canal is not removed. Consequently, removing the sedimentation from the main canal only may be of very little value in such a case.

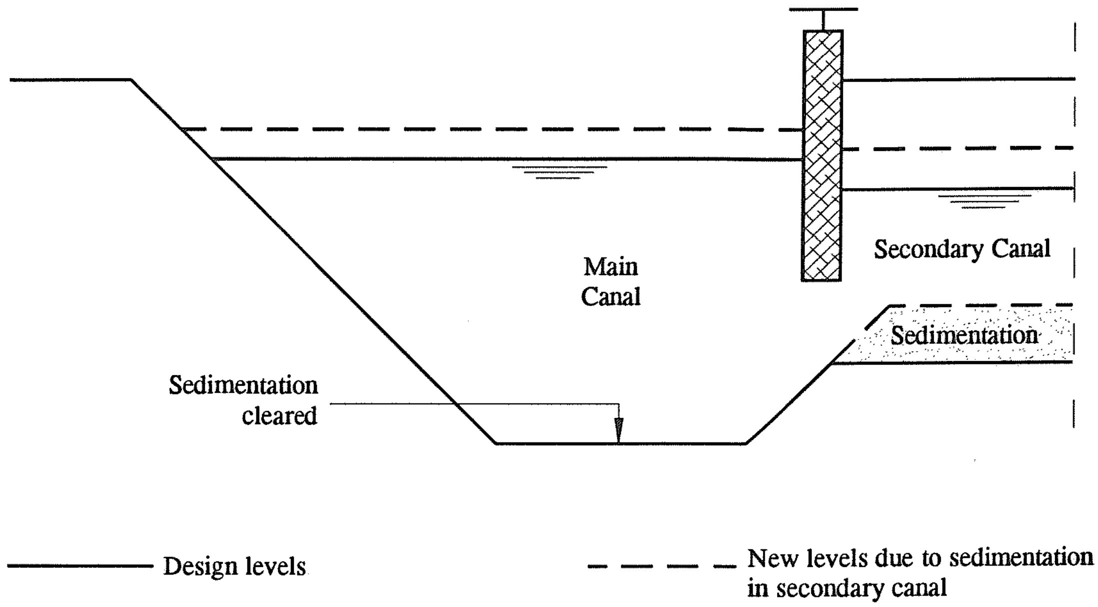

Figure 5.30 Effect of removing the sedimentation from high-order canals only on the water levels in the system

The reason for giving higher-order canals higher priorities over lower-order ones in procedures such as MARLIN originates from the way the priority index of a structure is calculated in such a procedure. MARLIN uses the area served by a structure as one of the criteria for working out its importance. Accordingly, high-level canals will have higher priority indexes than lower ones. While this approach might be appropriate in the absence of more accurate ones, it does not take into consideration the hydraulics of the different components of the system and how they interact together. On the other hand, this chapter showed that hydraulic modelling can be used in the planning and prioritisation of the expenditure on irrigation infrastructure and produces results which have more solid basis and therefore should be more accurate than other procedures. In fact it is possible to integrate the methodology developed in this chapter using hydraulic modelling with a computerised procedure like MARLIN. First, hydraulic modelling can be used to establish *typical* importance indexes for the different components in the system under consideration as has been demonstrated in this chapter. Those indexes can then be used with a maintenance planning procedure such as MARLIN. As the characteristics of the components of the irrigation system change with time the indexes worked out by hydraulic modelling may need updating. However, such integration can on the one hand still reduce the need for the frequent use of hydraulic modelling which still requires relatively special expertise, and on the other overcomes some of the simplifications made in existing asset management procedures.

- The modelling work by Nawazbhutta et al. (1996) concluded that cleaning the sediment from the top two-thirds of Lagar distributary canal was more effective than cleaning the lower parts of the canal (a review of their work is available in Chapter 2). Although their conclusion may seem to be the opposite to what is recommended in this research, that is removing the sediment from downstream canals (distributaries) should take high priority, it is important that the differences between the two studies be understood:

1) Nawazbhutta et al. (1996) investigated the prioritisation of sediment removal from one distributary canal (Lagar canal) only while this research covered the problem from the wider perspective of a whole irrigation system.

2) Lagar canal is relatively shallow (the design water depth varies between 0.64 m to 0.34 m) while the canals in system A have larger water depths (between 1.9 m and 0.75 m). Hence, sedimentation in the top-end of Lagar canal is likely to significantly impede the flow through the canal, which is

not the case in system A.

3) The prediction of the sediment profile in system A showed that most of the sediment deposited in the top-reaches of the canals (e.g. see Figure 5.4). Consequently, in the scenarios which simulated removing the sediment from the distributary canals in this research, most of the sediment was removed from the top-reaches of the canals, which is similar to the scenario recommended by Nawazbhutta et al. (1996).

- It seems that there is an inverse correlation between the impact of sedimentation on the hydraulic performance of irrigation systems and the capital investments made in those systems. The comparison between the performance of the same system under two modes of operation (automatic and manual) as presented in this chapter showed that the performance of the automated system was generally better. A summary of this comparison is given in Table 5.19. (Note that the table uses relative not absolute measures, so for example without the removal of sediment the performance of a fully automated system with respect to the equity of water distribution will be *higher* than the performance of an equivalent manual system. This does not imply that the performance of the automated system will always be high — it will just be higher than that of the manual system.) The table shows that the performance of the automated system was relatively higher than the performance of the manual system with respect to most of the criteria considered except for the loss of freeboard before sediment removal (the reasoning for the change in the behaviour of the two systems with regard to the loss of freeboard has been given in Section 5.6.2). Some conclusions can be drawn from these observations:

1. The partial removal of sedimentation (selective maintenance) from a canal network which is automatically operated can be a practical solution for solving the problem when enough resources are not available for full maintenance. The performance of the system after removing part of the sediment can be acceptable as has been shown in the scenarios presented in this chapter.

Table 5.19 The relative performance of automatic and manual systems with equal sedimentation problems

Condition	Criterion	Automatic System	Manual System
Without sediment removal	Discharge Capacity Ratio (DCR)	Higher	Lower
	Adequacy of supply	Higher	Lower
	Equity of water distribution	Higher	Lower
	Maintaining safe freeboard	Lower	Higher
After partial sediment removal	Adequacy of supply	Higher	Lower
	Equity of water distribution	Higher	Lower
	Maintaining safe freeboard	Lower	Higher

2. The results will not be as satisfactory, however, in a similar manually operated system. Consequently, for a manually operated system complete removal of almost all the sediment from the canal network may be the only possible solution which achieves good improvement in the performance. The expenditures required for the sediment removal tasks of a manually operated system might therefore be higher than the expenditures required for a similar automated system. This can initiate an argument that the large capital investments made in automatic systems might be offset by lower maintenance expenditures, while the relatively smaller capital investments made in manual systems are most likely to be compensated for by higher maintenance expenditures. A thorough financial analysis of some case studies will be required before this argument can be strongly advocated.

- Because the canals of the case study, system *A*, has uniform cross-sections it was reasonable to assume in the scenarios presented in this chapter that after removing the sedimentation from a canal, the cross-sections were restored to their original design dimensions (uniform). In practical terms however, the actual shapes of canal cross-sections may vary considerably from the original design uniform shapes after some years of operation due to sedimentation and other weathering factors. Removing the sediment from the canals does not necessarily mean that design cross-sections should be restored as well. The actual desilting needed to improve the condition and performance of a canal system should be based on performance measures rather than visual appearance or design drawings (Mott MacDonald, 1990). Hydraulic simulation techniques can be a useful tool in evaluating the improvement in performance gained by desilting alternatives, thus enabling a comparison between the effectiveness of each alternative to be made.

- Although system *A* has mainly undershot sluice gates as head and cross regulators on the distributary canals and only one weir cross regulator (overflow-type structure) on the main canal, it is expected that the results and conclusions presented in this chapter will largely be applicable to similar irrigation systems with overflow-type control structures. This suggestion is supported by the results of a study of the impact of the type of control structures on sedimentation in irrigation canals. Mendez (1998) used a hydraulic and sediment model to predict the sediment profile in a 10,000 m long hypothetical canal. Two scenarios were simulated, one with an undershot-type control structure at the tail end of the canal and another with an overflow-type control structure in place of the undershot one. The results showed that under the same initial hydraulic and sediment conditions, the volumes of sediment deposited in the first 9,600 m (96%) of the canal were very similar in both scenarios. Larger differences occurred nearby the locations of the structures (in the remaining 400 m of the canal) because of the difference in the capabilities of the two types of structures in transporting sediment bed load.

Nevertheless, the hydraulic performance of two similar irrigation systems with different types of control structures may be expected to vary under similar sediment conditions. Being usually movable, undershot-type structures can be more flexible than overflow-type structures, which are usually fixed. A change in the design dimensions of a canal is likely to have a direct impact on the water levels in the system when the control structures are fixed and cannot be adjusted to regulate the water levels/flow to cater for the new situation. Movable structures, on the other hand, will have some flexibility and might be able to cater for the changes in the canal dimensions within some limits.

6. Expenditure on Canal Regulators

6.1 General

The development of a methodology for planning and prioritising the expenditure on irrigation infrastructure has been presented in the previous chapter. The methodology was developed through the investigation of the problem of sedimentation in irrigation canals and the various alternatives for curing it.

As a continuation of the development and application of the methodology to other types of irrigation infrastructure, the current chapter focuses on water control structures. Irrigation control structures are crucial components in irrigation networks for achieving effective water control. The most common and therefore important component of an irrigation network after the canals is the regulators. They are either head regulators for controlling the flow or cross regulators for controlling the water levels. Due to their functionality, head regulators are almost always gated structures. Cross regulators may be gated or may not be. The focus in this chapter is on studying the impact of gated canal regulators on hydraulic performance and establishing the linkage between their condition and performance.

6.2 Loss of Control of Gated Structures

The loss of control of gated structures refers to the malfunctioning of one or more of the gates of a gated structure. A gate can be slightly damaged due to rust or some broken parts such that it stays jammed in one position and cannot be opened or closed. The damage can sometimes be severe to the extent that a gate gets removed but not replaced with another one due to lack of maintenance funds[8]. With average lives of one to five years for wooden gates, 10 to 15 years for cast iron gates and 50 to 100 years for the masonry and concrete parts of the structures; the gates are likely to need more frequent maintenance than the

8 This observation was made personally during a field study in Sri Lanka, 1998.

concrete parts. Brown (1989) reported that the main problems found in the water control structures of a three-year-old irrigation system in Africa were damaged gate wheels and stiff action during the operation of the structures.

The objective of the following sections is to investigate the problem of the loss of control of gated structures. The focus is on canal head and cross regulators as the main types of structures which affect operation and hence performance. The investigation covers the assessment of the impact on system performance and establishing a system of prioritisation of the expenditure on the repair work based on comparisons between the performance of different alternatives. Irrigation system *A* (Appendix III) was used as a case study for simulating all the scenarios presented in this chapter.

6.2.1 Head Regulators

The first type of structures to be investigated is canal head regulators. Irrigation system *A* has six gated head regulators at the intakes of its six distributary canals (see Figure 5.1). According to the layout of the system and the locations of the distributary canals along the main canal, they can be divided into two groups: top-end canals (M1C1, M1C4 & M1C6) and tail-end canals (M2C1, M2C2 & M2C4). Consequently, in order to study the difference between the loss of control of a top-end head regulator and a tail-end one, it is sufficient to simulate the loss of control of the head regulator of any one of the three top-end distributaries and any one of the three tail-end distributaries respectively. The head regulators of distributary canals M1C1 and M2C4 were chosen for investigation. In order to follow the same procedure adopted in investigating expenditure on canals (Chapter 5), different sets of simulations were used for studying the effects of the mode of system operation (manual vs automatic).

a. Manual Operation

The details of the simulation scenarios for investigating the impacts of malfunctioning canal head regulators on the performance of a manually-operated system can be outlined as follows:

1) Simulate the operation of irrigation system *A* during a whole growing year using a ranked supply pattern such as that shown in Figure III.3. The adaptation of this supply pattern in the current simulations is depicted in Figure 6.1. The supply is reduced in gradual steps. Each reduction in the flow is completed over 24 hours, after which the supply is maintained constant for 48 or 72 hours to allow the model to reach steady state before the next change in the supply takes place.

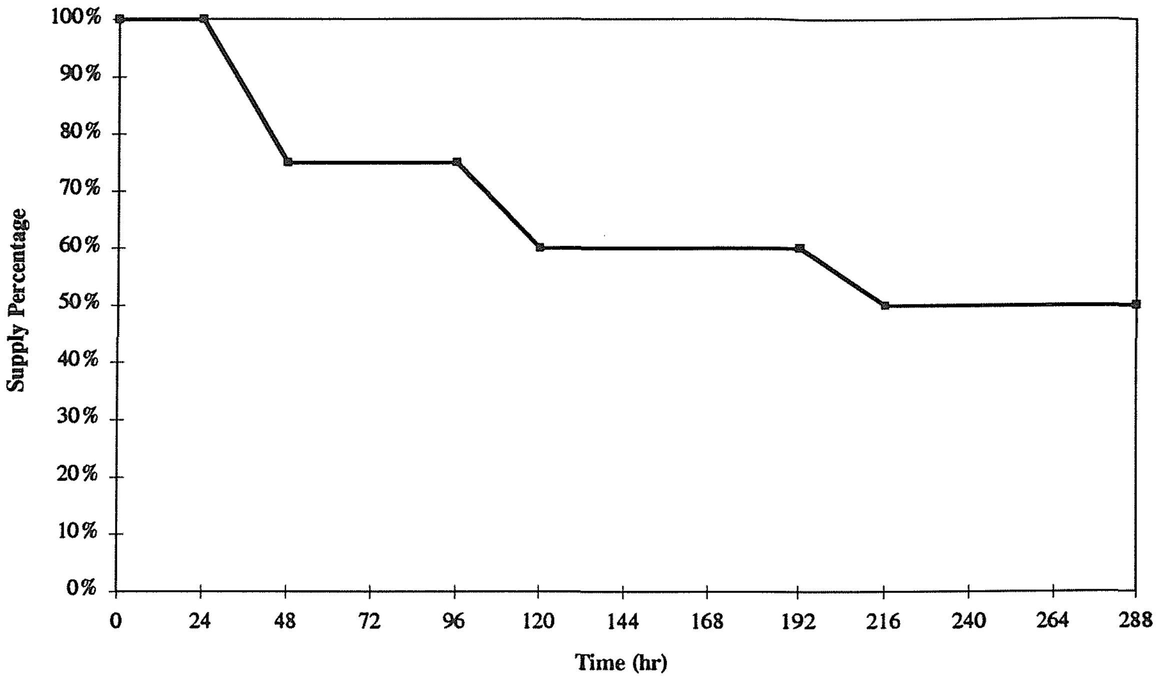

Figure 6.1 **The ranked pattern of the supply entering system *A* for simulating the operation of a whole year**

2) Allow the settings of the canal regulator gates to be adjusted along the run according to the change in the supply in order to maintain target discharges/water levels according to the function of each regulator. However, the settings of the gates of the malfunctioning head regulator are to be fixed at the full design discharge settings throughout the whole run. This procedure simulates a case where the gates of a canal head regulator are jammed fully open at the setting which should allow the maximum design flow to be withdrawn by that canal and hence the canal will withdraw more flow when the supply into the whole system is reduced.

To simulate the operation of a manual irrigation system, the gates of the field outlets are to be maintained fixed at full design discharge settings throughout the whole run. (It is common in manually operated irrigation systems that the gates of field outlets are not operated by irrigation staff and are left to the farmers to operate. They tend to leave them fully open at all times to divert as much water as is possible to their fields.)

3) Simulate other scenarios by changing the malfunctioning head regulator in each one.

4) Assess the performance of the different simulation scenarios to compare between them.

Three scenarios have been simulated, the specific features of each are summarised in Table 6.1.

Table 6.1 Brief description of the scenarios for investigating the loss of control of canal head regulators in manually operated systems

Scenario	Features
Str01m	Control scenario: the gates of all canal regulators function properly
Str03m	The gate of the head regulator of distributary canal M1C1 (top-end) is kept open at the full supply setting throughout the whole run (faulty gate)
Str04m	The gate of the head regulator of distributary canal M2C4 (tail-end) is kept open at the full supply setting throughout the whole run (faulty gate)

i. Simulation Results

In order to assess the overall performance of each scenario during a whole season, it is sufficient in the case of system *A* to evaluate the performance in four situations only, when the supply is 100%, 75%, 60% and 50% of the full design supply (Figure 6.2). Consequently, according to Figure 6.1, the performance should be assessed at 24, 96, 192 and 288 hr which correspond to the previously mentioned four supply categories respectively. Finally, the links between the output from the hydraulic simulations and the different periods of the growing season are given in Table 6.2.

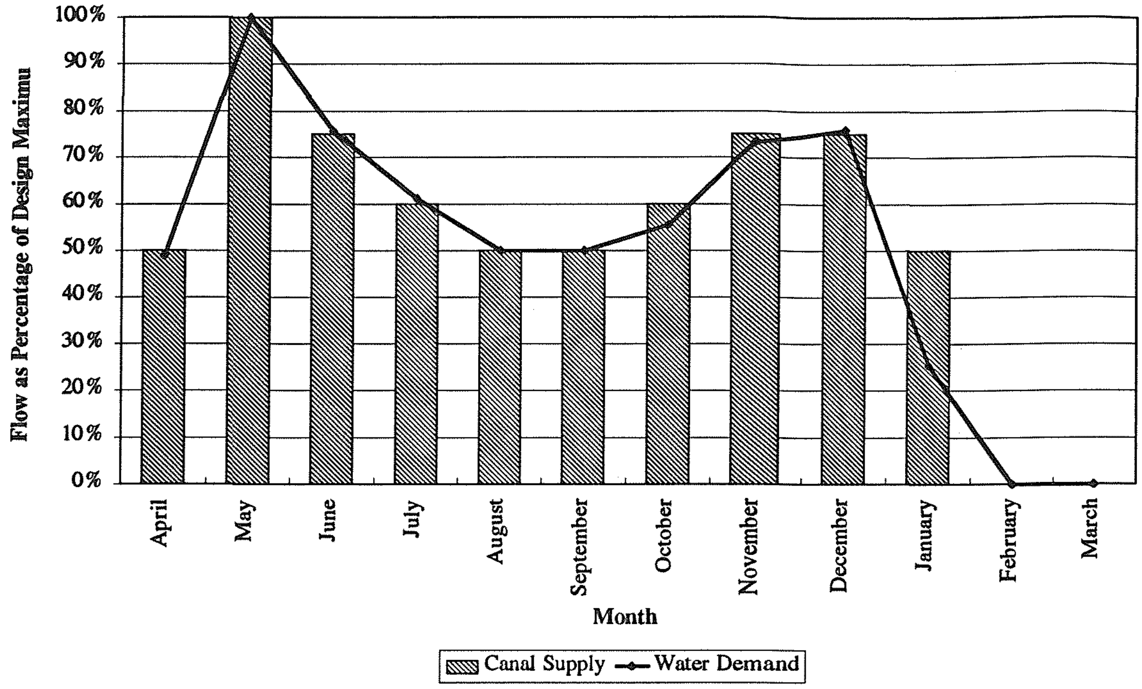

Figure 6.2 **The actual patterns of the water demand and canal supply for a typical year in irrigation scheme *A***

Table 6.2 **The linkage between the output from hydraulic modelling and the different months of the year**

Month	Corresponding Output Time	Month	Corresponding Output Time
April	288 hr	October	192 hr
May	24 hr	November	96 hr
June	96 hr	December	96 hr
July	192 hr	January	288 hr
August	288 hr	February	--
September	288 hr	March	--

Since the main function of head regulators is to control the discharge passing through them, it was expected that only those performance measures which reflect the distribution of water would be important to assess in the current scenarios. Performance measures such as the *adequacy* of the supply, the *equity* of water distribution and the *potential yield* of the scheme were therefore very important to assess. On the other hand, measures such as *command*, *freeboard* and the *variability* of the flow were not as important since in this case none of the scenarios could affect them directly and consequently they were not assessed.

The Delivery Performance Ratios (DPR) of the flow diverted to the field outlets were used to assess the adequacy of the supply. The equity of water distribution was measured by means of the interquartile ratio (IQR) of DPR. Finally, the potential yield of the scheme was worked out based on the functions defining crop yield response to water (Doorenbos and Kassam, 1979). The steps of these calculations can be summarised as follows:

- Obtain the actual flow diverted to each field outlet in the irrigation system during a whole season from the output of the hydraulic simulations.

- Using the actual flow diverted to each field outlet and the calculated crop water requirements, the potential yield of each crop grown in the area served by each outlet can be estimated based on the guidelines of Doorenbos and Kassam (1979).

- By summing up the yield of each crop from each field outlet, the total yield of that crop from the whole scheme can be determined.

With 76 field outlets and four main crops grown in scheme *A* these calculations were quite demanding and therefore could not be done manually. A special piece of software was developed to abstract the necessary output from the results of the hydraulic simulations and feed them into a spreadsheet which calculated the estimations of the crop yields. This software set is described in Appendix I.

The results are shown in Figures 6.3 to 6.5 respectively. An overall look at the results shows that the performance of the three scenarios is very similar and almost linear. The adequacy and equity deteriorated rapidly at low supply ratios in all three cases. The performance of scenario Str03m (malfunctioning of the head regulator of canal M1C1) was slightly poorer than the performance of the other two scenarios. Nevertheless, the performance of the control scenario itself (Str01m) was also poor. A brief description of what happens in each scenario can explain further.

First, the control scenario (Str01m): Although all the gates of the canal regulators were operable in this scenario, the gates of the field outlets were fixed at the full discharge settings to simulate a manually operated system. Consequently, at 100% supply percentage both the adequacy and equity were high (Figures 6.3 & 6.4). When the supply was reduced, the head regulators of the distributary canals were adjusted such that each canal withdrew its fair share of water. However, because the gates of the field outlets were not adjusted as well, the outlets at the top end of each distributary canal withdrew more water than they should; hence leaving the outlets at the tail end short of water and introducing high inequity of water distribution along each canal. Consequently, crop yields were low at certain locations in the system, reducing the overall average yields to between 60% to 75% (Figure 6.5).

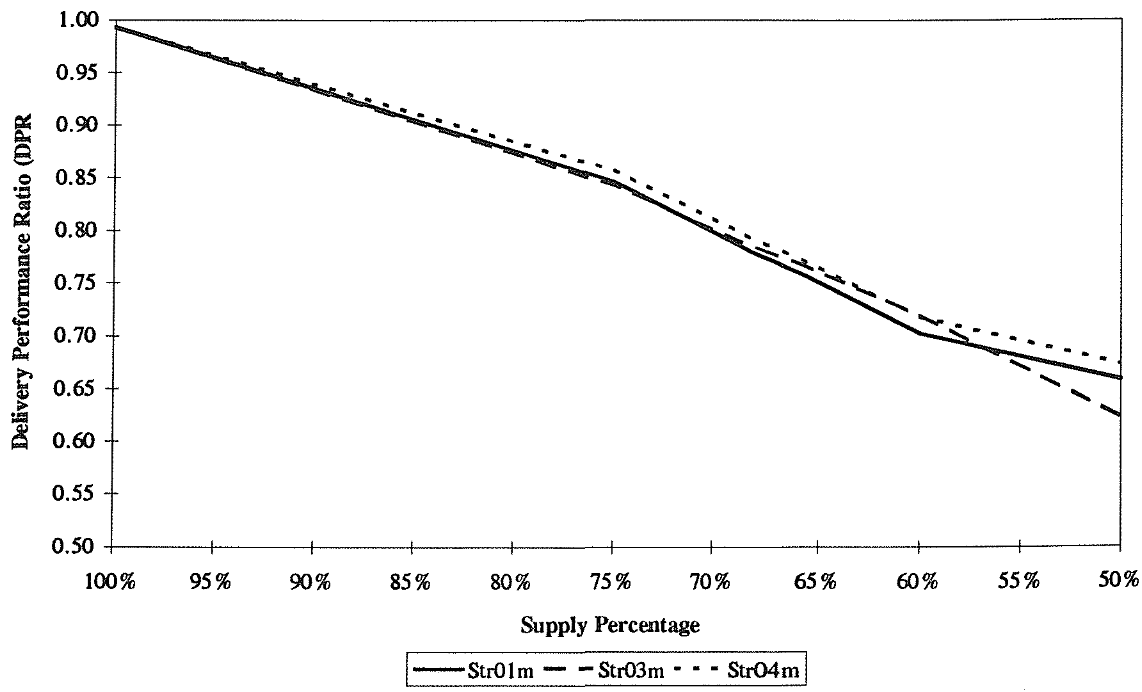

Figure 6.3 **The average adequacy of the supply in system *A* under manual operation in the case of malfunctioning canal head regulators**

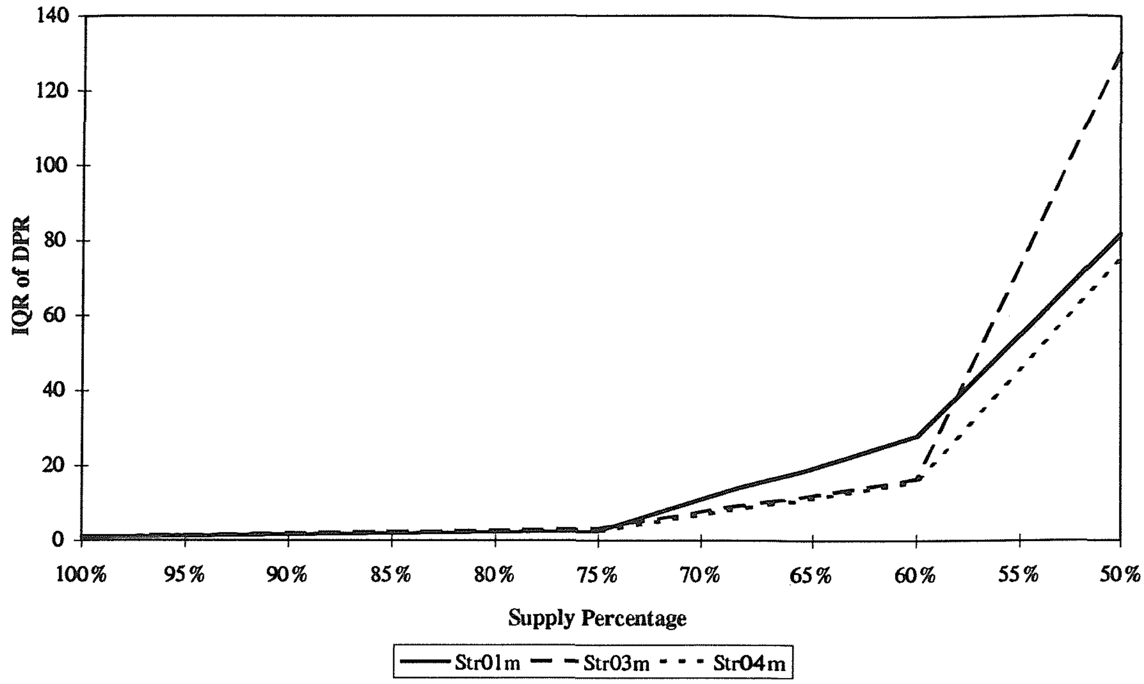

Figure 6.4 The average equity of water distribution in system *A* under manual operation in the case of malfunctioning canal head regulators

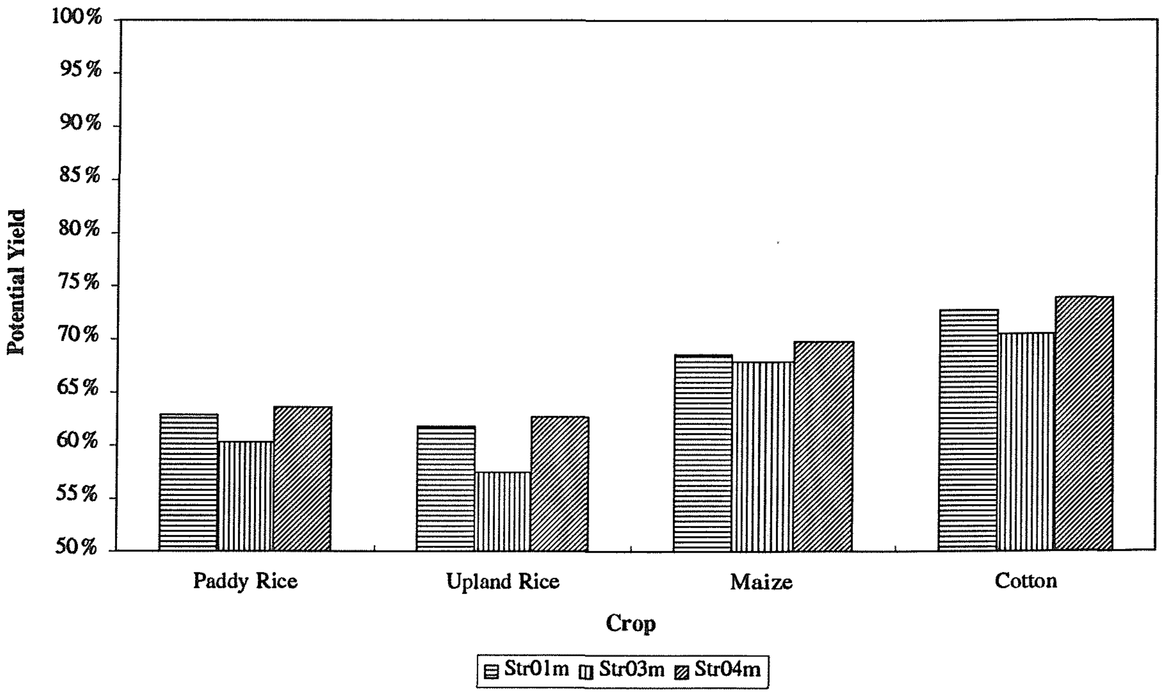

Figure 6.5 The potential yields of the main crops in scheme *A* under manual operation in the case of malfunctioning canal head regulators

Second, scenario Str03m in which the head regulator of canal M1C1 was inoperable. Again, when the flow into the system was reduced, the water distribution was neither adequate nor equitable because of the fixed settings of the field outlet gates. In addition, because the head regulator of the top-end canal M1C1 could not be adjusted as well, the inequity of water distribution between the distributary canals increased, lowering the performance even further. For example, Figure 6.4 shows that at 50% supply ratio the IQR increased from 82 in scenario Str01m to 130[9] in scenario Str03m.

Finally, scenario Str04m in which the head regulator of canal M2C4 was malfunctioning. The results show that the performance of this scenario is slightly better than that of the control scenario Str01m. As has been explained above, the water distribution in scenario Str01m was not equitable because of the upstream outlets abstracting more water than they should. In this scenario, because the head regulator of canal M2C4, located at the tail end of the system, was fully open all the time; the group of canals at the tail end of the system had the chance to withdraw more water, thus reducing the large surplus of water which was available to the canals at the top end of the system in scenario Str01m. Consequently, the water distribution in scenario Str04m was slightly more equitable than the distribution in scenarios Str01m and Str03m. In other words, the malfunctioning of a head regulator located at the tail-end of the system did not have adverse impacts on the performance of the system, unlike the case of the malfunctioning of a head regulator located at the top-end of the system.

The impact of all these imperfections in water distribution led to the loss of crop yields as estimated in Figure 6.5. It must be born in mind that when calculating the potential yield of the typical crops grown in the scheme, the quantities of water which exceeded the demand of any field outlet were considered as waste and that this water surplus could not be beneficially used by increasing the cropping intensity for example. Consequently, according to this method of calculation the maximum yield of a field outlet is 100% of the

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9 These very high IQR figures are due to the very low DPR figures of some field outlets (as low as 0.02) which when are divided by, result in numerically large figures.

potential, no matter how excessive the supply the outlet gets.

Figure 6.5 further proves that neither of the three scenarios could achieve the full potential yield (100%), and that there is a notable reduction in the total yield of scheme *A* in scenario Str03m (malfunctioning of a top-end canal head regulator). On the other hand, the agricultural production of scheme *A* in scenario Str04m is slightly higher than the productions in the other two scenarios for the same reasons explained earlier.

b. Automatic Operation

The three simulation scenarios presented above were repeated again with the main difference being the simulation of an automated system instead of a manual one. The primary objective of the current scenarios is therefore to investigate whether system automation can have additional impact on the performance of a system where some of its control structures are inoperable. The details of the new simulations can be outlined as follows:

1) Simulate the operation of irrigation system *A* during a whole growing year. The pattern of the water supply during a typical year is depicted in Figure 6.1.

2) Allow the settings of the gates of the canal regulators to be adjusted along the run according to the change in the supply in order to maintain target discharges/water levels according to the function of each regulator. However, the settings of the gates of the malfunctioning head regulator are to be fixed at the full design discharge settings throughout the whole run.

   To simulate the operation of a fully automated irrigation system, the gates of the field outlets should also be adjusted along the run according to the change in the supply in order to maintain target deliveries to the outlets.

3) Simulate other scenarios by changing the malfunctioning head regulator in each one.

4) Assess the performance of the different simulation scenarios to compare between them.

Three scenarios have been simulated, the specific features of each are summarised in Table 6.3.

Table 6.3 Brief description of the scenarios for investigating the loss of control of canal head regulators in automated systems

Scenario	Features
Str01a	Control scenario: the gates of all canal regulators function properly
Str03a	The gate of the head regulator of distributary canal M1C1 (top-end) is kept open at the full supply setting throughout the whole run (faulty gate)
Str04a	The gate of the head regulator of distributary canal M2C4 (tail-end) is kept open at the full supply setting throughout the whole run (faulty gate)

i. Simulation Results

The results of assessing the adequacy, equity and the total yield of the three scenarios are depicted in Figures 6.6 to 6.8 respectively (it should be noticed that Figures 6.6 & 6.7 have the same scales as Figures 6.3 & 6.4 respectively such that a visual comparison between the results of the two cases can be made easily). The figures clearly show that the overall performance of the automated system in the three scenarios is much better than the performance of the manual system. The performance of the control scenario (Str01a) is practically perfect. As was the case with the manual system, scenario Str03a (malfunctioning of a top-end canal head regulator) has the poorest performance. Another observation is that the performance of scenario Str04a (malfunctioning of a tail-end canal head regulator) is not so close to the performance of the control scenario (Str01a) as was the case in the manual system, but is still higher than the performance of scenario Str03a.

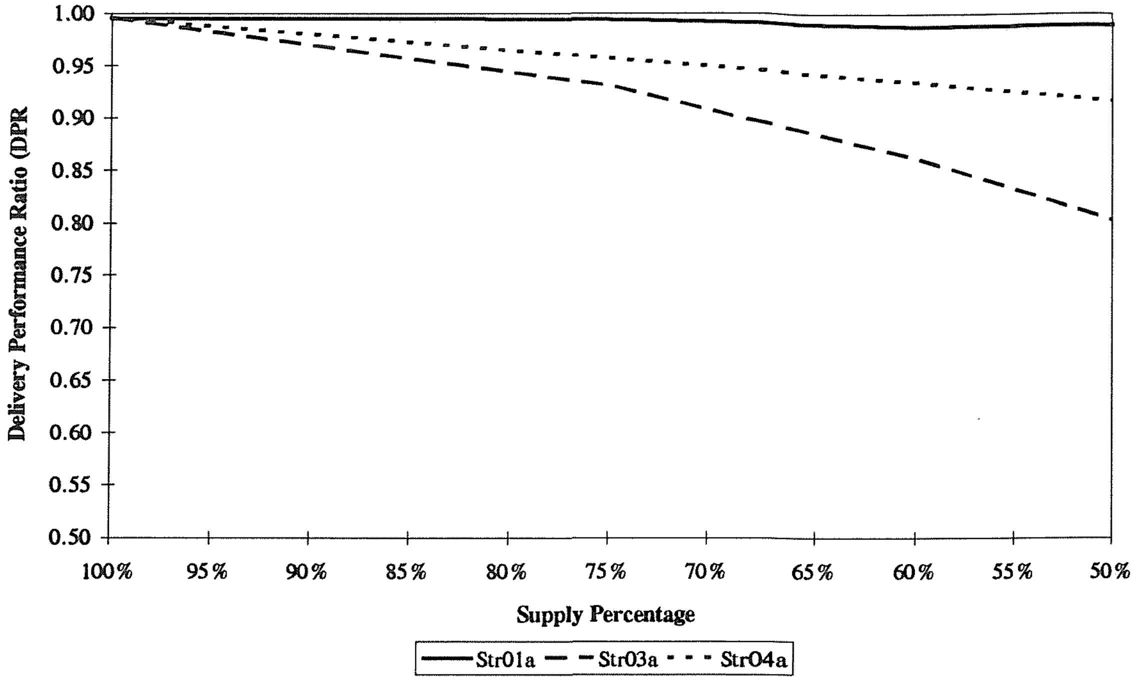

Figure 6.6 **The average adequacy of the supply in system *A* under automatic operation in the case of malfunctioning canal head regulators**

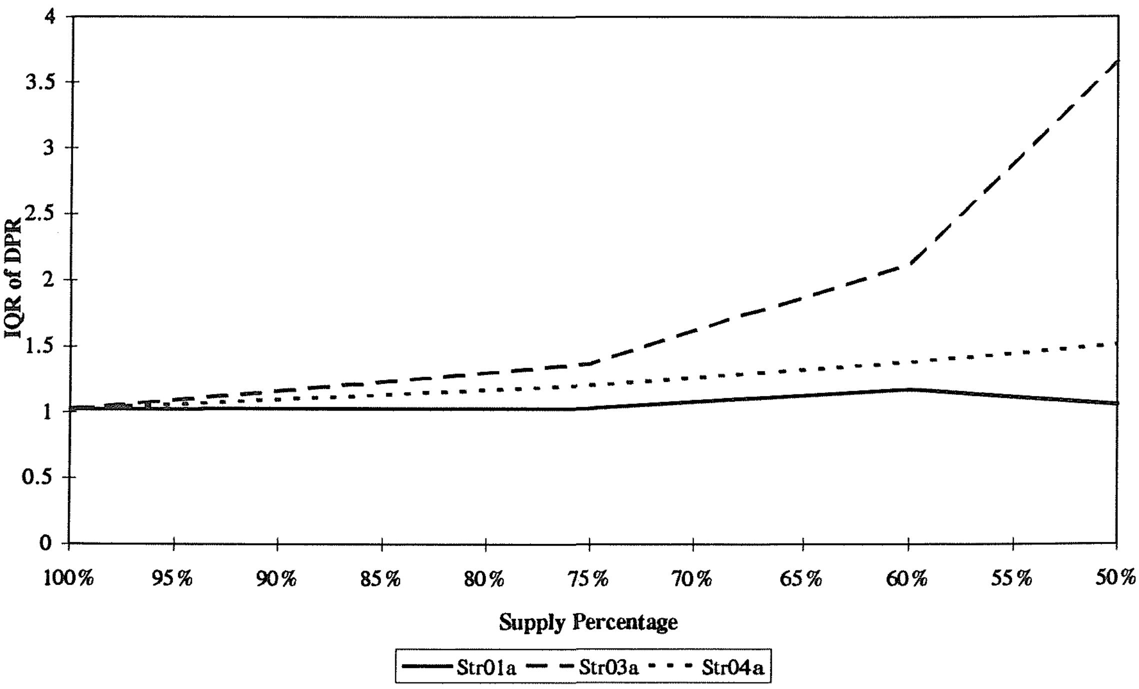

Figure 6.7 **The average equity of water distribution in system *A* under automatic operation in the case of malfunctioning canal head regulators**

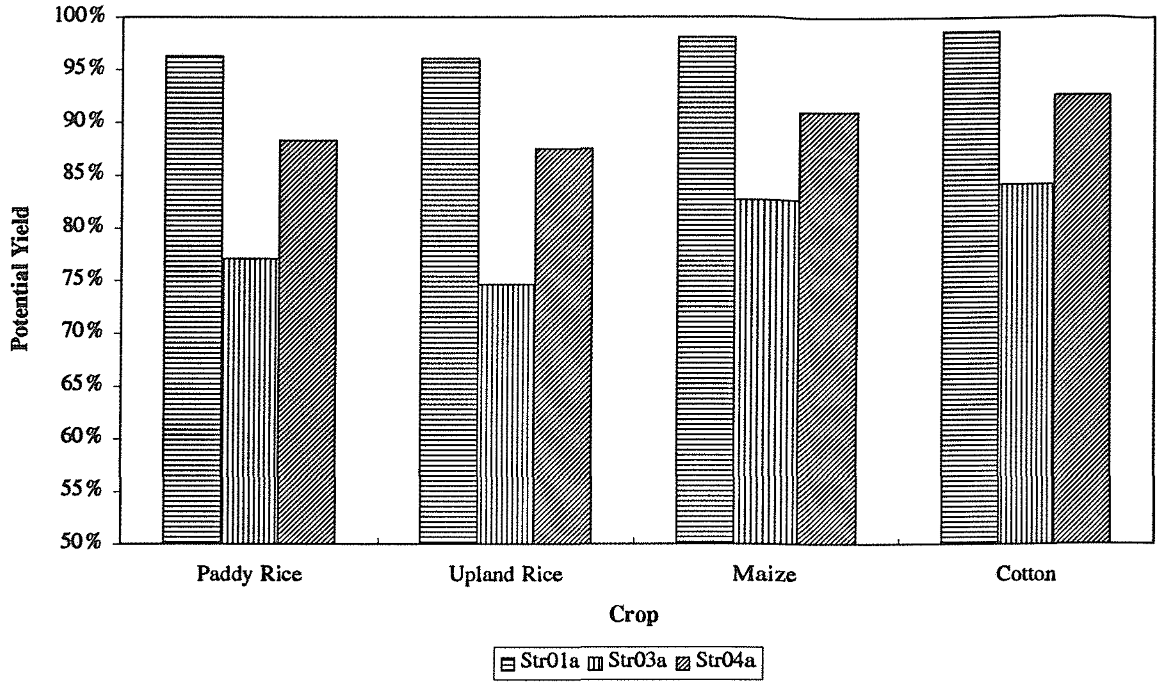

Figure 6.8 **The potential yields of the main crops in scheme *A* under automatic operation in the case of malfunctioning canal head regulators**

A detailed investigation of the output of the various hydraulic simulations is required such that an explanation of the results can be made. In scenario Str03a, where the head regulator of canal M1C1 was inoperable, canal M1C1 withdrew more flow than it should when the supply into the system was reduced below 100% of the design. However, unlike scenario Str03m, because the gates of the field outlets were automated, the additional water in the canal was not utilised by the field outlets and therefore went as a surplus to the tail escape of the canal. This reduced the flow available to the rest of the system and hence reduced both the adequacy of the supply and the equity of water distribution leading to a reduction of the total crop yield of the scheme.

The situation was better in scenario Str04a, where the head regulator of canal M2C4 was inoperable. Because the canal is located at the tail-end of the system, the loss of control of its head regulator did not have a significant impact on the rest of the irrigation system. Consequently, the water available to this canal did not change greatly from the target throughout the whole simulation and hence no water wastage took place as was the case in scenario Str03a. Additionally, the automation of the field outlets maintained relatively fair equitable distribution of water among the outlets on the canal. The impact on the performance was that this scenario achieved higher adequacy, equity and crop yields than scenarios Str03a and Str04m.

c. Conclusions

The overall conclusions which can be drawn from the previous two sets of simulations for investigating the impact of the loss of control of canal head regulators can be outlined as follows:

- The relative location of a malfunctioning head regulator within an irrigation system plays an important role in the performance of the system. In the two sets of simulations presented in the previous sections, a malfunctioning top-end head regulator caused more adverse impacts on performance than a malfunctioning tail-end head regulator. The area affected will vary however according to the nature of the problem in the gates of the regulators. For instance, if the gate of the head regulator of canal M1C1 was jammed in the fully-open position, the canal would withdraw more water than it should (which cannot be considered as a negative impact on canal M1C1) and hence the rest of the system downstream from the canal (canals M2C1, M2C2 & M2C4) would suffer from water shortage. The area adversely affected in this case would be the total area served by canals M2C1, M2C2 & M2C4. Figure 6.9 demonstrates this in terms of the total crop yield from the areas served by each distributary canal in system *A*. It is clear that in scenario Str03m, the yield of canal M1C1 increased (compare with the results of the control case Str01m) while the yields of canals M2C1, M2C2 & M2C4 all decreased.

  On the other hand, if the gate of the same head regulator was jammed almost fully closed, the area which would be adversely affected would be the area served by canal M1C1 itself, not the rest of the scheme. Using the area directly served by a canal as a proxy for its importance, and hence to prioritise the expenditure on the maintenance of control structures, will clearly not be correct in every scenario. A thorough examination of the different possible scenarios using hydraulic modelling techniques as has been demonstrated earlier is the ultimate procedure for planning and prioritising expenditures.

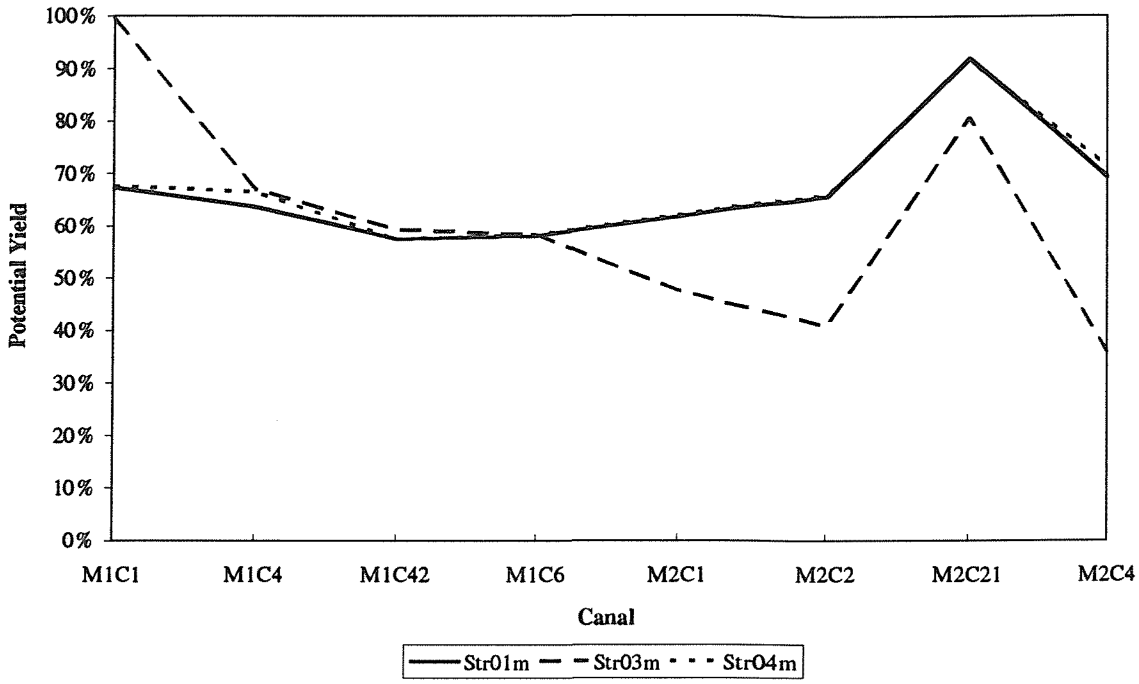

Figure 6.9 The potential total crop yield of each canal in scheme *A* under manual operation in the case of malfunctioning canal head regulators

- System automation down to the level of field outlets can maintain system performance at relatively high levels when problems in the control structures at higher levels in the system arise. For example, with reference to the scenarios in which the head regulator of canal M1C1 was malfunctioning, the estimated total crop yields from system *A* under manual control (scenario Str03m) varied between 60% and 65% (Figure 6.5). When the same system under the same conditions was fully automated (scenario Str03a), the total crop yields increased to about 75% to 80% (Figure 6.8), some 25% increase in the yields due to automation. These results can be the basis for a financial analysis to test whether the cost of automation can be justified by the corresponding increase in agricultural production or not. An example of such analysis is available in Section 7.5.4.

6.2.2 Cross Regulators

Usually, there are more cross regulators than head regulators in irrigation delivery and distribution systems (excluding field outlets). The case study (system *A*) has nine head regulators and 16 cross regulators on the main and distributary canals (see Appendix III). In larger systems, where conveyance and distribution canals run for quite a few kilometres, the cross regulators can significantly outnumber the head regulators. A large proportion of the expenditure on canal control structures might therefore be spent on cross regulators.

The objective of the following sections is to investigate the impact of the loss of control of canal cross regulators on the performance of irrigation systems through hydraulic modelling. One main objective amongst others is to compare this case (cross regulators) with the previous one (head regulators) in order to establish the relative importance of the two main types of canal regulators with respect to impact on performance. Two sets of simulations have been carried out, one simulated system *A* under manual operation and the second simulated the system under automatic operation.

a. Manual Operation

The details of the hydraulic simulations carried out for investigating the loss of control of canal cross regulators in manually-operated irrigation systems can be outlined as follows:

1) Simulate the operation of irrigation system *A* during a whole growing year. The pattern of the supply during a typical year is depicted in Figure 6.1.

2) Allow the settings of the gates of the canal regulators to be adjusted along the run according to the change in the supply in order to maintain target discharges/water levels according to the function of each regulator. However, the settings of the gates of the malfunctioning cross regulator are to be fixed at the full design discharge settings throughout the whole run.

To simulate the operation of a manual irrigation system, the gates of the field outlets are to be maintained fixed at full design discharge settings throughout the whole run.

3) Simulate other scenarios by changing the malfunctioning cross regulator in each one.

4) Assess the performance of the different simulation scenarios to compare between them.

The specific features of each of the scenarios simulated are summarised in Table 6.4.

Table 6.4 Brief description of the scenarios for investigating the loss of control of canal cross regulators in manually operated systems

Scenario	Features
Str01m	Control scenario: the gates of all canal regulators function properly
Str02m	The gate of the gated-weir cross regulator on the main canal MC is kept fully lowered (open) throughout the whole run (faulty gate)
Str10m	The gate of the first cross regulator (chainage 0.92 km) on distributary canal M1C1 (top-end) is kept open at the full supply setting throughout the whole run (faulty gate)
Str12m	The gate of the first cross regulator (chainage 0.56 km) on distributary canal M2C4 (tail-end) is kept open at the full supply setting throughout the whole run (faulty gate)
Str13m	The gate of the second cross regulator (chainage 2.06 km) on distributary canal M1C1 (top-end) is kept open at the full supply setting throughout the whole run (faulty gate)

i. Simulation Results

The performance assessment of the hydraulic simulations with reference to the adequacy of the supply, the equity of water distribution and the potential total crop yield is shown in

Figures 6.10 to 6.12 respectively. According to the figures, no significant distinction between the performance of the scenarios can be made. The direct impact of a cross regulator being jammed fully open is lower water levels upstream from the regulator. Since most of the cross regulators in irrigation system *A* serve between two to four field outlets only (out of a total number of 76 outlets), the overall impact on the performance of the whole system was insignificant. The relative location of the faulty cross regulator within the irrigation system did not have any significant impact on the performance as well.

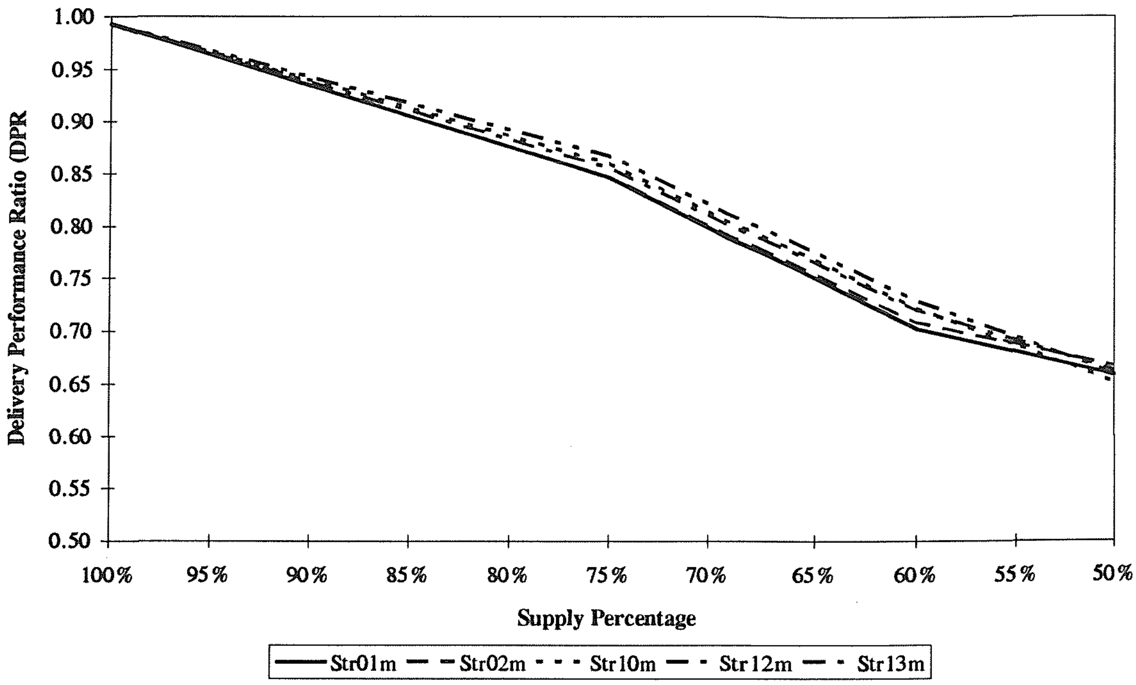

Figure 6.10 **The average adequacy of the supply in system *A* under manual operation in the case of malfunctioning canal cross regulators**

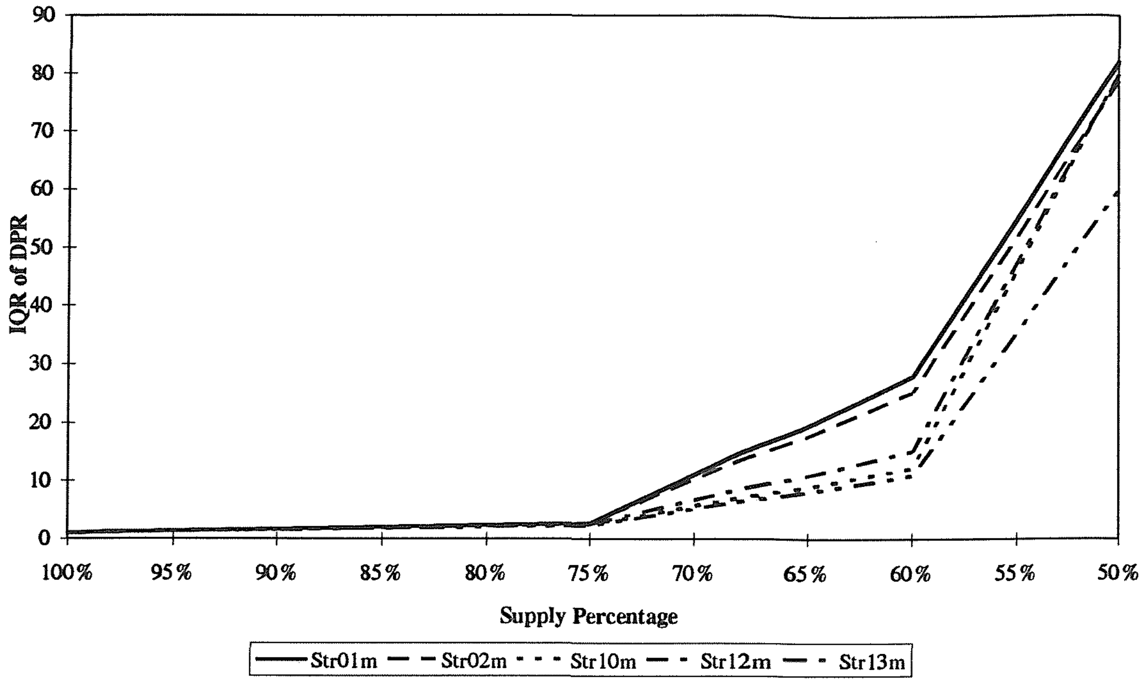

Figure 6.11 The average equity of water distribution in system *A* under manual operation in the case of malfunctioning canal cross regulators

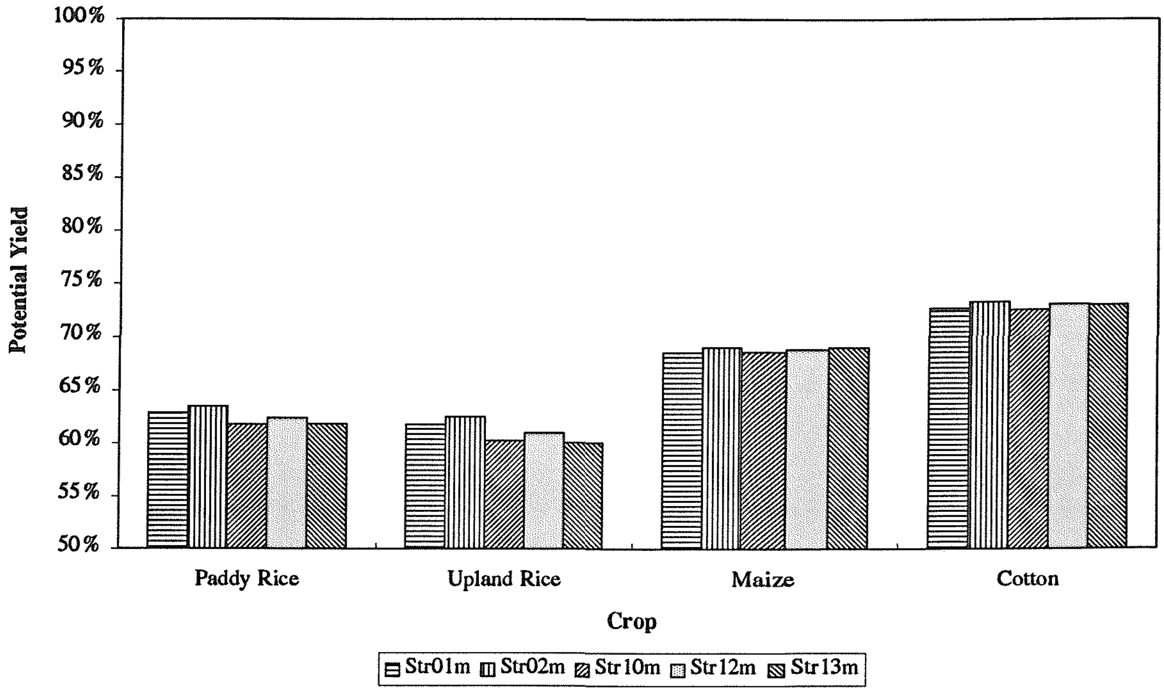

Figure 6.12 The potential yields of the main crops in scheme *A* under manual operation in the case of malfunctioning canal cross regulators

b. Automatic Operation

The previous set of simulations was repeated after changing the method of system control from manual to automatic in order to compare the performance of the two modes of operation under the same problem. The details of the current scenarios are outlined as follows:

1) Simulate the operation of irrigation system *A* during a whole growing year. The pattern of the supply during a typical year is depicted in Figure 6.1.

2) Allow the settings of the gates of the canal regulators to be adjusted along the run according to the change in the supply in order to maintain target discharges/water levels according to the function of each regulator. However, the setting of the gates of the malfunctioning cross regulator is to be fixed at the full design discharge setting throughout the whole run.

   To simulate the operation of a fully automated irrigation system, the gates of the field outlets should also be adjusted along the run according to the change in the supply in order to maintain target deliveries to the outlets.

3) Simulate other scenarios by changing the malfunctioning cross regulator in each one.

4) Assess the performance of the different simulation scenarios to compare between them.

The specific features of each of the scenarios simulated are summarised in Table 6.5.

Table 6.5 **Brief description of the scenarios for investigating the loss of control of canal cross regulators in automated systems**

Scenario	Features
Str01a	Control scenario: the gates of all canal regulators function properly
Str02a	The gate of the gated-weir cross regulator on the main canal MC is kept fully lowered (open) throughout the whole run (faulty gate)
Str10a	The gate of the first cross regulator (chainage 0.92 km) on distributary canal M1C1 (top-end) is kept open at the full supply setting throughout the whole run (faulty gate)
Str12a	The gate of the first cross regulator (chainage 0.56 km) on distributary canal M2C4 (tail-end) is kept open at the full supply setting throughout the whole run (faulty gate)
Str13a	The gate of the second cross regulator (chainage 2.06 km) on distributary canal M1C1 (top-end) is kept open at the full supply setting throughout the whole run (faulty gate)

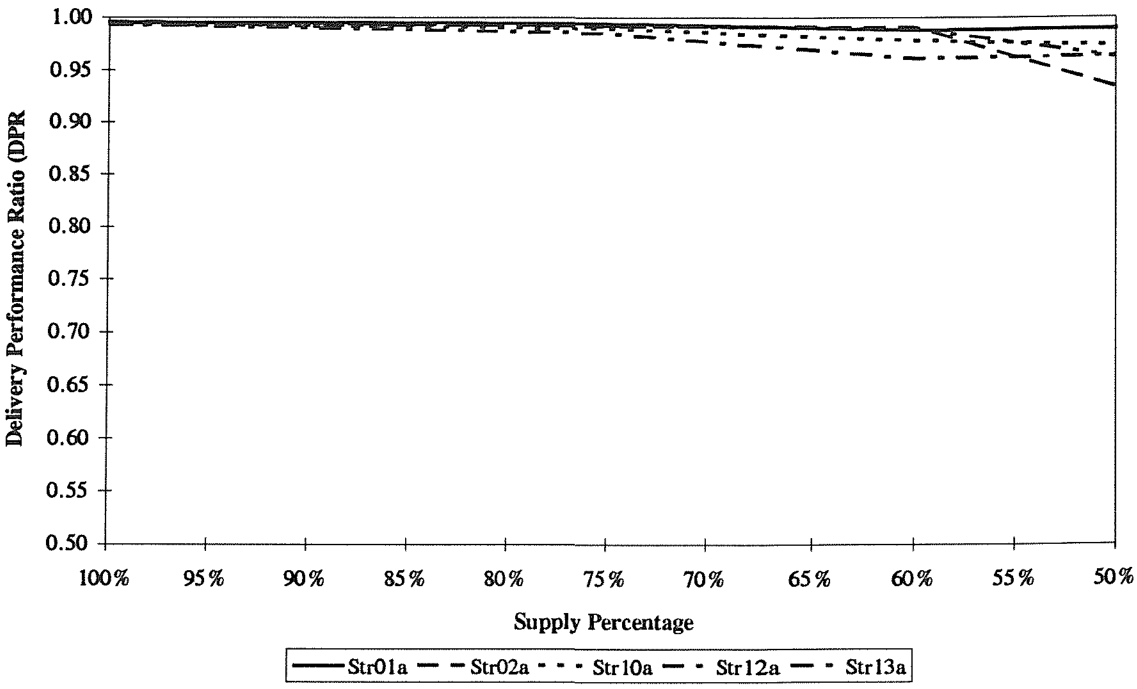

Figure 6.13 The average adequacy of the supply in system *A* under automatic operation in the case of malfunctioning canal cross regulators

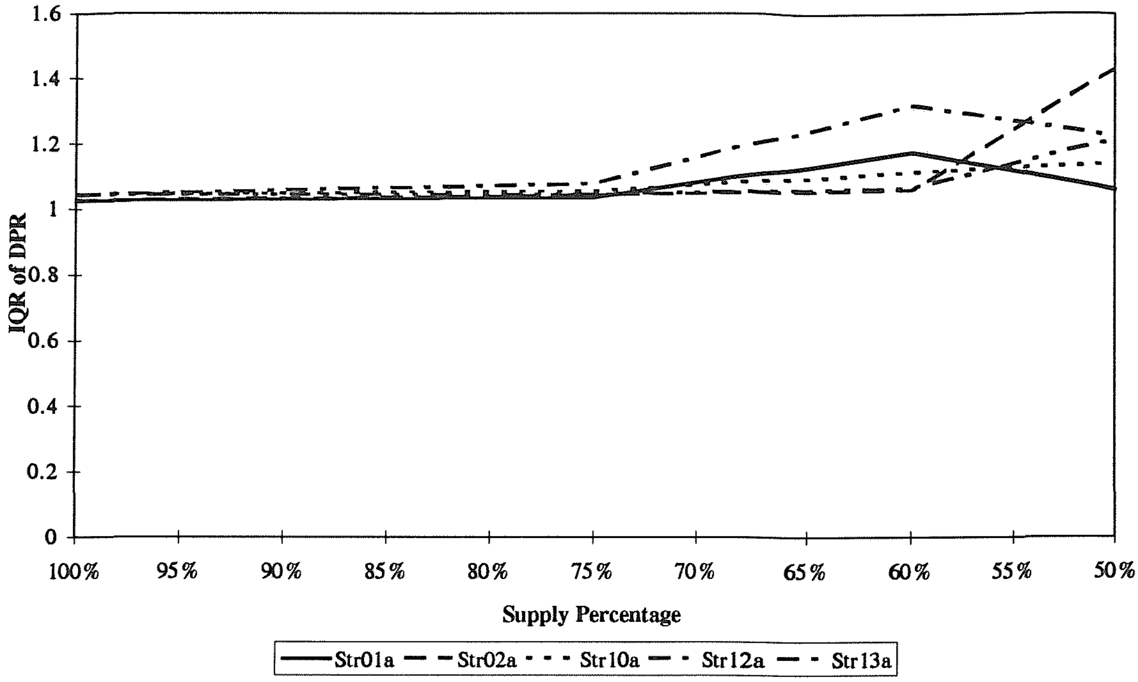

Figure 6.14 The average equity of water distribution in system *A* under automatic operation in the case of malfunctioning canal cross regulators

i. Simulation Results

Figures 6.13 to 6.15 depict the results of assessing the performance of the scenarios with respect to the adequacy, equity and total crop yields respectively. In general, the results show that the impact of the malfunctioning cross regulators was minimal on the automated system. Similar to the previous set of simulations (manual operation), the location of the malfunctioning cross regulator was not a significant factor.

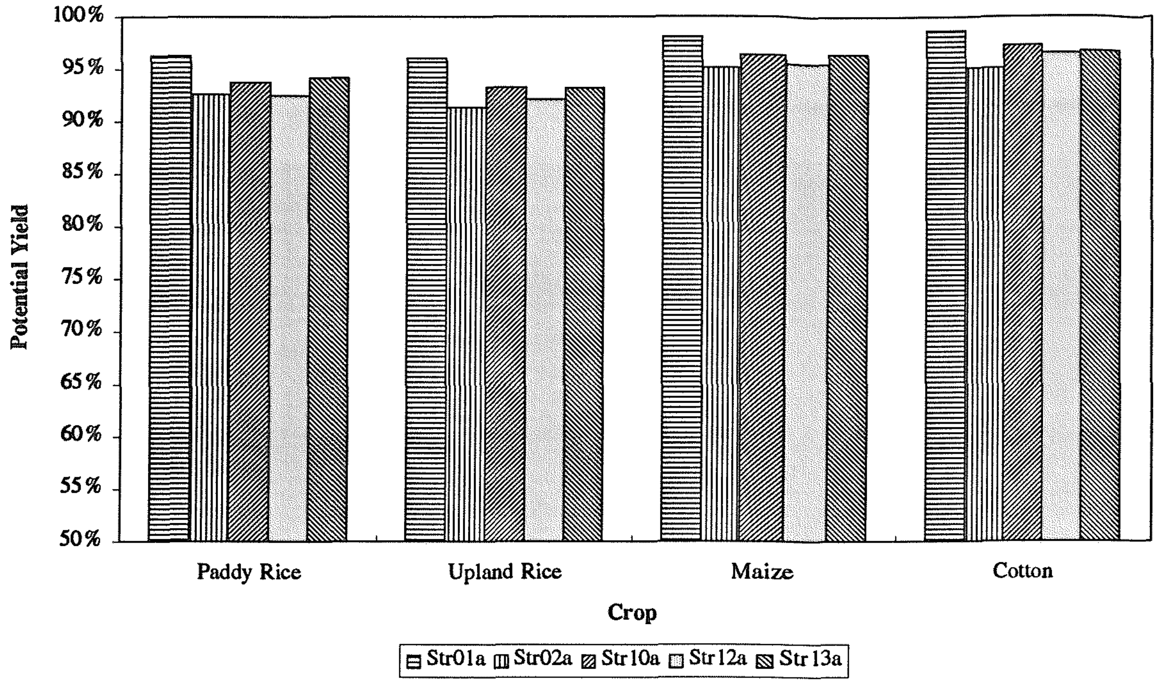

Figure 6.15 The potential yields of the main crops in scheme *A* under automatic operation in the case of malfunctioning canal cross regulators

6.3 Summary and Conclusions

Irrigation control structures are crucial components in any irrigation network for achieving effective water control. After the network of irrigation canals, the most common and therefore important components are the canal regulators. An investigation of the impact of the loss of control of gated canal regulators using hydraulic modelling techniques has been presented. The simulations focused on gated control structures as typical of many manual and automated systems. The main two types of canal regulators, namely head and cross regulators, have been studied. Among the factors which have been taken into consideration when setting up the scenarios were the mode of system operation (manual/automatic) and the relative locations of the regulators within the irrigation network (top-end/tail-end). A comparative assessment of the different scenarios simulated has been carried out in order to establish a system of prioritisation/importance ranking of the expenditure on the necessary repair work. Hydraulic performance indicators and the estimations of the potential total yield of the main crops in system *A* have been used in making the comparisons between the overall performance of the various scenarios. The findings of the investigation can be summarised in the following points:

- Due to their larger service areas and more crucial functions, faulty canal head regulators were found to have greater impact on the performance of the case study in both cases of manual and automatic operation than faulty cross regulators. Consequently, head regulators should generally be given higher priority than cross regulators when allocating the expenditure on maintenance and rehabilitation.

- The relative location of a faulty head regulator also had a significant role to play in determining the level of hydraulic performance. Generally speaking, top-end head regulators will usually be more important then tail-end ones. However, the technique of using the area served by a regulator as a proxy for its importance, as is currently implemented in some planning tools, might not yield accurate results in all situations. A more accurate technique is the use of hydraulic modelling as has been demonstrated in this chapter.

- System automation significantly improved the performance of the case study in the case of malfunctioning canal head regulators with all other factors being constant. For example, the total agricultural yield of scheme *A* was 25% higher when the system was fully automated than when it was manually operated. A financial analysis should be carried out in order to establish the viability of the investment on automation against the returns to the investment in terms of increased agricultural production. An example of such analysis is given in Section 7.5.4.

- The above results show that the utilisation of hydraulic modelling techniques in a methodology for expenditure and asset management planning is feasible and should yield better results than the methods which are currently used.

A summary of the points outlined above is given in Table 6.6.

Table 6.6 **The relative performance of manual and automatic systems in the case of malfunctioning control structures**

Criterion	Manual Operation	Automatic Operation
Malfunctioning of canal head regulators		
Impact on overall performance	High	Low
Impact of the relative location of the structure within the system	Significant	Significant
Malfunctioning of canal cross regulators		
Impact on overall performance	Low	Low
Impact of the relative location of the structure within the system	Insignificant	Insignificant

The following chapter covers the financial analysis of the problems investigated in this chapter and the utilisation of the output from hydraulic modelling in such analyses.

7. Cost-Benefit Analysis

7.1 Introduction

The preceding two chapters presented a procedure for linking hydraulic performance to infrastructure condition and prioritising expenditure options using hydraulic modelling techniques. The hydraulic performance of the tested scenarios was assessed using selected performance indicators and the scenarios were ranked according to their performance. After an expenditure/investment option has been tested from the hydraulic point of view, a cost-benefit analysis will usually be required in order to test the viability of that option from the economic and/or financial points of view. The purpose of this chapter is to:

- present a methodology for analysing the costs and benefits of options for expenditure on irrigation-structure maintenance/rehabilitation and investment in structure upgrading/modernisation;

- demonstrate the utilisation of hydraulic modelling in quantifying the costs and benefit; and,

- demonstrate the application of the methodology in asset management planning.

Comprehensive cost-benefit analyses would be complex and beyond the scope and purpose of this research. Simplified, but indicative, analyses will be presented. It must be emphasised again that the purpose of this chapter is to demonstrate the methodology for examining the returns to expenditures/investments in irrigation infrastructure, not to study the economics of a certain case study.

7.2 Types of Cost-Benefit Analysis

Although all cost-benefit analyses are similar, variations in the way the costs and benefits are valued do exist. Two main types of prices are used in cost-benefit analyses, namely *economic* and *financial* prices, depending on the type of the project being analysed. It is not the purpose of this chapter to explain the differences between these two types (reference is made to Snell, 1997, for such explanation). However, it is important to mention which type of analysis has been chosen for this work and why.

It is well acknowledged that most of the operation and maintenance budgets of many irrigation systems worldwide come from public funds (Skutsch, 1998). In this respect, an appropriate cost-benefit analysis should use economic prices. On the other hand, if a system has been privatised or turned over, financial analyses should be more appropriate in reflecting the interests of the farmers or the investors who run it as a business. Since irrigation-system turnover and privatisation have been on the increase since the 1970s (Vermillion & Garcés-Restrepo, 1998), it was decided to use financial prices in the cost-benefit analyses presented in this chapter. Nevertheless, the procedure of the analyses will not be much different if economic prices were used instead of financial prices.

7.3 Implementation of Cost-Benefit Analysis in Asset Management Planning

7.3.1 Determination of the Costs and Benefits

The first step in cost-benefit analysis is the determination of the costs and benefits of the process under consideration. For a successful determination of the costs and benefits of a process, all the elements involved in the process should be identified. With respect to expenditure and asset management planning, the *process* can be defined as a change in the condition/type of irrigation infrastructure. The cost involved is therefore the expenditure or investment required for the infrastructure change. The change might have impacts on the hydraulic performance of the irrigation system and/or the agricultural production of the scheme (Figure 7.1). Consequently, these two elements are the main potential benefits of the process.

However, two possibilities for how a change in the condition/type of irrigation infrastructure might impact hydraulic performance and agricultural production do exist.

The first possibility is that hydraulic performance and agricultural production are two benefits that are *necessarily* linked together. In this case, a change in agricultural production can *only* be caused by a change in hydraulic performance. The other possibility is that these two benefits are *not necessarily* linked together, i.e. either of them can change separately (Figure 7.1). These different possibilities significantly change the cost-benefit analysis process as explained below.

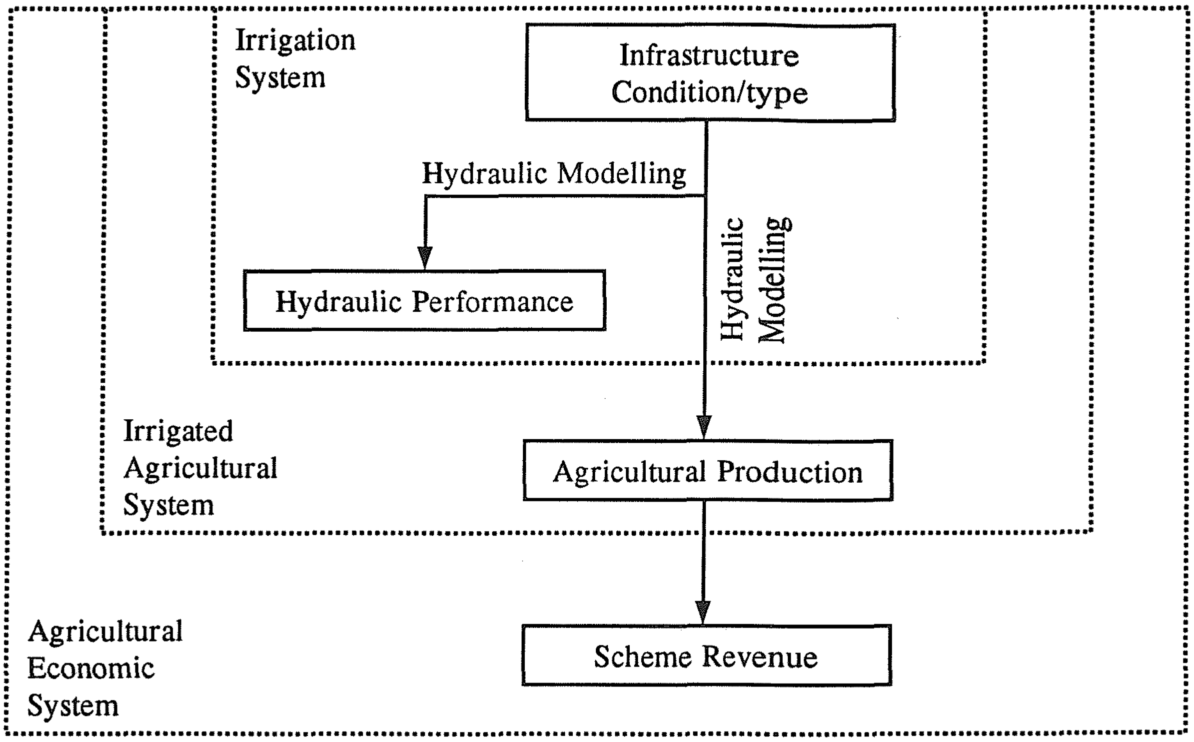

Figure 7.1 The chain of reactions of the change in the condition/type of irrigation infrastructure

If hydraulic performance and agricultural production are considered to be linked together, then these two elements should not be considered as two separate benefits. In such a case, the final benefit of the process is agricultural production and hence it is the only benefit that should be taken into consideration in cost-benefit analysis. Realising the two benefits in cost-benefit analysis in such a case is known as *double-counting* in economic terminology and is cautioned against (Snell, 1997).

If, on the other hand, hydraulic performance and agricultural production are considered as two separate elements (Figure 7.1), both benefits should be taken into consideration in the cost-benefit analysis. This approach has been adopted in this research since it is not difficult to acknowledge the possibility of an intervention to have an impact on either hydraulic performance or agricultural production. For example, an intervention might improve the equity of water distribution (a hydraulic performance criterion) without having significant impact on the overall agricultural production of a scheme. If water was the scarce resource and agricultural land was not a constraint, then improved water distribution equity might lead to more equitable distribution of the agricultural production of various parts of the scheme, but not necessarily an increase in the overall production.

The difficulty with this latter approach is that the evaluation of the monetary value of a change in hydraulic performance is not readily possible since hydraulic performance measures do not have prices. Such monetary valuation will be required if financial cost-benefit analysis is to be used as a single decision-making criterion. Although there are methods for such valuations, those methods are complicated and very approximate (Snell, 1997). A better approach for tackling the problem is the use of multi-criterion decision analysis such that financial analysis is used to evaluate only the benefits which can be easily valued in monetary units. Other more appropriate criteria are used to evaluate the non-marketed costs and benefits (e.g. performance indicators are used to evaluate hydraulic performance). Since the multi-criterion decision analysis approach has been adopted in this research, only the potential increase in agricultural production has been taken into consideration as the main benefit in the financial analyses presented in this chapter. The other benefits of the investigated scenarios in terms of improved hydraulic performance have been evaluated in the previous two chapters.

7.3.2 Outline of the Implementation

The steps of carrying out a cost-benefit analysis for any of the scenarios investigated in this research can be outlined as follows:

1) Model the case study in its ideal situation (assuming that all the irrigation infrastructure function properly) using hydraulic modelling techniques. Simulate the operation of the system during a whole year. Estimate the potential yield of

each of the main crops grown in the scheme according to the actual flow delivered to each field outlet from the output of hydraulic modelling using the guidelines of Doorenbos and Kassam (1979). Determine the expected gross revenue of the scheme by using the farm-gate prices of the crops. This revenue is considered as the potential ultimate (target) revenue of the scheme and therefore should be used as a reference for other scenarios.

This approach takes account of any design, construction or operational problems existing in the system and provides the best case scenario for the system under consideration. Note that production may not be 100% even in the best scenario. Examples of such cases can be found in the scenarios investigated in Section 6.2.1.

2) Model the case study again with the problem under investigation (e.g. sedimentation in the canals) being simulated this time. Estimate the total yield of each crop and determine the expected gross revenue of the scheme as outlined in the previous step.

3) The difference between the two gross revenues calculated in the previous two steps is an estimation for the impact of the problem under question on the production of the scheme being studied, which in most cases will cause a loss of revenue. It is also an indirect assessment of the impact of the problem on the hydraulic performance of the scheme.

4) Estimate the expenditure required to either totally or partially cure the problem (e.g. desilting all or some of the canals) or the investment required for upgrading the problematic structures. Compare this cost with the loss of revenue calculated in the previous step. The assumption in this case is that if the problem is totally cured, the performance of the system can be restored to its ideal level as per the modelling in Step 1. If the problem cannot be totally cured (due to lack of funds for example) then the situation after partially curing the problem should be modelled to assess the actual change in performance and hence the revenue of the scheme.

It should be noted that this procedure links the two nested systems, namely the irrigation and the irrigated agriculture systems, as viewed by Small and Svendsen (1992) (see Figure 3.2). First, hydraulic modelling is used to evaluate the output of the irrigation system down to the field outlet level (the conveyance and distribution system). Then, the actual water allocated to the field outlets is used to estimate crop yields from the irrigated agricultural system. The interaction between the two systems through the processes of converting the delivered water to crop production is assumed to have no constraints which impede production. Hence, any water that is delivered from the irrigation system to the irrigated agricultural system, which is not in excess to crop water requirements, is assumed to beneficially contribute to production.

The following sections present the application of the above procedure to some of the expenditure options investigated in previous chapters.

7.4 Financial Analysis of Sediment Removal Alternatives

The problem of sedimentation in irrigation networks has been dealt with in Chapter 5 from the perspective of its impacts on hydraulic performance. Several alternatives for tackling the problem under the restrictions of limited financial resources have been investigated. A comparison between the efficiencies of these alternatives have been made based on selected hydraulic performance criteria. In the following sections, the costs and benefits of these alternatives will be analysed. For consistency with the analyses presented in Chapter 5, each scenario has been modelled twice, once simulating the operation of an automated system and another simulating the manual operation. Irrigation system *A* (Appendix III) was used as a case study in all the simulations.

The adaptation of the steps outlined in Section 7.3.2 to the cost-benefit analysis of the sediment removal scenarios is as follows:

1) Simulate the operation of the case study during a whole growing year in the case of not removing any sediment from the canals to study the impact on crop yields. As has been shown in Section 5.7.1, the full design discharge should not be allowed in the system in such cases because it endangers the safety of the canals due to

encroachment on design freeboard. Consequently, the maximum flow allowed in the system should be taken from Figure 5.10. Based on the hydraulic output of the simulations, the total agricultural yield of the scheme can then be estimated such that the net income of the scheme can be determined.

2) Simulate the operation of the case study during a whole growing year after partially removing the sediment according to one of the alternatives outlined in Table 5.8. The results of modelling these alternatives in Section 5.7.2 showed that it is safe to allow the maximum design flow in the canal network after partial sediment removal. Estimate the potential agricultural yield and determine the net income of the scheme. The difference between this income and the one calculated in the previous step is the potential benefit from partially cleaning the sediment.

3) Estimate the cost of cleaning the sediment according to the alternative being investigated and compare it with the benefit worked out in the previous step to ascertain the return to the expenditure, hence evaluate the financial viability of the sediment removal scenario.

7.4.1 Details of the Hydraulic Simulations

The hydraulic analysis of the alternatives of partial sediment removal from system *A* showed that out of the seven alternatives investigated (Table 5.8), two alternatives achieved the best hydraulic performance (see Section 5.7.2). These two alternatives are to remove all the sediment in the distributary canals only (the sediment in the main canal is not removed, scenarios Sed30-34 & Sed30-54) or to clean half the sediment depth in the whole irrigation network (scenarios Sed30-39 & Sed30-59). Consequently, the focus in the cost-benefit analysis in this section is on these two alternatives only, and comparing them with the cases of full sediment removal. To avoid lengthy presentation, the results of the analyses of the cases of automatic and manual operations are presented together.

Table 7.1 gives a basic description of the various scenarios. Four simulations of each mode of operation (automatic/manual) have been carried out. The scenarios which have the code

Sed*xx*-3*x* simulate the operation of system *A* under full automation, while those which have the code Sed*xx*-5*x* simulate the system under manual operation. Scenarios Sed20-32, Sed20-52, Sed30-32 & Sed30-52 are the control scenarios which simulate the operation of the case study without sediment removal for comparison with the other scenarios. The other four scenarios simulate the two alternatives of partial sediment removal as mentioned above.

Table 7.1 Brief description of the scenarios for ascertaining the financial viability of partial sediment removal from system *A*

Scenario	Description	Volume of Removed Sediment (m^3)
Sed20-32 & Sed20-52	- 20% sedimentation in all the canals (no sediment removal). - Maximum allowed flow is 75% and 82% of the design in scenarios Sed20-32 and Sed20-52 respectively (Figure 5.10).	0
Sed30-32 & Sed30-52	- 30% sedimentation in all the canals (no sediment removal). - Maximum allowed flow is 60% and 65% of the design in scenarios Sed30-32 and Sed30-52 respectively (Figure 5.10).	0
Sed30-34 & Sed30-54	- Clean all the sediment in the distributary canals only (30% sedimentation remains in the main canal), Figure 5.13. - Maximum allowed flow is 100% of the design.	33,753
Sed30-39 & Sed30-59	- Clean half of the sediment depth in the whole network (15% sedimentation remains). - Maximum allowed flow is 100% of the design.	29,091

* Total volume of sediment at 30% sedimentation is 58,181 m^3.

7.4.2 Simulation Results

The main purpose of carrying out the simulations currently investigated is to estimate the potential total crop yields from system *A* (the hydraulic performance of these scenarios have already been assessed in Chapter 5). The steps of estimating the total yield of each crop are briefly as follows:

1) Obtain the actual flow diverted to each field outlet in the irrigation system during a whole growing season from the output of the hydraulic simulations.

2) Using the actual flow diverted to each field outlet and the calculated crop water requirements, the potential yield of each crop grown in the area served by each outlet can be estimated based on the yield response to water functions of Doorenbos and Kassam (1979).

3) By summing up the yields of each crop from each field outlet, the total yield of that crop from the whole scheme can be estimated.

The results of estimating the potential crop yields are summarised in Figure 7.2 for the automatic operation and Figure 7.3 for the manual case. The figures show that sedimentation can have serious impact on scheme production. For example, in scenario Sed30-32 (30% sedimentation in all the canals), the most two affected crops were the two varieties of rice whose potential yields were reduced to around 40%. With selective maintenance (scenarios Sed30-34 & Sed30-39) much of this lost production can be recovered (rice yield increased to more than 90% in the case of automated operation, Figure 7.2).

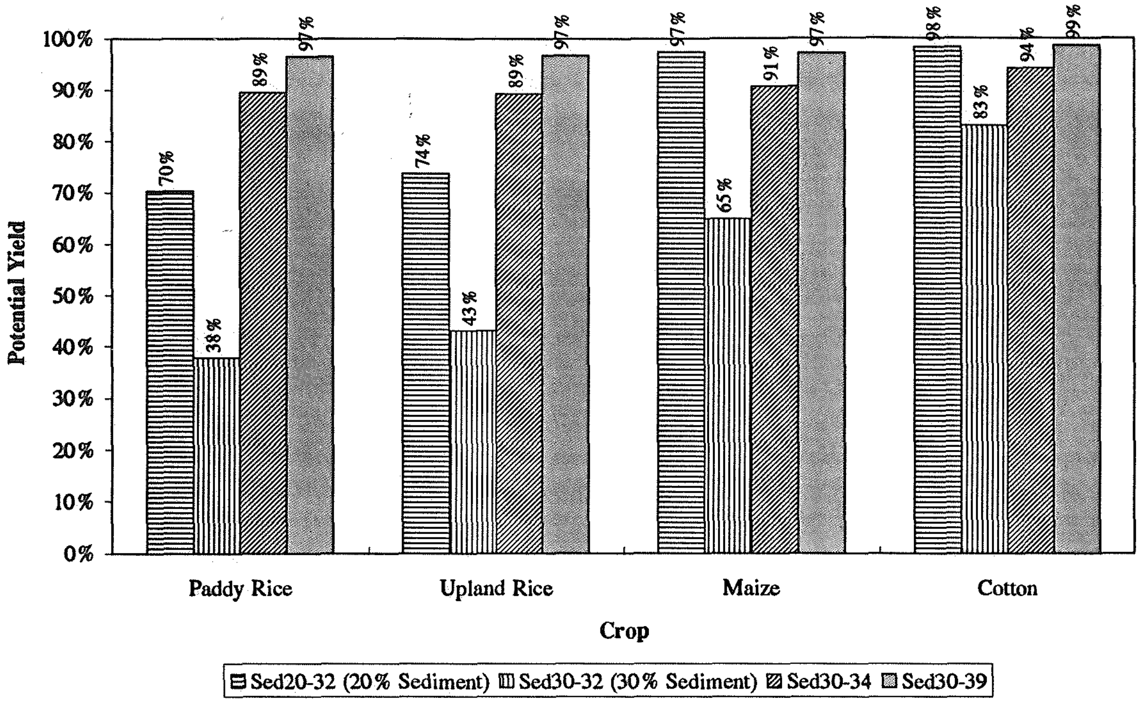

Figure 7.2 **The impact of sedimentation on the potential crop yields of scheme *A* under automatic operation**

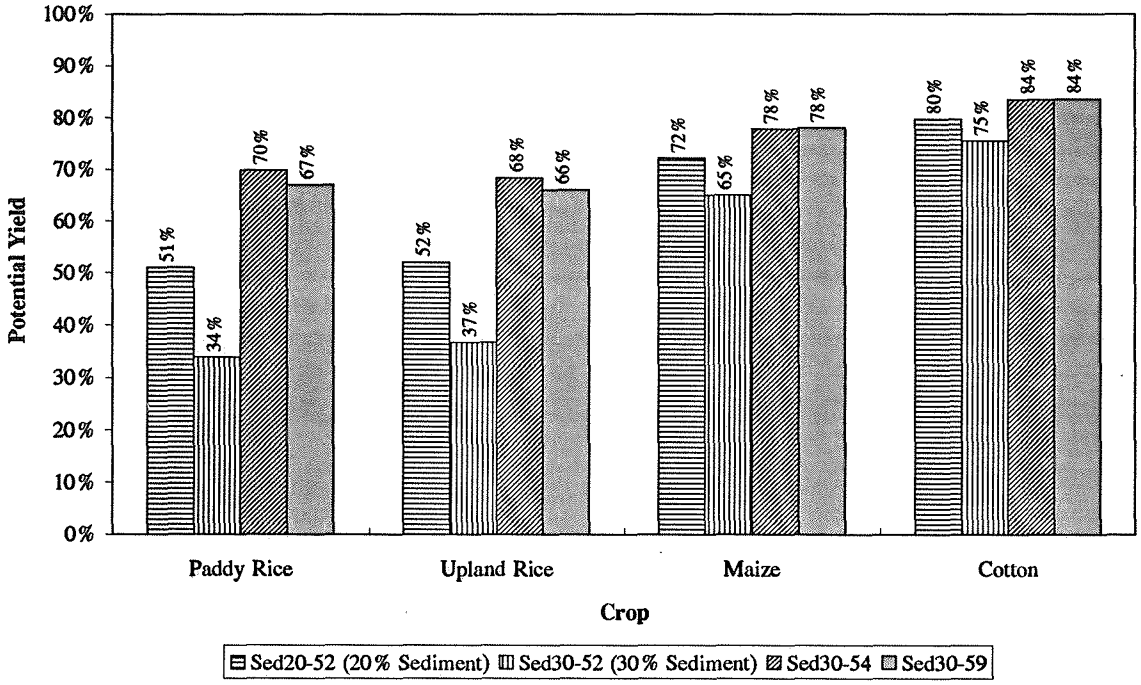

Figure 7.3 **The impact of sedimentation on the potential crop yields of scheme *A* under manual operation**

7.4.3 Cost-Benefit Analysis

For the purpose of analysing the costs and benefits of the current scenarios, the prices of sediment removal (costs) and the crops produced (benefits) must be determined. Table 7.2 summarises the calculations of the benefits in each scenario (all prices are in 1998 USD[10]). The total net income of each scenario is the sum of the net incomes of all the main crops. It is calculated as:

$$Total\ Net\ Income = \sum_{i=1}^{n_c} \frac{C_i * A_i * P_i}{100} \qquad \textbf{(7.1)}$$

where n_c = number of main crops = 4 in scheme *A*

C_i = estimated total production of crop *i* (%) (Figures 7.2 & 7.3)

A_i = total area of crop *i* in the whole scheme (ha) (Appendix III)

P_i = net income of crop *i* (USD/ha) (the crop budgets and average net incomes from the main crops grown in scheme *A* are calculated in Appendix V).

Accordingly, the maximum potential net income of scheme *A* under ideal conditions can be calculated by putting C = 100% in the above equation. This works out as \$4,191,320 (1998 prices).

The lost income in Table 7.2 is the income forgone due to sedimentation. It is calculated by subtracting the maximum potential net income (\$4,191,320) from the total net income of each scenario. It is an evaluation, in monetary units, of the level of performance of each scenario compared to the ultimate theoretical performance. The potential gross return to desilting is the potential gross return to the cost of desilting after sediment removal.

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10 United States Dollars

Table 7.2 The potential net income of scheme *A* under the scenarios of partial sediment removal

Scenario	Total Net Income (1998 USD)	Lost Income*		Potential Gross Return (1998 USD)	
		(1998 USD)	(%)	Partial Desilting	Full Desilting[§]
Automatic Operation					
Sed20-32	3,895,328	-295,992	7.1%	--	*295,992*
Sed30-32	2,755,579	-1,435,741	34.3%	--	*1,435,741*
Sed30-34	3,836,535	-354,785	8.5%	1,080,956[†]	*1,435,741*
Sed30-39	4,088,342	-102,978	2.5%	1,332,763[†]	*1,435,741*
Manual Operation					
Sed20-52	2,961,994	-1,229,326	29.3%	--	*1,229,326*
Sed30-52	2,627,228	-1,564,092	37.3%	--	*1,564,092*
Sed30-54	3,272,599	-918,721	21.9%	645,371[‡]	*1,564,092*
Sed30-59	3,259,828	-931,492	22.2%	632,600[‡]	*1,564,092*

* The maximum income of the scheme under ideal conditions is $4,191,320.

[§] If all the sediment was removed (full maintenance).

[†] In comparison with the total net income of scenario Sed30-32 (no desilting).

[‡] In comparison with the total net income of scenario Sed30-52 (no desilting).

The typical cost of removing 1 m^3 of sediment from irrigation canals has been estimated in Appendix V as $5.5 (1998 prices). The cost covers hauling the excavated material for 1 km and disposing of it as an average for the cost of removing the sediment from any location in the canal network. This approach is similar to what happens in practice where a contractor would generally be required to give one figure for sediment removal irrespective of the location. To apply this to the current scenarios, first the volume of the sediment to be removed in each scenario is calculated (Table 7.1) and then multiplied by the cost of removing 1 m^3 to obtain the total cost of desilting as shown in Table 7.3.

Table 7.3 The financial viability of the scenarios of partial sediment removal from system *A*

Scenario	Potential Gross Return to Desilting (1998 USD) *	Volume of Removed Sediment (m^3)	Cost of Desilting (1998 USD)	Benefit/Cost Ratio †
Automatic Operation				
Sed20-32	[*295,992*] §	[*38,788*] §	[*213,331*] §	[*1.4*] §
Sed30-32	[*1,435,741*] §	[*58,181*] §	[*319,997*] §	[*4.5*] §
Sed30-34	1,080,956	33,753	185,643	5.8
Sed30-39	1,332,763	29,091	159,998	8.3
Manual Operation				
Sed20-52	[*1,229,326*] §	[*38,788*] §	[*213,331*] §	[*5.8*] §
Sed30-52	[*1,564,092*] §	[*58,181*] §	[*319,997*] §	[*4.9*] §
Sed30-54	645,371	33,753	185,643	3.5
Sed30-59	632,600	29,091	159,998	4.0

* See Table 7.2.

† The larger the benefit/cost ratio the better the return on expenditure.

§ If all the sediment was removed (full maintenance).

The financial viability of the scenarios can be tested by using the benefit-cost ratio as shown in Table 7.3. The analysis considered that the "useful life" of the expenditure was one year only since canal desilting may be required each year. Hence, both the cost of desilting and the potential gross return to desilting are incurred each year.

Many important conclusions/decisions can be drawn/made from the results in Tables 7.2 and 7.3:

- Fully automated irrigation systems may be able to compensate for sedimentation better than manual systems due to the frequent adjustments to the control structures which is possible in the former case. Nevertheless, as the percentage of sedimentation in the irrigation network increases, the performance of the two types of controls becomes similar (Figure 7.4) because much of the control of the structures will be lost to sedimentation. In the case of system *A* for example, the loss of scheme income (loss of potential crop yield) at 20% sedimentation was estimated to be around only 7% when the system was fully automated compared to 29% when the system was manually operated (scenarios Sed20-32 & Sed20-52, Table 7.2). On the other hand, the loss of income at 30% sedimentation was much closer in the cases of automatic and manual operation at about 34% and 37% respectively (scenarios Sed30-32 and Sed30-52, Table 7.2).

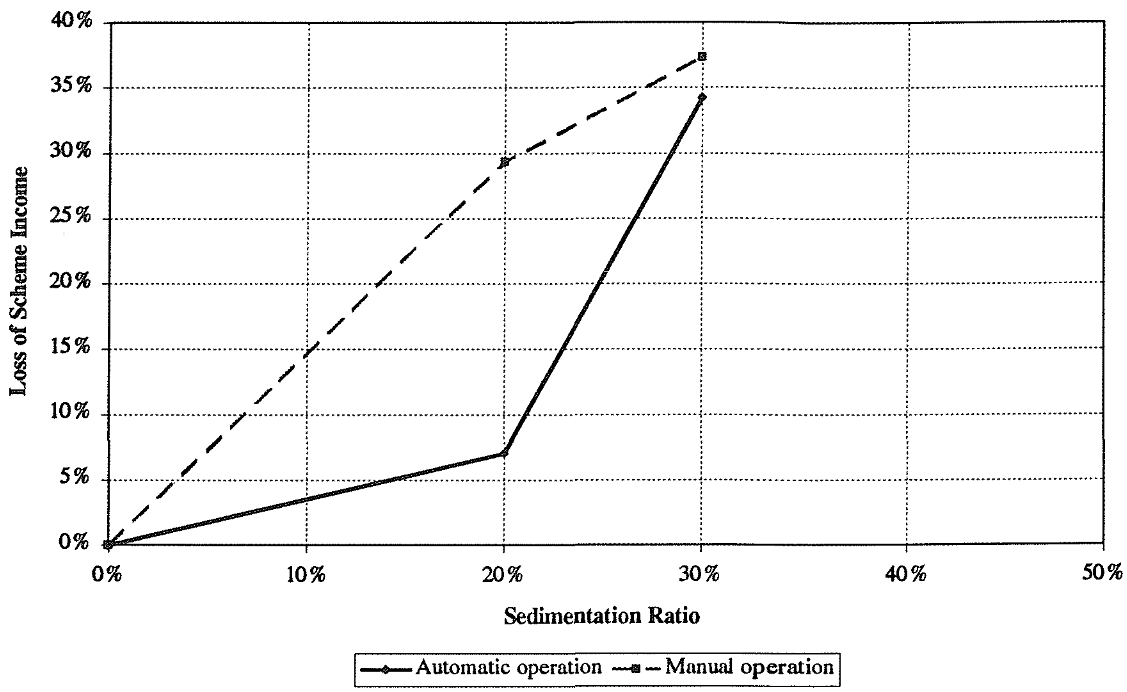


Figure 7.4 The impact of sedimentation on the income of scheme *A* under two modes of operation

- The financial analysis of the different scenarios, as outlined in Table 7.3, also suggests the advantage of automatic operation over manual operation. In the case of automatic operation, the most financially rewarding sediment removal alternative was to implement selective maintenance and clean half of the sediment depth in the whole irrigation network (scenario Sed30-39). The results show that this scenario was even better than total sediment removal (the figures between brackets in Table 7.3, scenario Sed30-32).

- The situation was different in the case of manual operation. The benefit/cost ratios if all the sediment was removed (the figures between brackets for scenarios Sed20-52 & Sed30-52) were slightly higher than those in the cases of selective maintenance (scenarios Sed30-54 & Sed30-59, Table 7.3). However, the scenarios of selective maintenance (partial sediment removal) were still financially viable since the benefit/cost ratios of these scenarios were high.

- The relationship between the percentage of sedimentation and the loss of income of scheme *A* is not linear, especially in the case of automatic operation (Figure 7.4). If the rate of sedimentation per year is known, Figure 7.4 can be reproduced to show the time on the horizontal scale versus the loss of scheme income. Such a chart should be useful to scheme managers in estimating the consequences of deferring sediment removal on scheme production and income, and in making decisions about when desilting ought to be done (expenditure planning).

7.4.4 Sensitivity Analysis

A number of sensitivity analyses have been carried out to test the sensitivity of the cost-benefit analysis of the sediment removal scenarios to variations in the parameters involved. The principle of the analyses was to work out the changes required to some parameters in order to make the benefit/cost ratio equate to 1.0 (break-even situation). The parameters tested were:

(i) the cost of removing 1 m^3 of sediment, which can also be considered as a proxy for the sensitivity of the variations in the volumes of the sediment to be removed;

(ii) the net income of all the main crops assuming that the prices of all the crops will change by the same percentage of the estimated price (e.g. due to local currency devaluation or other similar situations); and,

(iii) the net income of the cotton crop since it returns the highest income among all the main crops in scheme *A*.

A summary of the results of the sensitivity analyses is given in Table 7.4.

Table 7.4 Sensitivity analysis of the scenarios of sediment removal from system *A*

Scenario	Original Benefit/Cost Ratio *	New Price/Original Price
(i) Sensitivity of the cost of removing 1 m^3 of sediment		
Sed20-32	1.4	139%
Sed30-32	4.5	449%
Sed30-34	5.8	582%
Sed30-39	8.3	883%
Sed20-52	5.8	557%
Sed30-52	4.9	489%
Sed30-54	3.5	348%
Sed30-59	4.0	395%
(ii) Sensitivity of the total net income of all the main crops		
Sed20-32	1.4	72%
Sed30-32	4.5	22%
Sed30-34	5.8	17%
Sed30-39	8.3	12%
Sed20-52	5.8	17%
Sed30-52	4.9	20%
Sed30-54	3.5	29%
Sed30-59	4.0	25%
(iii) Sensitivity of the net income of cotton		
Sed20-32	1.4	-263%
Sed30-32	4.5	-413%
Sed30-34	5.8	-526%
Sed30-39	8.3	-484%
Sed20-52	5.8	-288%
Sed30-52	4.9	-294%
Sed30-54	3.5	-341%
Sed30-59	4.0	-348%

* See Table 7.3.

The sensitivity of each studied parameter is expressed in the table by means of the percentages of the new prices compared to the current prices which will reduce the benefit/cost ratios to 1.0. For example, the cost of removing 1 m^3 of sediment will have to increase by 39% over the cost used in the previous cost-benefit analysis ($5.5) for the benefit/cost ratio of scenario Sed20-32 to be equal to 1.0 (Table 7.4). Similarly, the net income of all the main crops will have to drop to 25% of the currently estimated net income (e.g. if 75% of the expected crop yield is lost) for scenario Sed30-39 to break even. Finally, the net income of cotton will have to drop to more than -263% of the current net income before scenario Sed20-32 starts to lose money. Such negative change in the net income of cotton can occur if the crop totally fails during the growing season such that the farmers will have already spent some money on the crop but will not have any return from it.

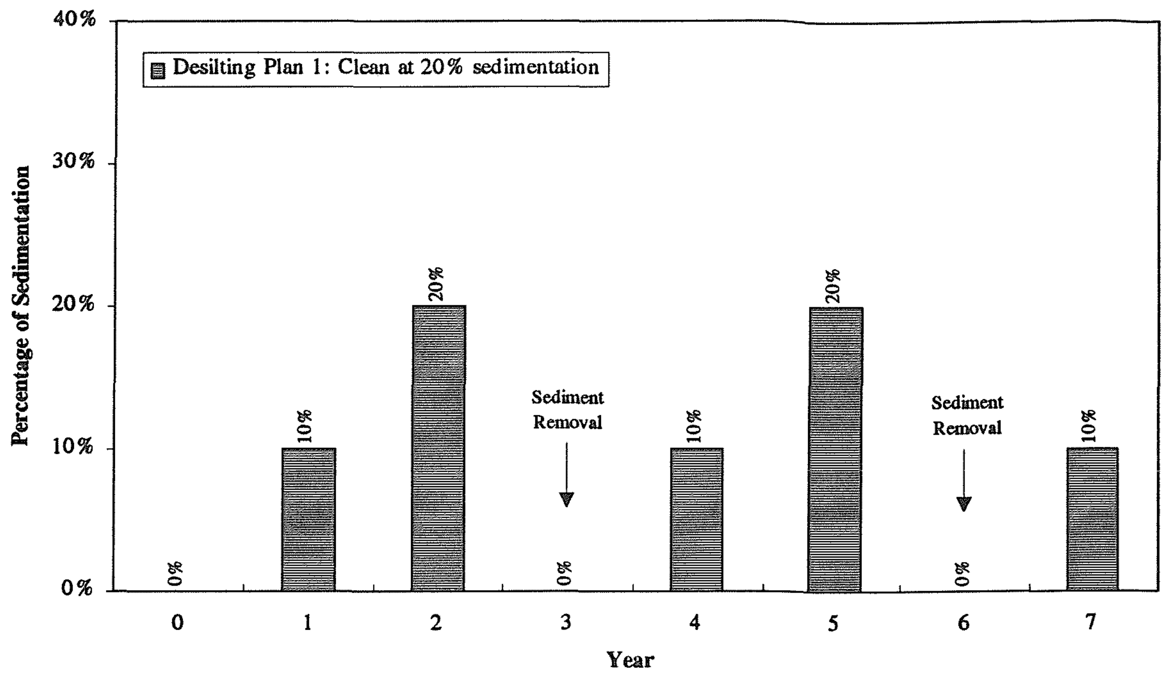
Accordingly, the results show that neither variations in the cost of sediment removal nor variations in the net income of all the crops are highly sensitive, with scenario Sed20-32 being the only exception (Table 7.4). The sensitivity of the net income of cotton is similarly very low since the hydraulic simulations showed that the expected yield of the crop is rather high in all the scenarios. A total loss of the crop due to water-delivery-related problems is therefore unlikely. (It should be noted that the possible loss of the cotton crop due to agricultural-related problems is beyond the scope of this research. However, such factors can easily be included in the analysis. For example, if it was forecast that the production of cotton will be low for other reasons, the previous sensitivity analysis will suggest that canal desilting may not be carried out before those problems are solved.)

7.4.5 Long-term Expenditure Planning

The financial analysis of the sediment removal alternatives presented above is suitable for making decisions on a yearly basis. For long-term expenditure and asset management planning (AMP), decisions regarding future expenditure on the assets need to be considered over a period of several years. The following analysis demonstrates the application of the output from hydraulic modelling and the other financial analyses presented in previous sections to long-term expenditure planning.

It has been shown in previous sections that some degradation to the performance of system *A* took place when 20% sedimentation existed in the canal network (Figure 7.4). When the sedimentation increased to 30%, the performance levels were significantly reduced. A scenario can therefore be foreseen where the management of an irrigation system needs to plan the desilting of the canals in their system. If, for example, the rate of sediment deposition in the canals is constant each year at 10%, then two possible desilting plans may be considered. Plan 1: to clean all the sediment whenever the sedimentation exceeds 20% (i.e. desilt the canals once every three years) and plan 2: to clean all the sediment whenever the sedimentation exceeds 30% (i.e. desilt the canals once every four years). The two plans are presented graphically in Figure 7.5. In the figure, year 0 can be the first year of operation after a full system rehabilitation where all the canals are clean of any sediment (sedimentation is 0%) and there is no cost of canal desilting incurred in this year. As the sediment deposits at 10% per year, desilting will be required in years 3, 6, etc. according to plan 1; or in years 4, 8, etc. according to plan 2. The objective of the expenditure planning exercise is to decide which plan to adopt based on financial merits.

The financial analyses of the costs and benefits that will be incurred in each desilting plan are given in Tables 7.5 & 7.6. These are related to the case of system *A* being automatically operated. Because the desilting operation has different frequencies in each plan (three/four years), the analysis of plan 1 covers a period of 14 years while that of plan 2 covers a period of 15 years. The costs considered in the analyses were mainly the costs of desilting the canals, while the benefits were the total net income of the scheme according to the crop budgets and estimated total productions in each scenario. All prices were discounted using 10% discount rate. The *net present benefit* and the *annual equivalent benefit* of each plan were used as the financial indicators of the analyses.



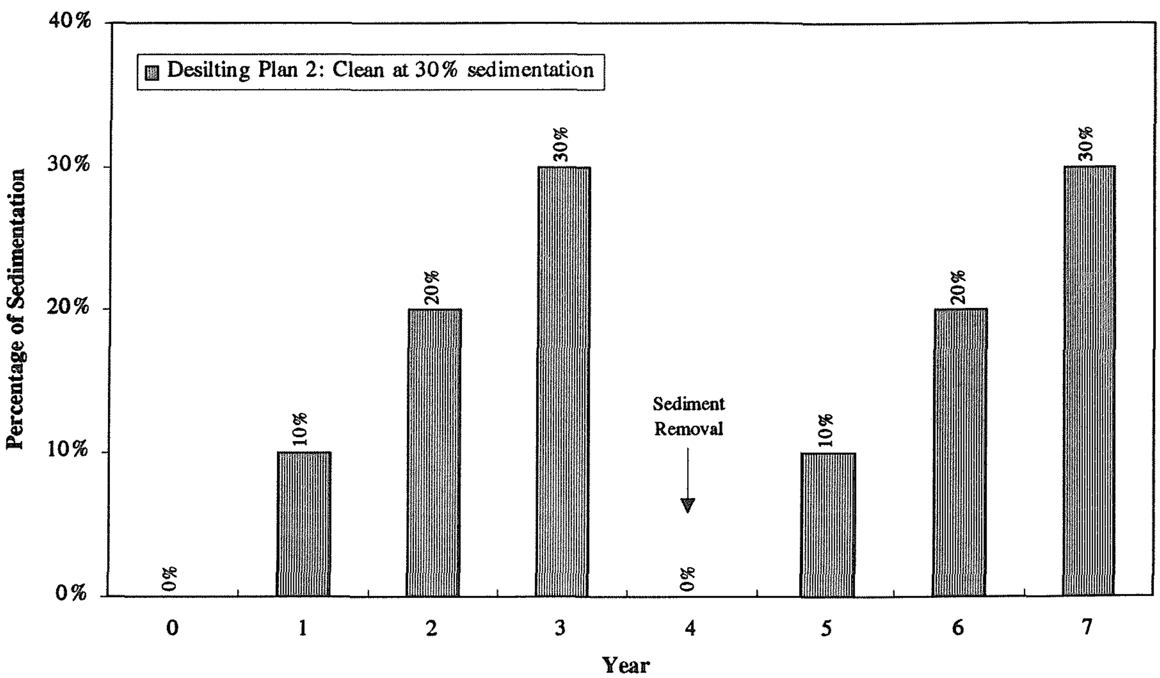


Figure 7.5 Possible sediment removal arrangements according to two different plans

Table 7.5 Financial analysis of desilting plan 1: clean at 20% sedimentation — system *A* under automatic operation

Year	Sediment (%)	Cost of Desilting (1998 USD) *	Total Net Income (1998 USD) §	Net Benefit (1998 USD)
0	0%	0	4,191,320	4,191,320
1	10%	0	4,191,320	4,191,320
2	20%	0	3,895,328	3,895,328
3	0%	213,331	4,191,320	3,977,989
4	10%	0	4,191,320	4,191,320
5	20%	0	3,895,328	3,895,328
6	0%	213,331	4,191,320	3,977,989
7	10%	0	4,191,320	4,191,320
8	20%	0	3,895,328	3,895,328
9	0%	213,331	4,191,320	3,977,989
10	10%	0	4,191,320	4,191,320
11	20%	0	3,895,328	3,895,328
12	0%	213,331	4,191,320	3,977,989
13	10%	0	4,191,320	4,191,320
14	20%	0	3,895,328	3,895,328
Net present benefit at 10% discount rate (1998 USD)				**33,880,140**
Annual equivalent benefit at 10% discount rate (1998 USD)				**4,599,101**

* See Table 7.3

§ See Table 7.2

Table 7.6 Financial analysis of desilting plan 2: clean at 30% sedimentation — system *A* under automatic operation

Year	Sediment (%)	Cost of Desilting (1998 USD) *	Total Net Income (1998 USD) §	Net Benefit (1998 USD)
0	0%	0	4,191,320	4,191,320
1	10%	0	4,191,320	4,191,320
2	20%	0	3,895,328	3,895,328
3	30%	0	2,755,579	2,755,579
4	0%	319,997	4,191,320	3,871,323
5	10%	0	4,191,320	4,191,320
6	20%	0	3,895,328	3,895,328
7	30%	0	2,755,579	2,755,579
8	0%	319,997	4,191,320	3,871,323
9	10%	0	4,191,320	4,191,320
10	20%	0	3,895,328	3,895,328
11	30%	0	2,755,579	2,755,579
12	0%	319,997	4,191,320	3,871,323
13	10%	0	4,191,320	4,191,320
14	20%	0	3,895,328	3,895,328
15	30%	0	2,755,579	2,755,579
Net present benefit at 10% discount rate (1998 USD)				**32,334,888**
Annual equivalent benefit at 10% discount rate (1998 USD)				**4,251,190**

* See Table 7.3

§ See Table 7.2

According to the net present benefit and the annual equivalent benefit, the plan which achieves the highest figures is the best from the financial point of view. However, since the two plans under investigation have different analysis periods (14 years for plan 1 and 15 years for plan 2), the annual equivalent benefit is the correct indicator to use in such a case[11] (Cassimatis, 1988). The results in Tables 7.5 & 7.6 indicate that plan 1 is financially better than plan 2 since its annual equivalent benefit is \$347,911 more than that of plan 2. This is equivalent to an increase in the annual benefit of scheme *A* of about 8% for each year during the analysis period if plan 1 is implemented instead of plan 2. The net present benefit estimates that plan 1 can achieve an additional benefit of around \$1.5 million over the whole analysis period.

The same analyses were repeated for the case when system *A* is manually operated. Since the methodology of the analysis is the same as this presented above, only a summary of the final results is given in Table 7.7. The figures in the table indicate that plan 1 is still financially more rewarding than plan 2 when system *A* is manually operated, although the net benefit will in this case be smaller than the benefit when the system is automated.

Table 7.7 Summary of the financial analyses of two desilting plans — system *A* under manual operation

Plan	Financial Indicator	Value (1998 USD)
1 (clean at 20% sedimentation)	Net present benefit at 10% discount rate	31,520,950
	Annual equivalent benefit at 10% discount rate	4,278,850
2 (clean at 30% sedimentation)	Net present benefit at 10% discount rate	30,193,068
	Annual equivalent benefit at 10% discount rate	3,969,597

7.4.6 Returns to Water

Calculating the returns to water is often one of the objectives of irrigation project

11 Because the analysis periods of the two plans are relatively similar, both the net present benefit and the annual equivalent benefit give the same indication (i.e. plan 1 is better) in this case.

evaluations. Irrigated agriculture is generally accused of not achieving high efficiencies in utilising the resources it uses, especially water. In the competition for the increasingly scare resource between different sectors, the return to water is often used as one of the indicators for comparing the performance of the different sectors.

The most common ways of calculating the return to water of a process is by comparing the quantity of the water used in that process to the mass and/or price of its final product. Since the final product of irrigated agriculture is crops, the return to water is calculated as the quantity of agricultural production per cubic metre of irrigation water (kg/m^3) or the value of agricultural production per cubic metre of irrigation water ($\$/m^3$). In the latter definition, the analysis is analogous to cost-benefit analysis with the "cost" being the quantity of the irrigation water used.

Table 7.8 gives the values of the return to irrigation water in the scenarios of sediment removal from system *A* (Table 7.1). The total net income of the scheme is calculated based on estimated agricultural production (the steps of the calculations have been given in earlier sections of this chapter). The total quantities of irrigation water supplied to scheme *A* in the scenarios are easily calculated from the output of the hydraulic simulations. Yet, two values for the return to water are given for some scenarios in the table. The *Return to Supplied Water* is calculated by using the actual quantity of irrigation water that was supplied to the scheme in each scenario. This indicator therefore takes into consideration the reductions in the actual supply below the full design supply due to sedimentation in some scenarios such as Sed20-32 and Sed30-32. The *Return to Full Supply* is the return to water if the full design supply had still been allowed in those scenarios, for example, because there were no other opportunities for utilising the saved water in other areas or other uses. Consequently, this indicator can be used to give indications of the benefits foregone in certain scenarios due to water wastage.

Table 7.8 The return to irrigation water in the scenarios of selective sediment removal from system *A*

Scenario	Total Net Income * (1998 USD)	Actual Scheme Irrigation Supply		Return to Supplied Water (USD/m^3)	Return to Full Supply (USD/m^3)
		(m^3/Year)	(mm/Year)		
Optimum	$4,191,320	112,981,262	1,165	$0.037	--
Automatic operation					
Sed20-32	$3,895,328	108,676,391	1,120	$0.036	$0.034
Sed30-32	$2,755,579	99,271,371	1,023	$0.028	$0.024
Sed30-34	$3,836,535	112,981,262	1,165	$0.034	--
Sed30-39	$4,088,342	112,981,262	1,165	$0.036	--
Manual operation					
Sed20-52	$2,961,994	109,869,307	1,133	$0.027	$0.026
Sed30-52	$2,627,228	99,305,948	1,024	$0.026	$0.023
Sed30-54	$3,272,599	112,981,262	1,165	$0.029	--
Sed30-59	$3,259,828	112,981,262	1,165	$0.029	--

* See Table 7.2.

Although the figures in Table 7.8 are generally comparable to figures from schemes in similar conditions worldwide (Molden et al., 1998), they are slightly lower than published figures due to the way the calculations in the table were made. Firstly, most published figures are based on the gross value of agricultural production, not on the net value of production as is the case in Table 7.8. Secondly, the quantity of available rainfall and the different water conveyance and application efficiencies are all important factors which can greatly affect the quantity of actual irrigation water supply and hence the return to water. The arid climate of scheme *A* (around 115 mm of rainfall per year) and the low conveyance and irrigation efficiencies meant that large volumes of irrigation water were required, thus lowering the return to irrigation water. This second point is rather important when comparing between the returns to water in schemes which are located in different climatic conditions.

Because the numerical values of the returns to water in Table 7.8 were quite small, the figures were provided down to the third decimal, i.e. to a fraction of the US cent. However, such accuracy was also required since the total volumes of irrigation water supply were large and hence a fraction of a cent can make a difference in the numerical value of the return to water from the whole scheme.

The figures show that the return to water is lower in the case of manual operation than it is in the case of automatic operation (with all other factors being constant). In either mode of system operation, the return to water remains relatively constant in most of the simulated scenarios except those with 30% sedimentation (scenarios Sed30-32 & Sed50-32). A significant reduction in the return to water might also occur if the full design supply was allowed when the system had high percentages of sedimentation (scenarios Sed30-32 & Sed50-32).

7.5 Financial Analysis of the Expenditure on Control Structures

As a continuation of the financial analysis of the scenarios tested using hydraulic modelling techniques, the following sections deal with the financial analysis of the expenditure on canal control structures. The hydraulic analysis of the problem of malfunctioning canal regulators has been covered in Chapter 6. The financial analyses presented in the following sections refer to the simulations in Section 6.2 by using the same name codes used in that section.

7.5.1 Cost Estimation

The parameters involved in the current financial analyses are mainly the costs of the repairs to the canal regulators and the income of scheme *A* according to the estimated crop productions in each scenario. The procedure of estimating the crop budgets and the net income of scheme *A* has been presented in Section 7.4. The costs of the repairs to the canal regulators have been obtained from contract documents of two rehabilitation projects in

Africa[12]. According to the scenarios investigated and the types of structures in system *A*, only two types of canal regulators were covered in the simulations. These are undershot sluice regulators and gated weirs. It was assumed in all the scenarios that the structures were inoperable and that all of them were fully open all the time due to failure to the gates of the structures. Consequently, the repair costs used were for the removal and disposal of existing gates and supply, erection, painting and testing of replacement gates for the regulators. The costs, after being converted from local currencies to 1998 USD, are as follows:

- sliding gates for sluice regulator = $1,530
- movable gate for gated-weir regulator = $2,850

7.5.2 Cost-Benefit Analysis

Tables 7.9 & 7.10 summarise the results of the cost-benefit analyses of the simulation scenarios presented in Section 6.2. Table 7.9 includes the cost-benefit analysis of the expenditure on head regulators, while Table 7.10 covers the cost-benefit analysis of the expenditure on cross regulators. A distinction between the cases of fully automated and manually operated systems is also made in the tables. The gross return to maintenance of a scenario is the expected return to the expenditure on repairing the structures after the repairs have been done. It is calculated as the difference between the net income of the control scenario (Str01m or Str01a as appropriate) and that of the scenario under investigation.

Generally, both tables show that the expenditures on the repairs/rehabilitation of canal head and cross regulators are highly rewarding, with exception of some scenarios. While the costs of the repairs are not significant, the impact on performance and hence on the production of the scheme is measurable. More specifically, the following points are important to note:

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12 The references to these documents are not published here based on the request of the supplier, but can be provided on request.

Table 7.9 The financial viability of the expenditure on selected head regulators in system *A*

Scenario	Total Net Income (1998 USD)	Potential Gross Return to Maintenance (1998 USD) *	Cost of Repairs (1998 USD)	Benefit/Cost Ratio
Automatic Operation				
Str01a	4,105,043	0	0	--
Str03a	3,437,109	667,934	1,530	436.6
Str04a	3,811,702	293,341	1,530	191.7
Manual Operation				
Str01m	2,880,760	0	0	--
Str03m	2,815,453	65,307	1,530	42.7
Str04m	2,927,156	-46,396	1,530	-30.3

* If the structures are repaired.

- The gross returns to maintenance in some scenarios, such as Str04m and Str02m, were negative and so were the benefit-cost ratios of those scenarios. This means that those scenarios, with some structures malfunctioning, performed better than the control scenario, with all the structures operating properly. In other words, no structure repairs should be carried out in such cases. This behaviour only occurred in some of the scenarios simulating system *A* under manual operation. This is partly related to the fact that the performance of the control scenario (Str01m) itself was low. As has been explained earlier in Section 6.2.1, the performance of the control scenario Str01m was poor because the gates of the field outlets were fixed at full supply settings throughout the whole simulation, thus the water distribution equity was not high at low supply percentages leading to low crop yields.

  Since this was not the case with the control scenario simulating automatic operation (Str01a), none of the scenarios simulating system *A* under automatic operation exhibited the same behaviour (see Tables 7.9 & 7.10).

Finally, this is one of the examples demonstrating the danger of depending on the output from mathematical models alone in making sensitive decisions without verifying them experimentally or in the field.

Table 7.10 The financial viability of the expenditure on selected cross regulators in system *A*

Scenario	Total Net Income (1998 USD)	Potential Gross Return to Maintenance (1998 USD) *	Cost of Repairs (1998 USD)	Benefit/Cost Ratio
Automatic Operation				
Str01a	4,105,043	0	0	--
Str02a	3,967,049	137,994	2,850	48.4
Str10a	4,030,310	74,733	1,530	48.8
Str12a	3,994,056	110,987	1,530	72.5
Str13a	4,027,380	77,663	1,530	50.8
Manual Operation				
Str01m	2,880,760	0	0	--
Str02m	2,903,215	-22,456	2,850	-7.9
Str10m	2,870,854	9,906	1,530	6.5
Str12m	2,888,985	-8,225	1,530	-5.4
Str13m	2,887,111	-6,352	1,530	-4.2

* If the structures are repaired.

- With all other factors being equal, the expenditure on canal head regulators are more rewarding than the expenditure on cross regulators. The analysis shows that a malfunctioning canal head regulator can have a greater impact on the water management and production of an irrigation system than a malfunctioning cross regulator. Hence, head regulators should be given higher priorities than cross regulators.

- The relative locations of head regulators within an irrigation system are important factors to take into consideration when prioritising the expenditure on those regulators. For example, Table 7.9 shows that in case of system *A* under automatic operation the benefit/cost ratio of scenario Str03a (repairing the malfunctioning head regulator of a top-end canal) was 2.2 times the benefit/cost ratio of scenario Str04a (repairing the malfunctioning head regulator of a tail-end canal). Cross regulators, on the other hand, are less or not sensitive to this factor.

- Fully automated systems are more sensitive to structure malfunctioning than manual ones. For example, in scenarios Str10a and Str10m (malfunctioning of the first cross regulator on canal M1C1 in system *A*) the losses in the income of the scheme if the cross regulator was not repaired were estimated as $74,733 and $9,906 in the automatic and manual operation modes respectively (Table 7.10). In other words, the automated system might lose around 7.5 times more money than the money which the manual system might lose. The expenditure on the repairs of automated systems should therefore be timely if production losses are to be avoided. This point is investigated further in Section 7.5.4.

7.5.3 Long-term Expenditure Planning

The cost-benefit analyses presented in the previous section combined with the results of the hydraulic simulations of those scenarios (see Section 6.2) can be implemented in long-term planning of the expenditure on canal regulators. The principal idea will be similar to the application to sediment removal alternatives as presented in Section 7.4.5. The results of the hydraulic simulations will be used to assess the hydraulic performance of the scenarios and estimate the potential crop yields of the scheme. The monetary value of the crop yields can then be determined and compared to the costs of necessary structure repairs or replacements. Numerous scenarios are possible, for example, whether to carry out a major repair to one cross regulator or minor repairs to two or more head regulators if funds are limited. The financial analyses should cover reasonable periods of time, preferably longer than the lives of the assets being investigated. Because of the similarity of the calculations in this case to those demonstrated in Section 7.4.5, no other numerical examples are presented in this section.

7.5.4 The Viability of System Automation

The investigation of the impact of the malfunctioning of canal regulators on the performance and production of the case study (system *A*) showed that there is a significant difference between the cases of manual and fully automatic operation. The results of the hydraulic performance assessments of the scenarios investigated in Section 6.2 as well as the financial analyses of those scenarios (Section 7.5) lead to the same conclusion. A final check before a definitive decision can be made is to analyse the financial viability of the investment on upgrading the system.

According to the simulation scenarios presented in Chapter 6, the main difference between the manual and the fully automated operations is in the operational procedure of the field outlet structures. In the scenarios simulating the manual operation, the gates of the field outlets were kept fully open at full design discharge settings throughout the whole simulations. For the simulation of the automatic operation, the gates of the field outlets were adjusted according to the actual crop water requirements and the flow available in the system. Consequently, a comparison between the finances of the two modes of operation can be made by comparing the difference in the potential net incomes of the case study under the two modes of operation to the cost of automation.

The potential net income of system *A* under ideal conditions in the cases of manual and automatic operations have been estimated from the outputs of scenarios Str01m and Str01a respectively (Table 7.10). The cost of automation is mainly the cost of replacing the manual field outlet structures with automatic/semi-automatic alternatives such as baffle distributors. The costs were obtained from a contract document for the rehabilitation of a scheme in Africa and then converted from the local currency to 1998 USD:

- Supply, erection, painting and testing of double baffle distributor for lateral turnouts including forming of box-outs in concrete work, levelling, fixing in position and building into existing structures complete with all embedded parts and fixings = $3,000
- Total number of field outlets to be upgraded in system *A* = 76 units

- Total cost of upgrading = **$228,000**

A cost-benefit analysis of the scenario over a 30 year period is given in Table 7.11. The analysis is based on relative prices instead of absolute ones, i.e. the benefits and costs used in the analysis are the differences between the benefits and costs which will be incurred in the two modes of operation. The baffle distributors are estimated to have an average useful life of 15 years (Verdier & Millo, 1992) and therefore during the 30 years of the financial analysis, the distributors will need to be replaced once in year 16. The relative costs in all other years were taken as zeros considering that there will be no significant difference between the costs of the regular maintenance of the manual structures and the baffle distributors.

The net present relative benefit of the scenario was used as the financial indicator for testing its viability. The analysis clearly shows that the automation option is highly viable in this case (the numerical value of the net present relative benefit is greater than zero, Table 7.11). An analysis of the sensitivity of the scenario to changes in the costs of the baffle distributors showed that the costs of the distributors will have to increase 45 fold in order to lower the net present benefit to zero (break-even situation) which means that the upgrade is unlikely to be unprofitable.

Table 7.11 The financial viability of automating the field outlet structures in system *A*

Year	Total Net Income (1998 USD)		Relative Net Income (1998 USD)	Relative Cost (1998 USD)	Relative Benefit (1998 USD)	Present Value of Relative Benefit (1998 USD)
	Manual System	Automated System				
1	2,880,760	4,105,043	1,224,283	-228,000	996,283	905,712
2	2,880,760	4,105,043	1,224,283	0	1,224,283	1,011,804
3	2,880,760	4,105,043	1,224,283	0	1,224,283	919,822
4	2,880,760	4,105,043	1,224,283	0	1,224,283	836,202
5	2,880,760	4,105,043	1,224,283	0	1,224,283	760,184
6	2,880,760	4,105,043	1,224,283	0	1,224,283	691,076
7	2,880,760	4,105,043	1,224,283	0	1,224,283	628,251
8	2,880,760	4,105,043	1,224,283	0	1,224,283	571,137
9	2,880,760	4,105,043	1,224,283	0	1,224,283	519,216
10	2,880,760	4,105,043	1,224,283	0	1,224,283	472,014
11	2,880,760	4,105,043	1,224,283	0	1,224,283	429,104
12	2,880,760	4,105,043	1,224,283	0	1,224,283	390,094
13	2,880,760	4,105,043	1,224,283	0	1,224,283	354,631
14	2,880,760	4,105,043	1,224,283	0	1,224,283	322,392
15	2,880,760	4,105,043	1,224,283	0	1,224,283	293,084
16	2,880,760	4,105,043	1,224,283	-228,000	996,283	216,820
17	2,880,760	4,105,043	1,224,283	0	1,224,283	242,218
18	2,880,760	4,105,043	1,224,283	0	1,224,283	220,198
19	2,880,760	4,105,043	1,224,283	0	1,224,283	200,180
20	2,880,760	4,105,043	1,224,283	0	1,224,283	181,982
21	2,880,760	4,105,043	1,224,283	0	1,224,283	165,438
22	2,880,760	4,105,043	1,224,283	0	1,224,283	150,398
23	2,880,760	4,105,043	1,224,283	0	1,224,283	136,726
24	2,880,760	4,105,043	1,224,283	0	1,224,283	124,296
25	2,880,760	4,105,043	1,224,283	0	1,224,283	112,996
26	2,880,760	4,105,043	1,224,283	0	1,224,283	102,724
27	2,880,760	4,105,043	1,224,283	0	1,224,283	93,385
28	2,880,760	4,105,043	1,224,283	0	1,224,283	84,896
29	2,880,760	4,105,043	1,224,283	0	1,224,283	77,178
30	2,880,760	4,105,043	1,224,283	0	1,224,283	70,162
Net present relative benefit at 10% discount rate (1998 USD)						11,284,322

7.6 Summary and Conclusions

With the rapid increase in the cases of irrigation-system turnover and privatisation in the past few decades, irrigated agriculture is being viewed as a business rather than a burden on public funds. As is the case with any business, the planning of expenditures and investments includes carefully studied financial analyses in order to test their returns.

A methodology for quantifying the costs and benefits of expenditure and investment scenarios on irrigation infrastructure has been presented in this chapter. The methodology utilises the output of hydraulic modelling for the quantification of the potential returns to expenditure/investment scenarios or the benefits which might be foregone if those scenarios are not implemented. Since the research adopts the multi-criterion decision analysis approach in assessing the overall impact of a scenario on performance, only the benefits that can be readily priced in monetary units have been included in the cost-benefit analyses. In all the scenarios investigated, the main benefit was the change in the potential agricultural production of the case study. Other benefits which are not quantifiable in monetary units, such as the change in hydraulic performance, have been assessed using appropriate performance indicators and have not been included in the cost-benefit analyses.

The outcome of the research shows that the proposed methodology of using hydraulic modelling techniques in the financial analysis of expenditure and investment scenarios is feasible. The analysis of some typical examples of alternative expenditure scenarios, such as those for removing the sediment from a canal network, demonstrated the application of the methodology in realistic cases.

Some findings regarding the prioritisation of expenditures on different types of irrigation infrastructure have been reached. However, those findings might be specific to the case study used in the research and therefore might not be valid for other cases. The emphasise is therefore on the methodology of the analysis rather than the specific results of the cases presented.

8. Methodology for Expenditure Planning

8.1 Introduction

The development of a methodology for using hydraulic modelling techniques for linking infrastructure condition to changes in performance has been presented in the previous chapters. Although the development of the methodology and the demonstration of some of its potential applications have been carried out through the investigation of some selected problems, the methodology is applicable to other problems. The purpose of this chapter is twofold:

(i) to outline and briefly describe the main general steps that should be followed when applying the methodology to any of the problems which it can handle; and,

(ii) to present how the methodology may be used for expenditure and asset management planning of irrigation infrastructure.

8.2 Outline of the Methodology

1) Select hydraulic modelling software using the following criteria:
   - Data units supported
   - Network size and layout
   - Canal sections and roughnesses
   - Structure library
   - Structure automation
   - Estimation of the initial conditions
   - Input data editing
   - Output extraction and presentation
   - Customer support
2) Collect data of the system to be modelled. The main data required is:

- Canal cross sections and roughness
- Control structure details (structure types, dimensions, design discharges, water levels, discharge coefficients, etc.)
- Environment data (climate, crops, etc.)
- System operational procedure

3) Set up performance assessment procedure:
   - Recommended performance criteria: equity, adequacy, variability, water level/free board and productivity.
   - Suitable indicators: Delivery Performance Ratio (DPR), Interquartile Ratio (IQR), Coefficient of Variation (Cv), Ratio of Lost Freeboard (LFb) and crop yield.
4) Define the control case and the target performance
5) List and screen the irrigation-infrastructure related cases
6) Model the identified cases

8.3 Description of the Methodology Steps

1) Select hydraulic modelling software

The first step in applying the methodology is the selection of the hydraulic modelling software which will be used in the simulations. Most of the currently available hydraulic modelling software have more or less the same capabilities and features. Nevertheless, it is important that the model selection guidelines in Section 4.3.1 are consulted before a model is chosen. Some training on the general use of the model will be required if no experience with it is already available.

2) Collect data of the system to be modelled

The data required for the developed methodology can be categorised as:

(i) Data required for hydraulic modelling: Section 4.2.3 lists the general data which is likely to be required for most hydraulic modelling work. The exact data required may however vary according to the specific requirements of hydraulic modelling software which will be used in the analysis.

(ii) Data required for cost-benefit and other financial analyses.

The following is a summary of all the data required:

- canal network layout including cross section dimensions, roughness, design discharges and water levels (down to field outlet level);
- control structure details such as structure types, dimensions, design discharges, water levels, discharge coefficients, etc (down to field outlet level);
- environment data including climate and rainfall;
- crops, cropping patterns, potential crop yields and area served by each field outlet;
- system operation procedure; and,
- cost of various system maintenance and structure replacement work and the revenue of the irrigation scheme (crop prices, water price, etc.).

3) Set up performance assessment procedure

Set up the procedure which will be used for assessing the performance of the investigated scenarios. The performance assessment framework outlined in Section 3.2 should provide some guidance in the process. The setup will typically include the selection of the measures (criteria) and indicators which will be used for performance assessment. A typical set of performance measures and indicators is listed in Table 8.1. However, it is recommended that this set is checked and modified if necessary according to the exact environment and the objectives of the scheme under consideration.

Most of the performance indicators which assess the performance at the irrigation system level can be worked out from the output of hydraulic modelling. Nevertheless, it is important to check that this is the case with the indicators and the model selected. If some of the selected indicators cannot be assessed from the output of the selected hydraulic modelling software, either those indicators or the hydraulic model should be replaced.

Table 8.1 **Recommended performance measures and indicators**

Performance Measure	Performance Indicators
Equity	Delivery Performance Ratio (DPR) and Interquartile Ratio (IQR)
Adequacy	Delivery Performance Ratio (DPR) Crop Yield
Variability	Delivery Performance Ratio (DPR) and Coefficient of Variation (Cv)
Water level/Freeboard	Ratio/Percentage of Lost Freeboard (LFb)
Productivity	Crop Yield

4) Define the control case and the target performance

A control case is required as a reference against which the performance of other scenarios can be compared. The control case will represent the irrigation system in its *ideal condition* (not necessarily its design condition) which should achieve its targeted objectives. In particular, the control case should define the following *targets*:

- the acceptable levels of service/performance;
- the maximum potential yield of the main crops in the scheme; and,
- the maximum potential revenue of the scheme according to crop prices, the price of water (if any), etc.

Each of the performance criteria selected in the previous step should have an acceptable level of service which will be used as a reference. For example, if water distribution equity is important, a definition of good/acceptable equity will be required. This might be, for example, that the differences between the quantities of the water delivered to the fields should not exceed 15% at all times. If the Interquartile Ratio (IQR) of the quantities of delivered water is the indicator used to assess equity, the previous definition of acceptable level of service can be translated to:

IQR:	1.0	1.15	>
Equity:	good/acceptable	|	poor/unacceptable

Because the levels of service and other targets are usually determined based on the demands of the system beneficiaries (the farmers), it is important to check that they are achievable when the irrigation system is in *ideal condition*. For instance, a target may be set too high that it cannot be achieved even if the infrastructure is in perfect condition because the technology used in the system simply cannot achieve it.

A system is said to be in ideal condition when all the components of the irrigation network are functioning properly, whether they are in design conditions or not. For example, if the cross sections of the earth canals of an old irrigation system no longer have the design prismatic shapes but can still deliver the maximum water demands safely then the canals can be considered to be in ideal condition.

Hydraulic modelling can be used to check whether a system can achieve its targets or not as follows:

- Model the irrigation system under consideration assuming that all the infrastructure are functioning properly, even if this is not the current situation in the field (to model the system in its ideal condition).

- Simulate the operation of the system throughout a whole growing year. In supply-orientated systems, such as manual upstream control, this can be done by ranking the supply into the system as described in Section 6.2.1 and Appendix III. The supply ranking process has the advantage of reducing both the modelling time and the potential instability of hydraulic models when simulating unsteady flow.

- Ensure that the hydraulic model is set up to closely simulate the procedure of operating the canal control structures in the field (manual or automatic).

- Assess the hydraulic performance of the simulation using the performance criteria selected before.

- Determine the maximum potential crop yields of the scheme as follows:
  - Obtain the actual flow diverted to each field outlet in the irrigation system during the whole season from the output of the hydraulic simulation.

  - Using the actual flow diverted to each field outlet and the calculated crop water requirements, work out the potential yield of each main crop grown in the area served by the outlet using the guidelines of Doorenbos and Kassam (1979).

  - By summing up the yield of each crop from each field outlet, the total yield of that crop from the whole scheme can be determined.

- Determine the maximum gross revenue of the scheme based on the maximum potential yield and the net crop prices.

- Compare the performance of the simulation with the acceptable levels of service and other set targets. If the performance levels are lower than the acceptable levels of service then the irrigation system cannot achieve the set targets, even when all the infrastructure are functioning properly. Two possible solutions exist in this situation:
  - to rehabilitate/upgrade the irrigation system; or
  - to change the acceptable levels of performance and other targets to match those obtained from the simulation results.

5) List and screen the irrigation-infrastructure related cases

List all the cases which require irrigation-infrastructure interventions and hence expenditure or investment. The interventions can be either maintenance/rehabilitation works or modernisation and structure upgrading options. Screen all the cases to identify where hydraulic modelling techniques may be applicable using the classification guidelines given in Table 4.1. If necessary, quick hydraulic simulations can be used to test the potential impact of the cases which are otherwise difficult to classify. Modify the list to include only those cases which were identified as suitable and worth modelling.

6) Model the identified cases

Model each of the cases identified in the previous step as worth modelling using hydraulic simulation techniques. The general steps of setting up the hydraulic simulations are:

- Introduce the case (problem) to be investigated in the digital model of the irrigation system. For example, for studying sedimentation problems enter the new dimensions of the canal cross sections after sediment deposition into the model. Additionally, the roughness coefficients of the canal sections (usually Manning n) can be changed according to the impact of sedimentation.

- Depending on the potential impact of the problem, run the hydraulic model to simulate the operation of the irrigation system throughout a whole growing season/year or during shorter time periods when the impact is significant (e.g. during the months when the problem of vegetation in the canals is so acute).

- Assess the hydraulic performance of the simulated scenarios using the previously-selected performance indicators and determine the potential agricultural yield and scheme revenue. Compare the current performance with the acceptable levels of service and other set targets to *quantify* the impacts of the problem and link infrastructure conditions to performance levels. The outcome of the comparison can be one of the following cases:

    - The performance of the system after the problem is still within acceptable levels and therefore no action may be taken if the funds available are not sufficient and other problems are more urgent to cure.

    - The performance has degraded below acceptable levels and hence corrective action is necessary. Depending on the funds available, the corrective action can be to carry out either full structure maintenance/rehabilitation/upgrading or selective maintenance (e.g. remove the sedimentation from selected canal sections only). In the latter case, hydraulic modelling can be used to test the efficiencies of the proposed selective maintenance scenarios to see whether they can raise the performance levels up to the accepted standards and to check their financial viabilities. Rank the scenarios according to their performance such that the most effective scenario can be chosen for implementation.

The exact details of the hydraulic simulations will vary according to the nature of the problem under consideration. Detailed simulations demonstrating the application of the methodology in investigating the sedimentation problem in irrigation canals and control structure malfunctioning are presented in Chapters 5 and 6.

8.4 Application in Asset Management Planning

The primary objective of most asset management planning procedures is the planning of the capital expenditures required for infrastructure replacement or upgrading over a period of time. They achieve this by monitoring the deterioration of the asset conditions with time, either by means of routine field inspection or by establishing deterioration rates for the different asset types, such that the replacement time of each asset can be forecasted. The cost of replacing an asset is sometimes referred to as the *modern equivalent asset* (MEA). The capital expenditure part of an asset management plan is therefore the profile of expenditures on MEA over time. In this process, asset management planning has the following shortcomings:

- although the conditions of the assets are routinely monitored and assessed, no linkage between the conditions and system performance is established; and,

- the procedure of prioritising the expenditures in case of lack of funds depends mainly on "perceived" asset importance according to asset type instead of using a more theoretically-based approach.

These shortcomings can be overcome by using the expenditure planning methodology which has been developed in this research. The methodology can be used for establishing the linkage between asset conditions, functionality and performance. Such linkages can be used to quantify the improvement in performance due to infrastructure interventions and hence choose the best alternative which achieves the highest return.

The following steps outline the application of the research methodology in long-term expenditure and investment planning:

1) Agree on acceptable levels of service with the users of the irrigation system. These will usually include hydraulic performance levels (e.g. adequacy, equity, reliability, etc.).

2) Using as-built drawings and other design charts, study the performance of the irrigation system in its ideal condition using hydraulic modelling techniques. Evaluate the hydraulic performance using appropriate performance indicators. Quantify the net revenue of the scheme from the agricultural and other main outputs. If the ideal performance of the system does not comply with the agreed levels of service, then immediate action is required.

3) Assess the current physical condition of the irrigation infrastructure and determine the rates of condition deterioration, thus forecast their conditions during the future planning period.

4) Use hydraulic modelling to simulate how the system will perform at any future point of time according to the forecasted infrastructure conditions. Assess the hydraulic performance from the output of hydraulic modelling and estimate the total agricultural production of the scheme. Compare the results with the agreed levels of service. If the performance will be still within the agreed levels of service then no expenditures/investments will be required in that future time. Otherwise some

or all of the deteriorated structures will need to be maintained/upgraded. If it is the latter case, list all possible alternatives and test them using hydraulic modelling as appropriate. Select the most appropriate interventions and determine their costs.

5) Update the forecast of the future infrastructure conditions after implementing the interventions selected in the previous step (if any) and then repeat the previous step to plan the expenditures required in other future time periods.

6) Finally, evaluate the resultant profile of future expenditures/investments (asset management plan) to see if it is consistent with the general expenditure policy of the organisation running the scheme. If not, the plan may be modified by changing the future levels of service, the types of irrigation infrastructure (e.g. by upgrading or downgrading them), the materials used in construction, etc.

9. Conclusions and Recommendations

9.1 Conclusions

A multi-criterion methodology for analysing and planning the expenditure and investment on irrigation infrastructure has been developed in this research. The main objective of developing the methodology was to overcome the shortcoming in the subjective methods which are currently used for linking expenditure on irrigation infrastructure interventions to return and performance improvement. In this respect, the research has shown that hydraulic modelling is a better alternative for establishing the said linkage. In particular, the research has shown that:

1) Hydraulic modelling is a suitable and powerful tool which can be used in irrigation expenditure and asset management planning procedures for directly *establishing* and *quantifying* the impacts of changes in infrastructure conditions on the hydraulic performance of irrigation systems. The research has shown that hydraulic modelling is superior to existing methods because it quantifies the impacts on the canal system as a whole.

   Furthermore, the above direct output from hydraulic modelling can be used to indirectly assess:

   - the potential agricultural production at the agricultural system level; and,
   - the scheme revenue and hence return to expenditure/investment at the agricultural-economic system level.

   The assessment of all these impacts is essential for effective and optimised asset management planning.

2) Hydraulic modelling has significant value in the field of irrigation engineering. With more potential applications being explored, as has been done in this research, and the continuous advancement in hydraulic modelling software, the utilisation of hydraulic modelling is set to increase.

Various applications of the research methodology have been demonstrated. These include:

1) Analysing the costs and benefits of expenditure (maintenance/rehabilitation) and investment (modernisation) options in order to test their financial viabilities in relation to the level of performance and increased production in both the short and long term.

2) Prioritising expenditures (e.g. on selective maintenance) when sufficient resources are not available.

3) Evaluating alternative expenditure and investment strategies and ranking them based on some criteria using multi-criterion analysis such that the best alternative can be selected. For example, the research compared between the performance of manual and automated systems in all the scenarios investigated. Such an analysis can be useful for testing the viability and advantage of modernisation as an upgrading option.

Because hydraulic modelling can be demanding in the resources and time it requires, initial *screening* of the cases which need to be studied should be carried out. The purpose of the screening process is to identify the cases which are likely to have significant impact on performance and hence are worth studying using hydraulic modelling techniques. Sometimes, it might be difficult to decide whether an infrastructure-related problem/intervention will cause significant impact on performance or not without carrying out quick and crude simulation test runs first.

The development of the research methodology and the demonstration of its applications were achieved through the investigation of two main problems related to open-channel networks and regulator structures. However, the developed methodology is generic and can be applied to other identified infrastructure interventions in any irrigation scheme. For example, it is possible to apply the research methodology to closed irrigation systems. Some of the details of the methodology, such as the interventions to be studied and the performance indicators, may be different in these cases.

The role asset management and expenditure planning play in irrigated agriculture is on the increase as more and more irrigation systems are privatised or turned over. Nevertheless, these tools will also have a key role to play in publicly-funded irrigation schemes due to the pressure on governments to be more *efficient* and *productive* with expenditure on irrigation infrastructure. The methodology which has been developed in this research can be used successfully in conjunction with existing asset management procedures to solve this challenge.

9.2 Recommendations and Further Research

It is recommended that the developed methodology be initially applied by consulting firms which are involved in the design and rehabilitation of irrigation projects. The methodology can be used to develop priority lists for the typical maintenance activities which will be required for the infrastructure of rehabilitated and new irrigation systems. Additionally, estimations of the potential impacts of not carrying out those works can also be prepared. Such information should become standards in operation and maintenance manuals. It should offer guidance to those who are/will be responsible for running the schemes in planning the expenditures and other activities. Hydraulic modelling has almost become a standard tool for the design of new and rehabilitated irrigation systems and therefore extending its application should not prove difficult to engineering consultants.

The methodology should also be used in conjunction with existing asset management planning procedures to provide theoretically-based linkage between structure conditions, interventions and performance instead of the subjective methods which are currently used. Since most asset management planning procedures handle large quantities of data, they are usually computerised. Integrating the research methodology with those procedures should therefore be possible to implement.

A large collection of different performance indicators can be found in the literature. They are usually advocated as being 'standards' that are suitable for any performance assessment exercise. A close inspection of those indicators reveals however that only a few of them do truly qualify. Many others can be specific and therefore might not be useful to other situations apart from those for which they have been designed.

Nevertheless, in spite of the long list of performance indicators currently available, sometimes especially designed indicators might be needed to assess certain performance measures in some specific situations. The Sediment Removal Effectiveness is an example of a performance indicator which have been developed in this research to fulfil some of its particular needs. The indicator could not be readily found in the literature and may be useful to other studies.

It is suggested that only a short and generic list of performance indicators for evaluating the most common performance measures be adopted and standardised in the literature. Those indicators should be useful to those who do not have the expertise and the knowledge to develop the most appropriate indicators for whatever purpose they might be assessing the performance for. More specific indicators can always be developed to suit the requirements of particular studies and therefore their development should be left to those studies rather than being forced for standardisation, as has been the case in this research.

The developed methodology is regrettably not so simple to implement; which is the reason why it was suggested earlier that the methodology be initially applied by consulting firms. Firstly, there is the difficulty of using hydraulic modelling. Although hydraulic models are becoming more user friendly through the implementation of graphical user interfaces and the like, they are still considered as specialised computer software. They still require the users to have reasonable computer and hydraulics skills before they can use them successfully and efficiently. When considering the utilisation of ISIS Flow in this research (and in irrigation engineering in general) some modifications had to be made to the software in order to achieve the research targets. In addition, there is the direct cost of obtaining the software which is still significantly higher than the cost of many other pieces of software.

Secondly, hydraulic modelling was not the only software used in the analysis. Other auxiliary pieces of software and spreadsheet applications were especially developed to satisfy the requirements of this research. Consequently, they are essential for the application of the complete methodology. Although those pieces of software are available for no charge, they were not designed for public use and therefore may not be so easy to use. It is therefore foreseen that for the methodology of the research to be practically implemented, a complete and integrated computer package should be developed. This package should link the output from hydraulic modelling to other models for assessing hydraulic performance, preparing production and cost estimations and carrying out cost-benefit analyses. It is not recommended that hydraulic models be extended to cover these additional capabilities since they are already complicated enough. Instead, it can be suggested that hydraulic modelling software be modified such that they can "talk" easily to other pieces of software. Accordingly, other models can be developed to feed into and get information from hydraulic models for further data processing.

Hydraulic modelling techniques cannot be used to quantify the impacts of interventions in irrigation infrastructure which do not have hydraulic functions, such as roads and bridges. However, efficient asset management plans must be able to optimise the expenditure on these types of structures as well. Further research is required in order to establish methods for linking the condition of and changes to these types of infrastructure to overall performance and levels of service.

Appendices

Appendix I: ISIS

The main purpose of this appendix is to give an overview of the computer software which have been used in the modelling work carried out in this research. An overview of the simulation model ISIS is given first and then followed by an outline of some other pieces of software which have been developed by the author for the specific purposes of this work.

I.1 ISIS Flow

I.1.1 Outline

ISIS Flow is a modular computer program which can be used for modelling steady and unsteady flows in open channels and flood plains. Any sensible looped or branched open channel network can be modelled using the program (Halcrow & HR Wallingford, 1997).

The channel network is modelled by breaking it down into hydraulic components referred to as units. In addition to channels and flood plains, ISIS Flow contains units to represent a wide variety of hydraulic structures including several types of sluices and weirs, side spills and head losses through bridges. Closed conduits and culverts are represented by cross sections and several standard shapes are available. Other units include Reservoirs (to represent flood storage areas, for example) and Junctions.

Free surface (flow depths and discharges) is computed using a method based on the equations for shallow water waves in open channels — the Saint Venant equations. Two methods are available for computation of steady flow problems: the Direct Method and the Pseudo Time-stepping Method.

In ISIS Flow, the model external boundaries are represented as either flow-time, stage-time or stage-flow (rating-curve) relationships including specifying tide curves and hydrological boundaries.

The usual way to create, edit, run and view the results for ISIS is through the graphical user interface — ISIS Workbench. A 'manual' method of editing data, running a simulation and tabulating results is also provided.

I.1.2 Application Areas

The following are the main sectors in which ISIS has applications:

- Flood defence and river engineering
- Environmental studies
- Urban pollution management
- Flow forecasting
- Sediment control
- Catchment management
- Development control
- Irrigation canal design and operation

I.1.3 Features

Topographic data, including channel cross-sections, structure dimensions etc., are assembled into a data file. This data file represents the channel network system which may be very complex; it may be looped or branched and contain many interacting flow paths. The data file will also contain the appropriate boundary conditions at the upstream and downstream extents of the model.

River sections are entered as pairs of (x, y) points defining the sections. Manning equation is used for frictional resistance calculation. A cross section may have more than one Manning roughness coefficient; allowing variable cross-sectional and longitudinal resistance variations to be modelled easily.

External boundary conditions are required at all terminal points in a model. For subcritical flow, which is of primary interest, these boundary conditions are specified as a discharge-time, stage-time or stage-discharge (rating curve) relationships. The following are known to lead to a properly posed system of equations:

- discharge hydrograph upstream and stage hydrograph downstream,
- discharge hydrograph upstream and rating curve downstream.

Although other combinations may work in certain circumstances, the specification of a discharge hydrograph at the downstream end may lead to problems.

In a one-dimensional network representation of a river (or conduit), reaches are separated by internal boundaries which may be control structures, losses, reservoirs or junctions (bifurcations or confluences). These boundary conditions impose a relationship between the stages and discharges at the nodes involved. The internal boundaries can be categorized as follows:

- Control Structures: a wide variety of control structures is readily available in ISIS:

  i. Weirs: sharp crested, round nosed, crump, spill (jagged), notional, triangular profile, and gated weirs (time varying crest elevation); may operate in dry mode (no flow), or free or drowned mode according to the modular limit.

  ii. Sluices: vertical lift and radial gated sluices. They operate in a variety of flow modes including weir equations when the gates are out of the water and the obvious orifice type flow for normal sluice operations. ISIS Flow considers many possible flow modes for some of the sluice units including no flow, free and drowned weir flow, free and drowned gate flow, free and drowned flow over the top of a sluice gate and combinations of flow both under and over gates. It is possible to control automatically the opening of sluice gates during a run, for example according to pre-specified times, or according to upstream or downstream water levels, or to relate gate openings to water level at a remote node to simulate actions initiated by a flood warning, for example. Control can be based on complex logical rules. An add-on module enables the user to model SCAPA systems, including PID controllers.

- Reservoirs

- Losses: the discrete energy losses such as those caused by a sudden contraction or expansion in the channel can be represented by a Bernoulli loss which relates the head loss to the square of the upstream velocity head. Bridges can be modelled explicitly using either the US Bureau of Public Roads method or the Arch bridge method devised by HR Wallingford.

- Junctions (Bifurcations and Confluences): in ISIS Flow junctions are represented by simply equating water levels at the nodes of the junction and conserving mass by applying Kirchhoff's Law to the flows.

Unsteady flow in open channels is modelled using the Saint-Venant equations, which express conservation of mass and momentum. Conservation of mass leads to the *continuity* equation which establishes a balance between the rate of rise of water level and wedge and prism storages. Conservation of momentum leads to the *dynamic* equation which establishes a balance between inertia, diffusion, gravity and friction forces. Some other forces, such as the effect of wind or meanders, may also be included but usually these are small.

The continuity equation is written as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \qquad \textbf{(I.1)}$$

where Q = flow (m^3/s)

x = distance (m)

A = cross-sectional area of the flow (m^2)

t = time (s)

q = lateral inflow ($m^3/s/m$).

The momentum equation can be written as:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{\beta Q^2}{A}\right) + gA\frac{\partial H}{\partial x} - gAS_f = 0 \qquad \textbf{(I.2)}$$

where S_f = friction slope

$$S_f = \frac{Q|Q|}{K^2} \tag{I.3}$$

where K = the channel conveyance calculated according Manning's equation:

$$K^2 = \frac{A^2 R^{\frac{4}{3}}}{n^2} \tag{I.4}$$

where R = the hydraulic radius (m)

n = Manning's roughness coefficient ($s/m^{1/3}$).

It is not possible to solve the Saint-Venant equations analytically - hence the need for numerical solution. ISIS Flow employs the Preissmann implicit scheme - which is usually referred to as the four-point Box scheme. The scheme is outlined below.

Let f be the value of depth or discharge or a function of depth or discharge at point ($i+½$, $j+\theta$) as shown in Figure I.1. The value of f or its continuous derivatives with respect to time or space can be discretised as:

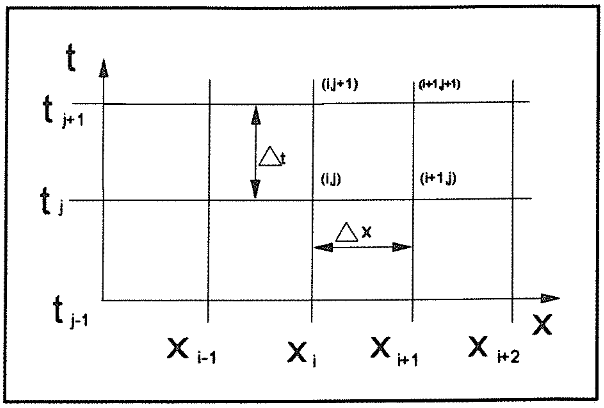

Figure I.1 Preissmann 4-point scheme

$$f(x,t) = \frac{1}{2}\left[\theta\left(f_{i+1}^{j+1} + f_i^{j+1}\right) + (1-\theta)\left(f_{i+1}^{j} + f_i^{j}\right)\right] \tag{I.5}$$

$$\frac{\partial f}{\partial x} = \frac{1}{2\Delta x}\left[\theta\left(f_{i+1}^{j+1} - f_i^{j+1}\right) + (1-\theta)\left(f_{i+1}^{j} - f_i^{j}\right)\right] \tag{I.6}$$

$$\frac{\partial f}{\partial t} = \frac{1}{2\Delta t}\left[\left(f_{i+1}^{j+1} - f_{i+1}^{j}\right) + \left(f_i^{j+1} - f_i^{j}\right)\right] \tag{I.7}$$

where θ = weighting factor ranging between 0.5 and 1

f_i^j = value of f evaluated at the point (x_i , t_j).

Using the above, both Saint-Venant equations can be transformed into the linear form:

$$aQ_i^{j+1} + bH_i^{j+1} + cQ_{i+1}^{j+1} + dH_{i+1}^{j+1} = e \qquad \textbf{(I.8)}$$

The values a, b, c, d and e are calculated for each iteration and each node in the open channel and depend on variables calculated at the previous iteration or time-step.

The coefficient matrix, which comprises largely of the a, b, c, d and e values described above must be inverted to solve the set of simultaneous difference equations for Q and H at the following iteration or time-step. ISIS Flow takes advantage of the banded structure of this matrix by employing a powerful sparse matrix solver.

To start an unsteady flow simulation, an estimate of the initial conditions (flow and stage) is required at every model node. This is most often obtained by carrying out a steady state run at the proposed start time. Two methods are available within the ISIS Flow program to compute steady flows, namely the Pseudo-time Stepping Method and the Direct Method.

The Pseudo-time Stepping Method uses the Preissmann four-point scheme described above. It requires initial guesses for flow and stage at each model node. These initial conditions are used for the steady state run and the boundary conditions are held constant at the time when the solution is required. The model is run until all the irregularities and inaccuracies in the guessed initial conditions have propagated or been dissipated out of the system. The monitoring parameters are the flow and head ratios. When these values become very small (usually less than 0.5×10^{-3}) and the time-step is large (usually greater than 200-500 seconds), then a steady solution should have been attained.

The Direct Method is faster and more accurate than the Pseudo-time Stepping Method and requires very little initial data. For steady state conditions the Saint-Venant equations can be reduced and written as ordinary differential equations; these are solved for individual reaches. The remaining problem is to solve the network so that Kirchhoff's law is satisfied and equal water elevations are obtained at junctions. This is done by an iterative scheme to solve the correction of the flow splits at channel confluences and bifurcations. Convergence is achieved when the maximum correction to a flow split is less than 0.1% and the maximum elevation difference is less than 1 mm at a junction.

The Direct Method deals accurately and consistently with the problem of cross section spacing. During the computation, the method checks whether the solution is "grid dependent", and if necessary will add extra interpolated nodes implicitly. The user is informed where this has been done so that extra surveyed sections can be added to the model if available, or extra nodes interpolated between the existing sections.

The time step of an unsteady run can vary from few seconds to minutes or even hours. During simulations, the program interface shows a progress meter and gives on-line information about the convergence status of the calculations. A run can be terminated at any time by the user if required.

I.1.4 Special Features

A special hydraulic feature in ISIS Flow is its capability on modelling supercritical flow. This is achieved by neglecting the part of the convective momentum term in the momentum equation when the Froude number exceeds a specified upper value. Between this upper value and a specified lower value, the term is gradually phased out so that a smooth transition is achieved.

For steady supercritical flow in a uniform channel, the method should be acceptably accurate but will become more approximate as the channel becomes more non-uniform.

This approach is adequate for problems where supercritical flow prevails locally in isolated areas of a model and when low flows are required as initial conditions for an unsteady run.

Additionally, ISIS Flow can relatively cope with dry-bed situations by introducing a minimum water depth when the depth approaches zero. However the model can be rather sensitive under these conditions.

I.1.5 Limitations

Since ISIS Flow solves the differential form of the momentum equation, the solution at a hydraulic jump or bore can never be accurate. Instead of a sharp change in stage, the change will be smeared over several nodes.

I.1.6 Data Entry and Output

The PC version of ISIS Flow has a graphical user interface (ISIS Workbench) that runs under Microsoft Windows. Through this interface the program can be controlled using standard Windows pull-down menus and a fast-access toolbar. The Workbench can be used to edit model data; view graphical presentations of model layouts, longitudinal sections, cross sections, time series graphs, and more. Graphs can be printed to the printer in draft mode or in engineering-style mode. Custom data can be entered into the program for comparison with modelled outputs. Comparisons between the results of two runs can be made in tabular or graphical form.

I.1.7 Developers

Halcrow Group Ltd.
Burderop Park
Swindon
Wiltshire SN4 0QD
UK

HR Wallingford Ltd.
Howbery Park
Wallingford
Oxfordshire OX10 8BA
UK

I.2 ISIS Sediment

I.2.1 Outline

The basic capabilities of the ISIS mobile bed module are to predict sediment transport rates, bed elevations and amounts of erosion/deposition throughout a channel system. In summary, this is achieved with the following calculations at each time step:

1) calculate the hydraulic variables of flow, stage, velocity in the usual way

2) starting at the upstream end of the system, loop around the nodes calculating the sediment transport capacity and solving the sediment continuity equation for depth of erosion/deposition

3) update the channel conveyance tables to allow for any calculated deposition or erosion ready for the next time step. A range of methods for updating the channel geometry are available: (i) no change in channel geometry (fully decoupled), (ii) move all section points uniformly by the Δz calculated for the particular section, (iii) move only those points at a section below water level uniformly by the Δz calculated for the particular section, and (iv) move those points below water level by a Δz distributed according to shear stress (scaling Δz at every data point).

Various options are available including: specification of dredging, cohesive sediment transport and rigid beds.

The main restrictions on the applicability of the software are that:

1) *local* effects may not be simulated (e.g. scour at bridge piers).

2) dunes and ripples are not explicitly modelled and therefore the effects of changes in form roughness on the hydraulic resistance is not simulated.

3) flow reversals are not accommodated at present.

4) reaches with zero flows are not permitted at present.

5) sediment transport computations in compound channels may be inaccurate due, in part, to cross section averaged velocities being unrepresentative of main channel velocities.

6) the maximum number of nodes in a mobile bed simulation is 250.

I.2.2 Sediment Continuity Equation

a. Channel Reaches

For channel units the form of the sediment continuity equation used is:

$$(1-\lambda)\, W \frac{\partial z}{\partial t} + \frac{\partial G}{\partial x} = 0 \tag{I.9}$$

where λ = bed porosity

W = water surface width (m)

z = bed elevation (m)

t = time (s)

G = sediment transport rate (m^3/s)

x = distance in flow direction (m)

The equation assumes that the rate of change of sediment entrained in the flow is negligible in comparison with the terms expressing rate of change in bed elevation and change in transport rate.

Equ. I.9 is discretised in the following form:

$$(1-\lambda)\, W_{k+1} \frac{\Delta z_{k+1}}{\Delta t} + \frac{G_{k+1}^{i+1} - G_{k}^{i+1}}{\Delta x} = 0 \tag{I.10}$$

where k = position index, which increases in the downstream direction

i = time index

Δz = change in bed elevation over time step Δt

Equ. I.10 can be solved explicitly for Δz as G_{k+1}^{i+1} is calculated by the sediment transport capacity equation and G_k^{i+1} will have been determined previously.

b. Boundaries

At upstream boundaries the user must specify the sediment inflow as rate against time, concentration against time or concentration against flow rate. The bed elevation is free to move at both the upstream and downstream boundaries (this includes the nodes immediately upstream and downstream of junctions and structures such as cross regulators).

At junctions the sediment outflow is equal to the sum of the sediment inflows (mass is conserved). All inflow node sediment transport rates are determined before the outflow nodes are calculated. If there is more than one outflow node then the concentrations at all outflows are assumed equal.

All hydraulic structures are considered simply as junctions with two nodes and thus the outflow sediment transport rate equals the inflow sediment transport rate.

I.2.3 Sediment Transport Equations

Four sediment transport equations are available: Engelund-Hansen (1967), Ackers-White (1973), revised Ackers-White and Westrich-Jurashek (1985) (details of the equations are available in the ISIS Sediment User Manual). All include a calibration factor which has the default value of unity to give the published form of the equations.

The revised Ackers-White equation is recommended in the absence of data to suggest that another equation is more accurate for that site. The Engelund-Hansen equation is also appropriate (the suggested applicability of the equation is for $\sqrt{(D_{75}/D_{25})} < 1.6$ and for a mean diameter greater than 0.15 mm). Use of the Westrich-Jurashek equation should be limited to cohesive sediments or for assessing deposition on rigid boundary channels. The 1973 Ackers-White equation should be used for sensitivity analyses or to ensure compatibility with earlier work using that equation.

The user can input a global sediment size distribution for the bed material, which consists of a table of size against proportion present of that size for between 1 and 10 sizes. The sediment transport is calculated for each size and the resulting total transport rate is calculated by multiplying the proportion of the size in the bed material by the calculated rate. The reported transport rate is the summation of the individual rates.

I.2.4 Results

The general results produced by the Sediment module include the following quantities:

(a) An overall sediment balance: the sediment mass entering the model, the mass leaving, the mass depositing and the mass dredged.

(b) The mass depositing converted to a volume, the total volume dredged and the overall volume change from the bed elevation changes. (The latter is found by summing the total volume between the river and the datum elevation at the beginning of the run and again at the end of the run.)

(c) Balance (a) repeated for each size fraction.

The time-varying results of the mobile bed run include:

- sediment transport rate (in m^3/s)
- bed elevation (in m)
- change in bed elevation during the last time step (erosion is negative and deposition is positive) (in m)
- sediment concentration (in ppm)
- net change in bed elevation (in m)

I.3 Problems Encountered

Most hydraulic simulation models are so complicated that it is almost inevitable that shortcomings, programming errors and other problems will be found. Using those models, as with most computer software, should therefore not be taken for granted. It is important that spot checks are done on several outputs of a model before it is used in any study.

Table I.1 includes a list of the important problems which were encountered while using ISIS in this research and the status of those problems up to date. All the problems which were caused by programming errors were reported by the author to the software developers and were fixed.

I.4 Auxiliary Software

Although ISIS Flow did all the main hydraulic simulations, much work was still required both before the simulations for preparing the input data files of the model and after the simulations for processing the output data. From the input data side, the nature of this research dictated that many changes to some of the data files were required for each simulation scenario. For example, changing the mode of operation of the simulated irrigation control structures from manual to automatic required a few changes to the input data of each structure. Since the number of control structures including the field outlets in the case study, system *A*, exceeded 100 the changes required in such a case could not be done by hand correctly and efficiently.

Similar efforts were required for processing the output data of ISIS Flow. The output of a simulation in ISIS Flow consists of the values of six hydraulic variables for each node in the simulated model and each simulation time step. With around 900 nodes in the model of system *A*, a very basic simulation which has just five time steps for example will produce around 27,000 figures in the output file of the simulation. The capabilities already built into ISIS allowed the abstraction of user-selected output data for tabular listing as well as graphical presentation. Nevertheless, processing of the output data, for example the calculation of the volume of water delivered to any node, was not available in ISIS. This necessitated the development of auxiliary pieces of software to cover the various needs of the research.

Table I.1 Problems encountered while using ISIS in this research

Problem Description	Impact	Corrected?
ISIS Flow could not simulate negative (reverse) flows over *Notional Weir* units.	Total model crash.	Yes
ISIS Flow did not set the gates of the *Gated Weir* units according to the *Move* instruction in the logical rules.	Simulations did not crash but the results were incorrect because the *Gated Weirs* were not set as was intended.	Yes
ISIS Flow reported incorrect *Rule* numbers in the output file.	Only the reported rule number was incorrect - ISIS Flow executed the correct rule and therefore only part of the output was incorrect. The rest of the output data was correct.	Yes
ISIS Flow cannot automatically simulate the operation of automated irrigation control structures.	Automation of control structures has to be simulated by using ISIS Control module or sets of logical rules which are not very easy to use. For example, it requires 20 or more logical rules to simulate the automation of one control structure during a whole season.	No

Table I.1 Problems encountered while using ISIS in this research

Problem Description	Impact	Corrected?
ISIS Flow cannot handle situations when the flow in any part of the network is zero or near zero.	Modelling the closure of irrigation canals as in the case of rotational flow is very difficult to implement.	No
Seepage losses cannot be simulated by ISIS Flow.	An approximate approach for simulating the effect of seepage losses is to estimate the quantity of the water lost from each canal reach and use an equivalent boundary condition to extract that quantity from the end of the reach.	No
ISIS Flow cannot readily evaluate any performance indicators from the output of the runs.	The evaluation of the performance indicators have to be done externally. This was done in this research by writing a special piece of software for carrying out the calculations.	No
ISIS Sediment stopped running shortly after starting sediment simulations without giving any warning.	Total model crash.	Yes

Problem Description	Impact	Corrected?
The capacity of ISIS Sediment (250 nodes) is much lower than the capacity of ISIS Flow (2000 nodes).	Large models, which can be modelled as one piece in ISIS Flow, have to be broken down into smaller ones if sediment modelling is required.	No

I.4.1 ISIS Mate

ISIS Mate is a collection of utilities which were developed by the author to satisfy the needs of this research in terms of providing greater flexibility in the manipulation of ISIS Flow input and output data. The package contains more than 15 utilities divided into four main groups for output processing, model building, control mode alteration and some generic utilities. The output processing group contains one of the most important utilities in the package for post processing ISIS output data. The utility facilitates exporting ISIS output data to spreadsheet applications for various sorts of analyses, uses ISIS output data to calculate some standard hydraulic performance indicators such as the Delivery Performance Ratio (DPR) and the coefficient of variation (Cv) of the flow delivery, and prepares output data for graphical presentation of canal longitudinal sections including water surface profiles and structure settings.

Although originally developed for the exclusive use of this research, ISIS Mate was purchased by the ISIS developers and is now distributed with the ISIS software.

I.4.2 Spreadsheet Applications

In addition to ISIS Mate various spreadsheet applications were developed mainly for processing ISIS output data. One of the examples worth mentioning is the calculation of the crop yields according to the functions of yield response to water (Doorenbos and Kassam, 1979). These calculations required an integration between ISIS Mate for abstracting the flow data from the output of the simulations and putting the data in a proper format to be fed into a spreadsheet which calculates crop yields.

The main parameter in the functions describing yield response to water is the ratio of the actual water available to a crop to the full water requirements of that crop. The full crop water requirements are dependent on factors such as the climate, the type of the crop and the planting date, and not dependent on the actual supply to the crops. Since all the simulations investigated in this research used the same case study, the crop water requirements of the main crops did not change from a simulation to another and therefore were fixed in the spreadsheet which calculated crop yields. On the other hand, the actual water available to the crops varied in each simulation and hence had to be fed into the spreadsheet after each run. The steps of the calculations procedure were as follows:

- A list of all the model nodes which represent the outlets to the fields was stored in a file for ISIS Mate to be able to read them.

- After each simulation, ISIS Mate read the output data of the simulation produced by ISIS, read the list of nodes which represent the field outlets and extracted the data of the discharge actually delivered to each field outlets during the whole simulation. The extracted data was saved in a format that was readable by the spreadsheet application.

- The discharge data was read and inserted into the proper fields in the spreadsheet which calculates crop yields using a specially written macro.

- The spreadsheet calculated the estimations of the yields of the four main crops for each of the 76 field outlets and exported the accumulated total crop yields to a summary sheet for final manipulation.

Appendix II: Review of Hydraulic Modelling Software

II.1 Types of Available Hydraulic Simulation Software

There is a wide variety of canal flow simulation programs that deal with the different types of flow in open channels (Lenselink & Jurriens, 1993). The main differences between those programs can be seen as the size of the irrigation network that can be simulated in one run, and the types of flow that a program can solve. While some programs handle single canal reaches, others deal with much more complicated networks that have many branches and loops. On the other hand, some programs can solve the uniform flow type only, while others can solve both the uniform and the unsteady flow types.

Eight hydraulic simulation models that can handle both steady and unsteady flow are reviewed below. The models are listed in alphabetical order.

II.2 BRANCH: Branch-Network Dynamic Flow Model

II.2.1 BRANCH: Developer

United States Geological Survey (USGS)
Hydrologic Analysis Software Team
R. Steven Regan
437 National Center
Reston, VA 20192
USA

II.2.2 BRANCH: Outline

The Branch-Network Dynamic Flow Model (BRANCH) is used to simulate steady or unsteady flow in a single open-channel reach (branch) or throughout a system of branches (network) connected in a dendritic or looped pattern. It is applicable to a wide range of hydrologic situations wherein flow and transport are governed by time-dependent forcing functions. BRANCH is particularly suitable for simulation of flow in complex geometric configurations involving regular or irregular cross sections of channels having multiple interconnections, but can be easily used to simulate flow in a single, uniform open-channel reach. Time-varying water levels, flow discharges, velocities, and volumes can be computed at any location within the open-channel network. Stream flow routing and computation by the BRANCH model is superior to simplified-routing methods in open-channel reaches wherein severe backwater and/or hysteretic conditions prevail.

Typical uses of the model encompass the assessment of flow and transport in upland rivers in which flows are highly regulated or backwater effects are evident, or in coastal networks of open channels wherein flow and transport are governed by the interaction of freshwater inflows, tidal action, and meteorological conditions. Surface- and ground-water interactions can be simulated by the coupled BRANCH and USGS modular, three-dimensional, finite-difference ground-water flow (MODFLOW) models, referred to as MODBRNCH.

The first version of the program was released in the early eighties. The latest version (4.1) was released in August 1996.

II.2.3 BRANCH: Method

The BRANCH model uses a weighted four-point, implicit, finite-difference approximation of the unsteady-flow equations. Flow equations are formulated, using water level and discharge as dependent variables, to account for nonuniform velocity distributions through the momentum Boussinesq coefficient, to accommodate flow storage and conveyance separation, to treat pressure differentials due to density variations, and to include wind shear as a forcing function. The extended form of the de Saint Venant equations is formulated so as to provide a high degree of flexibility for simulating diverse flow conditions produced by varied forcing functions in channels of variable cross-sectional properties. Subdivision of branches into segments of unequal lengths is accommodated by the finite-difference technique and the implicit solution scheme permits computations at large time steps. The effects of hydraulic control structures within the model domain are treated by a multi-parameter rating method. The model accommodates tributary inflows and diversions as well as lateral over-bank flows, and includes a Lagrangian, particle-tracking scheme for conservative constituents. Transformation equations are formulated that describe the relationship between unknowns at the ends of branches thereby reducing the order of the coefficient matrices and producing a significant saving of execution time and computer memory. The resultant matrix of BRANCH-transformation and boundary-condition equations is solved by Gaussian elimination using maximum pivot strategy.

II.2.4 BRANCH: Data Requirements

Input data consist of channel geometry and initial flow conditions defined at all cross-section locations and boundary conditions defined at channel extremities. Cross-sectional data, in the form of tables of top-width and area as functions of water level, describing the open-channel reaches can be manually prepared and formatted for input to the model or interactively entered, processed, and formatted using the Channel Geometry Analysis Program (CGAP). Initial flow conditions can be measured, assumed, or interpolated values. Boundary conditions can be specified by equation, functional relations, or time-series values. Time series of boundary conditions, i.e., water levels or discharges, can be input directly via formatted sequential files or automatically retrieved from the data base of either the Time-Dependent Data System (TDDS) or the Watershed Data Management (WDM) system. Input values can be either in metric or English units.

II.2.5 BRANCH: Output Options

Time series of computed flow results can be directly output in tabular or graphical form at all, or selected, cross-section locations. Tabular output options include discrete flow results at every time step or iteration; daily summaries of minimum, maximum, and average flow conditions; monthly flow-volume summaries; or river-mile locations of injected particles. Digital or line-printer graphical options include hydrograph plots of computed water levels or discharges or comparative plots of computed results versus measured data. Graphical plots can be produced on CRT devices, directly, and/or in CGM, PostScript, or HPGL formatted files for postprocessing. Computed results can be stored directly in text files or in the data base of either the TDDS or WDM. Interfaces are available for the USGS/WRD National Water Information System (NWIS) and the Branched Lagrangian Transport Model (BLTM). Output results can be either in metric or inch-pound units.

II.2.6 BRANCH: System Requirements

BRANCH is coded in Fortran 77. Executable versions of the model are available for UNIX-based systems (supported: Data General AViiON with PRIOR Graphical Kernel System (GKS)) and DOS-based personal computers. The PC executable requires a 386 or higher with 2MB memory and a math coprocessor. Executables requiring less memory can be produced and tailored to a computer system by reducing array dimensions to suit application needs and then recompiling. In general, the model is readily adaptable to a host of other computer systems.

The program is optionally supported by CGAP for preparation of channel cross-sectional data and TDDS for preprocessing of boundary-value data and postprocessing of simulation results. Graphical plots of particle-tracking results are produced by the TRKPLOT support program included with the BRANCH model distribution software.

II.3 CANALMAN: CANAL MANagement

II.3.1 CANALMAN: Developer

Software Engineering Division
Department of Biological & Irrigation Engineering
Utah State University
Logan, UT 84322-4105
USA

II.3.2 CANALMAN: Description

CANALMAN is a hydraulic model designed to simulate unsteady flow in branching canal systems with trapezoidal cross sections. Canal reaches are separated by in-line structures such as gates, weirs, etc. Turnouts from a simulated canal can discharge directly onto irrigated fields, into waste-ways, or into laterals within the canal system.

CANALMAN uses an implicit-solution technique to solve the complete Saint Venant equations. The simulation time step can be varied from one to 10 minutes in steps of a whole minute.

II.3.3 CANALMAN: Features

CANALMAN will model the topology of most canal systems, including branching canals. A maximum of four branches, with a total maximum of 40 reaches, can be simulated with a single model set-up. Channel friction gradients are computed using Manning's equation. Hydraulic roughness is constant within a reach; it cannot vary with depth, flow rate, or longitudinal distance.

Canal reaches are separated by in-line structures which include pumps, rectangular weirs, circular, rectangular, and radial sluice gates. Several types of other boundary conditions cannot be directly modeled by the program, including broad crested weirs, inverted siphons, reservoirs, rating curves, discharge hydrographs, and hydraulic losses. The boundary at the upstream end of a canal system is always modeled as an inflow hydrograph.

Boundary-condition analysis in the model is of average accuracy. Mathematical representation of structures is relatively simple and straightforward. However the simplification of structure representation makes modelling of complicated structures not possible.

The model has three modes of operation: manual, preset, and automation. The manual mode simulates canal flow with user-specified operation of the control structures, and the model calculates water levels and flow rates. In this mode the user must interrupt a simulation to change a gate setting, and then continue the simulation again. The preset mode is similar to the manual mode; however, instead of interrupting the simulation, the user specifies structure settings for each 5 minutes time interval before the simulation begins. In the automation mode CANALMAN controls in-line structures according to specified automation algorithms. Those algorithms include: AVIO, AVIS, EL-FLO, BIVAL, Littleman, Colvin, Zimbelman, CARDD, and UMA.

Turnouts are always operated by the model during a simulation. The user specifies turnouts demands and the model changes the turnouts settings according to the demand and the hydraulic conditions upstream and downstream them.

Duplicating emergency operations is more difficult. CANALMAN was not designed to perform design studies that must simulate worst-case scenarios.

II.3.4 CANALMAN: Special Hydraulic Conditions

A unique and valuable feature in CANALMAN is its capability to analyze water advance on a dry bed, which is a problem to most other simulation software. Canal filling analysis was calibrated with empirical data. Initial filling of an empty canal can be studied with reasonable accuracy, which should be particularly valuable to operators of small canals and laterals that are frequently drained and filled. On the other hand, the program will not analyze channel dewatering, rapid flow changes, negative flow at structures, hydraulic jumps, and supercritical flow.

II.3.5 CANALMAN: Data Entry and Output

CANALMAN excels in the simplicity and user-friendliness of data entry and model building. The program uses Windows graphical user interface to interactively build models and edit their data. A global check of entered data can be performed at any time to check the integrity of all entered data.

Simulation results can be viewed in numerical and graphical formats on-screen, and can be written to text files. Direct printouts of the tabular and graphical results can also be obtained. Changes in selected reach depth profiles, downstream target levels, gate settings, and other information can be monitored during a simulation through graphical flow profile windows.

II.3.6 CANALMAN: System Requirements

The minimum recommended hardware configuration includes an IBM-PC or compatible with 386 processor, a hard disk, and at least 4MB of memory. The operating system should be MS-DOS 3.1 or later with Microsoft Windows.

II.4 CARIMA: CAlcul des RIvieres MAillees

II.4.1 CARIMA: Developer

SOGREAH Consulting Engineers
Grenoble
France

II.4.2 CARIMA: Description

Although CARIMA was originally developed for flood-propagation studies, it has been used for regulation problems in irrigation canal systems. The program solves the complete de Saint Venant equations with the unconditionally stable, convergent Preissmann method to analyze unsteady flow conditions. The nonlinear algebraic equations resulting from application of the Preissmann methods are solved in a Newton-Raphson context.

II.4.3 CARIMA: Features

The initial conditions required for starting any simulation run can be taken from the results of a previous run, calculated by CARIMA using an automatic steady flow stabilization procedure, or directly entered by the user. A zero-discharge initial condition is perfectly admissible; however, a zero-depth condition (dry bed) cannot be accommodated.

CARIMA routinely treats simple, branched, or looped systems, including interconnected quasi-toe-dimensional floodplain cells. Computational points in a model are given unique four-character names. Channel sections can be trapezoidal, circular, and general (defined by either elevation and width or distance from bank and elevation). The program provides automatic detection of the number of data pairs, no pre-counting required.

CARIMA uses the Manning/Strickler or Chezy equations. The resistance coefficient can be specified as constant within an entire cross section or within a subsection, and can also be specified as varying with water-surface elevation, although this practice is discouraged.

The different types of external and internal boundary conditions supported by CARIMA are: discharge hydrographs, stage hydrographs, rating curves, pumps, composite rectangular weirs, composite rectangular gates, local head loss, storage basins, culverts, inclined weirs, idealised flood-control dams, and automatic regulators (Proportional-Integral-Differential, ideal BIVAL, manual BIVAL, constant level difference, and simple local control). Special user-defined regulators are also available. Both internal and external boundary conditions are treated in fully implicit form.

The program routinely treats negative flow at structures, and generally can accommodate rapid flow change. However, it contains no specific treatment of hydraulic jumps or bores, and cannot normally accommodate dry-bed situations (currently under development).

II.4.4 CARIMA: Data Entry and Output

The CARIMA system comprises three batch-mode programs, each with a formatted input data file. These files and their respective programs dealing with model data assembly, simulation execution, and results postprocessing, are based on automatic recognition of four-character record identifiers. Editing facilities are provided for model construction input data files, whereby the user can modify topological or physical features of an existing model without having to reread the entire data set.

The program produces logging and diagnostic output files for model assembly, simulation execution, and results post-processing, in English or in French. The postprocessing program also produces batch graphs of temporal and spatial variation of water levels, depths, velocities, energy slopes, volumes, etc. Output of structure settings and other nonstandard variables is customised by the user through the regulation interface.

II.4.5 CARIMA: System Requirements

Minimum hardware requirements are a 640 KB DOS based machine, with 20 MB hard disk space and a coprocessor. The program is routinely run on Apollo, VAX, and IBM (mvs) systems, though the graphics capability is not fully developed for all these systems.

II.5 DUFLOW: DUtch FLOW

II.5.1 DUFLOW: Developers

Delft University, The Tidal Water Division; The International Institute for Hydraulic and Environmental Engineering (IHE); and the public-works department (Rijkswaterstaat) in the Netherlands.

II.5.2 DUFLOW: Outline

DUFLOW is a user-orientated package for unsteady flow computations in networks of open water courses. Apart from uniform and non-uniform flow calculations, it can address, for example, propagation of tidal waves in estuaries, flood waves in rivers and operation of irrigation and drainage systems. Free flow in open channels is simulated and control structures like weirs, culverts, siphons, and pumps can be included. A simple rainfall-runoff relationship is part of the model. DUFLOW can be used for large river systems, but also for simpler irrigation and drainage networks, for which input hydrographs can be specified. Both graphical and numerical output are available. Version 2.0 of the program (1992) includes a water quality module.

II.5.3 DUFLOW: Features

A four-point implicit Preissmann scheme is used to solve the complete Saint Venant equations of continuity and momentum. The user can select solution of linearized or fully nonlinear versions of the equations, the latter being solved with a Newton-Raphson type scheme that starts from the linearized results. The linearized option nearly always gives results very close to the nonlinear results.

Initial conditions must be defined by the user at all model nodes. However it is recommended that initial discharge values are not specified by the user and left for the program to calculate. DUFLOW does not include a separate steady-flow solution procedure and uses the unsteady procedure to handle both types of flow.

There are no real limitations on the layout that can be employed in DUFLOW. The user defines nodes and is free to connect any nodes together by any number of channel reaches or structures. Looping is automatically accounted for. Multiple structures at a location also pose no difficulty.

Cross sections are defined at each node in terms of top width of flow at given depths. DUFLOW has the capability of water storage, which does not contribute to flow cross section. This water storage is included in the continuity equations but does not contribute to flow rate, velocity, or momentum. The Chezy equation is used as the standard frictional resistance equation for channels, culverts, and siphons. Manning-Strickler equation can be used for channel resistance. DUFLOW has the capability of adding wind shear, which can be applied in any direction relative to the channel reach.

Boundary conditions can be defined at nodes at any location within the network. External boundary conditions can include a fixed water level; a fixed inflow or outflow; a rating curve; rain, which adds flow to the network according to precipitation, catchment area, and runoff proportion of rainfall. Internal boundary conditions are classified as structures which include orifices, weirs, and entrance and exit losses of culverts and siphons.

Structures can be operated in a number of ways; a structure setting can be changed to a new constant value or to a time series of values. The trigger conditions that cause the setting to change values are: 1) Time when depth at a node changes with respect to a specified water level; 2) When the difference between the water levels at two nodes exceeds a given amount. Automatic controls are not supported by DUFLOW. Also the program has no interface whereby a user could simulate a gate-control algorithm.

DUFLOW has some limitations in modelling canal networks. The maximum number of channel sections and structures in a model is 250. Cross sections can be defined with up to 15 depth-width pairs. The number of boundary conditions multiplied by the number of time steps may not exceed 50,000. The discharge-water level relationships for boundary conditions are limited to 20 pairs of values. Each structure operation can have up to 99 triggers, but the number of operations times the number of structures per operation cannot exceed 16.

Besides these limitations on model configuration, DUFLOW cannot simulate critical flow, hydraulic jumps, and dry-bed channels.

II.5.4 DUFLOW: Data Entry and Output

DUFLOW has a menu driven interface. The menus are arranged in hierarchical fashion so that the user can easily navigate through the system. This menu system is mainly targeted at novice users but experienced users might find it cumbersome to travel up and down the menu tree to make simple changes. All data are entered in tables on the screen and then saved in data files. The layout of a model can be plotted for double checking.

Output results can be viewed in either tabular or graphical form. Tabular data can be printed to a printer or to files, while graphs can only printed to a printer. Field measured data can be entered for comparison with simulated results.

II.5.5 DUFLOW: System Requirements

The program requires an IBM-AT machine or compatible with a 640 KB of memory and a graphics card. A math coprocessor is highly recommended. The program runs under

MS-DOS from a hard drive or two floppy drives instead.

II.6 MODIS: MOdelling Drainage and Irrigation Systems

II.6.1 MODIS: Developer

Delft University of Technology
P.O. Box 50400
2600 GA Delft
The Netherlands

II.6.2 MODIS: Outline

MODIS is an implicit hydrodynamic modelling package that computes the unsteady water flow in open channels. The model can run in various computational modes varying from steady-state mode to a full-dynamic mode. Branched and looped open channel networks can be modelled by the program. The simulation of structure operational plans and control are easy to perform in MODIS. Furthermore the model has performance indicators that allow for a fast diagnostic interpretation of the results.

II.6.3 MODIS: Features

The MODIS model can run in various computational modes, varying from steady-state mode to full dynamic mode, in which the complete Saint Venant equations are solved. The applied numerical solution technique is always based on finite differences using the four-point Preissmann implicit scheme. The minimum computational time step is 0.001 second.

The canal layout in MODIS is modeled by using branches and nodes. Branches represent conveyance elements such as pools and reaches. Nodes are used to link branches together and to indicate a branch end. No restrictions are imposed on branch lengths, and both branched and looped canal networks can be modelled. Structures can be placed anywhere within a branch. Hence one branch can be divided into several pools separated by structures. Composite structures can be modelled by locating several structures at the same location.

Canal sections can either be trapezoidal or irregular. The friction term of the Saint Venant equations is represented by Manning-Strickler resistance formula. The resistance value can be varied with height in a section and with the longitudinal distance.

External boundary conditions are imposed on nodes indicating branch ends. Water level, discharge, or stage-discharge relationships can be used as external boundary conditions.

Internal boundary conditions are needed to link branches. At these links, water level compatibility is assumed. The boundary conditions can be specified as fixed values, as time series, or as a function of a user-written FORTRAN routine.

Structures are not treated as boundary conditions in MODIS because they are placed within branches. Instead, structure equations are rewritten in the same format as the momentum equation and thus are fully incorporated in the implicit solution procedure. Consequently, supercritical flow in the vicinity of structures can be handled.

The standard structure library included in MODIS comprises pumps, weirs, orifices, pipes, head-loss structures, and NEYRTEC baffle distributors. Furthermore user-defined structures can be included by writing their equations in FORTRAN-defined format. During a simulation, the flow condition can fluctuate from free to submerged for example. Also reversal of flow directions causes no problems.

MODIS can simulate all modes of structure operation. Structure settings can be defined as fixed values, functions of time, or water levels and flows. It is also possible to specify the water level or the flow a structure should maintain. The model computes structure parameters to realize the water level or the discharge.

Real-time controlled canals can be simulated in MODIS. Several control algorithms are standard in the model: multiple speed control, proportional-integral-differential (PID) control, CARDD, BIVAL, and EL-FLO controls. User-defined control algorithms can be easily added by using the FORTRAN function facility.

The initial conditions required for the solution of the unsteady flow equations can be taken from previous runs, or entered at the ends of the branches and MODIS then interpolates the initial values for the intermediate grid points inside the branches.

II.6.4 MODIS: Special Features

To avoid program termination in case of dry-bed flow, a Preissmann slot is automatically added to trapezoidal cross sections. A routine prevents the slot from falling dry be continuously checking the water levels. If the water level drops below the bed level, the water depth at that location is increased to 0.01 m and a base flow of 0.001 m^3/s is generated. A warning is given when this routine is activated.

To evaluate the simulated water distribution in irrigation canal networks, two operation-related performance indicators can be automatically computed by MODIS: a Delivery Performance Ratio (DPR) and an operation efficiency (e_o).

DPR is calculated as the ratio between the actual volume of effective water delivered to the target volume. Water is assumed to be effectively delivered when it is delivered during a specified period of time and within allowable limits of flow rate.

The e_o indicator is used to monitor the water use efficiency, and is calculated as the ratio of the volume of effectively delivered water to the total volume of water delivered to an offtake. The indicator can be calculated for single offtakes or for a canal system with multiple offtakes.

II.6.5 MODIS: Data Entry and Output

MODIS is operated from an interface that utilises drop-down menus, but the menu is not fully developed yet. The actual data entry is file oriented, using a file editor. Input data are organized in tables supported by explanatory comments lines.

The computational results can be presented both graphically and in tables. All hydraulic variables or combinations of variables can be printed on printers and plotters. It is also possible to print only maximum, mean, and minimum data values. Furthermore, the percentage of time of exceedence of user-defined minimum and maximum values can be tabulated.

II.6.6 MODIS: System Requirements

The program can run on any IBM-compatible computer with a minimum of 640 KB of RAM. A 386 or higher processor is required. For the graphical output, the commercial package HALO is needed.

II.7 SIC: Simulation of Irrigation Canals

II.7.1 SIC: Developer

CEMAGREF
B.P. 5095
34033 Montpellier Cedex 1
France

II.7.2 SIC: Description

The software, SIC, has been developed by the Irrigation Division of CEMAGREF, Montpellier, France. SIC can provide a detailed simulation of flow in a canal system and thus allows to make studies in order, for example, to reduce water losses and inequity of supply to users. The model is based on one-dimensional hydraulic analyses for transitional and steady-state flows. It is divided into three parts: a topographical unit to generate the topography and topology of the scheme, and two separate computational units for steady and unsteady flow.

Special features include a calibration module to compute both Manning's and discharge coefficients, given measured flows and water levels. It is also possible, for example, to calculate structure settings to achieve the required flow at off-takes and proportion of flow in the canal. Seepage and inflows can also be accounted for. But it cannot be applied in the cases of supercritical flow and dry beds.

SIC may be used to test different manual or automatic management rules and to evaluate their performance. The efficiency of the water delivery at off-takes is quantified in terms of water volumes and delivery delays in relation to pre-defined objectives. The results are available in graphical and tabular form giving a time series of water levels and discharges. SIC has also been used to design canals and to determine maintenance schedules by, for example, working out priority reaches for desilting work.

The software has an interactive interface, including an on-line help, which works in several languages (English, French and Spanish). Input and output data are in SI units. The time scale of flow simulation can be as small as few minutes. The current version of the program is dated May 1992.

II.7.3 SIC: System Requirements

SIC will run on any IBM compatible PC with installed math coprocessor under MS-DOS or Windows 3.1. The program requires 1 MB RAM for the DOS version or 8 MB RAM for the Windows version. An EGA/VGA colour display is also required.

II.8 USM: UnSteady Model

II.8.1 USM: Developer

U.S. Bureau of Reclamation
P.O. Box 25007,
Denver,
CO 80225,

USA

II.8.2 USM: Outline

The computer program USM is a hydraulic simulation software that models gradually varied unsteady flow in canal systems. The primary purpose and application of the program has been the hydraulic analysis during the design of new canals and canal-control systems. Often, design studies require analysis of emergency operations so that canal structures can withstand worst-case scenarios.

USM uses the method of characteristics to calculate a numerical solution to the complete Saint Venant equations of unsteady open channel flow. Two variations of the solution method are available: complete grid of characteristics and specified time interval. In the first solution, the calculation time step varies as water depth and wave speed change. The specified time interval solution uses a fixed time step. The method of characteristics yields an accurate numerical solution, but requires a large number of computations since the calculation time step must be small. Therefore USM is more efficient for solving problems of short duration involving rapid flow changes than for problems of long duration with gradual changes.

USM topology is limited to a linear series of up to 40 canal pools separated by structures or boundary conditions. Branching or looping cannot be modeled directly. The program can model trapezoidal, rectangular, circular, U-shaped, trapezoidal with one vertical side, and trapezoidal membrane-lined sections. Different values of cross section, roughness, slope, etc., may be entered for each canal pool, but they are assumed to be constant for each pool. Computational nodes are spaced in equal user-defined intervals within a pool.

Each pool must have an upstream and a downstream boundary condition, which can be a depth-discharge relationship or a discharge hydrograph. In-line structures include check gates, weirs, siphons, pumps, transitions, drops, and check-siphons combinations. Pumps are simulated with discharge hydrographs. Multiple independently operated gates in parallel cannot be modelled. Also the program assumes that submerged flow occurs at drop structures, so free flow is problematic.

USM can model various gate schedules. The program also simulates various types of local automatic control. Several automatic control algorithms are included in the program. Furthermore, USM can be used with a companion program called GSM (Gate Stroking Model). GSM can be used to produce initial gate positions for USM; and USM can be used to confirm and refine a gate-stroking solution produced by GSM.

One turnout is allowed at the downstream end of each pool. Turnout operation is simulated by specifying a discharge hydrograph for the water leaving the canal at the turnout location.

II.8.3 USM: Limitations

USM does not simulate advance on a dry bed, canal dewatering, hydraulic jumps, bore waves, supercritical, flow, or negative flow through structures. The maximum time span for a single simulation is 24 hours.

II.8.4 USM: Data Entry and Output

Two separate programs for data entry are provided with USM to guide the user through the data-entry procedure. Data is saved in ASCII files on disk which can be edited directly for faster actions. During a simulation, the program does not show a progress message. Results are written to output disk files after each time step. USM is provided with a conversion program to reformat output data into columns that can be read by commercial spreadsheet and graphics software.

II.8.5 USM: System Requirements

USM was developed on a mainframe computer, but versions for VAX and IBM-compatible computers are now available. The microcomputer version runs under the MS-DOS operating system. The minimum required configuration is a 386 processor with a math coprocessor.

II.9 Summary and Conclusions

The review of hydraulic modelling software presented above illustrates a great variety of models which cover a wide spectrum of features and capabilities that indeed emphasize the importance of such modelling tool in the study and design of irrigation systems. A summary of the features of the reviewed programs is outlined in Table II.1. Some points of interest to highlight are:

- Some models like CARIMA and ISIS were originally designed for purposes such as flood routing and river modelling and were then modified to be able to model irrigation networks as well, while the rest of the reviewed models were specifically designed to model irrigation networks except BRANCH which is suitable for river modelling only as it cannot model irrigation structures.

- Some models were designed to satisfy particular needs in the design and operation of irrigation networks: CANALMAN focuses mainly on the management of irrigation systems and excels in the analysis of canal filling procedures. USM was designed to focus on the analysis of emergency operations of newly designed canal structures. ISIS Flow includes a large built-in structure library ready for the user to choose from. MODIS targets controlled irrigation canals with its capability of duplicating most manual and automatic structure operations besides the calculation of some performance indicators which help in monitoring system performance.

- All reviewed models solve the complete Saint Venant equations for unsteady flow calculations. The four-point Preissmann implicit scheme is used for solving the equations except in USM where the method of characteristics is used instead. The four-point Preissmann implicit scheme is usually more robust and can cope with more varied hydraulic conditions at the expenses of accuracy, while the method of characteristics achieves the opposite.

Table II.1 Summary of the features of hydraulic modelling software

Feature/Capability	BRANCH	CANALMAN	CARIMA	DUFLOW
Solution of unsteady equations	Preissmann	Preissmann	Preissmann	Preissmann
Calculation of initial conditions	No	No	Yes	No
Branches and loops modelling	Yes	Yes	Yes	Yes
Network size	No limit	Up to 40 reaches		Up to 250 sections
Canal cross sections	Any	Trapezoidal/ Circular	Trapezoidal/ Any	Any
Variable roughness		For reaches	Within section	
Duplication of control automation	No	All algorithms		No
Negative flow at structures	N/A	No	Yes	
Simulation time step	Any	1 - 10 min		1 min
Data units	SI/Imperial	SI/Imperial	SI	SI
Special features	* Communicates with MODFLOW	* Graphical model building * Canal filling simulation	* User-defined structures	* Accepts rainfall data * Wind shear effects can be studied
Limitations	No structures	* No complex structures * No reservoirs		* Many on network configuration
Interface		Graphical	Yes	Menus
Graphical output	Yes	Yes	Yes	Yes
Additional software required	No	No	GKS	No
Operating system	MS-DOS	MS Windows	MS-DOS	MS-DOS
Price range		$100 - $150	$5,000 - $10,000	$200 - $300

Table II.1 Summary of the features of hydraulic modelling software (cont.)

Feature/Capability	ISIS	MODIS	SIC	USM
Solution of unsteady equations	Preissmann	Preissmann	Preissmann	Characteristics
Calculation of initial conditions	Two methods	No		Yes
Branches and loops modelling	Yes	Yes		No
Network size	2000 nodes	No limit		Max. 40 pools, 1 turnout per pool
Canal cross sections	Any	Trapezoidal/ Any		Prismatic
Variable roughness	Within section	Within section		For pools
Duplication of control automation	Control Module	All algorithms	Yes	All algorithms
Negative flow at structures	Yes	Yes		No
Simulation time step	Any	Any	1 min	Max 60 sec
Data units	SI	SI	SI	SI/Imperial
Special features	* Large structures library * Partly handles supercritical flow	* Dry-bed routine * User-defined structures * Built-in performance indicators	* Accounts for seepage * Built-in performance indicators	* Emergency operations/ rapidly varied flow simulation * Communicates with GSM
Limitations				* Not robust * Cannot handle free flow
Interface	Graphical/DOS	Menus	Graphical	Yes
Graphical output	Yes	Yes	Yes	No
Additional software required	No	HALO	No	No
Operating system	MS Windows	MS-DOS	MS Windows	MS-DOS
Price range	£5,000 - £14,000	$15,000 - $20,000	FF 80,000	Public

- It is interesting to note that most of the models reviewed have almost the same limitations in their modelling capabilities:

  - supercritical flow cannot be handled (except in ISIS);
  - zero water levels (dry beds) cannot be solved (except in CANALMAN & MODIS);
  - canal dewatering cannot be simulated;
  - hydraulic jumps and bore waves cannot be simulated.

Appendix III: Irrigation System *A*

III.1 General

The case study used in the various hydraulic simulations which were carried out in this research (referred to as system *A*[13]) is a real-life irrigation scheme located in eastern Africa. The scheme covers a gross area of 8,000 ha out of which the total net irrigable area is about 6,400 ha. The soils of the scheme can be divided into two broad classifications, namely basin clays, for which surface irrigation in basins is suitable, and the meander complex levee soils, for which overhead irrigation is more appropriate. The primary crops grown are rice (two varieties), maize and cotton.

III.2 Climate Data

The climate of the scheme area is semi-arid with around 500 mm of rainfall annually, which is also irregular and inconsistent. Rainfall cannot therefore be relied upon as a primary source of crop water requirements but it can supplement irrigation supplies.

Monthly reference evapotranspiration (ETo) was calculated using the method of Doorenbos and Pruitt (1977). The criterion of the US Bureau of Reclamation was applied to monthly rainfall with an 80% probability of exceedence to calculate monthly effective rainfall, Table III.3.

III.3 The Irrigation System

A schematic layout of the irrigation system, showing the components which have been modelled in details, is depicted in Figure III.1. Irrigation supplies for the scheme are pumped from a river into the main canal (MC) of the system. Water is then conveyed to

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13 The phrases 'system *A*' and 'scheme *A*' have been used as identifiers of this irrigation system/scheme to emphasise the generic nature of this research.

the irrigated areas in a system of six distributary and two tertiary canals. Finally, water is delivered to the fields for surface irrigation by gravity through 76 field outlets or pumped into the overhead irrigation networks. An inventory of the main components of the irrigation system is given in Table III.4.

Table III.3 Average monthly evapotranspiration and effective rainfall data for irrigation scheme *A*

Month	ETo (mm)	Effective Rainfall (mm)	Month	ETo (mm)	Effective Rainfall (mm)
April	157	17	October	160	0
May	145	22	November	147	0
June	129	36	December	161	0
July	132	32	January	179	0
August	147	8	February	170	0
September	157	0	March	195	0

Table III.4 The main components of irrigation system *A*

- Intake channel
- Main pump station
- Settling basin
- 2 underpass culverts
- 9 km main canal supply
- 25 km distributary canals
- 9 head-regulators and 16 cross-regulators on the main and distributary canals
- Approximately 100 km feeder canals
- 76 gated field outlets and 2 field pumping units
- Numerous other water control structures

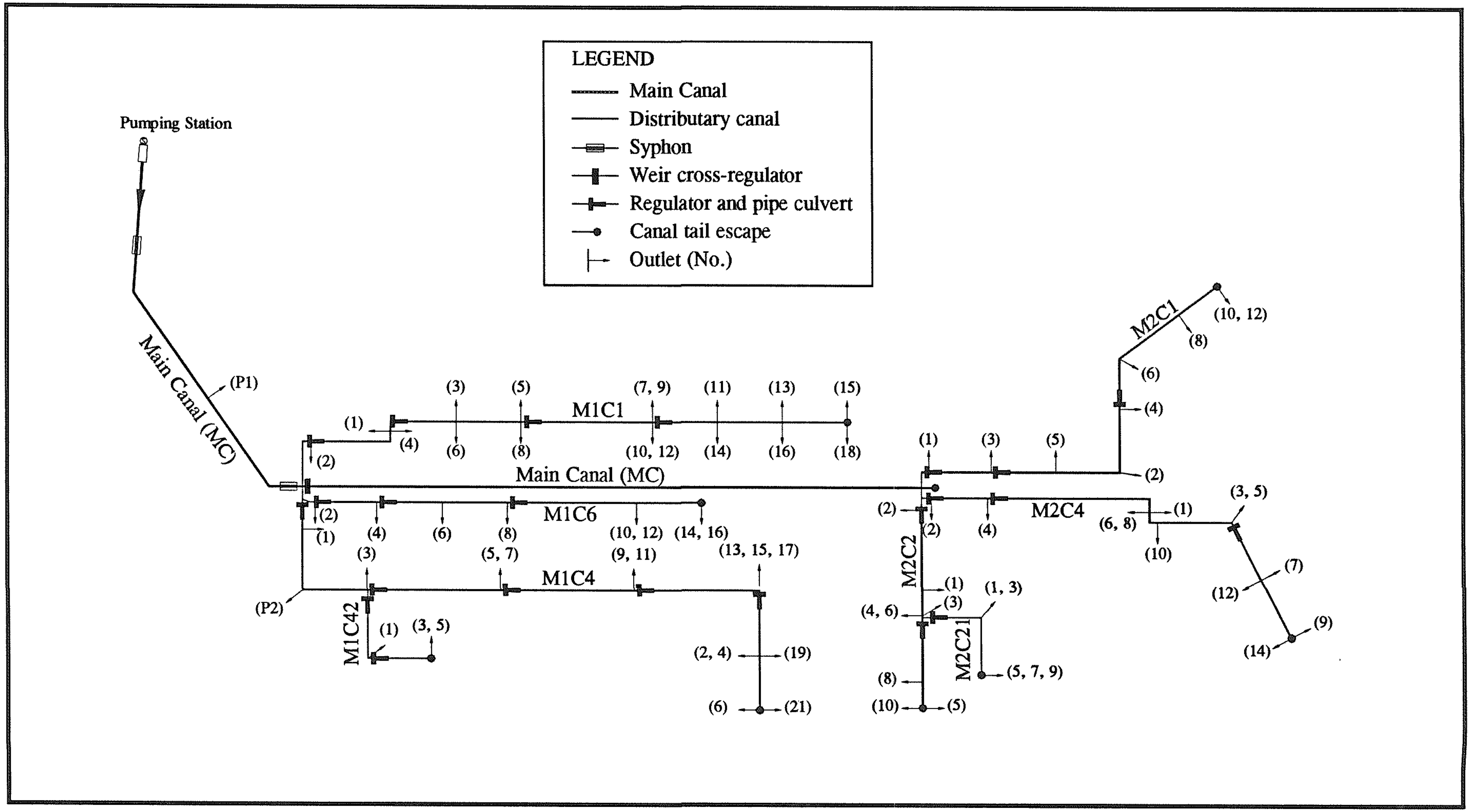

Figure III.1 Schematic layout of irrigation system *A*

All canals are made of trapezoidal earth cross-sections with 2:1 side slopes. The design Manning n of the main canal is 0.022 and for the other canal types is 0.03. A brief of the important design data of the canals is given in Table III.5.

Table III.5 Summary of the main design data of the canals in irrigation system *A*

Canal	Length (m)	Area Served (ha)	Design Discharge (m^3/s)	Bed Width (m)	Section Depth* (m)	Bed Slope (cm/km)
MC	8,900	122	6.67	8.2 - 6.5	1.9 - 1.7	8 - 9
M1C1	4,930	1,394	1.43	4.0 - 1.5	1.45 - 0.85	10 - 22
M1C4	5,870	1,189	1.48	4.0 - 1.5	1.45 - 1.0	10 - 11
M1C42	1,150	246	0.25	2.5 - 1.5	1.2 - 0.9	10
M1C6	3,420	656	0.67	2.5 - 1.5	1.2 - 0.75	5 - 11
M2C1	3,800	738	0.75	2.5 - 1.5	1.3 - 0.9	9 - 11
M2C2	1,790	656	1.11	3.5 - 1.5	1.3 - 0.9	9 - 14
M2C21	1,020	410	0.43	2.0 - 1.5	1.1 - 0.9	10 - 21
M2C4	4,050	984	1.00	3.0 - 1.5	1.35 - 0.75	10 - 50

* Including design freeboard of 0.5 m for the main canal (MC) and 0.4 m for the rest.

Manual upstream water level control is practised in the system. There is only one gated weir cross-regulator on the main canal (MC). The rest of the canals have composite head and cross-regulators, each consisting of one gated sluice-type structure and pipe culverts. The structures are constructed of concrete and the gates are made of cast iron.

The field outlets are also provided with similar composite structures of smaller sizes. The gates of the structures should be adjusted to allow the required flow to be diverted to the fields. Each field outlet serves an area of about 82 ha. The details of the parts of the irrigation system below the field outlets have not been built into the hydraulic models used in this work. The assessment of the performance of the system was carried out assuming that the flow diverted to any field outlet will be evenly applied to the whole area served by that outlet.

III.4 The Cropping Pattern and Water Demands

The cropping pattern contains four main crops: paddy rice, upland rice, maize and cotton. A typical crop calendar is given in Table III.6. The crop water requirements were calculated using the climate data given earlier and the crop calendar in the table. Those requirements were then converted to canal flows by allowing for 60% to 80% field application efficiency and 90% conveyance efficiency. The peak demand is about 6.67 m^3/s and occurs in May. Figure III.2 depicts the water demand of the scheme and the proposed canal supply for a typical year. Both the supply and the demand are presented in the figure in percentages rather than water quantities or rates. These percentages, which are referred to in this work as **supply ratios/percentages**, are *the ratios of the supply/demand flows to the maximum design flow* (6.67 m^3/s). For example, a 60% supply percentage indicates that the water supply through the intake of the main canal of the system is about $6.67 * 0.6 = 4.0$ m^3/s.

Table III.6 Typical crop calendar of irrigation scheme *A*

Crop	Total Area (ha)	First Planting	Last Planting	First Harvest	Last Harvest
Paddy rice	3,321	Mid Apr	End of May	Begin. Aug	Mid Sep
Upland rice	998	Mid May	Mid Jun	Begin. Sep	End of Sep
Maize	4,301	Mid Sep	Mid Oct	Begin. Jan	End of Jan
Cotton	1,103	Begin. Aug	End of Aug.	Begin. Feb	End of Feb

Despite the fact that the demand varies continuously during the whole year, the supply pattern was approximated and fixed during each month (Figure III.2). The pattern of the supply can be characterised as having four discrete supply percentages of 100%, 75%, 60% and 50%. The reason for this simplification is that it is practically difficult to match the variable demand pattern with a very close supply pattern in manually operated systems. This requires large quantities of data to be collected frequently and a lot of control structure operations which are not intended for such systems. A closer match between the supply and the demand may be possible in fully automated systems.

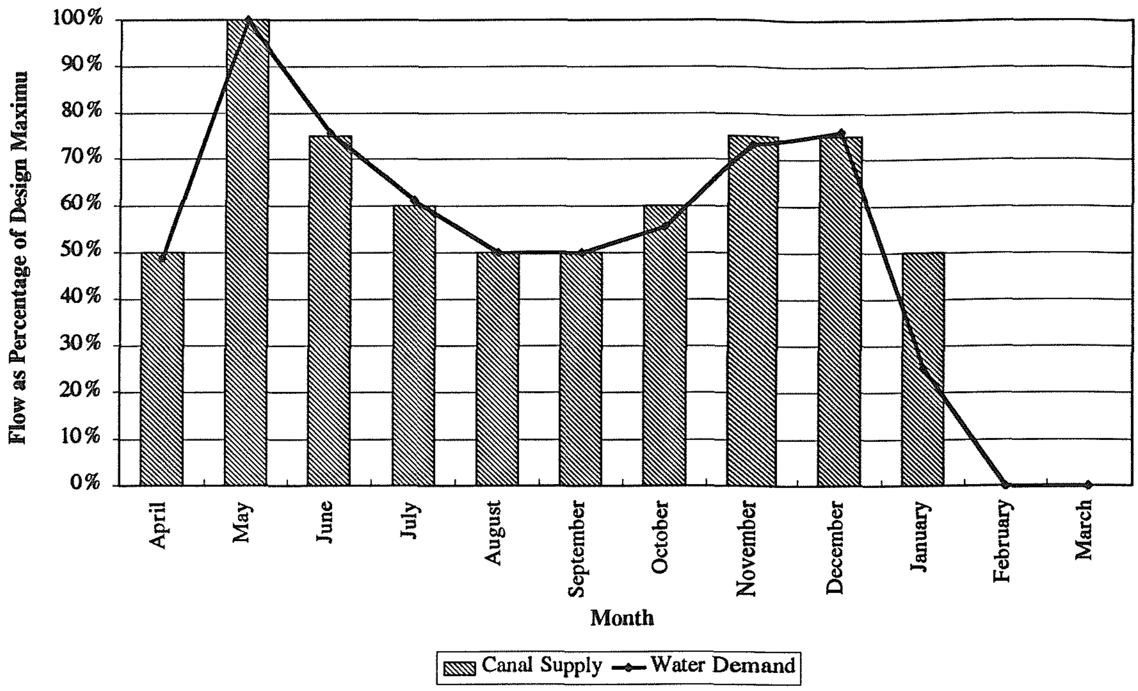

Figure III.2 The actual patterns of the water demand and canal supply for a typical year in irrigation scheme *A*

III.5 Simulating System Operation Using Hydraulic Modelling

For the purpose of simulating the operation of the system throughout a typical growing season using hydraulic modelling, it may be beneficial to rearrange the pattern of the supply in order to simplify the modelling process. For example, Figure III.2 shows that the actual supply pattern starts in April (start of the growing season) with 50% supply, then the supply increases to 100% in May and then drops to 75% in June, etc. In cases when what happens during the transition periods (during which the supply is changed) is not of importance, there is no need for the actual pattern to be modelled; a ranked supply pattern as that shown in Figure III.3 can be used instead. The idea is to minimise the number of times when the flow in the model changes and also, and more importantly, to change the flow on gradual steps as shown in the figure in order to minimise the instability of hydraulic models to large changes in the flow. The time scale in Figure III.3 can be changed to suit the exact requirements of any study so it can be hours, days, months, etc. It is recommended that the time periods during which the supply does not change are maintained long enough to allow the hydraulic simulation to reach steady state condition before the next change in the flow takes place. Although the actual flow in irrigation canals is unlikely to be steady most of the time, it may be desirable to allow this condition to develop in hydraulic simulations in order to avoid results which do not reflect the objective of their studies. For example, if the objective is to investigate the situation at 60% supply and the supply pattern shown in Figure III.3 is used, the output from hydraulic modelling at time step 6 should be used instead of the output at time step 5. For time step 5 may be too soon after the reduction of the flow from 75% to 60% and there might be some water stored in the canals. That storage may give false indication of the true situation at 60% supply when no storage is available. Evaluating the output at time step 6 when the storage has been depleted and the model stabilised should therefore be more accurate.

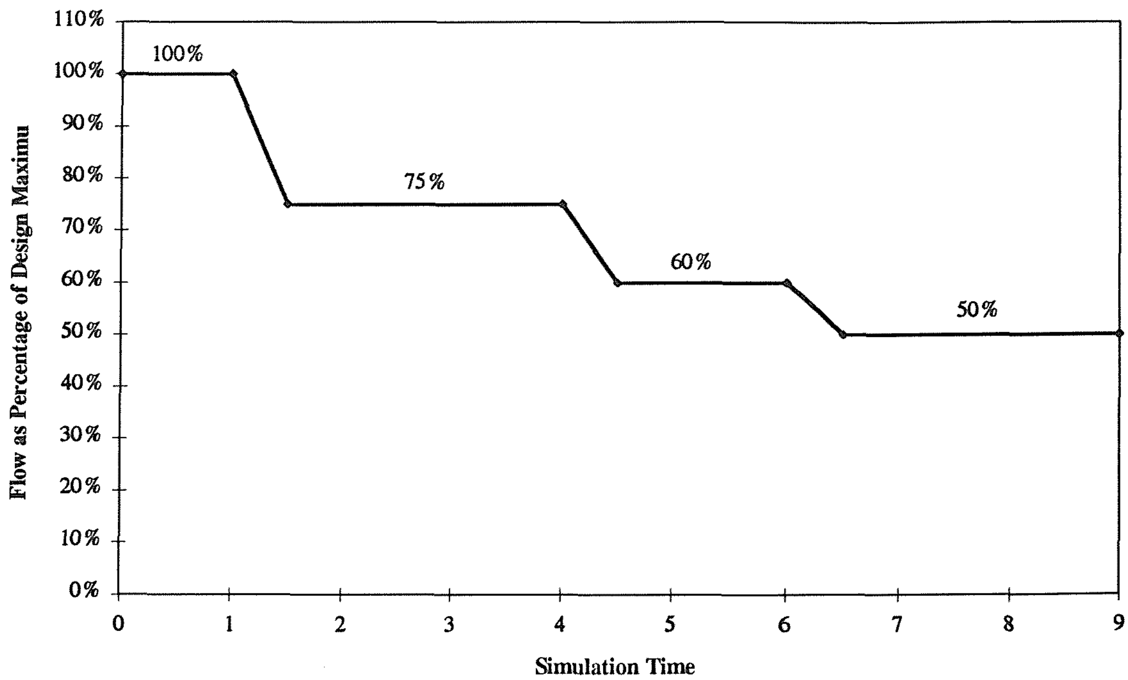

Figure III.3 Using ranked water supply for modelling annual system operation

When the modelling has been carried out, the results should be interpreted using Figures III.2 and III.3 together in order to provide linkage between the real time and the simulation time steps. For example, to get the simulation results for the month of October, Figure III.2 is first used to find out the planned canal supply in this month (60% is this case) and then the last time step of the period simulating 60% canal supply is obtained from Figure III.3 (time step 6 in this case). Consequently, the output of the hydraulic model at time step 6 is used to represent the water delivery and other hydraulic conditions in the irrigation system during the month of October.

Appendix IV: Overall Performance

The following sections describe in detail the implementation of the weighted-additive value function (weighted averages) in this research for calculating the average overall performance score of any scenario based on the values of a group of performance indicators.

IV.1 The Weighted-Averages Method

In this method, a weighted average of some criteria is derived by multiplying the score of each criterion by the weight assigned to that criterion. The weights should be chosen to reflect the relative importance of each criterion within the group. In application to performance assessment, the criteria can be a group of performance indicators which are used to assess the performance of a scenario or a case study. The weighted average will in this case be a numerical representation of the overall performance of that scenario. It can be calculated as follows:

$$OPS = \frac{\sum_{i=1}^{n} PI_i * w_i}{\sum_{i=1}^{n} w_i} \qquad \textbf{(IV.1)}$$

where OPS = the overall performance score

PI_i = the value of performance indicator i

w_i = the weight assigned to performance indicator i

n = the number of performance indicators

For clarity and ease of use, the overall performance score (OPS) should have the following characteristics:

1) The range of possible values should be between 0.0 and 1.0 (or 0% and 100%).
2) The upper limit of the range (1.0 or 100%) should represent the best performance.

According to the equation above, this can only be achieved if each individual indicator (PI) has the same characteristics too.

In the example of calculating the overall performance of the scenarios investigating selective sediment removal from system *A* (see Figure 5.26) the overall performance was calculated based on the following indicators:

1) The mode of the Delivery Performance Ratio (DPR) as a measure of adequacy.
2) The Interquartile Ratio of DPR (IQR) as a measure of equity.
3) The ratio of canal sections whose Lost Freeboard (LFb) exceeded 0.25 (*unacceptable category*).

The results of evaluating these indicators in the scenarios referenced are given in Table IV.1.

Table IV.1 The original values of the indicators used for assessing the performance of the scenarios of selective sediment removal from system *A*

Scenario	DPR Mode	IQR	LFb
Sed30-53	0.92	2.98	0.23
Sed30-54	1.01	1.04	0.14
Sed30-55	1.06	2.27	0.21
Sed30-56	0.84	1.68	0.18
Sed30-57	1.00	1.54	0.20
Sed30-58	0.92	2.23	0.25
Sed30-59	0.98	1.47	0.12

According to the formulae governing the evaluations of these indicators, their ranges and optimum values are as given in Table IV.2. The table shows that the three indicators needed adjustments in order to satisfy the two requirements of the overall performance indicator. The mode of DPR needed adjustment such that its upper limit was 1.0. This was achieved by truncating the values of the indicator which were larger than 1.0. It must be mentioned however that this adjustment did not affect the meaning of the indicator in such cases. When assessing the adequacy of the supply using DPR, any value that is equal to or larger than 1.0 indicates that the supply is adequate. Although it is true that when the value of DPR is much larger than 1.0 this indicates oversupply which causes water wastage, from the sole perspective of adequacy the supply is considered to be not deficient.

Table IV.2 Selected characteristics of performance indicators

Characteristic	DPR Mode	IQR	LFb
Lower limit	0.0	1.0	0.0
Upper limit	[undefined]	[undefined]	1.0
Optimum value	1.0	1.0	0.0

The IQR needed adjustment to its lower and upper limits. Both adjustments were made by inverting the original values of the indicator (1/x). Consequently, after adjustment low IQR values indicated inequitable water distribution.

Table IV.3 The adjusted values of the indicators used for calculating the overall performance of the scenarios of selective sediment removal from system *A*

Scenario	Adjusted DPR Mode	Adjusted IQR	Adjusted LFb	Overall Performance
Sed30-53	0.92	0.34	0.77	0.68
Sed30-54	1.00	0.96	0.86	0.94
Sed30-55	1.00	0.44	0.79	0.74
Sed30-56	0.84	0.59	0.82	0.75
Sed30-57	1.00	0.65	0.80	0.82
Sed30-58	0.92	0.45	0.75	0.71
Sed30-59	0.98	0.68	0.88	0.85

Finally, the ratios of LFb within the *unacceptable category* (LFb > 0.25) needed to be adjusted such that the optimum value was 1.0 instead of 0.0. This was achieved by subtracting the original values of the indicator from 1.0. In other words, the adjusted indicator measured the ratio of canal sections whose LFb were less or equal to 0.25, i.e. the sections which fill within the *acceptable category* of Lost Freeboard.

The adjusted values of the indicators of the scenarios in Table IV.1 are given in Table IV.3. In this example, equal weights were given to the three performance indicators when calculating the overall performance.

Appendix V: Cost Estimations

V.1 General

The following sections summarise the estimation of the costs used in the financial analysis part of this research. The prices used in the cost estimates came from different countries and therefore different environments. This was allowed for in order to (1) emphasize the generic nature of the research and (2) minimise the influence of any local economic or financial situations (for example, manual labour being very cheap in certain areas) on the results of the financial analysis. However, one inconsistency in the data used was that the prices did not have the same time reference; some dated back to 1991 while others were quite recent (1998). This inconsistency was corrected by converting all prices from their original currencies to US dollars (USD) according to the exchange rates during their respective base years, and then converting the USD figures from whatever time references to 1998 prices (chosen to be the time reference in this work) by applying the appropriate inflation rates of the USD. Historical information about currency exchange rates and the rates of inflation of the USD were obtained and verified from different sources on the Internet (OANDA, 1999; U.S. Bureau of Labor Statistics, 1999).

V.2 Crop Budgets

Typical financial crop budgets for the main crops grown in system *A* have been obtained from different sources, including the budgets used in the feasibility study of irrigation system *A* itself. Typical crop budgets for paddy rice were obtained from Pakistan[14] (Tables V.1 & V.2) and Indonesia (Table V.3 - Malik, 1995). For maize two budgets were used, Table V.4 taken from Snell (1997) and another from Somalia (Table V.5 - Mott MacDonald, 1979). And finally, one crop budget for cotton was available from Somalia (Table V.6 - Mott MacDonald, 1979).

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14 The reference of this data is not published here based on the request of the supplier, but can be provided on request.

As was the case with other costs, the prices in those crop budgets have different time references. To convert all the prices to the same time reference (1998), the prices were first converted to USD according to the appropriate exchange rates and then the inflation rates of the USD were applied to estimate the 1998 prices.

Table V.1 Financial budget of 1 ha of paddy rice in Pakistan

Item	Unit	Quantity	Prices (1998)	
			Rupees	USD
Input Costs:				
- Ploughing			2,800	43.75
- Levelling			741	11.58
- Seeds	kg	50	220	3.44
- FYM	tonne	2.5	1,875	29.30
- Urea	kg	105	761	11.89
- DAP	kg	119	1,238	19.34
- Agro-chemicals (rice)	kg	10	555	8.67
- Abiana			193	3.02
Labour Costs:				
- Labour	day	39	2,340	36.56
- Harvest	kg	446	1,797	28.08
Revenue:				
- Main product	kg	4,460	17,974	280.84
- By-product (rice straw)	kg	4,460	4,460	69.69
Net Income in 1998 Prices			9,914	154.91

The figures in Tables V.1 to V.6 show good agreement between the net incomes of each crop type. For example, the net income from 1 ha of paddy rice in 1998 varies between around \$155 to \$200. Similarly, Tables V.4 & V.5 show that the net income from 1 ha of maize ranges between \$450 to \$500. These agreements in the figures increase the confidence levels in the budget data collected and more importantly in the method of estimating the 1998 prices from earlier ones. The average net incomes from the main crops grown in system *A* are summarised in Table V.7.

Table V.2 Financial budget of 1 ha of Basmati rice in Pakistan

Item	Unit	Quantity	Prices (1998)	
			Rupees	USD
Input Costs:				
- Ploughing			865	13.52
- Puddling			1,605	25.08
- Seeds	kg	12	86	1.34
- Urea	kg	100	725	11.33
- DAP	kg	77	801	12.52
- Agro-chemicals (rice)	kg	10	555	8.67
- Abiana			181	2.83
Labour Costs:				
- Labour	day	39	2,340	36.56
- Harvest	kg	220	1,047	16.36
Revenue:				
- Main product	kg	2,598	18,394	287.41
- By-product (rice straw)	kg	2,598	2,598	40.59
Net Income in 1998 Prices			12,787	199.80

Table V.3 Financial budget of 1 ha of paddy rice in Indonesia

Item	Unit	Quantity	Prices (1991)	
			Rupees	USD
Input Costs:				
- Urea			40,000	20.51
- TSP			28,000	14.36
- KCL			27,000	13.85
- Agro-chemicals (rice)			30,000	15.38
Labour Costs:				
- Land preparation			100,000	51.28
- sowing/transplanting			80,000	41.03
- mid season work			80,000	41.03
- harvesting			72,000	36.92
- Threshing, etc.	kg	9,670	26,109	13.39
Revenue:				
- Main product	kg	2,700	756,000	387.69
Net Income			272,891	139.94
Net Income in 1998 Prices *				157.57

* Inflation rate in the USD from 1991 to 1998 = 19.69%

Table V.4 **Financial budget of 1 ha of maize (after Snell, 1997)**

Item	Unit	Quantity	Prices (1998)		
			Pesos	Pesos	USD
Production Costs:					
- Seed	kg	20	0.20	4	1.58
- Fertilizer, N	kg	110	2.86	315	124.35
- Fertilizer, P2O5	kg	60	2.30	138	54.55
- Fertilizer, K2O	kg	90	2.39	215	85.02
- Agro-chemicals	Pesos	40		40	15.81
- Machinery, Tractor	h	2	25.00	50	19.76
- Machinery, Combine	h	0		0	0.00
- Animal power	Pesos	10		10	3.95
- Sacks, etc.	Pesos	0		0	0.00
Labour Costs:					
- Hired	Person-day	8	12.00	96	37.94
- Family	Person-day	60	0.00	0	0.00
Gross Return:					
- Main product	kg	4,000	565.00	2,260	893.28
Net Income in 1998 Prices				1,392	550.32

Table V.5 Financial budget of 1 ha of maize in Somalia

Item	Prices (1979)	
	Somali Shilling	USD
Materials:		
- Seed - 20 kg	19	3
- Fertiliser - 110 kg N	315	50
- Fertiliser - 25 kg P2O5	79	13
- Herbicide - 5 lit Primagram 500 FW	307	49
- Pesticide - 5 lit Nuvacron Combi	184	29
- Aerial spraying - 3 applications	105	17
Machinery Operations (excluding operators):		
- 150 hp tractor - 2.23 h	282	45
- 110 hp tractor - 2.41 h	155	25
- 75 hp tractor - 1.56 h	61	10
- Combine - 1.25 h	241	39
- Equipment costs	59	9
Machine Operators	45	7
Machinery Depreciation	491	78
Unskilled labour (22 man days)	264	42
Processing:		
- Drying and storing	35	6
Returns (40 quintals)	3,880	620
Net Income	1,238	198
Net Income in 1998 Prices *		445.50

* Inflation rate in the USD from 1979 to 1998 = 125%

Table V.6 Financial budget of 1 ha of cotton in Somalia

Item	Prices (1979)	
	Somali Shilling	USD
Materials:		
- Seed - 30 kg (undelinted)	17	3
- Fertiliser - 80 kg N	229	37
- Fertiliser - 25 kg P2O5	79	13
- Herbicide - 2.8 lit Treflan	154	25
- Pesticide - 2.5 lit Nuvacron Combi C500	734	117
- Aerial spraying - 10 applications	350	56
Machinery Operations (excluding operators):		
- 150 hp tractor - 1.89 h	239	38
- 110 hp tractor - 2.41 h	155	25
- 75 hp tractor - 1.83 h	71	11
- Equipment costs	57	9
- Transportation to Jamama	275	44
Machine Operators	37	6
Machinery Depreciation	272	43
Unskilled labour (103 man days)	1,236	197
Returns (25 quintals)	7,150	1,142
Net Income	3,245	518
Net Income in 1998 Prices *		1165.50

* Inflation rate in the USD from 1979 to 1998 = 125%

Table V.7 Average net incomes from the main crops grown in system *A*

Crop	Average Net Income	References
Rice	$175	Tables 9.1 to 9.3
Maize	$500	Tables 9.4 & 9.5
Cotton	$1,165	Table 9.6

V.3 Cost of Sediment Cleaning

The cost of cleaning the sediment from irrigation canals have been estimated using data from three different sources:

(1) The unit rates for construction works in Sri Lanka (Ministry of Irrigation, Power and Energy, 1996) - Table V.8.

(2) The composite schedule of rates of the Punjab Province in Pakistan (Standing Rates Committee for the Punjab, 1998) - Table V.9.

(3) Unit costs from Indonesia (Horner, 1991) - Table V.10.

To standardise the calculations, the cost of cleaning 1 m^3 of sediment from irrigation canals and hauling the excavated material for a distance of 1 km was estimated using data from each of the three sources. Sediment cleaning was assumed to be carried out using manual labour, while hauling would be carried out using dump trucks.

Table V.8 Estimation of the costs of cleaning 1 m^3 of sediment according to the unit rates of Sri Lanka (1996)

Work Description	Cost (1996 Prices)	
	Sri Lankan Rupees	USD
Desilting in canals and spoil to waste, cut 0-1.5 m, lift 1.5 m & haul 30 m	120.00	2.17
Haul earth for 1 km	192.16	3.48
Total	312.16	5.65
Total Cost in 1998 Prices *		5.87

* Inflation rate in the USD from 1996 to 1998 = 3.93%

It should be noticed that the total cost estimates in Tables V.8 & V.10 agree together to a large degree, unlike the estimate in Table V.9. Although the information in Table V.9 were taken from an official government source, it was considered unlikely that there will be such a large difference in the costs of carrying out the same type of work in three countries which have more or less the same characteristics. Consequently, the final estimation for the cost of cleaning and hauling 1 m^3 of sediment from irrigation canals was taken as the average of the total costs in Tables V.8 & V.10, i.e. **$5.5 (1998 prices).**

Table V.9 Estimation of the costs of cleaning 1 m^3 of sediment according to the schedule of rates of Pakistan (1998)

Work Description	Cost (1998 Prices)	
	Pakistani Rupees	USD
Earthwork excavation in irrigation channels, drains, etc. to designed section grades and profiles, excavated material disposed off and dressed within 15 m lead	20.85	0.43
Transportation of all types of earth for up to 400 m	13.00	0.27
Transportation of all types of earth for 600 m beyond 400 m	16.50	0.34
Total	50.35	1.04
Total Cost in 1998 Prices		1.04

Table V.10 Estimation of the costs of cleaning 1 m^3 of sediment according to the prices in Indonesia (Horner, 1991)

Work Description	Cost (1991 Prices)	
	Indonesian Rupees	USD
Labour for cleaning 1 m^3 of sediment	6,000	3.08
Foreman	225	0.12
Running cost of 4 m^3 capacity dump truck	1,965	1.01
Drivers for dump trucks	111	0.06
Total	8,301	4.26
Total Cost in 1998 Prices *		5.10

* Inflation rate in the USD from 1991 to 1998 = 19.69%

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