DISSERTATION

CONJUNCTIVE USE OF SURFACE WATER AND GROUNDWATER WITH DIFFERENT SALINITIES IN THE INDUS BASIN OF PAKISTAN

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ABSTRACT

CONJUNCTIVE USE OF SURFACE WATER AND GROUNDWATER WITH DIFFERENT SALINITIES IN THE INDUS BASIN OF PAKISTAN

A mathematical model for optimal conjunctive use of surface water and groundwater is developed to determine canal and tubewell installed capacities in three different groundwater salinity zones. The objective is to minimize the total capital investment, and the operational and maintenance costs, for the system to satisfy a given irrigation water requirement.

The Lower Jhelum canal command, one of many similar hydrologic areas in the Indus Basin, is selected as the area for testing the mathematical model. The system is decomposed into a two-level approach for easier problem solving by separating the design variables and the operational variables. In the design level, the flexible tolerance algorithm is used to search iteratively for the optimal design alternative. Each time a design alternative is chosen, the design variables are considered as fixed parameters and a sequential decision process is used to determine the optimal operational decisions within a time interval. During each subperiod, direct river diversion will be the most feasible solution whenever the available river flow can satisfy the water requirement without causing water logging in the three areas and lateral salt water movement to the relatively fresh water area. Otherwise linear programming is adopted to allocate the available river flow and usable groundwater subject to constraints of water availability, canal capacity, water logging, salt water coning, lateral salt water movement and the water requirement.

The study shows that through conjunctive use of groundwater and surface water, an irrigation system can be designed as an "on demand" system providing sufficient water to meet a cropping intensity of at least 150 percent without waterlogging and salt water contamination. An optimal conjunctive use policy would transfer available surface water to the more saline groundwater areas, and the existing canal capacity would have to be expanded. Generally groundwater in each of the three different areas would be pumped for their own use except the amount which must be exported for salt balance and control of the water table.

The mathematical model is applicable to other canal commanded areas in the Indus Basin, Pakistan and other areas with similar groundwater salinity problems.

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CHAPTER I

INTRODUCTION

General

Water shortage due to an inadequate water supply has been a key factor in the low agricultural production and the shortage of food that occurs in Pakistan. The population increase - at about three percent a year - has intensified the food problem and calls for the development of additional water supplies and proper water management.

Rainfall and river run-off in Pakistan are highly variable and concentrated in a short period during the summer months when water availability is more than adequate for irrigation, while in the other months water becomes very scarce or unavailable. Due to the small land surface slope existing over most of the area, river flow cannot be regulated at this time due to limited numbers of reservoirs for storage of excess water during the monsoon season. Accentuating the problem for the future are the limited number and high cost of suitable reservoir sites. Groundwater is another important source of water for irrigation in Pakistan, and the groundwater aquifer can provide the huge storage capacity needed for regulating the water supply.

Improper management and poor practice connected with the use of groundwater might lead to some serious problems such as waterlogging due to the rise of water table from recharge of surface water. The maximum conservation utilization and regulation of the available water supply must be through the proper management of the conjunctive use of ground water and surface water.

Water quality is another important factor for proper management. A conjunctive use system which considers quantity alone might not produce the optimum results, since water salinity will put a constraint on the use of some water and thus reduce the amount of water which might be considered available when quality is not considered. Hence, in areas where some parts of the groundwater are too saline to be used, it is necessary to allocate the available fresh water from surface water sources and the fresh groundwater aquifer while preventing the salt water contamination due to salt water coning and lateral salt water movement. It will also be necessary to export portions of the recycled water out of the area to achieve a long term salt balance.

Objective and Scope

This study involves the redesign and operation of an irrigation system for optimizing conjunctive use of ground and surface water resources of part of the Indus Basin in Pakistan with consideration of groundwater quality. Due to the complexity of the existing irrigation system of the Indus Basin, the overall system is decomposed into individual canal subsystems for easier study and problem solving. This study will thus be limited to lower level optimization of the individual canal subsystem. The available surface water at the head of the canal and groundwater with different salinities beneath the area are allocated optimally to satisfy the given water demands during each subperiod. The optimal decisions for the individual canal subsystem can then be fed back for the overall system optimization which is defined as the master problem and is not included in this research.

Within each canal subsystem, the area is divided into three different groundwater quality zones, i.e. nonsaline, intermediate and saline zones, according to the quality-of-water standards adopted by Tipton and Kalmbach, Inc. U.S.A. (T&K, 1967). These zones are classified on the basis of the mineral content of the water defined from the water quality data available at depths from 100 to 600 feet. The mineral concentrations of the groundwater, commonly referred to as salinity, is expressed in terms of parts per million (ppm) of total dissolved solids (TDS). The three zones are defined as follows:

1. Nonsaline zone, TDS $\leq$ 1500 ppm ;
2. Intermediate zone, 1500 ppm < TDS $\leq$ 4000 ppm ;
3. Saline zone, TDS > 4000 ppm .

The groundwater can also be divided vertically into two layers; i.e. the upper fresh water layer and the lower saline water. The fresh water layer varies in thickness within each of the three zones. In the nonsaline zone the fresh water layer is sufficiently thick to support withdrawal by tubewells of large volumes of water having a TDS content of less than 1500 ppm . In the intermediate zone the fresh water layer is thinner and of poorer quality and water pumped by tubewells must be diluted with surface water prior to use for irrigation. In the saline zone most of the water pumped is from the lower layer and is discharged as drainage water as it is unfit for irrigation. Special low capacity skimming wells are being developed to skim off the shallow fresh water layer in the saline zone and this water is mixed with other surface water for irrigation.

To reach the objective of this study it is essential to formulate a mathematical model to determine the optimal design capacities of the canals and tubewells in a canal commanded area having three different ground water quality zones. The optimal solution would minimize the total cost of the system, including capital, and operational and maintenance costs within the given time span to meet irrigation water demands.

The Lower Jhelum canal commanded area has been selected for testing the mathematical model. It covers a total cultivable area of 1.5 million acres, and has a serious groundwater salinity problem. Data related to the three groundwater quality zones are available for formulating the mathematical model.

The available surface water from the three main rivers of the Indus Basin is assumed to be allocated to the area in proportion to the mean historical withdrawal of the area. Monthly irrigation water demands at heads of watercourses are available from a T&K study based on a cropping intensity of 150 percent (i.e. 1.5 crops per year).

The recharge to the aquifer takes place by seepage from the canal distribution system and watercourses, and from the deep percolation of irrigation water and rainfall. Additional recharge of the aquifer in the nonsaline area can be provided by increasing paddy rice acreage, over-irrigation of other crops or flooding the fallow lands to store the surplus water from rivers.

The objective function and constraints are assumed linear based on the physical model developed in this study for the Lower Jhelum canal commanded area. The optimization problem, however, can not be efficiently solved directly by linear programming due to the large number of variables involved. It is necessary to simplify the problem using some intuitive judgements. The simplified problem is still too large and needed to be decomposed. The problem has been further decomposed into a two level problem, i.e. the design problem and the inner operational problem. The design alternatives are searched in an optimal manner, while within each alternative the inner operational problem has been divided into a number of independent sub-periods, and determined optimally within each subperiod. Direct diversion of the available surface water turns out to be the most feasible scheme during the high flow seasons as long as the constraints of the system can be satisfied. Otherwise, a linear programming subroutine is used to determine the optimal allocation of surface water and tubewell water for satisfying given water demands in three different groundwater quality zones.

The mathematical model developed can also be applied to other canal commanded areas in Pakistan or other areas of the world with similar groundwater salinity problems.

In this dissertation, Chapter II reviews briefly the literature of the conjunctive use of groundwater and surface water. Chapter III describes briefly the Indus Basin Irrigation System, its problems and considerations toward conjunctive use of groundwater and surface water. A physical model then is defined and described in Chapter IV. The cost functions for defining the objective function is also described in detail. The fifth chapter presents the mathematical model including simplifications and descriptions of the objective function and constraints. Solution techniques and procedures are also described. The sixth chapter presents the results of the computation. The last chapter summarizes the research endeavor, and suggests items for future studies. Symbols, notations and special terms used in this research are summarized in Appendix A. Appendix B gives a review on the flexible tolerance method which was used for searching the optimal design alternative. Some of the associated analysis and developments such as salt water coning, lateral salt-water movement, recharge coefficients and development of cost functions related to the formulation of the problem are presented in Appendix C. A listing of the computer program is included as Appendix D. Appendix E provides the results for four of the different computer runs.

CHAPTER II

LITERATURE REVIEW

The increasing pressures throughout the world for better management and higher efficiency of water use to satisfy the increasing food demands have called for comprehensive development of water resources and also the consideration of aquifer development for conjunctive use of surface water and groundwater. With more understanding of the characteristics of the groundwater aquifer and the recent advance in system analysis and computer programming techniques providing more efficient tools, the subject has received much attention in water development and management project.

The literature on conjunctive use of surface water and groundwater covers a very broad spectrum from concepts to actual field applications. System analysis and optimization techniques have been applied to conjunctive water use since 1960. The mathematical models developed deal mostly with concepts rather than with actual field application.

Development of General Concepts

Conkling (1946), Kazmann (1951), Banks (1953), Thomas (1955), Todd (1959), and ASCE Committee on Groundwater (1961) are among many of the prominent hydrologists and organizations who have discussed the potential of conjunctive use in general terms. The physical, engineering, financial, economical and legal complexities of the problem had been explored and delineated. The important aspects of accessibility, availability and dependability have been identified with respect to groundwater use. The advantages and methods of artificial recharge have been discussed. The prevention and control of seawater intrusion have also been considered (Todd, 1959; ASCE Committee, 1961).

Fowler (1964) emphasized the need of conjunctive use for optimum water resource management and the need for knowledge of the geology, hydrology, available water supplies, existing water supply facilities, and future water demands for the area under consideration. Furthermore, he emphasized the need for adequate institutional arrangements to control and coordinate the system.

Kazmann (1965) classified aquifers in accordance with their primary function. He recognized that an aquifer can function as a filter plant, a reservoir and a mine. Hall and Dracup (1967) explained further that a groundwater aquifer has six properties which must be considered. They are:

1. Safe yield to ensure a balance between inflows and withdrawals.
2. Volume of groundwater which is capable of being mined.
3. Reservoir for long term storage.
4. The ability of the basin to act as a water distribution system. That is aquifers have economic value as a transmission system in partial replacement for surface distribution systems.
5. Energy resource represented by modified pumping lift through management (i.e. conserving energy by reducing pumping lift).
6. Water quality management through use of the filtering characteristics of the aquifer.

Chaudhry (1973) cited that a seventh property as a recycling facility can be added. However, deteriorating water quality will be a factor which will put a constraint on the amount of recycling.

Economic Approaches

Economics is considered the major factor in the study of a project for optimal water resources allocations. In a study of the Coastal Plain of Los Angeles County, California, for the conjunctive use of ground and surface water (Chun, Mitchell and Mido, 1964), a general cost equation was derived to obtain the most economical combination of pumping and storage facilities. Alternative plans were studied and the one with the least total cost was selected as the most economical plan. The approach used was actually a trial and error procedure and, since it is impossible to try all possibilities, one cannot be sure that the final solution was the one with the lowest cost.

Renshaw (1963) argued that the decisions concerning the use of groundwater resources should be based on their cost. The problem deals with the comparison of present values associated with present use, i.e. mining of groundwater and the value of groundwater left in the ground. It is noted that the water left in the ground has a greater value than can be obtained for certain low-value uses above the ground. Kelso (1961) provided another example with the same reasoning. On the contrary Koenig (1963) stated that the current rate of withdrawals in the USA is too conservative and argued that groundwater overdrawn in an area is compensated through import from ample groundwater elsewhere. However, problems such as seawater intrusion and land subsidence were not considered.

Legal and Organizational Considerations

Groundwater law is much less developed than that for surface water. This is in general due to the lack of thorough understanding of the mechanics of groundwater movement; the lack of specific information on the physical characteristics of the aquifer; slow development of groundwater use; and the lack of effective control over the movement of groundwater. The conjunctive use of groundwater and surface water undoubtedly creates other legal problems in addition to the existing ones, such as water rights and adjudication.

For an efficient groundwater management program, the governing agency must have the legal authority (ASCE committee, 1961) to do the following:

1. Purchase water supplies.
2. Spread water for recharge.
3. Acquire lands and improvements by eminent domain.
4. Protect the basin with regard to water level and water quality.
5. Influence pumping practice.
6. Obtain revenue.

Water Qaulity Considerations

In the past, the investigations and planning of water resources have been quantity-oriented to develop additional water supplies for meeting water demands, often disregarding water quality. But as more and more water resources are developed, quality becomes important and inseparable from quantity. Water must be of suitable quality for the specific beneficial use. The quantity of water used may be limited due to quality constraints. Deterioration of water quality results from, and depends on, both natural and man-made causes. Agriculture land use and waste water discharge to the basin affect and degrade both surface and groundwater quality. Loss of water through evaporation from the ground surface and transpiration through plants leave salts underground and cause the increase of groundwater salinity. The degree of salinity is further increased as the groundwater is recycled. The quality of groundwater also changes gradually by natural mineral solution or chemical reaction in the aquifer, or by contamination due to lateral salt water movement from more saline areas to fresh water areas, or due to sea water intrusion into coastal aquifers.

The Upper Santa Ana River Basin groundwater quality simulation model was developed in 1967 through the joint efforts of California State Department of Water Resources and Water Resources Engineers, Inc. to study the change in water quality as a function of time and space. The measure of total dissolved solids was used to represent the water quality (California Department of Water Resources, 1967).

The Harvard Water Resource Group (1965) constructed a salt flow simulation model for Pakistan determining the build-up characteristics of salt in the irrigation waters for various values of the well field parameters including well spacing; well depth; percentage of tubewell effluent to drainage; pumping rate; initial groundwater concentration; and amount of salt on or near the surface of the ground. They concluded that, in general, drainage should be provided at a rate of about 10% of the pumped water in all cases to keep the concentration of applied irrigation water within reasonable limits. The assumption made in this study for a long term salt balance was based on this conclusion.

They also developed a mathematical model for determining optimum allocations of surface and ground water supplies between two areas of high and low groundwater salinity in the Punjab and Bahawalpur region, Indus River Basin, Pakistan. The simulation model mentioned above and several simulation models, with their hydraulic interactions under various pumping schemes, were introduced as a foundation for the optimization model. Salinity, sodium, mixing, mining-export, and areal loading constraints were defined. The nonlinear objective function was linearized in the vicinity of a feasible solution and linear programming was used to maximize the net return. This problem is relevant to this research. But it is oversimplified in a large complex area by simply deciding how much water should be transferred from the nonsaline area to the saline area on a yearly basis. The decision of how much water to be pumped from the nonsaline area was predetermined. This excluded the possibility of optimum conjunctive use. The study was based on time intervals of one year, but the availability of water during the wet season and the dry season is quite different and this will greatly affect the allocation policy and the groundwater pumping decision.

The water quality problem has also drawn great attention in Israel in connection with its water resources development (Buras, 1963b, 1967). The use of an aquifer with good quality groundwater in conjunction with more saline surface water was analyzed. The system state included consideration of the amount of water stored in a surface reservoir, the amount of water in the aquifer and the salinity of surface water.

Application of Optimization Techniques

The conjunctive use system has been analyzed as a lumped or as a distributed system. In the distributed models the aquifer parameters are distributed into nodes throughout the basin. In the lumped models the parameters of the system are considered as aggregated for the entire basin. Simulation techniques have been widely used for the distributed models while other mathematical programming techniques such as linear programming and dynamic programming have been mainly used for the lumped ones.

A general purpose analog and digital computer model representing the water supply, distribution and replenishment system of the Los Angeles Basin in California was developed by applying a simulation technique in the early 1960's (Tyson and Weber, 1964, Weber, 1968). The basin was divided into polygons for the detailed simulation study. From the results it was concluded that the electronic differential analyzer or analog computer were advantageous in the modeling phase, while the digital computer was best suited for operation analysis of the model.

Eshett and Bittinger (1965) prepared a computer simulation program to analyze the stream aquifer system. Useful relationships between the components of the system were developed for analyses and design purposes.

Applying linear programming, Castle and Lindeborg (1961) tried to allocate water between two agricultural areas for maximizing beneficial use of the resource. The benefit function was assumed linearly proportional to the amount of water use.

Dracup (1966) used parametric linear programming to find the optimal groundwater and surface water allocation for a 30-year period in the San Gabriel Valley of California. Five sources of water were utilized optimally to satisfy three water requirements. Cost coefficients and water demands were varied for parametric analysis.

Milligan (1969) formulated several linear programming models for groundwater and surface water systems in order to maximize the net-return from the system. The models which were developed included a general deterministic model, a general stochastic model in which hydrologic inputs were allowed to be probabilistic, and models of two simple, but real, river basins. The aquifer was divided into several layers so that the pumping lift from each layer could be assumed constant.

Rogers and Smith (1970) formulated a deterministic linear programming model for planning an irrigation system. The objective was to maximize the annual net return considering crop return and project cost. The canal, tubewell and drainage capacities, project size and cropping pattern were selected from the program. The operation was based on a monthly schedule and extended only for one year. Mining of groundwater was not allowed between years. Their sensitivity studies showed that the optimal solution was insensitive to a wide range of canal and tubewell capital and maintenance costs, but was sensitive to the cost of energy for pumping. The inclusion of surface reservoirs, recharge facilities, water quality and salinity intrusion were discussed but were not considered in their study.

Longenbaugh (1970) formulated a linear programming model for the stream-aquifer system. Instead of using lumped parameters for the aquifer, he divided the basin into blocks and finite difference equations were used to define the set of constraints. A small hypothetical problem with only four blocks was demonstrated.

Buras and Hall (1961) first introduced dynamic programming to the conjunctive use aspect of groundwater and surface water. The problem was to determine operational allocations from surface and ground water reservoirs and evaluate surface storage requirements. It was assumed that demands were known over the life of the project. In their study, the operational problem was first considered on the basis of whether to allocate water to the surface reservoir or to the groundwater reservoir. It was shown that allocation of water to both storage facilities should be an "all or nothing" decision. Secondly, they demonstrated the use of dynamic programming to determine the optimum surface storage capacity.

Buras (1963a) postulated a simplified one reservoir - one aquifer system each with an independent irrigation area and benefit function. Three states which represent the amount of water available in the surface reservoir, in the aquifer, and the amount of water in transit to the aquifer were involved in determining the optimum operating policy. The model is far from the real situation where both sources of water need to be used on the same area. This application of dynamic programming to the water resource problem was a major contribution. It was also pointed out that by changing the design parameters, the optimum system design as well as its optimal policy could be determined through comparison of all design alternatives. Buras also extended his work to other similar problems (Buras 1963b, 1965).

Burt (1964a) used dynamic programming to derive approximate decision rules in the form of a functional equation for optimal resource allocation with a fixed or only partially renewable groundwater resource over time. The decision rules in general specified that production should be expanded until marginal net output equals marginal recovery cost. This was defined as optimal safe yield of the aquifer. Several other problems including the temporal allocation of groundwater and stochastic considerations were also studied by Burt (1964b, 1966, 1967a, 1967b, 1970).

Aron (1969) developed a model for regional water conservation and distribution with conjunctive use of groundwater and surface water. The northern portion of the Santa Clara Valley was chosen as the physical model. The whole system was decomposed into several subsystems which were preoptimized to give optimal parameters in overall system optimization, There were three state and 12 decision variables. The multidimensional character of the decision vector required inner optimization by a steepest descent method within each stage of the dynamic programming solution.

Clausen (1970) applied quadratic programming to solve the water supply problem in the Tucson Basin, Arizona. The objective was to maximize the gains from the sale of water from surface and underground to the users within the basin. The concept of economic demand was used to estimate the amount of water that different users would purchase at different prices. The objective functions were in quadratic forms.

Chaudhry (1973) formulated a mathematical model for an area within the Marala-Ravi Link Canal system in the Indus Basin, Pakistan. The objective of his study was to determine the size of the canal, the capacity of the surface reservoir and the tubewell installation capacity so that the overall capital and operation and maintenance costs of meeting the given monthly irrigation water requirements were minimized. He emphasized the need for integrating an empirical approach into the theoretical optimization techniques in order to simplify a complicated water resources system. The problem was divided into a two-level problem, the design problem and the inner operational problem. The inner operational problem was optimized through the use of dynamic programming for the wet and dry seasons separately. Some apriori decisions were made from physical considerations. He pointed out that the irrigation water obtained by direct river diversion into the canal system is the cheapest. For a given capacity of the canal system the optimal policy is to divert water directly from rivers to the maximum possible extent so that the cost of groundwater pumping can be minimized. The model area was underlain by a fresh water aquifer and he assumed groundwater quality was satisfactory for irrigation. A systematic search method was developed to obtain the optimal design alternative.

Other literature which relates to the conjunctive use of ground and surface water but not cited here is included in the bibliography.

CHAPTER III

CONJUNCTIVE USE OF GROUNDWATER AND SURFACE WATER IN THE INDUS BASIN OF PAKISTAN

General Description of the Indus Basin.

The Indus Basin, a vast and flat alluvial plain extending south from the Himalaya Mountains, is traversed by the Indus River and its tributaries - the Kabul, Jhelum, Chenab, Ravi, Beas and Sutlej. These tributaries converge gradually and ultimately join the main Indus in the northern part of Pakistan. Below Gudu, the Indus River extends southward to the Arabian Sea. The basin is formed of alluvium, deposited by the rivers to depths of several thousand feet, forming an essentially featureless level plain with an average slope of about one foot per mile toward the sea. Figure 3.1 is a map showing the Indus River System and locations of barrages, links and canals.

The climate of the Indus Plain is arid to semi-arid and is characterized by large seasonal fluctuations in temperature. Maximum daily temperatures of 100 to 120 °F are common during the summer months. Winter months are generally cool with daily temperatures ranging from 35 to 75 °F . Evaporation is high, and the highest evaporation occurs during the summer season from April to September. The annual lake evaporation varies from 57 to 75 inches in the north and from 72 to 87 inches in the south. Rainfall is highly variable with respect to time and location, and therefore, not a dependable source for crop moisture. Annual rainfall ranges from more than 30 inches at the foothills of the Himalayas in the north to less than 6 inches in the south. About 50 percent of the total annual rainfall falls in the

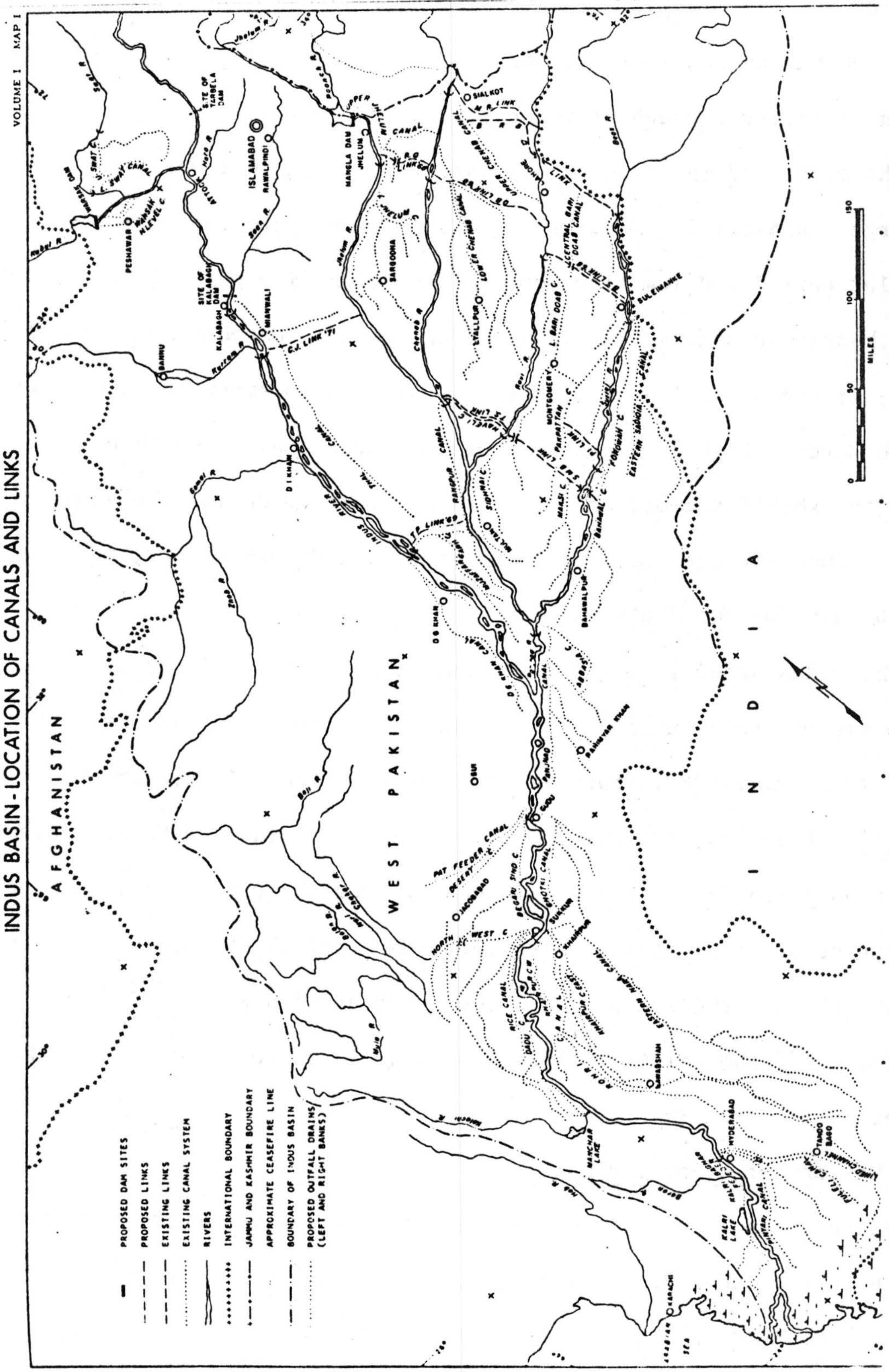


Figure 3.1. (Adopted from Lieftinck, et al., 1968).

months of July and August. As a consequence of the relative high temperature; low and uneven seasonal and areal variation of rainfall, irrigation is necessary throughout the entire basin.

The rivers of the Indus system have great seasonal variation in discharge. Run-off in the rivers can be divided into two periods, i.e. high flow period and low flow period. The high flow period, or the so-called kharif season, starts in April and ends in October. The low flow period, or the so-called rabi season, starts in November and continues through March. Eighty percent or more of the annual run-off occurs during the kharif season, and 50 to 60 percent during the summer months - June through August for the Indus and Chenab Rivers, and May through July for the Jhelum River.

The Indus Water Treaty of 1960 entitles India to divert all flows of the eastern tributaries - Ravi, Beas and Sutlej - for her own use after 1970. Pakistan in turn has the right to the full use of the Indus itself and the two western tributaries - Jhelum and Chenab. The total flow potentially available annually from these three rivers is on the average of 135 million acre feet (MAF), of which 40 MAF presently runs off into the sea unused. Construction of new reservoirs and enlargement of the existing canal system would be required to provide greater use.

Brief History of Canal Irrigation.

Since irrigation is a prerequisite for extensive agriculture production in the Indus Basin, throughout the recorded history of the area man has contrived ways to divert water to the lands. In the early period irrigation was restricted to the active flood plains by utilizing flood water. About the end of the seventeenth century irrigation was extended with the so-called inundation canals which drew water from rivers during periods of high flow stage to convey water to lands lying along the rivers above the flood plains. This kind of irrigation, however, was limited to the summer season and to a relatively narrow belt along the rivers. Late in the middle of the nineteenth century when the British entered the subcontinent, they conducted extensive experiments and research to enhance the usage of river flows. This led to the construction of the largest irrigation system in the world. Permanent headworks and barrages were constructed on the rivers to place the innundation canals under weir control. This made it possible to divert large quantities of river water out onto a broader area and some canals were able to divert continuously throughout the year.

The partition of India into two sovereign states, Pakistan and India, in 1947 resulted in a long dispute of the water rights on the Indus River System. A plan was established in 1960 along with the signing of the Indus River Treaty. It included the construction of two major storage reservoirs, Mangla Dam on the Jhelum River and Tarbela Dam on the Indus River; and, construction of new or remodeling of existing barrages and link canals to transfer water from the western rivers to meet the irrigation water requirements of the eastern portion of Pakistan. This project is now virtually complete with the last phase, Tarbela, to be commissioned in 1975.

At present, the irrigation system of the Indus Plains commands a gross area of about 38 million acres and comprises some 38 thousand miles of canals and a series of river barrages and canal headworks which control the diversion of river flows into the canals. The total irrigated area is covered essentially by 42 principal canal commands which cover 33.5 million acres of the culturable commanded area (CCA). Table 3.1 gives the data on principal canal commands.

It was realized in 1950 that a basin-wide comprehensive plan was necessary for the development of water and power resources for the area and international efforts have been involved. In 1958 the Water and Power Development Authority (WAPDA) of West Pakistan was organized to take charge and unify the resources development of the area. The Indus Basin Replacement Plan mentioned above was one of the first efforts to implement the concept of the comprehensive development. Harza (1963, 1968), Revelle Group of the U. S. White House Panel (1964), Huntings (1966), Irrigation and Agricultural Consultants Association (IACA, 1966); Tipton and Kalmbach (T&K, 1967), Lieftinck Group of World Bank (1969) and USAID are the major organizations which have contributed to the development of the area. The present research and studies conducted by Colorado State University under USAID sponsorship continue the multi-dimensional international efforts. A multi-disciplinary approach is being utilized to combine the efforts of engineers, economists, agronomists, and other experts to improve the use of the agricultural water of the basin.

<u>The Problems.</u>

Despite huge investments which have been made over the years in irrigation works, agricultural production - especially food grains - has increased quite slowly. The rate of increase in population has exceeded the rate of increase in food production. In Pakistan, agriculture is the major economic factor and more than 50 percent of the labor force is employed by agriculture, but food still has to be imported to provide an adequate diet for 60 million people.

TABLE 3.1

Data on Principal Canal Irrigation Systems
West Pakistan
(in 1000 Acres)

River	Headworks(2)	Canals	Capacity Cusecs	Operating Since	Commanded Area Gross	Culturable Perennial	Culturable Non-perennial	Culturable Total	Area irrigated 1960/61 Kharif	Rabi	Total
(1) Peshawar Vale											
Swat	Amandara	Upper Swat	.. 1,800	1915	319	276	—	276	192	136	328
	Munda	Lower Swat	.. 800	1890	147	134	—	134	116	72	188
		Sub-totals	.. 2,600		466	410	—	410	308	208	516
		High level									
Kabul	Warsak dam	Left bank	.. 45	1962	13	11	—	11	—	—	—
		Right bank	.. 455	1962	125	108	—	108	—	—	—
	—	Kabul River	450	1890	92	77	—	77	38	30	68
		Sub-total	.. 950		230	196	—	196	38	30	68
	Total Peshawar Vale		.. 3,550		696	606	—	606	346	238	584
(2) Northern Zone—Indus Plans											
Jhelum	Mangla	.. Upper Jhelum	.. 1,900(1)	1915	580	367	174	541	292	247	539
	Rasul	.. Lower Jhelum	.. 5,300	1901	1,622	1,284	215	1,499	658	732	1,390
		Sub-total	.. 7,200	—	2,202	1,651	389	2,040	950	979	1,929
	Marala	.. M-R Link	.. 2,000(1)	1956	179	160	—	160	31	15	46
	,,	.. Upper Chenab	.. 4,100(1)	1912	1,511	613	832	1,445	549	341	890
Chenab	Khanki	Lower Chenab	.. 11,500	1892	3,703	2,831	156	2,987	1,424	1,654	3,078
	Trimmu	.. Rangpur	.. 2,700	1939	380	—	347	347	105	126	231
	,,	.. Haveli-Sidhnai	.. 5,200	1939	1,123	668	343	1,011	538	547	1,085
		Sub-total	.. 25,500	—	6,896	4,272	1,678	5,950	2,647	2,683	5,330
	Madhopur (2)	Central Bari Doab	2,600	1859	704	642	—	642	321	249	570
Ravi	Balloki	.. Lower Bari Doab	.. 7,000	1913	1,822	1,417	43	1,460	827	811	1,638
	Sidhnai	.. Sidhnai	.. (4,500)	1887	—	—	(Included in Haveli data)			—	—
		Sub-total	.. 9,600	—	2,526	2,059	43	2,102	1,148	1,060	2,208
	Ferozepore(2)	Dipalpur	.. 6,100	1928	1,045	—	983	983	321	256	577
	Suleimanke	Pakpattan	.. 6,600	1927	1,396	920	341	1,261	525	535	1,060
	,,	Fordwah	.. 3,400	1927	465	60	365	425	138	122	260
Sutlej	,,	Eastern Sadiqia	.. 4,900	1926	1,134	915	22	937	429	355	784
	Islam	.. Mailsi	.. 4,900	1928	751	—	688	688	287	215	502
	,,	Qaimpur	.. 600	1927	45	—	42	42	14	15	29
	,,	Bahawal	.. 5,400	1927	791	274	374	648	248	228	476
		Sub-total	.. 31,900	—	5,627	2,169	2,815	4,984	1,962	1,726	3,688
Panjnad	Panjnad	.. Panjnad	.. 9,000	1929	1,505	444	895	1,339	624	515	1,139
	,,	Abbasia	1,100	1929	131	68	42	110	48	41	89
		Sub-total	.. 10,100	—	1,636	512	937	1,449	672	556	1,228
Indus	Jinnah	.. Thal	.. 10,000(3)	1947	1,855	1,473	—	1,473	275	474	749
		Sub-total	.. 10,000	—	1,855	1,473	—	1,473	275	474	749
Indus	—	.. Paharpur	.. 500	1909	102	—	100	100	24	41	65
		Sub-total	.. 500	—	102	—	100	100	24	41	65
Indus	Taunsa	.. D. G. Khan	.. 8,800	1958	730	729	—	729	160	210	370
		Muzaffargarh	.. 7,300	1958	721	714	—	714	164	266	430
		Sub-total	.. 16,100	—	1,451	1,443	—	1,443	324	476	800
	Total Northern Zone		.. 110,900	—	22,295	13,579	5,962	19,541	8,002	7,995	15,997

(1) Internal uses.
(2) Madhopur and Ferozepore headworks are in India.
(3) Ultimate capacity 10,000 cusecs; present capacity 6,000 cusecs.

(Continued)

TABLE 3.1 continued

Data on Principal Canal Irrigation Systems

West Pakistan

(in 1000 Acres)

River	Headworks	Canals	Capacity Cusecs	Operating Since	Commanded Area Gross	Culturable Perennial	Culturable Non-perennial	Total	Area Irrigated 1960/61 Kharif	Rabi	Total
(3) Southern Zone—Indus Plains											
		Pat	.. 8,300(4)	1962	766	—	712	712(5)	—	—	—
		Desert	.. 12,900	1962	479	—	420	420	159	150	309
Indus	Gudu	Begari-Sind	.. 15,500	1962	1,019	—	890	890	426	419	845
		Ghotki	.. 8,500	1962	1,004	—	995	995	138	130	268
		Sub-total	.. 45,200	—	3,268	—	3,017	3,017	723	699(6)	1,422(6)
		North West	.. 5,100	1932	946	928	—	928	214	403	617
		Rice	.. 10,200	1932	537	—	520	520	340	230	570
		Dadu	.. 3,200	1932	593	549	—	549	119	241	360
Indus	Sukkur	Khairpur West	.. 1,900	1932	323	304	—	304	91	166	257
		Rohri	.. 11,200	1932	2,614	2,604	—	2,604	845	1,010	1,855
		Khairpur East	.. 2,700	1932	531	335	—	385	150	184	334
		Eastern Nara	.. 13,400	1932	2,381	2,237	—	2,237	739	569	1,308
		Sub-total	.. 47,700	—	7,925	6,957	520	7,477	2,498	2,803	5,301
		Pinyari	.. 14,400	1955	802	—	786	786	217	15	232
	Ghulam	Fuleli	.. 13,800	1955	1,065	—	929	929	413	44	457
Indus	Mohammed	Lined Channel	.. 4,100	1955	675	487	—	487	29	30	59
		Kalri-Baghar	.. 9,000	1955	733	352	252	604	71	72	143
		Sub-total	.. 41,300	—	3,275	839	1,967	2,806	730	161	891
	Total Southern Zone		.. 134,200	—	14,468	7,796	5,504	13,300	3,951	3,663	7,614
	Total Indus Plains (2)+(3)		.. 245,100	—	36,763	21,375	11,466	32,841	11,953	11,658	23,611
	Total West Pakistan (1)+(2)+(3)		.. 248,650	—	37,459	21,981	11,466	33,447	12,299	11,896	24,195

(4) Ultimate capacity 8,300 cusecs; present capacity 6,300 cusecs.
(5) New area 509,000 acres; old area 203,000 acres.
(6) Partially irrigated.

Summary

Headworks – 20 including Warsak Dam; and Madhopur and Ferozepore in India. The Kabul River and Paharpur canals have only minor diversion facilities.
Canal systems—43, of which several function mainly as links but supply irrigated area directly.
Canal capacities—represent authorized full-supply discharges.

(Adopted from HARZA, 1963)

Water shortage is a major reason for the low production rate and low cropping intensities experienced in Pakistan. There is no doubt that other kinds of farm inputs such as fertilizer and pesticides for plant protection and growth are also essential for an increase of agricultural production. However, effective utilization of these inputs will be limited until more water becomes available. The farmer must be assured of the reliability of the available water before he will increase his investment in fertilizer and other farm inputs. Consequently, inadequate water supply is the primary constraint on crop production in Pakistan.

The lack of storage capacity and the inadequacy of parts of the present canal system are major factors causing the general water shortage. The present irrigation system was designed on the basis of water scarcity for a very low cropping intensity. The canal system was set up by and large to prevent famine, at a time when it seemed more effective to spread the water thinly to provide each area a measure of famine insurance. Thus the existing canals are not able, at their full capacities, to divert and deliver sufficient water throughout the commanded areas to support high levels of crop production on all the land even when sufficient surface water is available at the diversion points during the kharif season.

Water logging and salinity of the lands commanded by the existing irrigation system is another important factor contributing to the very low crop yield. But this was not always so. Prior to the construction of the canal system, groundwater tables in the Indus Basin were at a considerable depth ranging from 60 to 80 feet below the ground surface. The infiltration of water from rivers and the deep percolation of rainfall within any particular area was in equilibrium with the underground outflow from the area.

However, once the irrigation system came into operation, the pre-irrigation hydrological equilibrium was destroyed. The permeable soil which favors canal seepage had dissipated about 50 percent of canal diversions within the irrigation distribution system. The deep percolation of seepage from canals not only caused the losses of supply available for irrigation, but also formed a new increment of recharge. The overall recharge from the irrigation system, river and rainfall exceeded the rate at which water could flow out of the aquifer. As a result, water tables have steadily risen over the years at a rate of 1 to 2 feet per year. This trend persisted until the water table rose to within a few feet of the land surface and established a new equilibrium under which recharge from seepage losses is balanced by discharge to evaporation. The poor drainage condition and the upward evaporation of water from the water table resulted in a progressive salinization and waterlogging of the soil.

The salinity and waterlogging hazards were amplified by the man-made irrigation practice due to the application of insufficient water to a broader area as mentioned previously. The water applied was transpired by the crops leaving very little water to pass below the root zone with the result that most of the salts contained in the irrigation water remain in the uppermost soil layer. By 1958, about 5 million acres of the culturable commanded area was seriously affected by waterlogging and soil salinity problems (Revell, 1964). Furthermore, the hazards of waterloggins and salinity are increasing at the rate of 50,000 to 100,000 acres per year, of which about half goes out of production, and the rest is affected sufficiently to reduce crop production severely.

Groundwater Utilization.

Groundwater has, in fact, been a traditional source of water to help satisfy the need for irrigation water. Persian wheels, normally powered by animals, have always made an important contribution to irrigation especially in the rabi season. It is estimated that there are about 200,000 Persian wheels in the basin, but the discharge is so small and the operating time is so short that more efficient techniques and equipment are needed to draw more water from underground aquifers to cope with increasing demands. Installation of tubewells to pump more groundwater from a deeper depth for irrigation has increased in the last twenty years. IACA (1966) estimated that about 32,000 private tubewells, with an average capacity of about 1 cfs each, had been installed in the Indus Basin by 1965. About one-third of the tubewells are operated by electric power and the remainder by diesel engines.

A comprehensive program of groundwater and soils investigations was begun in 1954 under a cooperative agreement between the Government of Pakistan and USICA the predecessor of USAID. The investigators were to inventory the water and soil resources of the Punjab and to describe the relationships between irrigation activities, natural hydrologic factors, and the incidence of waterlogging and subsurface drainage problem. As a result, several salinity and reclamation projects (SCARP) were formulated and constructed. More than 6000 public tubewells with capacity ranging from 3 to 5 cfs have been installed.

Alluvial Aquifer Characteristics.

About 1030 test holes drilled by WASID over 47,000 square miles of the Punjab region during the 1950's defined the nature of the alluvium to depths of about 600 feet and provided water quality data to depths of 400 to 500 feet. Since 1962 WASID has also drilled about 95 deep test holes, 600 to 1500 feet deep, in the Punjab area (Bennett, 1967).

Geologic studies show that virtually all of the Indus plains are underlain to a depth of 1000 feet or more with unconsolidated sediment of alluvial origin. Scattered hills and bedrock outcrops have been found in some of the area. But, in general, sediments vary in texture from medium-grained sand to silty clay, with the sandy sediments predominating. The alluvial deposits are heterogeneous and anisotropic due to the random distribution of clay strata, but, generally, have the characteristics of an unconfined aquifer. According to WASID's experience, large capacity wells yielding 4 cfs or more can be developed almost everywhere.

The horizontal permeabilities ranged from 0.001 to 0.008 cfs per square foot and are commonly between 0.0025 to 0.004 . The vertical permeabilities are considerably less than the lateral permeabilities. In general, the ratio of the horizontal permeability to that of the vertical is on the order of 50 to 100 . The few calculations of vertical permeability which could be made indicate that the vertical permeability is in the range of 0.00001 to 0.001 cfs per square foot.

The storage coefficient, equivalent to the specific yield in an unconfined aquifer, is an important parameter in estimating the storage capacity of the groundwater aquifer, and the rise and fall of the water table due to tubewell pumping and recharge. For a broad aquifer area, a small change in the storage coefficient will have a great effect on the estimate of the volume of water stored and water table changes.

In 106 tests made by WASID, 90 percent of the storage coefficients were in the range of 0.02 to 0.26 and the average value was 0.14. According to Greenman, et al. (1967), assuming an effective porosity of 20 percent for the saturated sediment, the volume of usable groundwater in storage in the Indus Basin is on the order of 2 billion acre-feet.

Groundwater Quality and Its Distribution.

The quality of groundwater is best considered in two contexts: that of the native or the deep water which occurred in the alluvial aquifer prior to the inception of irrigation, and that of the shallow groundwater due to seepage from the irrigation system.

Data from the extensive groundwater quality investigations indicate a gradual increase in mineralization of groundwater with depth and distance from sources of fresh water recharge. Thus, even extensive fresh water areas appear to be underlain at different depths by saline groundwater in most of the Indus Basin.

There are factors affecting the distribution and concentration of highly mineralized groundwater. They include not only variation of the recharge from the river bounding the doab, and the areal pattern of rainfall and evaporation but also the physiographic characteristics such as direction, slope, symmetry and width of the doabs, size and position of bar land and the abandoned flood plains. Some of the local factors also affect the regional distribution pattern. For example, the presence of clay deposits within the alluvium are normally associated with higher salt concentrations surrounding that area.

The pattern of the chemical composition of groundwater reflects the geochemical evolution of the ground water in the hydrologic environment. Near the source of recharge, groundwater is of the calcium-magnesium bicarbonate type which commonly has a total dissolved solids content of 200 to 500 ppm. Away from the recharge source, sodium content gradually increases. Groundwater from 500 to 1000 ppm commonly contains a large amount of sodium bicarbonate. With increasing mineralization from 1000 to 3000 ppm, the relative proportion of chloride and sulphate increases. The salt in highly mineralized groundwater, containing 4000 ppm or more is generally a dominantly sodium chloride water.

The horizontal distribution of the groundwater quality in the aquifer of the Northern Indus Plain can be described from the contours shown on figure 3.2 which were drawn to represent the average conditions at depths of about 100 to 450 feet according to samples collected between 1957 and 1965 by WASID. In general, the groundwater quality varies from less than 200 ppm of the total dissolved solids (TDS) adjacent to the river and increases with the distance from the river to over 20,000 ppm TDS in the central part of the doab.

The quality of the shallow groundwater up to about 100 feet depth is largely controlled by local recharge and the depth to the water table. In general, the quality of the shallow groundwater supplies tends to have a regional pattern similar to that of the deep groundwater. However, the shallow groundwater in the saline area is of considerably better quality than the underlying deep groundwater. It appears that there may be considerable scope for developing irrigation water from parts of the area with deep saline groundwater by means of low capacity skimming wells. Skimming wells in the saline area would have two advantages. They would lower the water table in the saline area while supplementing the irrigation water supplies.

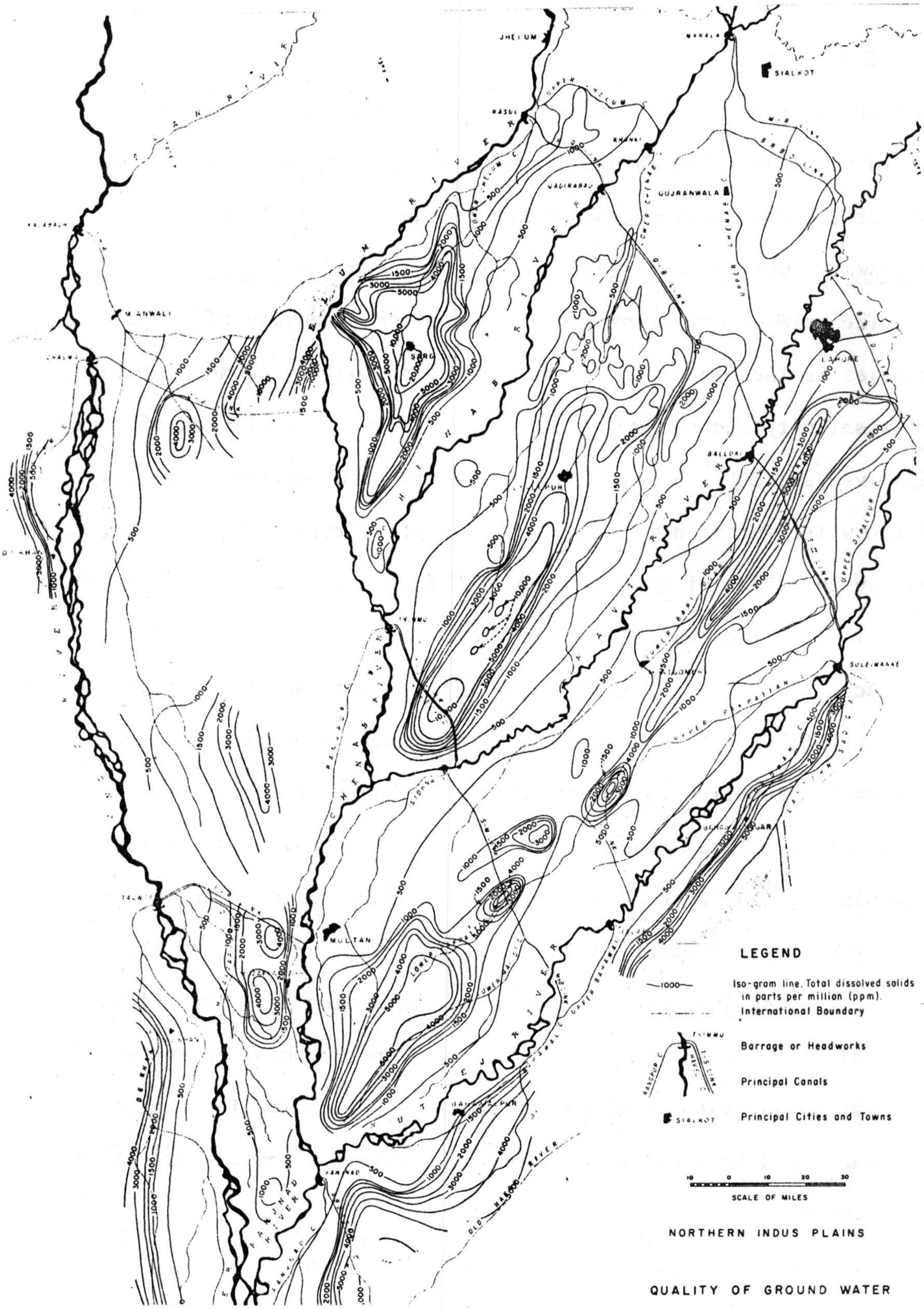


Figure 3.2. Area Distribution of Ground Water Quality (Adopted from Tipton and Kalmbach, 1967).

The general pattern of groundwater quality distribution in the southern zone is one of a band of good quality water immediately adjacent to the Indus River and of increasing salinity away from the river. Some of the most saline groundwater of the area is found in the delta.

Groundwater Quality Zones and Quality Criteria for Irrigation.

The primary criterion for classifying the quality of irrigation supplies is the mineral concentration of the water, commonly referred to as salinity and expressed in terms of "parts per million of total dissolved solids". Secondary criteria are based on the ionic composition of the water - commonly the sodium adsorption ratio (SAR) and the residual concentration of sodium carbonate" (RSC), and the concentration of toxic ions, principally boron.

According to the quality-of-water standards which have been adopted by T&K (1967) for the Northern Indus Plains, the utilization of the groundwater supplies is classified on the basis of the mineral content of the water. Three general zones have been established horizontally according to the TDS contours which were drawn based on groundwater quality at depth greater than 100 feet. They are defined as:

1. Nonsaline Zone - Groundwater containing less than 1500 ppm salinity, classified as safe for use under normal approved irrigation and water management practices, which implies that about one-third of the applied irrigation water is derived from canal supplies.

2. Intermediate Zone - Groundwater containing 1500 ppm to 4000 ppm salinity classified as marginal which requires dilution with canal supplies or special water and soil management practices.

3. <u>Saline Zone</u> - Groundwater containing more than 4000 ppm salinity classified as unfit for economic development for irrigation supplies under present or assumed future conditions.

Invariably, a lower layer of more saline groundwater exists underneath the relative fresh water in the nonsaline and intermediate zones. The shallow groundwater in the upper layer of the saline zone can also be withdrawn and mixed with surface water for irrigation use. It is assumed that an abrupt interface exists between the upper relative fresh water layer and the lower more saline water layer in each zone.

For the southern zone, T&K (1967) also suggested that water can be used safely with salinity less than 1500 ppm TDS and sodium absorption ratio (SAR) less than 7.5 . Waters that are more saline and alkaline can be used only after mixing with surface water so that the resultant mixture meets the above criteria.

IACA (1966), according to their own experiments, derived a set of criteria for classifying irrigation water which is slightly different from that of T&K. For this study, the groundwater zones set up by T&K were adopted. The primary reason for selecting the T&K classification was that data are available on areas and distributary capacities for the three different zones in the model area mentioned in the next chapter. The mixing ratio for the intermediate zone and the upper layer of the saline zone will be assumed at 1:1 according to the analysis of SCARP 1 data made by the Harvard Water Resource Group in 1964.

It is not possible to specify a definite groundwater quality criteria for irrigation application based on water quality data available. If the criteria is set too loosely, crop production might be reduced due to the application of more saline water which exceeds the tolerance of plants. On the contrary, if the criteria is set too tightly, portions of the groundwater will not be made available for use to meet the water demand. Further research on these criteria will be necessary so that groundwater can be utilized as much as possible without reducing crop production.

Groundwater Recharge.

Recharge is the input to the groundwater aquifer. It is an important factor in evaluating groundwater resources and potential utilization of the aquifer as a reservoir. The sources of recharge can be from the percolation of rainfall, from losses through line sources such as rivers, canal distribution systems, watercourses on the farm lands, and percolation of irrigation water.

Many factors affect the magnitude of recharge. Among them are the characteristics of the soil and other deposits above and below the groundwater table, especially permeability, thickness of soils, the topography, the depth to water table; hydraulic gradient, land use and vegetative cover, rainfall intensity, duration and seasonal variation, temperature, and also, the man-made pumping activities and water diversion through the conveyance system.

Artificial recharge is the other possibility in adding water to the aquifer. Many kinds of methods have been developed, including recharge through modified streambed, percolation basin, ditches and furrows, pits, excavations, shafts, injection wells, and pumping to induce recharge from surface water bodies (ASCE Committee, 1961). The importance and the need for artificial recharge have been brought about by an increasing demand for groundwater as a source of water. Artificial recharge can serve the purposes of conserving and disposing of runoff and flood waters to prevent floods, supplementing available groundwater, reducing or eliminating the decline of groundwater level to prevent land-subsidence and reducing costs of pumping and piping, reducing or preventing salt-water intrusion, and disposal of solid waste (Walton, 1970). There are also many problems encountered in using the artificial recharge facilities. Siltation and plugging of the recharge surface reducing infiltration, and the high maintenance cost involved are but two of the major problems.

Recharge through natural rainfall and river runoff, and through the irrigation system in Pakistan have been reported by various agents such as Harza (1963), IACA (1966) T&K (1967). In general, their results are similar and can be summarized as follows:

1. <u>Recharge from Rainfall and River</u> - Deep percolation of rainfall is considered not to be a significant contributor of recharge in Pakistan. On the average, it varies from 1 inch to 5.6 inches per year. River losses probably also make a comparatively small contribution to recharge at the present time due to the high water table. A series of empirical coefficients for each river reach relating loss or gain in the reach to discharge at its head had been investigated and derived by Harza (1963) for WAPDA. T&K (1967) reported the same method. T&K (1967) also estimated that the overall recharge from rainfall and rivers is on the average 0.2 feet per year.

2. <u>Recharge from Canal System and Irrigation Field</u> - Seepage losses from the canal system have made the most significant contribution to recharge and the recent rise of the water table. There will be a tendency for the net addition of recharge to increase as the water table is drawn down to more than 10 feet below the

surface by tubewell pumping. Seepage losses are generally expressed as a percentage of discharge at the diversion point, and recharge is also expressed as a percentage of seepage loss. Table 3.2 shows the recharge criteria proposed by Harza (1963), IACA (1966) and T&K (1967) in their respective studies and summarized by Chaudhry (1973). The overall recharge to the canal command area in the Northern Indus Plains can be estimated as the sum of 54% of the volume of water delivered to the heads of watercourses, 22% of tubewell supply at the heads of watercourses and 0.2 feet per year from other sources such as rainfall and river runoff.

Aquifer Storage and Conjunctive Use of Surface and Ground Water.

The fresh groundwater aquifer represents a large natural subsurface storage reservoir which will play an important role in the development of water resources in the Indus plains. The total volume of water stored in the aquifer will depend on the gross area, depth and specific yield of the fresh groundwater aquifer. In the intermediate zones, tubewells cannot operate without surface water for mixing, and in the saline groundwater areas tubewells can only skim the relatively fresh water from the upper layer for mixing. The remaining needs will be dependent on transfer of surface water or water from adjoining fresh groundwater area.

The reasons and advantages for the conjunctive use of groundwater and surface water in Pakistan can be summarized as follows:

1. Only half of the canal commanded areas proposed for development in the Indus Basin is underlain by fresh groundwater which can be applied directly to the crops, but surface water supplies could be improved throughout the remainder of the canal commanded areas by transfer from fresh groundwater areas. A further 15% of the

TABLE 3.2

COMPARISON OF WATER LOSSES AND GROUND WATER RECHARGE ESTIMATES USED IN THE VARIOUS STUDIES

Item	Harza	IACA	T&K
A-1. Losses from link canals in cfs/ million ft^2 of wetted perimeter for			
a. lined canals	6	-	2
b. unlined canals			8
2. Recharge as percentage of above losses	90%		90%
B-1. Losses from irrigation canal system as percentage of water supply at the head of the system			
a. Main canals and branches			As in A-1.b above
b. Distributaries and minors			15%
c. Total canal system up to water course head	30%	20 to 30%	(28%)
2. Recharge as percentage of above losses	80%	80%	80%
C-1. Losses from water courses as percentage of water supply at water course head	10%	10%	-
2. Recharge as percentage of the above losses	50%	50%	-
D-1. Farm losses as percentage of water delivered at the field	25%	30%	-
2. Recharge as percentage of the above losses	75%	67%	

canal commanded areas is underlain by groundwater which will require mixing with surface water before being applied to crops. The integrated control of surface and groundwater is necessary to ensure the good quality irrigation water (IACA, 1966).

2. From the viewpoint of short term development, tubewells can be installed relatively rapidly and will provide a large amount of additional water. From the long term viewpoint tubewells provide means of regulating the huge aquifer of the Northern Indus Plains. In this sense, tubewells should not be regarded merely as accessories to irrigation works that may be used for supplemental supply, but rather as major devices that make possible a much more complete integration and ultimate control of the entire hydrological regime.

3. Due to the flatness of the land, suitable reservoir sites for surface storage are rare and the storage capacities are small and can only be used as storage regulation within a year. The reservoirs are remote from the areas of water use, and they are relatively short lived because of the high sediment load of the river. Groundwater storage is the alternative for water storage and because of its vast natural storage capacity it will provide long term storage. It is near the demand areas, thus, the length of conveyance is largely reduced.

4. Canal seepage becomes less of a problem in usable groundwater areas because ground water pumpage can control both the effect of leakage and salvage the losses from the canal.

5. Storage underground eliminates the evaporation losses encountered in surface storage.

6. Use of aquifers provides flexibility in the timing of water supplies and increases the irrigation water supply.

7. Waterlogging and salinity can be controlled more effectively by lowering water table and reducing salt concentration in the root zone.

8. Tubewell pumping provides a more flexible and controllable drainage scheme.

Remodeling of the existing canal system is a necessity under this conjunctive use policy. Especially in the saline area where groundwater is too saline to be used, irrigation must be accomplished largely with surface water or usable groundwater brought in and distributed through the canal system. Additional canal enlargement beyond that required to meet the water requirement for the given cropping intensity also can deliver water for artificial recharge. This applies only to the non-saline area where the quality of groundwater is suitable for direct use. The canal in this nonsaline zone can be enlarged so that the recharge from the surface water deliveries during the summer months would be capable of supplying the irrigation water demands during the rabi season from tubewell pumping without inducing possible salt water intrusion from the saline groundwater area.

CHAPTER IV

DEVELOPMENT OF THE PHYSICAL MODEL

In an optimization study, the formulation of a mathematical model depends on the selected physical system. The objective criteria in the mathematical model must be defined in terms of the system variables. Constraints must also be expressed in terms of system variables and parameters so that the physical system is closely described. Modifications and simplifications are often necessary so that the system can be expressed mathematically and systematically in order to devise a feasible solution technique for the mathematical model.

System Decomposition and Multilevel Approach.

The irrigation system of the Indus Basin consists of more than 33 million acres of culturable commanded area, 43 major canal systems including link canals, and several major reservoirs. It is one of the largest and most complex systems in the world. The complexity of the system gives rise to the need for developing a method of optimal analysis of the system and the need for the decomposition of the entire system into several subsystems. Chaudhry (1973) proposed that a multi-level optimization scheme be employed to sub-optimize the subsystems and the results combined to obtain an optimal solution for the overall system.

The basic idea inherent in the decomposition and multi-level approach is to decompose the large scale system into the more or less independent subsystem (Lasdon and Schoeffler, 1966; Haims, etc., 1968). Instead of optimizing the entire system with large dimensionality, each subsystem with smaller dimensionality can be solved more easily and rapidly by the available optimization techniques and within the limit of existing computer capacity. These lower level optimal subsystems are tied together through some coordinating parameters, which are responsible for the whole system optimization and defined as the master problem. Through the master problem, a set of parameters are released to each subsystem. Then each individual subsystem will be optimized accordingly and fed back to the master problem. The master problem evaluates the overall results from each individual system and releases another set of parameters in order to improve the solution for the overall system. This process is iterated until the overall system is optimized.

The Indus Basin is decomposed into subsystems for each individual canal. The reservoirs are also treated as a separate subsystem which is not considered in this study. Water delivered from the rivers to the head of each canal will be treated as coordinate parameters. For each canal subsystem the available surface water at the head of the canal and the groundwater beneath the area is allocated optimally to minimize cost. This solution is fed back to the master problem, which evaluates another surface water release pattern. This procedure is iterated until the optimal solution is found.

The objective of this study will be limited to the lower level optimization of the subsystem, i.e., the optimal conjunctive use of groundwater and surface water for each canal subsystem. Results could be utilized in the overall system optimization.

Requirements of the Physical Model Area.

Water distribution for conjunctive use of ground and surface water requires the following general conditions:

1. The area chosen should be as independent as possible so that the interaction between the chosen area and its neighboring areas

can be neglected. The inflow and outflow of water into and out of the area is well defined. If the subsystem cannot be isolated, reasonable estimates of interflow bewteen neighboring areas must be made.

2. The system is underlain by an aquifer of sufficient yield and storage capacity and can be pumped out readily. The aquifer is recharged naturally with water or is capable of being recharged artificially. The data concerning the aquifer characteristics are available. The groundwater quality zones according to the criteria mentioned previously are well defined.

3. There are sufficient river flow and precipitation data.

4. Conveyance systems exist which could be remodeled to transfer surface and ground water to the demand areas.

5. Water demands are well recorded, or the planned water demands are well estimated.

Selected Study Area.

The canal systems in the Indus Basin irrigation system have many similarities. The development of the physical model will be for a selected area in particular, but to a large extent, it is applicable to other canal systems in the Basin.

The Lower Jhelum Canal within the Indus River Basin was chosen as a typical subsystem. The reasons for choosing the area are as follows:

1. The area is surrounded by the two rivers and a main canal, the Lower Jhelum, to form a more or less natural hydrologic subregion. Uncontrolled or unmeasured surface and ground water outflow is negligible. The surface inflow is well controlled and delivered entirely by canal.

2. The area is underlain by a groundwater aquifer. It has been under study in the project SCARP 2. The Mona pilot project is being conducted within this area. Data from investigation of the installed tubewell performance are available. The groundwater quality changes gradually from the sources of recharge to the central part of the area. The total dissolved solids (TDS) concentration and the three groundwater quality zones are well defined. Data concerning thickness of fresh water, depth to water table, and area of each zone are all available.

3. Data on mean annual diversions to the canal and river flows of the Indus Basin have been recorded.

4. Total capacities at the heads of watercourses within each groundwater quality zone are available.

5. The water demands can be computed from the designed cropping pattern and cropping intensity. Certain cropping patterns with the cropping intensity of 150% have been proposed by some consultants to Pakistan. The water requirements at this level of cropping intensity have been calculated and are available.

General Description of the Model Area.

The Lower Jhelum Canal command area is located in the Chaj Doab of the Northern Indus Plain. It covers about three quarters of the doab and is separated from the northeastern part by the Lower Jhelum Canal.

The climate is fairly uniform over the area, except that humidity is slightly higher in the north, and the temperature is a few degrees lower. The mean day maximum temperature varies from 106°F in summer to 65°F in winter. The mean annual rainfall varies from 10 inches in the south to 20 inches in the north (two-thirds of which falls in the summer). From June to October it varies from 6 to 16 inches. Mean annual evaporation from a free water surface is about 60 to 65 inches.

The canal commands a total culturable area of 1.5 million acres (MA) and is supplied from the Rasul Barrage on the Jhelum river and through the Rasul Hydroelectric Plant. The canal command and its water distribution network are shown on figure 4.1.

The soils have adequate water holding properties for irrigated agriculture. They are potentially fertile and the texture varies from heavy, on which rice is grown, to light, on which crops are usually not irrigated but dependent mainly on rainfall. Soil salinity problems occur in about 24% of the area and are usually associated with a high water table and waterlogging.

The Lower Jhelum Canal command area is considered to be one of the more advanced agricultural areas of the Punjab. Cotton is the main cash crop in the area, and wheat is the most important food crop grown during the rabi (winter) season. In the nonperennial areas, where water is normally delivered only during the summer period, grain is sown in rabi. Fodders, particularly rabi fodders, are grown over large areas. The most important perennial crop which grows all year round is sugarcane whereas fruit is of minor importance. The average cropping intensity at present is about 105%.

The whole of the Lower Jhelum Canal area is already covered by a network of surface drains. New drains or extension and enlargement of existing drains will be necessary to convey the excess effluent from drainage tubewells.

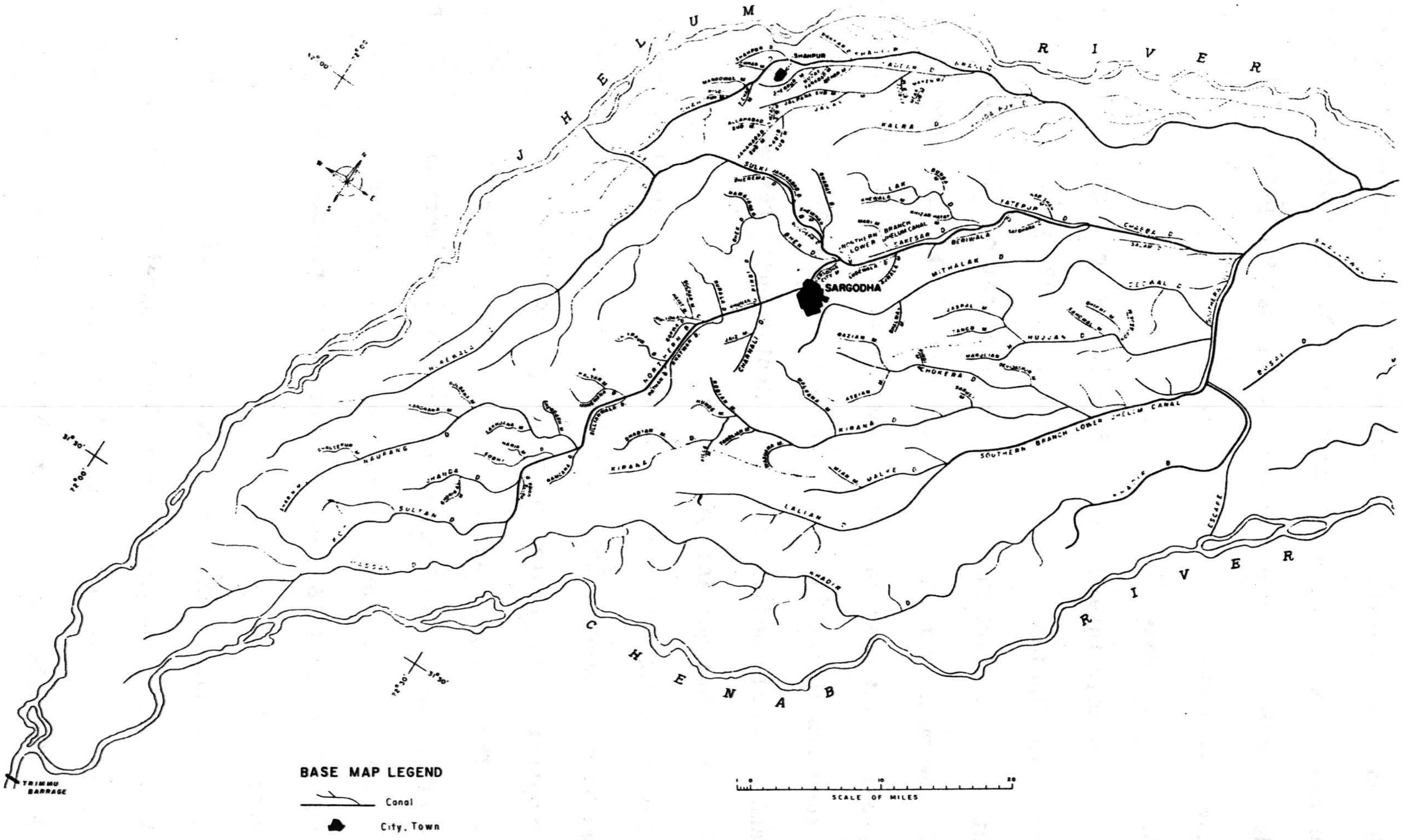


Figure 4.1. Lower Jhelum Canal Command.

System Components.

The major components of the system are a surface water supply, an aquifer and tubewell system, a canal distribution system, and water demands. Figure 4.2 shows the schematic diagram representing the simplified physical system.

A. Surface Water.

The only three sources of surface water supply to the plains are the Indus, Jhelum and Chenab Rivers. A 42 year record (1922-1963) of monthly runoff from these rivers at rim stations is available (IACA, 1966). How much of the available water should be allocated to each area is a problem that must take into consideration the known obligations for water supplies in the various parts of the basin, the tubewell development and seasonal needs. It must be continuously reviewed whenever a new project is brought into operation.

The following criteria will be assumed for allocating surface water to each area. However, the final decision on allocation of water is the master optimization problem mentioned previously.

1. The available water supply from records will be adjusted by a coefficient for each month to account for upstream reservoir regulation. These coefficients are roughly calculated according to the T&K study (1967) on their reservoir release plans in a median year.

2. The amount of water allocated to the Lower Indus Basin will be based on the required flow at Gudu suggested by T&K (1967) in their study. The rest of the flow is allocated to the Northern Indus Basin.

3. The allocated water in the respective Northern and Lower Indus Basin then is divided proportionally to the mean historical diversion to each canal commanded area.

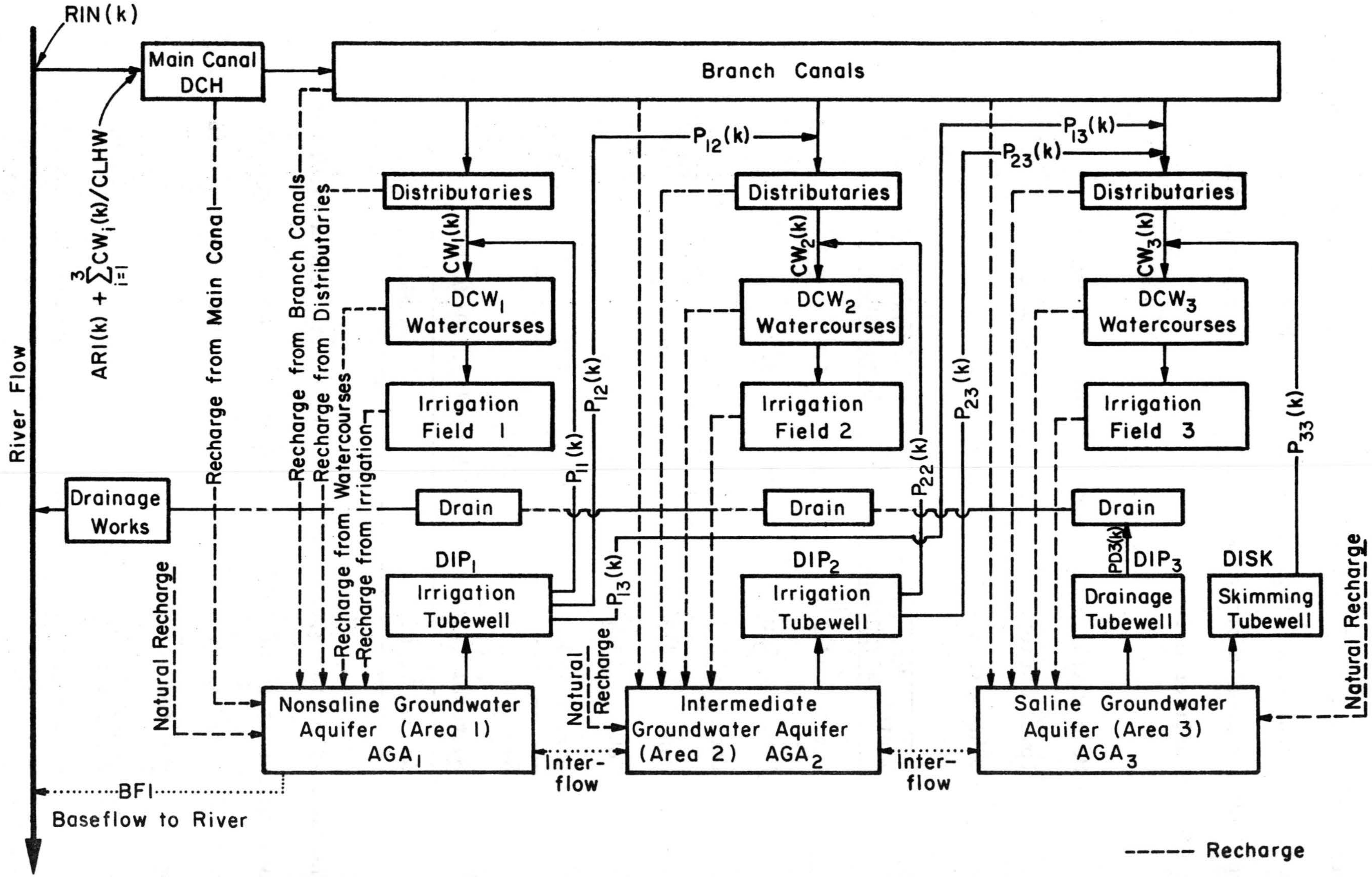


Figure 4.2. Schematic Diagram of the Physical System.

Mean historical surface diversion to the canal for the internal use of the Lower Jhelum Canal command is shown in Table 4.1. A period from 1947 to 1960 has been adopted as being generally representative for recent surface deliveries.

TABLE 4.1

Mean Historic Diversion for Internal Use of Lower Jhelum Canal (in 1,000 cfs unless otherwise noted)(T&K, 1967).

Month	Mean Historic Withdrawal	Month	Mean Historic Withdrawal
Oct.	4.9	Apr.	4.5
Nov.	3.6	May	5.1
Dec.	3.4	Jun.	5.3
Jan.	3.3	Jul.	4.6
Feb.	3.4	Aug.	4.6
Mar.	3.8	Sep.	4.9
Rabi		Kharif	
Subtotal	1.361(MAF)	Subtotal	1.769(MAF)
Annual Total			3,130(MAF)

The quality of the river flows are excellent. The average total dissolved solids is about 250 ppm, and will be assumed constant at this value for this study. At this level of quality, there is no restriction on their use for irrigation.

B. <u>Canal Distribution System.</u>

The existing canal command and its distribution network is shown in figure 4.1. The river flow is diverted from the main canal through branch canals and then delivered through distributaries to the heads of watercourses in each of the three different groundwater quality zones. The capacities at heads of distributaries and watercourses will be assumed aggregated and considered lumped within each area. These aggregated capacities will be the decision variables to be determined in the mathematical model. The existing capacities at heads of distributaries and water courses for the three different zones are shown in Table 4.2.

TABLE 4.2

Area Distribution and Capacities at Heads of Distributaries and Watercourses for Divided Zones, Lower Jhelum Canal Command (T&K, 1967).

	Nonsaline Zone (0-1500 ppm)	Intermediate Zone (1500-4000 ppm)	Saline Zone (4000 ppm)	Total Command
Gross area (acres)	1,077,100	330,100	330,000	1,737,200
Culturable Area (acres)	990,000	304,400	303,000	1,596,700
Culturable Commanded Area (acres)	929,800	285,000	284,900	1,499,700
Capacity at Heads of Distributary (cfs)	2,534	963	963	4,460
Capacity at Heads of Watercourse (cfs)	2,154	819	819	3,791

C. Ground Water.

C.1. The Alluvial Aquifer.

The area is underlain to a depth of at least 1,000 feet by an alluvium consisting of unconsolidated fine sands and silts with intermittent clay layers. The aquifer is anisotropic with the higher permeability in the horizontal plane. The horizontal permeability ranges from 0.0018 to 0.0034 , and averages about 0.0028 feet per second from tests carried out by WASID, Pakistan (Bennett, 1967). Very few vertical permeability tests have been made, but they indicate a ratio of 25-50 to 1 between horizontal and vertical permeabilities for this area. In general, the aquifer can be considered unconfined and the mean specific yield from tests is about 0.16 (IACA, 1966).

C.2. Tubewells Development.

There were about 410 private wells in operation in 1965. The public tubewells have been constructed under the SCARP 2 project in the usable groundwater area, and about 514 wells with a total capacity of about 2,000 cfs have been completed (IACA, 1966).

C.3. Ground Water Quality Zones.

The area is divided into three groundwater quality zones, i.e., the nonsaline, intermediate and saline zones, according to criteria mentioned in Chapter III. The area covered for each water quality zone will be assumed unchanged within the time span studied. The average salt concentrations within each zone are estimated as 500 , 2,600 , and 9,000 ppm respectively. Table 4.2 also shows the gross area, culturable area and culturable command area within each zone. Figure 4.3 shows the area distribution of the three groundwater quality zones.

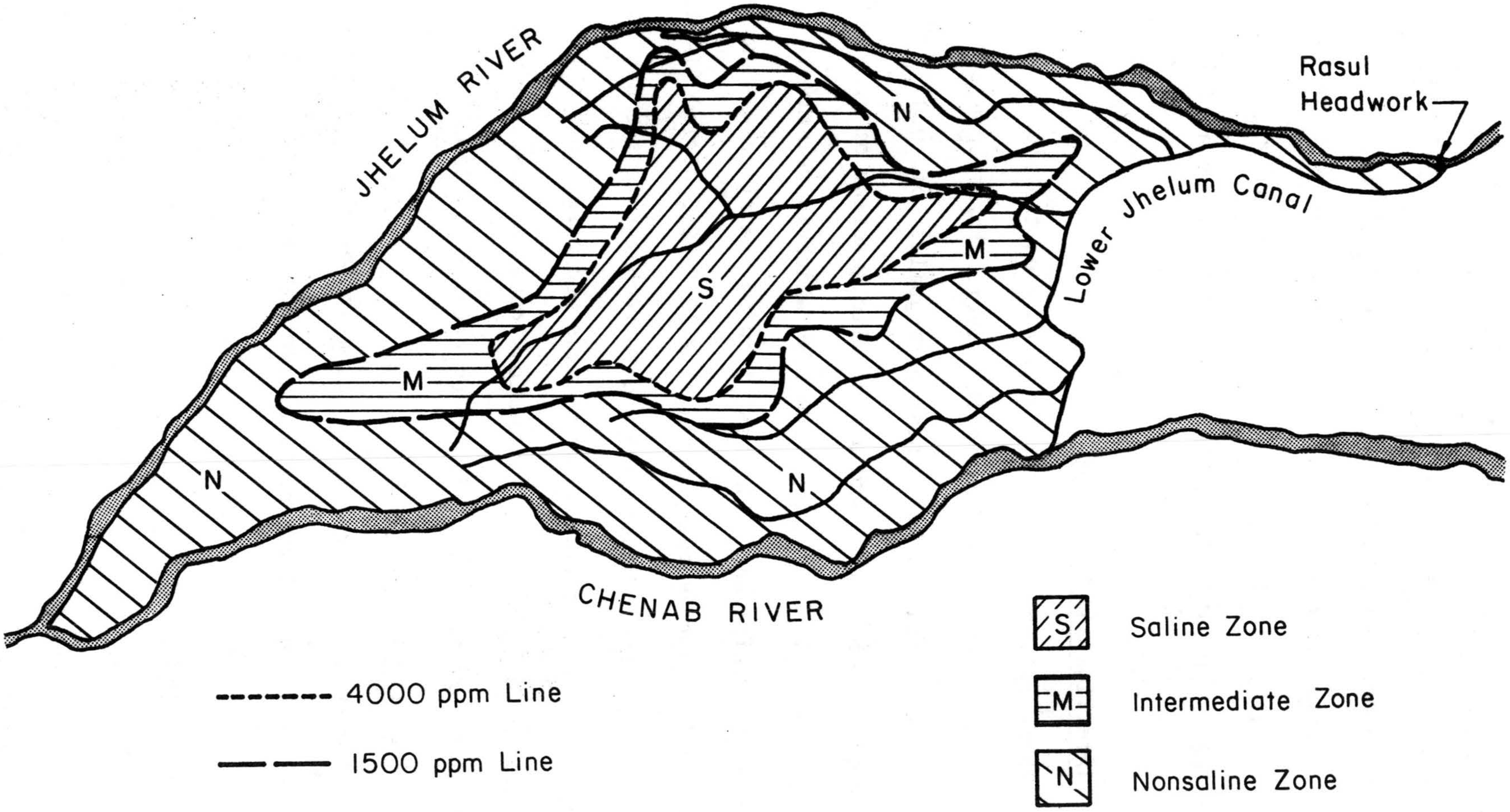


Figure 4.3. Ground Water Quality Zones - Lower Jhelum Canal Commanded Area.

The nonsaline zone, can be pumped out for use directly as long as contamination due to salt water coning does not occur. In the intermediate zone and the upper layer of the saline zone, the water can be pumped for use only when mixed with surface water with a mixing ratio of at least 1 to 1 . It is assumed that the relatively fresh water can be transferred to the relatively more saline areas through the heads of distributaries.

C.4. Interflow and Base Flow.

There is very little data available on the amount of interflow between areas and base flow to the river. For a boundary of 200 miles between two areas, a horizontal permeability of 0.003 feet per second, a depth of 500 feet and a hydraulic gradient of 1 foot per mile, the interflow is only about 300 cfs which is relatively small and assumed insignificant. The base flow to the river was estimated to be around 150 cfs for February during the 1947-1955 period (IACA, 1966) and is also relatively small compared to recharge to the aquifer. For this study both the interflow between areas and base flow to the river will be neglected. However, these flows could be easily added to the model whenever more reliable information becomes available.

C.5. Recharge to the Aquifer.

Recharge is derived mainly from losses through line sources such as the river and the canal system and from deep percolation of irrigation and rainfall. The other possibility is through artificial recharge. It is assumed that recharge to the aquifer is uniformly distributed over each area. Due to the possible evaporation and consumptive use of the crop, recharge to the aquifer will only be a fraction of the seepage loss. Recharge criteria adopted for this study are as follows:

	Seepage Loss	Recharge
Canal System up to Watercourse Heads	30% of water diverted from river	80% of loss
Distributary Heads up to Watercourse Heads	15% of water diverted at distributary heads	85% of loss
Watercourses	*10% of water delivered at heads	50% of loss
Irrigation Fields	*25% of water delivered at fields	75% of loss

*Recent field studies indicated that the values are greater than the value shown here.

The above criteria are based on HARZA's study (1963) except that from the head of distributary to the heads of watercourse they are based on a T&K study (1967). Since main canal and branches cover three different areas, it is assumed that recharge from these sources to the respective aquifer will be proportional to the length of the main canal and branches within each of the three areas. This proportion is approximately 5:1:2 for the nonsaline, intermediate and saline zones. Deep percolation of rainfall and other sources of recharge is estimated approximately as 0.2 feet per year.

D. Irrigation Water Requirement.

The only water demand for this study is water for irrigation. The irrigation water requirement at heads of watercourses is determined by consumptive use of the crop, the cropping pattern and cropping intensity, pre-planting irrigation requirements, effective precipitation, water use efficiency on the farm, leaching requirement, depletion of soil moisture, watercourse losses and the size of the area. The water requirements for each of the three groundwater quality areas would be different due to the different leaching requirements.

An optimal cropping pattern and intensity can be determined by maximizing the total net return of the crop yields subject to constraints of available water supply, water requirement for each crop, total available area, area required for subsistence food and other agricultural constraints. This would be a lower level subsystem optimization of the present problem. For this study, a final level of cropping pattern and intensity is assumed. A 150% cropping intensity is used as suggested by T&K (1967) in their Northern Regional Plan study. Their study showed that considerable change can be made in the cropping patterns and in the Kharif-Rabi ratios without significantly affecting the total irrigation water requirement or the net value of harvested crops. Accordingly, differences in cropping patterns are not of great significance, and great precision in predicting the details of future cropping patterns, even if it were possible, is not essential.

IACA (1966) also suggested in their proposals for development that an intensity of 150% would be approaching the optimal level of cropping, when an additional supply of irrigation water from tubewell water and surface water becomes available.

The monthly water requirements at heads of watercourse for three water quality zones of the Lower Jhelum Canal commanded area are shown in Table 4.3 according to T&K (1967). They were adopted for this study.

E. Cost Functions.

The two kinds of costs involved in the system are fixed costs and variable costs. In order that the system be comparable, all the fixed costs must be converted to an annual basis by multiplying various

TABLE 4.3

Monthly Water Requirements at Heads of Watercourses for the Lower Jhelum Canal Commanded Area at 150% Cropping Intensity (T&K, 1967).

Month	Water Requirements (1,000 cfs) Nonsaline Zone	Intermediate Zone	Saline Zone
Oct.	6.49	1.87	1.87
Nov.	4.67	1.34	1.34
Dec.	2.43	0.70	0.70
Jan.	3.51	1.00	1.00
Feb.	5.54	1.59	1.59
Mar.	5.82	1.67	1.67
Rabi Total (MAF)	1.722	0.494	0.348
Apr.	2.98	0.85	0.85
May	3.92	1.12	1.12
Jun.	5.36	1.54	1.54
Jul.	4.16	1.17	1.17
Aug.	5.22	1.45	1.45
Sep.	7.72	2.21	2.21
Kharif Total (MAF)	1.788	0.508	0.309
Annual Total (MAF)	3.510	1.002	0.657

capital recovery factors which depend on the interest rate and the lives of various structures. The cost figures indicated here are based on 1966 to 1967 data.

E.1. Cost of Canal Remodeling.

The remodeling of a canal cross-section to increase its delivery capacity also requires the remodeling of existing structures including regulators, offtakes, falls, crossing structures and outlets. The remodeling costs will be either due to enlargement of the existing capacity or the construction of a new canal along with the existing canal. Cost will vary considerably depending on the increase in capacity, the condition of the existing system and structures and other factors.

IACA (1966) established a general relation between cost per acre of canal commanded area and percentage of enlargement of capacity at heads of watercourses for distributary and minor canals as shown in figure 4.4. This relationship was based on the calculations for the Vahn distributary at the tail of the central Bari Doab canal in the Indus Basin with a commanded area of 56,000 acres and capacity of 200 cfs. The design of the enlarged canal cross-section was based on Lacey's regime theory. The cost of enlargement for the main canal and branch canals are estimated to be about 50% of the costs estimated for enlarging the distributary and minor canals obtained from the curve of figure 4.4. The curve indicates that it will be cheaper to enlarge the existing canal cross-section up to 60% and from then on a second canal will be cheaper.

For an interest rate of 8% and a 100 year canal life, a capital recovery factor of 0.08 is obtained and the annual costs of remodeling according to the curve are as follows:

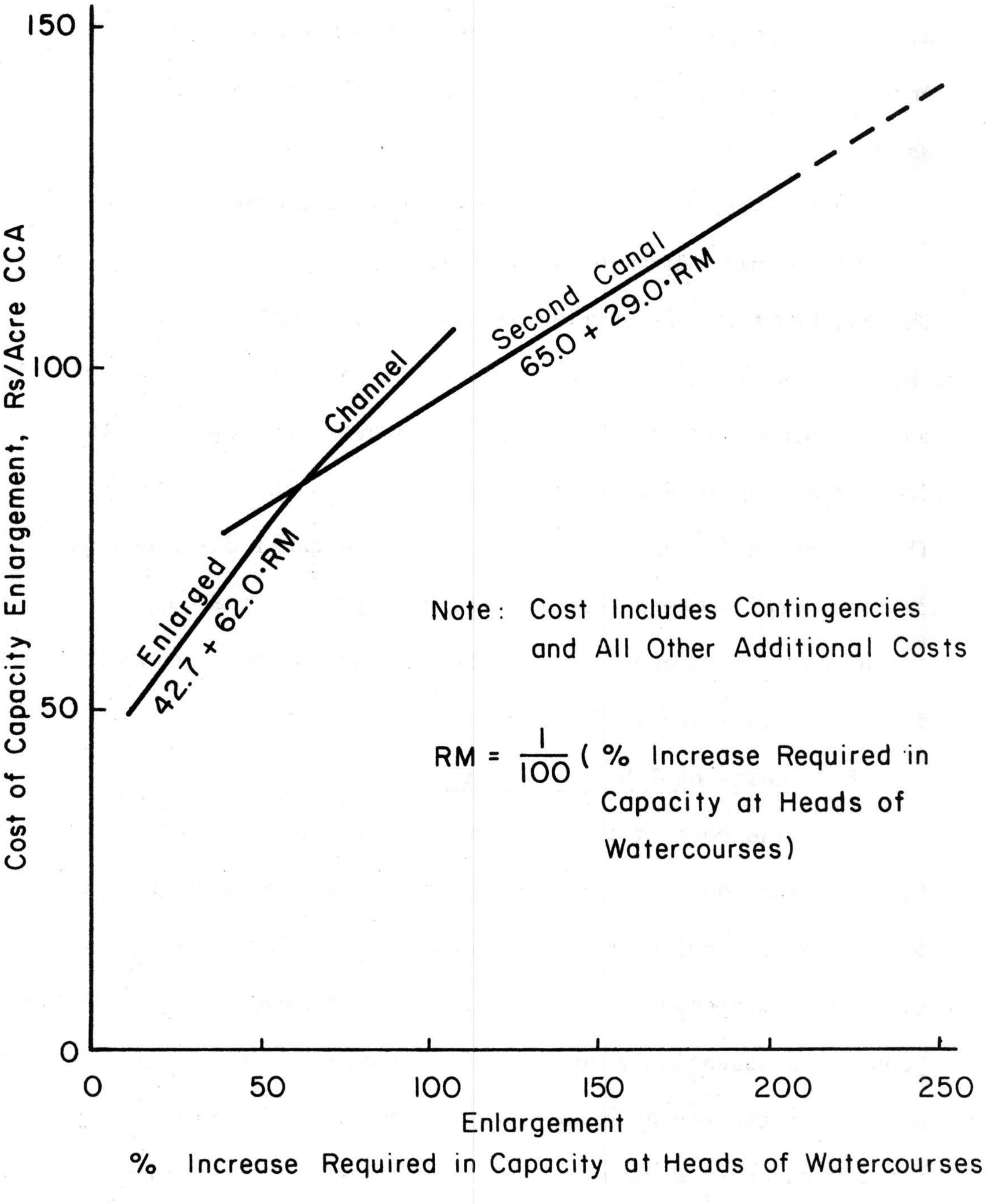


Figure 4.4. Enlargement Cost Curve for Distributary and Minor Canals (Northern Zone only). Adopted from IACA, 1966.

1. For enlargement at watercourse head up to 60%:

$$\text{Annual Capital Cost (RS)} = 97.5 \cdot \text{CCA} + 43.5 \cdot \text{CCA} \cdot \text{RM} \quad .$$

2. For enlargement at watercourse head greater than 60%:

$$\text{Annual Capital Cost (RS)} = 64.1 \cdot \text{CCA} + 93.0 \cdot \text{CCA} \cdot \text{RM} \quad ,$$

where

RM = Percentage of enlargement at watercourse heads.

CCA = Canal commanded area in acres.

The construction of distributary and minor canals throughout the Indus Basin is similar since there is little variation in slope within the canal commanded areas. Therefore the above estimation of general equation can be applicable to all distributaries and minor canals.

The operation and maintenance costs of the canal distribution system are considered somewhat constant with respect to the quantity of water supplied. These costs tend to be related to the size of the area served, and are included in fixed costs for this study.

E.2. Costs of Tubewell Pumping.

The cost of tubewell pumping includes fixed costs of tubewell installation and variable cost of energy for pumping. The fixed cost of a tubewell installation includes amortization, depreciation, operation, and maintenance costs. For a given tubewell installation fixed costs are essentially constant irrespective of the volume of water pumped, whereas the variable costs of energy are a function of the volume of water pumped and the pumping lift. The size of public tubewells range from 2 to 5 cfs and serve an area ranging from 200 to 600 acres. The average size for a public tubewell used in this study is 4 cfs, based on IACA (1966) estimation.

E.2.1. Capital, Operational and Maintenance Costs.

The capital costs of the public tubewell for 2 to 5 cfs have been estimated by IACA (1966) based on the information from the contracts for tubewells in the Khairpur and SCARP 2 and 3 areas in the Indus Basin.

The capital cost for a 4 cfs tubewell is RS (Rupees) 90,000 which includes construction cost and is increased 30% to take into account contingencies and engineering, local currency and foreign exchange. The annual cost then is RS 9,167 based on the 8% interest rate and a life of 20 years, with a capital recovery factor of 0.10 .

The annual operation and maintenance costs for public tubewells based on the IACA (1966) study is equal to RS 3,000 per well, including costs of additional engineering staff, repairs and maintenance of tubewells and maintenance of transportation.

The total annual costs for a 4 cfs well including annual capital and annual operation and maintenance costs then is RS 12,167 .

E.2.2. Annual Power Cost.

The annual power cost depends on the unit charge of the power, total volume of water pumped, pumping head and the overall pumping efficiency of the tubewell. Since the water table changes from period to period due to recharge and pumping, the cost will vary from time to time. A general formula for calculating power cost for any period, k , is:

$$\text{Power Cost} = U \cdot VP(k) \cdot H(k) \quad ,$$

where U = Power cost for pumping per acre foot of water per foot of lift, and is equal to RS 0.184 for this study (T&K, 1967).

VP(k) = Total volume of water pumped during period k (AF)

H(k) = Total pumping head during period k (feet) .

Detailed derivation of H(k) and power cost are presented in Appendix C.

This cost function is a nonlinear quadratic equation. For a pumping period as short as one to three months, it is reasonable to assume that the pumping head is more or less constant during that subperiod. The pumping head at the beginning of each subperiod will be used as the constant pumping head throughout that subperiod, and the pumping cost, therefore, becomes linearly related to the pumping rate.

E.3. Cost of Drainage Works.

A tubewell drainage system is adopted for this study. According to T&K (1967), for a typical 6 cfs drainage tubewell with a rated head of 60 feet, the total capital cost will be RS 115,400 . For an average life of 20 years and 8% interest rate, the annual capital cost is RS 10,250 . The annual operation and maintenance costs of the drainage tubewell are assumed the same as the irrigation tubewell at a cost of RS 3,000 . So the total annual fixed cost of a drainage tubewell will be RS 13,250 .

The annual power cost can be estimated in the same way as it is for the irrigation tubewell.

The annual cost of extra drainage capacity for drainage tubewell effluent beyond that of the existing drainage works was estimated at RS 4,600 per cfs of the well capacity according to T&K (1967).

E.4. Cost of Artificial Recharge.

No special artificial recharge facility is stipulated in this study. It is assumed that the extra water for artificial recharge will be delivered through the main canal and branches to distributaries and watercourses in the nonsaline zone. Besides the increase of recharge in the canal distribution system, recharge is assumed to be performed by over-irrigation and flooding of the fallow land. The operational and maintenance costs for artificial recharge diversion are not available and will be assumed constant, and thus, not necessary in the study.

E.5. Cost of Shortage.

The cost of shortage can be measured by the cost of water for agricultural development. Little quantitative data are available for determining the cost of water in the Indus Basin. For the present it is necessary to rely entirely on estimates of crop yields, crop values, and costs of production. T&K (1967) has studied the value of water at present and in the future from the annual value of agricultural production and irrigation supplies for different agriculture zones. For the Lower Jhelum Canal command, the average future annual value of water with cropping intensity at 150% for an average depth of water applied at 3.6 feet per year, is RS 177 per acre-foot. This figure will be adopted as the annual cost of shortage for this study.

Summary of Assumptions for Development of the Physical Model.

1. The total available surface water from river runoffs in the Northern Indus Basin will be allocated to the model area in proportion to its historical withdrawal which is shown in Table 4.1.

2. The irrigation water requirements during each time period are given and determined from an assumed level of cropping pattern and 150% intensity (see Table 4.3). Irrigation is the only beneficial water use considered.

3. Due to the salinity of groundwater, the aquifer in the model area is decomposed into three zones horizontally according to the

quality-of-water standard adopted by T&K, i.e., nonsaline, intermediate and saline zones. The area of each zone is considered constant with time.

4. Vertically, the groundwater within each zone is also divided into upper fresh-water and lower salt-water layers and it is assumed that an abrupt interface exists.

5. Assume uniform properties within each zone of the aquifer such as storage coefficient, permeability, water quality, cropping pattern, soil property, groundwater level, ground surface slope, and so on.

6. Assume recharge to aquifer through the seepage loss from the canal distribution system, deep percolation of rainfall and irrigation water are uniformly distributed to the whole area within each zone.

7. Some of the components in the system are considered as aggregated. For example, capacities at heads of watercourses and tubewell installation capacities within each zone will be considered lumped.

8. Assume a constant quality of surface water supply at 250 ppm TDS. The average quality of groundwater in each zone are also assumed constant at 450, 2,600, and 9,000 ppm TDS for the respective nonsaline, intermediate and saline zones.

9. Assume that only the aquifer in the nonsaline zone will receive artificial recharge.

10. Assume an average size of four cfs irrigation tubewells in the nonsaline and intermediate zones, six cfs drainage tubewells and 0.25 cfs skimming wells in the saline zone. The calculation of

tubewell installation costs and energy costs will be based on these average sizes.

11. There is no constraint on available electric power for tubewell pumping during any time interval and power rates remain constant.

12. Assume groundwater interflow between areas and base flow to river are small and insignificant, and will be neglected.

CHAPTER V

FORMULATION AND SOLUTION OF THE MATHEMATICAL MODEL

The objective of this study was to determine the optimal design capacity and operational decisions for the conjunctive use of groundwater and surface water to satisfy the water requirements in the canal subsystem. Since water requirements are specified according to some predetermined cropping pattern and intensity, the objective is to minimize the capital, operational and maintenance costs of the system.

In this chapter, a mathematical programming problem is formulated based on the physical model developed previously. The objective function and constraints are all linear according to the assumptions made in the physical model. The optimization problem, however, cannot be efficiently solved directly by linear programming due to the large number of variables involved, and therefore must be decomposed. Spatially speaking, the model area was decomposed into three zones, according to the water quality standards adopted by T&K (1967) for the Northern Indus Basin, as mentioned in the previous chapter on development of the physical model. The modeling period was also divided into a number of independent subperiods of monthly duration.

Two kinds of decision variables are involved in the problem: design capacity variables and operational variables. There are eight design capacity variables and seventeen operational variables within each subperiod. The problem was further decomposed into two optimization problems by separating the design capacity variables and operational variables. The first level problem is defined as the inner operational problem and the second level problem is the design problem. The flexible tolerance method (Paviani and Himmelblau, 1969) was used to search the optimal design alternative iteratively. Each time a setof design capacity variables were chosen, they were treated as known parameters for the inner operational problem. During each subperiod, the operational decisions were determined independently, and linear programming was used to allocate the available surface water and usable groundwater subject to the applicable constraints described later. Engineering judgements were used to simplify the problem and reduce the computational time.

General Formulation of the Mathematical Programming Problem.

A. Decision Variables.

The schematic diagram of the physical model, figure 4.2, illustrates both the design capacity and operational variables. The eight design capacity variables are:

DCW_i - The lumped capacity at heads of watercourses for zone i $(i=1,2,3)$.

DCH - Capacity at the head of the main canal.

DIP_i - Total tubewell installation capacity for zone i $(i=1,2,3)$.

DISK - Total skimming well installation capacity for zone 3.

(Note: zones 1 , 2 , 3 correspond to the nonsaline, intermediate and saline zones.)

The seventeen operational variables occurring during each subperiod k , for $k=1,2,\ldots,n$, are:

$CW_i(k)$ - Delivery rate to heads of watercourses for zone i $(i=1,2,3)$.

$AR1(k)$ - Delivery rate at the head of the main canal for artificial recharge to zone 1.

$P_{ii}(k)$ - Rate of tubewell or skimming well pumping from zone i delivered to heads of watercourses in the same zone $(i=1,2,3)$.

$P_{ij}(k)$ - Rate of tubewell pumping from zone i delivered to heads of distributaries of zone j (i=1,2 ; j=2,3).

PD3(k) - Rate of pumping for drainage from zone 3.

$SHRT_i(k)$ - Shortage of water in zone i (i=1,2,3).

$DGW_i(k)$ - Depth to water table in zone i (i=1,2,3).

B. Objective Function.

The various assumptions previously made for the physical model result in a linear model, which can be solved by linear programming. As pointed out later, direct solution is infeasible and the problem must be decomposed. The objective function minimizes the capital, operational and maintenance costs of the total conjunctive use system over the chosen time period for the total model area, including all three groundwater quality zones. It can be stated as

$$\underset{y,x}{\text{Minimize}}\ Z = \underset{y,x}{\text{Minimize}}\ [CD \cdot y + CO \cdot x] \qquad (5.1)$$

where Z = Total capital, operational and maintenance costs of the system over the chosen time period.

y = Vector of design variables.

$$= [(DCW_i,\ DCH,\ DIP_i,\ DISK),\ i=1,2,3)]' \qquad (5.2)$$

x = Vector of operational variables.

$$= [CW_i(k),\ SHRT_i(k),\ DGW_i(k),\ AR1(k),\ PD3(k),\ (P_{ij}(k),\ j=i,\ldots,3);\ i=1,2,3\ ;\ k=1,2,\ldots,n]' \qquad (5.3)$$

CD = Row vector of cost coefficients for the design variables.

CO = Row vector of cost coefficients for operational variables over the chosen time period (k=1,2,...,n).

(Note: all vectors are column vector unless otherwise stated and superscript, ' , means transpose of the vector.)

C. Constraints.

The objective function is minimized subject to both physical and management constraints.

C.1. Non-negativity Constraints.

All the design capacity and operational decision variables must be greater than or equal to zero.

C.2. Design Capacity Constraints.

The design capacities must be greater than or equal to the existing capacities, since the proposed cropping intensity will be greater than the present cropping intensity. Described in vector notation, $y \geq b_\ell$, where b_ℓ is the column vector representing existing capacities of the design capacity variables.

The design capacities must also be less than or equal to some selected upper limits. These limits are based on judgement of the necessary capacities of the canal system and/or tubewells in each zone needed to satisfy the maximum water requirements within each zone. Expressed in vector notation, $y \leq b_u$, where b_u is the vector of the upper limits of the design capacity variables.

C.3. Operational Constraints.

Operational constraints include both physical and management limitations.

C.3.1. Constraints on canal and tubewell operational decisions.

The total delivery rate at heads of watercourses must be less than or equal to the design capacity.

$$CW_1(k) + CLHW \cdot AR1(k) - DCW_1 \leq 0$$
$$CW_2(k) + CLFD \cdot P_{12}(k) - DCW_2 \leq 0 \qquad (5.4)$$
$$CW_3(k) + CLFD \cdot P_{13}(k) - CLFD \cdot P_{23}(k) - DCW_3 \leq 0$$

where CLFD and CLHW are the seepage loss factors from the head of the main canal down to the heads of distributaries and watercourses.

The total delivery rate at the head of the main canal must be less than or equal to the design capacity at the head of the main canal.

$$\left[\sum_{i=1}^{3} CW_i(k)/CLHW\right] + AR1(k) - DCH \leq 0 \quad . \qquad (5.5)$$

The total pumping rate must be less than or equal to the tubewell installed capacity within each zone.

$$P_{11}(k) + P_{12}(k) + P_{13}(k) - DIP_1 \leq 0$$
$$P_{22}(k) + P_{23}(k) - DIP_2 \leq 0$$
$$P_{33}(k) - DISK \leq 0$$
$$PD3(k) - DIP_3 \leq 0 \quad . \qquad (5.6)$$

C.3.2. Constraints on availability of river water.

The total delivery rate at the head of the main canal must be less than or equal to the available surface water from the river.

$$\left[\sum_{i=1}^{3} CW_i(k)/CLHW\right] + AR1(k) \leq RIN(k) \qquad (5.7)$$

where RIN(k) is the available river flow allocated to the model area during period k .

C.3.3. Constraints on water requirements.

The total water delivered from the river and tubewells plus shortage during each period must be equal to the water requirement during the same period.

$$CW_1(k) + P_{11}(k) + SHRT_1(k) = WR_1(k) \tag{5.8}$$

$$CW_2(k) + CLFD \cdot P_{12}(k) + P_{22}(k) + SHRT_2(k) = WR_2(k)$$

$$CW_3(k) + CLFD \cdot P_{13}(k) + P_{23}(k) + P_{33}(k) + SHRT_3(k) = WR_3(k)$$

where $WR_i(k)$, $i=1,2,3$ are water requirements in zones 1, 2, and 3 during subperiod k .

C.3.4. <u>Constraints on water shortage.</u>

It is assumed that water shortage during any period can not exceed a certain limit due to the need to fulfill the goal of the proposed cropping intensity. Chaudhry (1973) assumed that a shortage, not to exceed 10% of the water requirement during each subperiod, would not substantially affect the crop production. The same assumption is used in this study. These constraints are not applied in the operational study, but are used as a check. Whenever water shortage in any of the three zones is greater than 10% , the design alternative is considered infeasible and other design alternatives must be chosen.

$$SHRT_i(k) \leq 0.10\ WR_i(k) \text{ , for } i=1,2,3 \quad . \tag{5.9}$$

C.3.5. <u>Constraints on mixing requirement.</u>

The water pumped from the aquifer in zones 2 and 3 must be mixed with surface water or fresh groundwater at certain mixing ratios in order to maintain proper salinity control.

$$P_{22}(k) - RMIX2 \cdot [CW_2(k) + P_{12}(k)] \leq 0$$

$$P_{23}(k) + P_{33}(k) - RMIX3 \cdot [CW_3(k) + P_{13}(k)] \leq 0 \tag{5.10}$$

where RMIX2 and RMIX3 are the mixing ratios of the pumped ground water in zones 2 and 3, with respect to the total surface and ground water from zone 1.

C.3.6. Constraints on prevention of salt concentration.

For a long term salt balance, the input of salt must be equal to the output of salt. A salt balance equation can be formulated with respect to the average salt concentrations and volumes of surface water, rainfall and groundwater. According to the study made by the Harvard Water Resources Group (1964), at least 10% of the pumped water must be exported out of the area to preserve the long term salt balance. Due to the lack of complete information on groundwater quality, and for simplicity, their criteria will be used in this study.

$$\begin{aligned} &PER1 \cdot P_{11}(k) - P_{12}(k) - P_{13}(k) \leq 0 \\ &PER2 \cdot P_{22}(k) - P_{23}(k) \leq 0 \end{aligned} \tag{5.11}$$

where PER1 and PER2 are the fractions of water pumped from zones 1 and 2 to be exported for long term salt balance and assumed to be equal to 0.10 in this study.

C.3.7. Continuity constraints.

Depth to water table at the end of any period $k+1$ is equal to the depth to water table at the end of the previous period $k-1$ plus the change of water table due to pumping and recharge during period k.

$$DGW_i(k) = DGW_i(k-1) - RECH_i(k) + PDC_i(k) \quad \text{for } i=1,2,3 \tag{5.12}$$

where $RECH_i(k)$ = Rise of water table due to recharge during period k for zone i, $i=1,2,3$.

$PDC_i(k)$ = Decline of water table due to pumping during period k for zone i, $i=1,2,3$.

Detailed equations for obtaining values of $RECH_i(k)$ and $PDC_i(k)$ are in Appendix C.

C.3.8. Constraints on the water table related to waterlogging.

The depth to water table must be greater than or equal to the minimum allowable depth to water table.

$$DGW_i(k) \geq DMI_i \text{ , for } i=1,2,3 \quad . \tag{5.13}$$

C.3.9. Constraints on maximum allowable depth to water table.

The depth to water table must be less than or equal to the maximum allowable depth to water table considered to be economically feasible for pumping of tubewells. According to the T&K (1967) study, this value should be set at 90 feet.

$$DGW_i(k) \leq DMA_i \text{ , for } i=1,2,3 \quad . \tag{5.14}$$

C.3.10. Constraints on lateral salt water movement.

Control on the groundwater gradient between different aquifer zones will prevent salt contamination of the fresher water. Constraints on the relative groundwater elevations in each zone are as follows:

$$DGW_1(k) = DGW_2(k) \leq RT_{12}(k)$$

$$DGW_2(k) - DGW_3(k) \leq RT_{23}(k) \tag{5.15}$$

where $RT_{12}(k)$ and $RT_{23}(k)$ are the limits of the relative difference between zones 1 and 2, and zones 2 and 3.

D. Magnitude and Structure of the Problem.

The objective function and constraints described above are all linearly related to the decision variables. The problem consists of eight design capacity variables and seventeen operational variables for each subperiod k. The constraints include 15 upper and lower limits for the design variables, spanning the entire chosen time period, and 30 constraints related to the operational variables for each subperiod.

As previously stated, linear programming can be directly applied to the problem. An advantage of the direct application of linear programming is that the design variables and operational variables can both be included in the model and solved simultaneously. The primary disadvantage is that only a few operational periods can be included; otherwise, the size of the problem becomes excessively large. Take, for example, a ten year planning time period for monthly operation, there are 120 subperiods. This would result in a total of 2,048 decision variables (including design capacity variables and operational variables) and the total number of constraints would be 3,617 . This shows how large the problem can become, and the need for decomposition of the entire period into independent subperiods.

Simplification of the Problem.

Problem simplification must be based on sound judgement and practical engineering experience. An approach which combined the scientific and empirical points of view was emphasized repeatedly in recent research by Chaudhry (1973). The following are the major simplifications made in this study.

A. Strategies for Surface Water Diversions.

If water requirements during the period, after making allowance for seepage losses, are less than the available river inflow and the design capacity at the head of the main canal, then it is possible to satisfy all the demands from the available surface water. This will always be the least-cost alternative, since the operational and maintenance cost for the diversion of surface water in the canal distribution system is relatively small and constant compared to the pumping cost from tubewells. Thus,

$$CW_i(k) = WR_i(k) \ , \quad \text{for} \quad i=1,2,3 \quad \text{and} \quad k=1,2,\ldots,n \quad . \qquad (5.16)$$

Diversion for artificial recharge is possible for certain situations and will depend on the following three conditions:

1. The available river flow is greater than the amount required to satisfy the water demand in the three zones.

2. The capacity of the main canal is greater than that needed for the diversion requirements plus the seepage allowance.

3. Aquifer space is available for storing recharged water in the nonsaline zone.

If direct diversion of surface water to meet the water demand and artificial recharge will cause the water table in the three zones to rise higher than the allowable limit for preventing waterlogging, then it is necessary to consider pumping from the aquifer. Thus, artificial recharge is no longer feasible.

In considering direct river diversion, the lateral salt water movement should also be taken into account. If artificial recharge in the nonsaline area cannot raise the water level high enough to satisfy the relative water level constraints, it is necessary to pump some water from the intermediate and saline zones to reduce the water level to meet these constraints.

If the design capacity is less than the water requirement and the available river flow, then the water inflow is limited by the capacity of the canal system. Pumping from tubewells is necessary to try to satisfy the total water requirements.

When the available water is less than the water requirement, it is always necessary to pump water from aquifer to meet the requirements.

Once groundwater pumping is necessary, such as described above, or when the water table is higher than the allowable limit causing water-logging, it is necessary to allocate the available surface water and usable groundwater to satisfy the irrigation water requirements, subject to the applicable constraints. A linear programming subroutine is used to determine these optimal operational decisions within each subperiod.

B. Relation Between Artificial Recharge and Pumping in the Non-saline Zone.

During the dry season when the available water from the river is less than the water requirement, artificial recharge is not feasible. Water pumped from the aquifer must be supplied to satisfy the demand. Artificial recharge and pumping from the nonsaline zone thus becomes mutually exclusive.

$$AR1(k) \cdot P_{1i}(k) = 0 \ , \quad \text{for} \quad i=1,2,3 \quad . \tag{5.17}$$

C. Capacity and Pumping Rate of Skimming Wells.

In the saline zone, due to the recharge of fresh water from the canal distribution system, the quality of the groundwater in the upper layer, depth of 100 to 150 feet, is usable if mixed with other good quality water. Pumping by skimming wells from this relatively fresh water is feasible as long as local saltwater coning from the lower saline groundwater layer does not occur.

The rate of pumping from skimming wells during each period depends on the rate of recharge to the aquifer in this area. It is shown in Appendix C that a well capacity of 0.25 cfs will be safe from salt water contamination. Assuming that the wells are uniformly distributed and each covers and area of 200 acres, the total allowable installed capacity for the 330,000 acres in the saline zone is about 400 cfs.

This is within the estimated minimum recharge from surface water delivery to this area. For safety purposes it was assumed that the total installed capacity of the wells was 300 cfs, and a pumping rate of 200 cfs was used to allow possible failure of some of the wells. The design capacity of skimming wells and the operational decision for pumping during each period were determined apriori, and were excluded from decisions to be made in the optimization formulation.

D. Design Capacity at Heads of Watercourses - Saline Zone.

In addition to the available water pumped from skimming wells in the saline zone, an amount of water must be supplied either from surface water or groundwater imports from the nonsaline or intermediate zones. This water is delivered through the heads of distributaries. For a fixed amount of pumping from the skimming well, the design capacity at the heads of watercourses in the saline zone can be readily determined from the maximum water requirements during the subperiods. Expressed mathematically,

$$DCW_3 = \text{Maxima of } [WR_3(k) - P_{33}(k)] \qquad k=1,n \qquad (5.18)$$

where $P_{33}(k)$ was determined apriori, as mentioned above.

E. Relative Water Levels Between the Three Zones.

One of the methods for preventing salt water contamination due to lateral salt water movement is to control the water levels between the three zones (see Appendix C). During the dry season when there is insufficient surface water for irrigation, withdrawal of groundwater from the nonsaline zone is necessary in order to satisfy the irrigation water requirement. The water level in the nonsaline zone can thus be temporarily lowered below that in the intermediate zone within a certain limit which will prevent the contamination of the relatively fresh water in the nonsaline zone. The same situation also applies between the intermediate and saline zones.

During the wet season, when there is excess water from the river, delivery to the nonsaline zone should be increased and artificial recharge initiated in order to raise the water level and reduce intrusion of salt water. In some cases, when it is not possible to raise the water level in the relatively fresh water area high enough to prevent salt water movement, pumping of salt water from the saline zone for drainage must be undertaken to satisfy the water level constraints. After all the pumping decisions have been made for the entire study period, the design capacity of the drainage tubewell, DIP_3 , is determined to be the maximum drainage pumping rate among all subperiods and is excluded from the design capacity variables to be chosen.

F. Groundwater Consideration.

In the operational study, the optimal least cost policy for a given set of design parameters must minimize the groundwater pumping, with the condition that water requirements must be satisfied within some allowable limits (Chaudhry, 1973). For a number of consecutive subperiods, as long as groundwater pumping during each subperiod is minimized, the lowering of the groundwater table, and thus the total pumping head, is the least during that subperiod. It is obvious that when the lowering of the groundwater table is less in the previous subperiods, then the cost of pumping will be less for the current subperiod due to the reduced pumping head. In this way, the optimal operational decision for the entire period is to optimize the decision during each subperiod separately by keeping the groundwater table as high as possible. The groundwater level then will not need to be considered as a state variable, and the optimal operational decisions during each subperiod can be determined independently.

It is assumed that recharge from the diversion of surface water in the canal distribution system and diversion for artificial recharge to the nonsaline zone will keep the water table as high as possible without waterlogging. Under this condition the cost of pumping will be small compared to the water shortage, which is within the allowable limit. Only the groundwater aquifer in the nonsaline zone will receive artificial recharge, and only the water level in this area is considered in minimizing the cost of groundwater pumping. The depth to water table in the intermediate and saline zones are restricted by limits relative to that of the nonsaline zone (see Appendix C).

Under this operational policy, the constraints on upper and lower limit on depth to the ground water table still must be considered. If the depth to the groundwater table exceeds the maximum allowable pumping depth, there is no way to satisfy the demands from pumping underground. The design alternative under consideration is then infeasible and the capacity of the canal system must be increased to supply more surface water to the area. For this study the maximum depth to water table constraint (90 feet according to T&K) is neglected. Since it will seldom occur when the aquifer is considered as a storage reservoir to keep the water table as high as possible within the waterlogging constraint. A check can be made in the final solution. If the water table is higher than that allowable then it is necessary to reduce the supply from the river flow and increase the pumping from tubewells to lower the water table to the required minimum allowable depth.

G. Mixing Criteria.

The proposed water requirements in each zone were assumed to include the leaching requirements for preventing salt accumulation in the root zone in each area. Under this assumption it is not necessary to have that mixing criteria satisfied during each subperiod. Since the main sources of water supply will be from the relatively fresh surface water and groundwater in the nonsaline zone, it can be expected that the overall mixing of surface water and groundwater in a longer period, such as three months or a year, will meet the required criteria. This constraint will therefore be neglected in the operational study. A check can be made to see if the constraint has been violated under an optimal policy.

H. Design Capacity at the Head of the Main Canal.

The capacity at the head of the main canal can be reasonably expressed as follows:

$$DCH = \sum_{i=1}^{3} DCW_i/CLHW \quad . \tag{5.19}$$

The assumption here is that during some of the wet summer seasons, the diversion of surface water to the three zones will be at their full capacities in order to satisfy the water demand and necessary artificial recharge in the nonsaline zone.

General Formulation of the Simplified Problem.

A. Decision Variables.

There are four design capacity variables after simplification:

DCW_i - The lumped capacity at heads of watercourses for area i (i=1,2).

DIP_i - Total tubewell installation capacity for area i $(i=1,2)$.

There are still sixteen operational decision variables to be determined for optimal operational policy within each subperiod. The pumping rate of skimming wells, $P_{33}(k)$, in area 3 was determined apriori as stated previously.

B. Objective Function.

The simplified objective function is:

$$\text{Minimize } Z = \text{Minimize } (CD \cdot y + CO \cdot x) \tag{5.20}$$

where y = column vector of design variables,

$= [DCW_i, DIP_i, i=1,2]$

x = column vector of operational variables,

$$= [CW_i(k), SHRT_i(k), DGW_i(k), AR1(k), PD3(k), P_{ij}(k)] \tag{5.21}$$

$i = 1,2,3$

$j = 1,\ldots,3$

$k = 1,2,\ldots,n$

and Z, CD and CO are the same as previously defined.

C. Constraints.

All decision variables must be greater than or equal to zero. These are referred to as the non-negativity constraints.

Two types of design capacity constraints must be considered to assure that the capacities are greater than existing capacities but less than or equal to some preselected upper limit. They are expressed as $y \geq b_\ell$ assuring the capacity is greater than the existing and $y \leq b_u$ for the upper limit.

C.1. Operational constraints for each subperiod k, $k=1,2,\ldots,n$.

C.1.1. Constraints on canal and tubewell operation decisions.

The total delivery rate at heads of watercourses must be less than or equal to the design capacity,

$$CW_1(k) - CLHW \cdot AR1(k) - DCW_1 \leq 0$$
$$CW_2(k) - CLFD \cdot P_{12}(k) - DCW_2 \leq 0$$
$$CW_3(k) - CLFD \cdot P_{13}(k) - CLFD \cdot P_{23}(k) - DCW_3 \leq 0 \quad . \qquad (5.22)$$

Similarly the total delivery rate at the head of the main canal must be less than or equal to the design capacity at the head of the main canal,

$$\sum_{i=1}^{3} CW_i(k) + CLHW \cdot AR1(k) - \sum_{i=1}^{3} DCW_i \leq 0 \qquad (5.23)$$

and the total pumping rate must be less than or equal to the tubewell installed capacity within each area,

$$P_{11}(k) + P_{12}(k) + P_{13}(k) - DIP_1 \leq 0$$
$$P_{22}(k) + P_{23}(k) \qquad - DIP_2 \leq 0 \quad . \qquad (5.24)$$

C.1.2. Constraints on availability of river water.

$$\left[\sum_{i=1}^{3} CW_i(k)/CLHW\right] + AR1(k) - RIN(k) \leq 0 \quad . \qquad (5.25)$$

C.1.3. Constraints on water requirements.

$$CW_1(k) + P_{11}(k) + SHRT_1(k) = WR_1(k)$$
$$CW_2(k) + CLFD \cdot P_{12}(k) + P_{22}(k) + SHRT_2(k) = WR_2(k)$$
$$CW_3(k) + CLFD \cdot P_{13}(k) + CLFD \cdot P_{23}(k) + SHRT_3(k)$$
$$= WR_3(k) - P_{33}(k) \quad . \qquad (5.26)$$

C.1.4. Constraints on water shortage.

$$SHRT_i(k) \leq SLMIT_i(k) \quad , \quad \text{for } i=1,2,3 \quad . \qquad (5.27)$$

C.1.5. Constraints on prevention of salt concentration.

$$PER1 \cdot P_{11}(k) - P_{12}(k) - P_{13}(k) \leq 0$$

$$PER2 \cdot P_{22}(k) - P_{23}(k) \leq 0 \quad . \qquad (5.28)$$

C.1.6. Continuity of water tables.

$$DGW_i(k) = DGW_i(k-1) - RECH_i(k) + PDC_i(k) \quad , \text{ for } i=1,2,3 \quad . \qquad (5.29)$$

C.1.7. Constraints on water table related to waterlogging.

$$DGW_i(k) \geq DMI_i \quad , \text{ for } i=1,2,3 \quad . \qquad (5.30)$$

C.1.8. Constraints on water table related to lateral salt water movement.

$$DGW_1(k) - DGW_2(k) \leq RT_{12}(k)$$

$$DGW_2(k) - DGW_3(k) \leq RT_{23}(k) \quad . \qquad (5.31)$$

Reformulation of the Problem.

In order to deal with the large scale linear programming problem, it was necessary to simplify the problem using some intuitive insight and judgement. The simplified problem presented in the above section was still too large, so it was necessary to decompose the problem into two levels of optimization. The large problem was replaced with an equivalent problem involving solution of several smaller problems. The design and operational aspects were separated and the operational problem associated with each subperiod was solved independently. A more detailed description is presented in the following sections.

Projection is a problem manipulation device which takes advantage of the relative simplicity introduced by temporarily fixing the value of certain variables (Geoffrion, 1969). Observing the structure of this problem, it can be seen that if the design variables are fixed temporarily, then the problem reduces to an operational problem. The inner operational problem is a time sequential type problem which can be reformulated as a multi-stage sequential decision process resembling dynamic programming.

The problem is restated in the following general form:

$$\min_{y,x} [CD \cdot y + CO \cdot x]$$

Subject to:

$$G(y) + g(x) \leq 0$$

$$x \in X$$

$$y \in Y \tag{5.32}$$

where $G(y)$ and $g(x)$ are vector-valued functions, X and Y are the sets which define the upper and lower bounds of x and y, and the constraints stated above are equivalent to the constraints stated in the previous section.

Through projection onto the space of design capacity variables y alone, the result is

$$\min_{y \in Y \cap W} w(y) = \min_{y \in Y} \; CD \cdot y + V(y) \tag{5.33}$$

where $$V(y) = \min_{x \in X} CO \cdot x \tag{5.34}$$

Subject to:

$$g(x) \leq - G(y) \quad .$$

Assuming a minimum exists for $y \varepsilon Y$ which defines the greatest lower bound of $CO \cdot x$ for given y, over x. Define $W(y) = \infty$, if the inner problem is infeasible. So y must be in the set

$$W \equiv \{ y \mid w(y) < \infty \}$$

$$\equiv \{ y \mid g(x) \leq - G(y) \text{ for some } x \varepsilon X \} \quad . \tag{5.35}$$

The problem now has two parts: the design problem and the inner operational problem. The design capacity variables in the design problem are treated as parameters in the inner operational problem. Each time the design problem assigns a set of design capacity variables, the inner operational problem is solved optimally. This result is fed back to the design problem to obtain the overall results. Another set of design capacity variables are assigned to the inner operational problem to find another set of results, and so on, until the best result is found according to some convergence criteria.

Details of the solution procedures will be described in subsequent sections. The inner operational problem is discussed first, assuming a given design alternative, followed by discussion of the overall optimal solution with regard to design capacity variables and operational decisions. Figures 5.1 and 5.2 show the computational procedures.

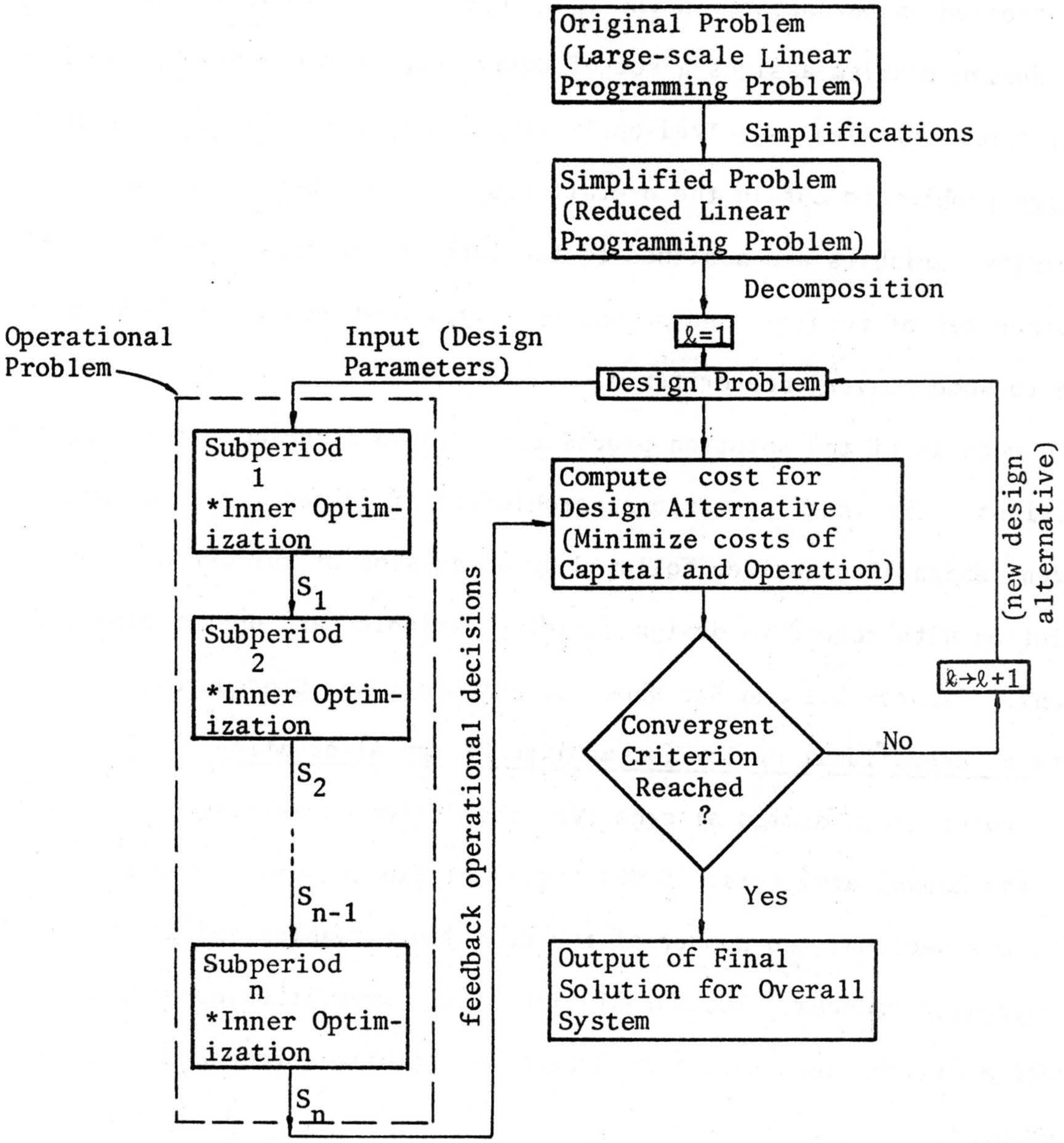
Optimal Operational Policy for a Given Design Alternative.

For a given design alternative, the design capacities of the system are the known parameters. Under the simplified assumptions made in the previous section, the number of the decision variables and constraints are greatly reduced. The formulation of the simplified operational problem will be described first, then the solution procedures will be presented.

A. General Statement of the Operational Problem.

The operational problem including constraints can be expressed mathematically as:

$$\underset{x \varepsilon X}{\text{Min}}\ C = \underset{\substack{x_k \varepsilon X_k \\ k=1,\ldots,n}}{\text{Min}} \sum_{k=1}^{n} CO \cdot x_k$$



State transformation:

$S_k = T_{k-1}\,(S_{k-1}, x_k)$, i.e.

$DGW_i(k) = DGW_i(k-1) - RECH_i(k) + PDC_i(k)$

*See figure 5.2.

Figure 5.1. Problem Decomposition and Solution Procedures.

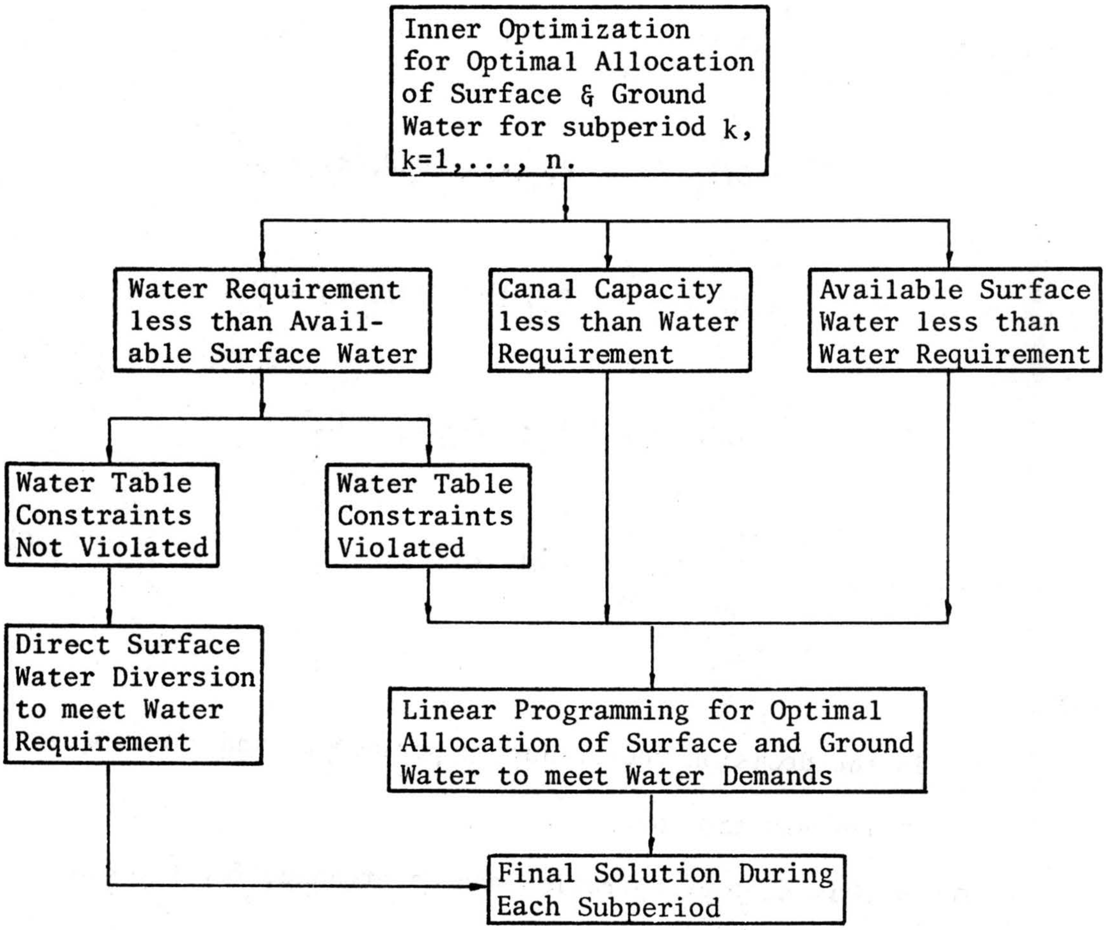


Figure 5.2. Inner Operational Decisions During Each Subperiod.

$$\underset{x \in X}{\text{Min}}\ C = \underset{x_k \in X_k}{\text{Min}} \sum_{k=1}^{n} \{\ COS \cdot [\ \sum_{i=1}^{3} CW_i(k)] + CAR \cdot AR1(k)$$

$$+ CTI_1 \cdot [\ \sum_{j=1}^{3} P_{1j}(k)] \cdot (DGW_1(k) + H_1)$$

$$+ CTI_2 \cdot [\ \sum_{j=2}^{3} P_{2j}(k)] \cdot (DGW_2(k) + H_2)$$

$$+ CTI_3 \cdot PD3(k) \cdot (DGW_3(k) + H_3)$$

$$+ CST \cdot [\ \sum_{i=1}^{3} SHRT_i(k)]\ \} \qquad (5.36)$$

where:

x_k is the decision vector during period k and set X_k represents the bounds on x_k.

COS = Unit cost of operation and maintenance for the canal distribution system.

CAR = Unit cost of operation and maintenance for artificial recharge.

CTI1, CTI2, CTI3 = Unit cost of energy for pumping from zones 1,2,3.

CST = Unit cost of shortage in zones 1,2, and 3, assumed to be the same in each zone.

H_1, H_2, H_3 = Dynamic heads including drawdown and head loss for tubewells pumping in zones 1,2, and 3.

This formulation is subject to the operational constraints described in the general formulation of the simplified system, except that design capacity variables are now put on the right hand side of the equations and considered as known parameters.

B. Formulation as a Multi-Stage Sequential Process.

The general operational problem stated above can be reformulated as a sequential decision problem. The whole operational problem is divided into stages with each subperiod as a stage. Within each stage, there are decisions on the allocation of available surface water and groundwater to meet irrigation water demands in the three different groundwater quality zones. Decisions on surface water diversions and groundwater pumping in each zone for a particular period will transform the groundwater levels at the beginning of the period to a new level at the end of the period. Figure 5.3 shows the diagram describing this process. Figure 5.4 shows the transformation of groundwater table between two consecutive periods, k-1 and k . The state transformations are

$$DGW_i(k) = DGW_i(k-1) - DT_i(k) \quad ,$$
$$DT_i(k) = RECH_i(k) - PDC_i(k) \tag{5.37}$$

where $DT_i(k)$ = Net change of water table during period k ,

$RECH_i(k)$ = Rise of water table due to recharge during period k and is dependent on operational variables $x(k)$,

$PDC_i(k)$ = Decline of water table due to pumping during period k and is dependent on operational variables $x(k)$.

As mentioned previously, the optimal least cost operational policy is to minimize the groundwater pumping and yet satisfy the water requirements. As long as groundwater pumping during each period is minimized, the net lowering of the groundwater level will be minimized during that period and the groundwater table will remain as high as possible within the waterlogging constraint. In this way, the operational decisions can be determined independently within each period. The operational problem can now be simplified as

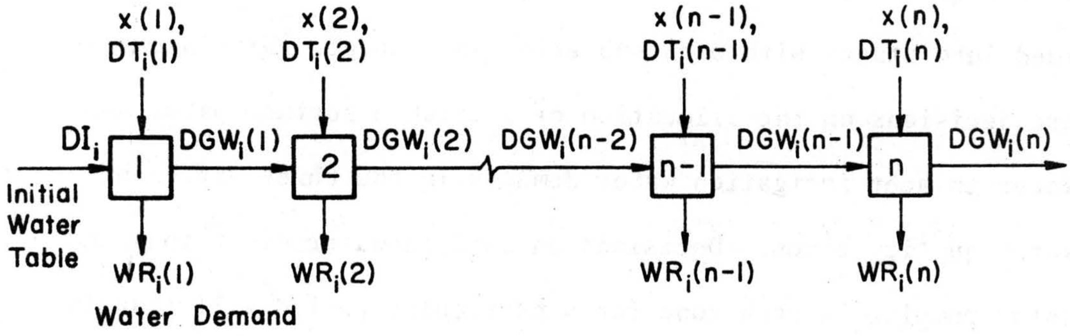


Figure 5.3. Sequential decision process.

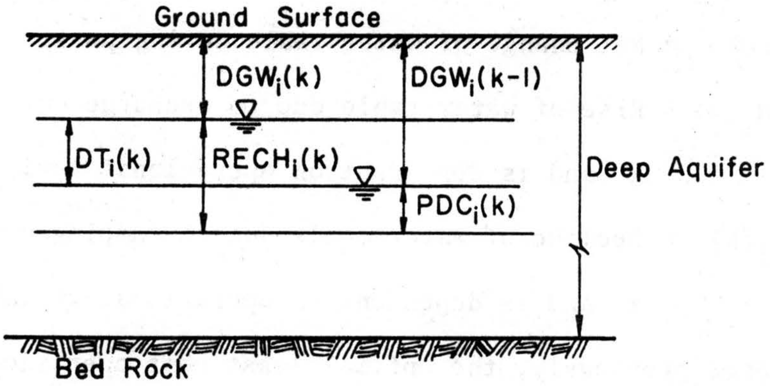


Figure 5.4. State transformation of water table.

$$\underset{z \varepsilon X}{\text{Min}}\ Z = \sum_{k=1}^{n} \underset{x_k \varepsilon X_k}{\text{Min}}\ CO \cdot x_k \quad . \tag{5.38}$$

This objective function is subject to the operational constraints stated in the general formulation of the simplified problem.

C. Optimal Solution Procedures.

C.1. Available Surface Water Exceeds the Water Requirement.

During the summer season when the available surface water from the river is greater than the total water requirement, the following policy will be followed:

1. If the capacity of the canal system is greater than the water requirement, then , direct diversion of river flows to the three zones is the most feasible.

$$CW_i(k) = WR_i(k) \quad , \quad i=1,2$$

$$CW_3(k) = WR_3(k) - P_{33}(k) \tag{5.39}$$

where $P_{33}(k)$ is constant throughout the whole study period, and the other decision variables are zero.

2. The extra available surface water then will be diverted for artificial recharge in the nonsaline area. The amount of artificial recharge will be limited to:

$$AR1(k) = \text{Minima of } \{[RIN(k) - (\sum_{i=1}^{3} WR_i(k) - P_{33}(k)]/CLHW\} ,$$

$$[DCW_1 - WR_1(k)] \ , \ ASPACE(k)\} \tag{5.40}$$

where ASPACE(k) is the available aquifer space in the nonsaline aquifer during period k .

3. It is necessary to check the relative groundwater levels in the three zones. If the constraints are violated, pumping from zone 2 and/or 3 is needed to lower the water table to satisfy the relative water level constraints.

4. If capacity of the canal system is less than the water requirement, there is no extra capacity available for artificial recharge and AR1(k) = 0 . Groundwater is pumped to supplement the irrigation water requirement. A linear programming subroutine, described below, is then used to determine the optimal decisions.

C.2. Available Surface Water Less Than the Water Requirement.

During some of the periods, especially the dry season, when the available surface water from the river is less than the total water requirement, tubewell pumping is required to satisfy the water demand. The optimal decisions are determined by solving a linear programming problem. The decision variables, objective function and constraints are those representing the multistage sequential process problem.

C.2.1. Decision variables.

The decision variables for the inner operational problem are the same as stated in the decision variables of the simplified problem considering only one period, except that AR1(k) is equal to zero. Totally, there are fifteen decision variables.

C.2.2. Objective function.

$$\min_{x_k \in X_k} C = CO \cdot x_k \quad . \tag{5.41}$$

C.2.3. Constraints.

There are sixteen constraints for the simplified operational problem which can be grouped as follows:

1. Design capacity constraints.

$$\begin{aligned}
&CW_1(k) \le DCW_1 \\
&CW_2(k) + CLFD \cdot P_{12}(k) \le DCW_2 \\
&CW_3(k) + CLFD \cdot P_{13}(k) + CLFD \cdot P_{23}(k) \le DCW_3 \\
&P_{11}(k) + P_{12}(k) + P_{13}(k) \le DIP_1 \\
&P_{22}(k) + P_{23}(k) \le DIP_2
\end{aligned} \tag{5.42}$$

2. Capacity or available river flow constraint.

$$\sum_{i=1}^{3} CW_i(k) \le \text{Minima of } [CLHW \cdot RIN(k) \ , \ \sum_{i=1}^{3} DCW_i] \tag{5.43}$$

3. Constraints for preventing salt concentration.

$$\begin{aligned}
&PER_1 \cdot P_{11}(k) - P_{12}(k) - P_{13}(k) \le 0 \\
&PER_2 \cdot P_{22}(k) - P_{23}(k) \le 0
\end{aligned} \tag{5.44}$$

4. Relative water level constraints.

$$\begin{aligned}
&DGW_1(k) - DGW_2(k) \le RT_{12}(k) \\
&DGW_2(k) - DGW_3(k) \le RT_{23}(k)
\end{aligned} \tag{5.45}$$

5. Water requirement constraints.

$$\begin{aligned}
&CW_1(k) + P_{11}(k) + SHRT_1(k) = WR_1(k) \\
&CW_2(k) + CLFD \cdot P_{12}(k) + P_{22}(k) + SHRT_2(k) = WR_2(k) \\
&CW_3(k) + CLFD \cdot P_{13}(k) + CLFD \cdot P_{23}(k) + SHRT_3(k) = WR_3(k) - P_{33}(k) \ .
\end{aligned} \tag{5.46}$$

6. Continuity constraints.

$$RECH_i(k) - PDC_i(k) + DGW_i(k) = DGW_i(k-1) \quad , \quad \text{for } i=1,2,3. \tag{5.47}$$

The above procedures are performed for each subperiod through the entire planning period depending on the conditions of the available river flow, the design capacity of the canal system and water requirements. The connection between two consecutuve subperiods is calculated through the continuity equation stated in equation 5.47.

The conjunctive use system of surface water and groundwater is designed to satisfy the irrigation water requirements. If water deficiency does occur at times, it must be less than the specified 10% limit, otherwise the chosen design alternative is considered infeasible. Once this situation occurs, the operational study will be stopped and other design alternatives must be chosen. This helps to save computational time in searching for the optimal design alternative.

Numerical Search Techniques for Design Alternatives.

As previously described, operational costs are evaluated at the first level for a particular design alternative. There is no functional way to express this operational cost related to the design variables, and it is not possible to evaluate derivatives of the objective function explicitly. Because of this, gradient search methods requiring analytic derivatives are not applicable. Search methods that numerically estimate the gradient from objective function values must be utilized.

The constrained optimization problem can be attacked by one of two approaches. First, the search for an optimum, constrained by some inequalities, is carried out ensuring that each new point is a feasible one, and to direct the search in a feasible direction when a nonfeasible point is found. The second approach is to convert the constrained problem into the form of an unrestricted problem and use an unconstrained optimization technique directly.

Penalty function methods have been widely used to transform a constrained problem into an unconstrained one, when the penalty terms are used to incorporate the constraints into a modified objective function. An example penalty function (Himmelblau, 1972) is:

$$\text{Min } Z(y) = f(y) + \sum_{i=1}^{m} H(g_i)\cdot P_i\cdot(g_i(y))^2 \tag{5.48}$$

where $f(y)$ = original objective function.

$g_i(y)$ = original ith constraint function.

P_i = a positive-valued constant for constraints, for $i=1,2,\ldots,m$.

$$H(g_i) = 1, \text{ if } g_i(y) \geq 0$$
$$= 0, \text{ if } g_i(y) < 0 \quad . \tag{5.49}$$

During the minimization process performed by any of the unconstrained solution methods, the decision vector, y , is forced by the penalty to satisfy the constraints to some degree, depending on how the value of the penalty factor, P_i , is chosen. Clearly, as long as $g(y)$ is satisfied, and, as y reaches the optimal value, the value of P_i becomes negligible and the minimum $Z(y)$ approaches the minimum of $f(y)$.

The general procedure for use of this method is:

1. Choose the penalty factors P_i , $i=1,\ldots,m$, starting with small values.

2. Use a suitable unconstrained search technique to determine the optimum of $Z(y)$.

3. Adjust values of P_i , $i=1,\ldots,m$, based on some other search techniques. In general, these values are gradually increased.

4. Repeat steps 2 and 3 until some convergence criteria are satisfied.

For this study, the modified objective function is

$$\begin{aligned} \min_{y \in Y} Z(y) = {} & CD \cdot y + C^*(x) \\ & + \sum_{i=1}^{2} P_i(DCW_i - EDCW_i)^2 + \sum_{i=1}^{2} P_{i+2}(DCW_i - DMCW_i)^2 \\ & + \sum_{i=1}^{2} P_{i+4}(DIP_i - EDIP_i)^2 + \sum_{i=1}^{2} P_{i+6}(DIP_i - DMIP_i)^2 \\ & + \sum_{i=1}^{2} P_{i+8}(DCW_i + (1-PER_i) \cdot DIP_i - WRM_i)^2 \qquad (5.50) \end{aligned}$$

where $C^*(x) = \sum_{k=1}^{n} \min CO \cdot x_k$ is the minimum cost of operation under a certain design alternative, and P_i for $i=1,\ldots,10$ are penalty factors for each constraint. Parameters WRM_1 and WRM_2 are the maximum water requirements during a subperiod for area 1 and 2 respectively.

Once the constrained problem had been transformed to an unconstrained problem, Powell's method (Powell, 1964) for minimizing unconstrained problem was applied. In addition, direct solution of the constrained problem was carried out by the flexible tolerance method (Paviani and Himmelblau, 1969). A systematic search method (Chaudhry, 1973), based on intuitive judgement to exclude the obviously nonoptimal solution, was also attempted.

Powell's method is based on the properties of conjugate directions, and does not require analytical derivatives. Beginning from an initial point, a series of linearly independent directions for searches are generated. Before proceeding from one direction to another, a check is made to see if the new direction is more effective or not. If it is not, the old direction is followed again. Starting from the last best point, the procedure is repeated again until the required convergence is reached. Detailed development of the theory is available in the publication by Powell (1964).

The flexible tolerance method is an extension of the flexible polyhedron search for unconstrained optimization proposed by Nelder and Mead (1964). Additional rules are added to take care of the equality and/or inequality constraints. A flexible tolerance criterion which combines all restrictions into a single tolerance is set up for constraint violation throughout the search. New points are checked to ensure that they improve the objective function and that they satisfy the tolerance. The tolerance limits are gradually decreased and become more restrictive as the search progresses toward the final solution. A review of the development and theory are presented in Appendix B.

The systematic search method is the straight forward enumeration search. But based on physical reasoning, a large number of design alternatives can be excluded in the course of the search. The criteria of search are:

1. Grid points are set up for each design capacity variable. Starting from the lowest value of each variable, the possible design alternatives are obtained through different combinations of grid points by an iterative procedure.
2. Before the operational study, the combined design alternative is checked to see if the alternative is feasible or not. If it is not, another feasible one must be found by increasing the value of one of the variables successively.

3. During the operational study of any feasible design, if shortages of water requirement exceed the specified limits during any period, the design is dropped immediately. The next higher value of the variable is chosen.

4. For any alternative which is feasible in both design and operation, if increase of any variable results in a higher cost, further increase of that variable should not be made.

One of the particular features of the solution procedure is that in the course of searching the best design alternative, the operational study will be stopped immediately when the selected design alternative is unfeasible for system operation (i.e. when the shortage of water requirement exceeds the allowable limit). Under this condition, it is not possible to evaluate the actual operational cost. A very large cost value is then assigned to this alternative to ensure the exclusion of this infeasible solution.

With Powell's method there is the possibility of obtaining negative values during the search, and the assignment of a large cost to assure the exclusion of the infeasible design has caused irregularity in evaluating search gradients in the solution procedure. The systematic search method does not have the disadvantage of Powell's method, but with more design variables, the search becomes prolonged if there are too many grid points for each variable. A large number of grids may be required for accuracy. The flexible tolerance method does not have either of the above disadvantages and it appears to be the most promising technique for the solution of this problem. It was adopted for this study.

CHAPTER VI

COMPUTATION AND RESULTS

The simplified mathematical model developed in Chapter V was programed in Fortran IV language and was solved using the CDC 6400 computer available at Colorado State University. A description of the computer program and the program itself are included as Appendix D. In this chapter, the general computational procedures are described, selected results are presented and a general discussion of all results follows.

General Computational Procedures for Overall System Optimization.

The general computational procedures are depicted in the flow chart of figure 6.1. The computer program was adopted from Himmelblau (1972) for the flexible tolerance method and was modified to include two other subroutines for optimizing the inner operational policy. The original program which includes the main program and five subroutines - WRITEX, SUMR, PROBLEM, START and FRASBL was used mainly for searching the optimal design alternative. They calculate the overall capital and operational and maintenance costs, compute the sum of the squares of the violated constraints which are compared with the tolerance criterion and search for the new design variables which minimize the sum of the square value for the violated constraints. For each design alternative, subroutines DYP and SIMPLEX were used to determine the optimal operational decisions during each subperiod. These are described in detail in Appendix D.
The following general computational procedures are performed in the computer program:

1. The main program will read input data, calculate the required parameters and also make the initial guess of the design capacity

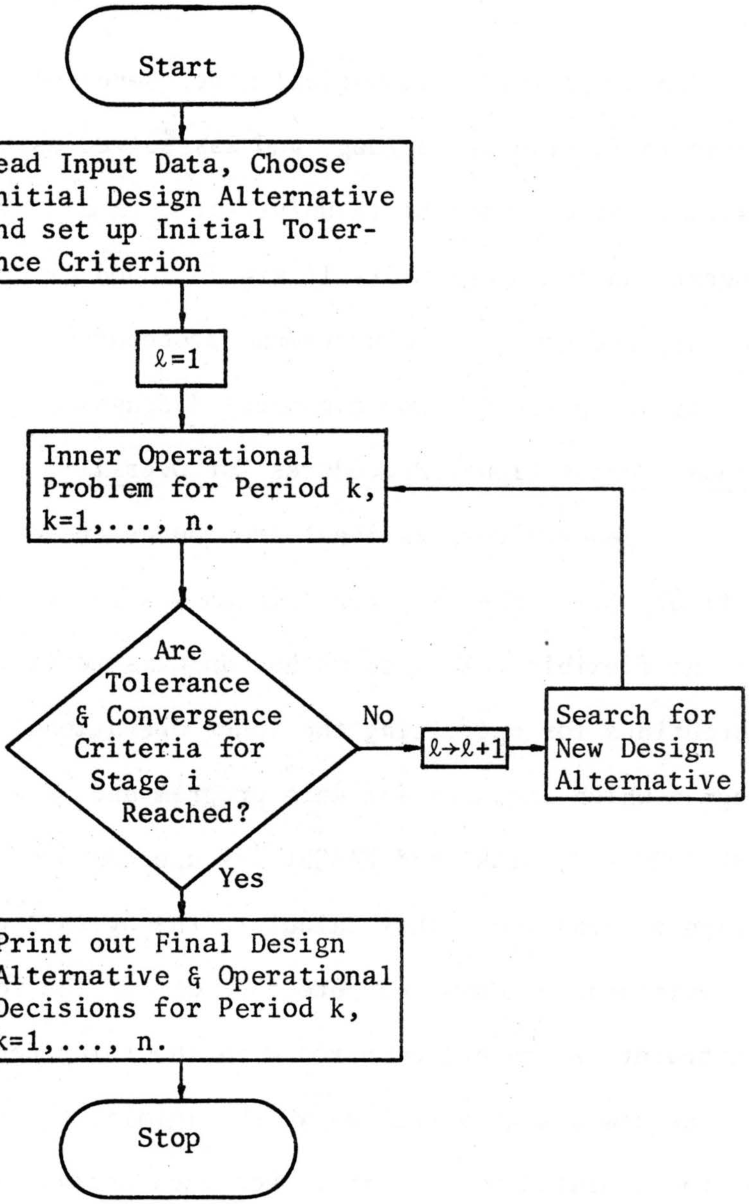


Figure 6.1. Flow Chart of General Computational Procedures.

variables. The tolerance criterion used during each iterative stage of calculation is assigned in the main program.

2. Determine the optimal inner operational decisions during each subperiod k(k=1,2,...,n) for the current design alternative by calling subroutines DYP and SIMPLEX.

3. Compute total capital, operational and maintenance costs for the current design alternative.

4. Search for a new design alternative (outer design problem) by utilizing the flexible tolerance method. The search procedures are included in the main program and subroutines are called whenever they are needed.

5. In each iterative stage of search, the new design alternative will be considered as given parameters and steps 2 and 3 above for determining the optimal inner operational policy and computing the overall capital and operational costs will be repeated. The new tolerance criterion is also calculated during each new iterative stage of calculation for the new improved design alternative. The tolerance criterion is reduced with each iteration as mentioned in the numerical search techniques of Chapter V.

6. The search is continued until the required tolerance criterion is reached. The final optimal design alternative and the final optimal operational policy are printed out.

<u>Application to the Lower Jhelum Canal Commanded Area.</u>

The Lower Jhelum Canal Commanded area was selected to test the mathematical model and the computer program. Several computer runs were made for this specific area including different hydrological inputs for the available surface water depicting the high flow and low flow situations. Water demands in the saline area were varied for cropping intensities of 100 and 150 percent. Monthly operational studies were made for a three year period. The tolerance and convergent criteria for the search of the optimal design alternative were set to be within 40 cfs of the design capacity variables. Detailed descriptions of the various outputs for the different computer runs are presented in Appendix E. Selected results for a low river flow situation with cropping intensity of 150% for each of the three areas are as follows:

Run E.1. Cropping intensities in the three areas are all equal to 150% , low river flows. Minimized total cost including fixed and operational costs = RS 166 million.

Design Capacities

Item	Zones: Nonsaline	Intermediate	Saline
Capacity at heads of watercourses	5,156 cfs	915 cfs	2,010 cfs
Watercourse Remodeling Ratio*	2.39	1.12	2.45
Installed Tubewell Capacities	6,058	1,650	1045 cfs (drainage) 300 cfs (skimming)

*The modeling ratio is defined as the ratio of the new design capacity with respect to the existing design capacity.

Design capacity at the head of the main canal = 11,544 cfs with a remodeling ratio of 2.13 .

The operational decisions were determined on a monthly basis for a total period of three years. These monthly operational decisions are listed in Appendix E and are graphed as lumped sums in figure 6.2. The most significant features of this solution can be summarized as follows:

1. The total design capacity of the overall canal system must be enlarged to increase surface water diversions to meet the water demands and provide water for artificial recharge during the summer months as indicated in the figure for the months of June to August of the first year and months of July and August of the second year.

2. The optimal operational decisions show that no water shortage occurs in any month within the three year period. Generally speaking, more groundwater withdrawal is necessary during the dry season (October to March).

3. In some of the high flow months such as June and August, there is enough surface water to satisfy the water requirement. But due to the limited capacity of the canal design, pumping of groundwater is still necessary. In the month of July, due to the lower demand, canal capacity is large enough to carry part of the surface water for recharge and no pumping of groundwater from the nonsaline and intermediate areas is required.

4. During the months of April in each year and May of the first and third year, the extra amount of surface water available was diverted for artificial recharge.

5. In August of the third year, there is enough surface water to satisfy the demand, however, groundwater still must be pumped and

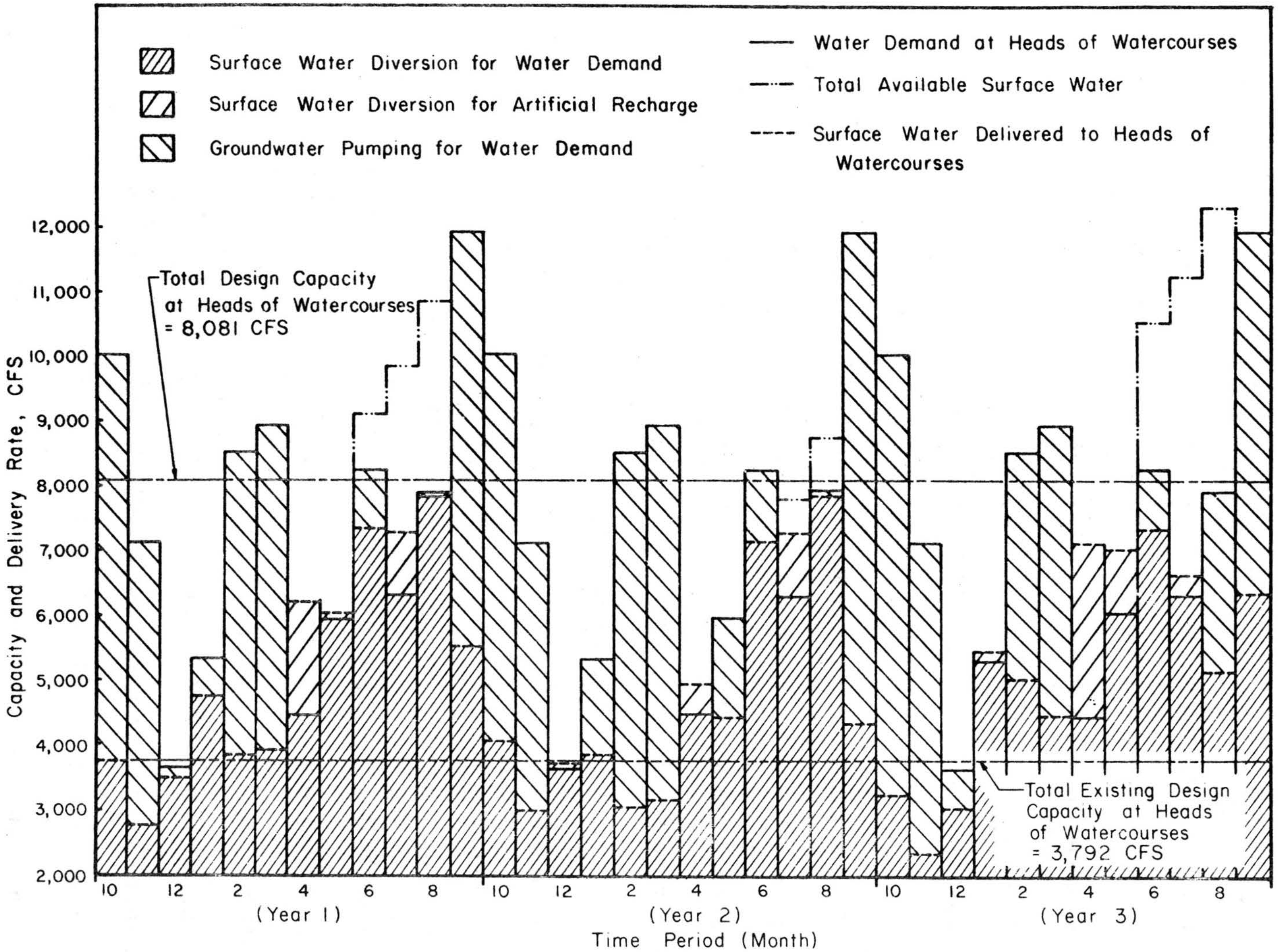


Figure 6.2. Allocation of available surface water and groundwater to the total model area (150% cropping intensity in all three zones).

used in order to maintain the depth to water table in the non-saline area in excess of 10 feet, thus preventing waterlogging.

6. In some months such as December of year 1 and 2, May of year 1 and 3, and April in each year, surface water can satisfy water requirements except in the saline area where a fixed amount of water is constantly pumped out from skimming wells. This is probably due to the low water demand for the months of December and April and the rising river flows beginning in May.

Figures 6.3 to 6.5 illustrate the monthly allocation of both surface and ground water to each individual subarea for the three year period. Data for the nonsaline area is presented in figure 6.3 and can be summarized as follows:

1. The optimal operational decisions indicated that the expansion of the existing canal capacity in this area serves both the purposes of diverting more surface water to meet water demands and provide artificial recharge.

2. Groundwater withdrawal is necessary in the high flow month of August of the third year in order to lower the groundwater table to staisfy the groundwater table constraint.

3. In the month of July, water demand for the area is small compared to the other high flow months of June and August, so there is extra canal capacity for diverting water for artificial recharge.

4. The available surface water for April is not large compared to the other high flow months, but the water demand is also comparatively small so that diversion of water for artificial recharge occurred.

Figure 6.4 shows the water allocated from surface and ground water for the intermediate area. It is summarized as follows:

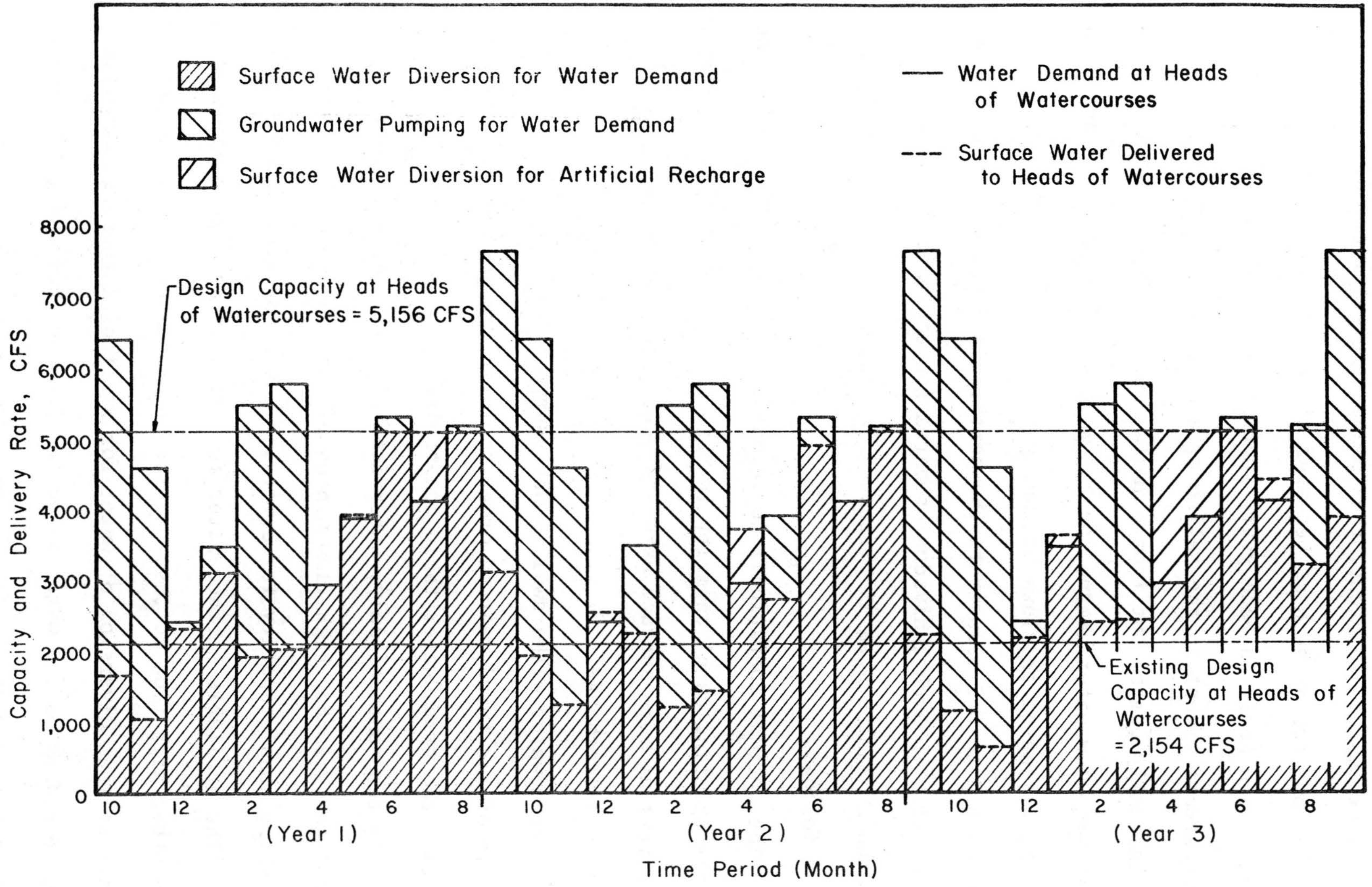


Figure 6.3. Allocation of available surface and ground water to the nonsaline zone (150% cropping intensity in all three zones).

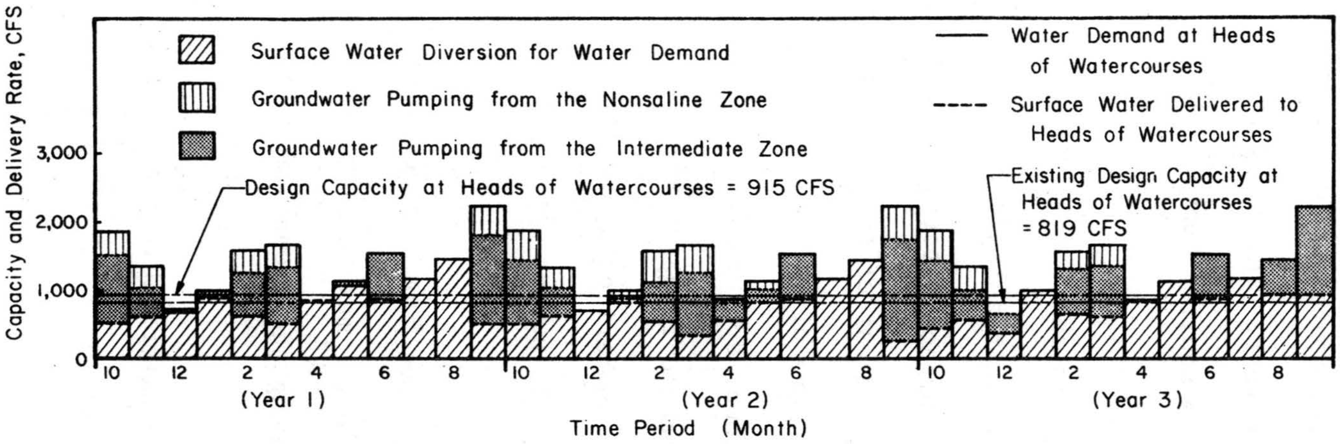


Figure 6.4. Allocation of available surface water and ground water to the intermediate zone (150% cropping intensity in all three zones).

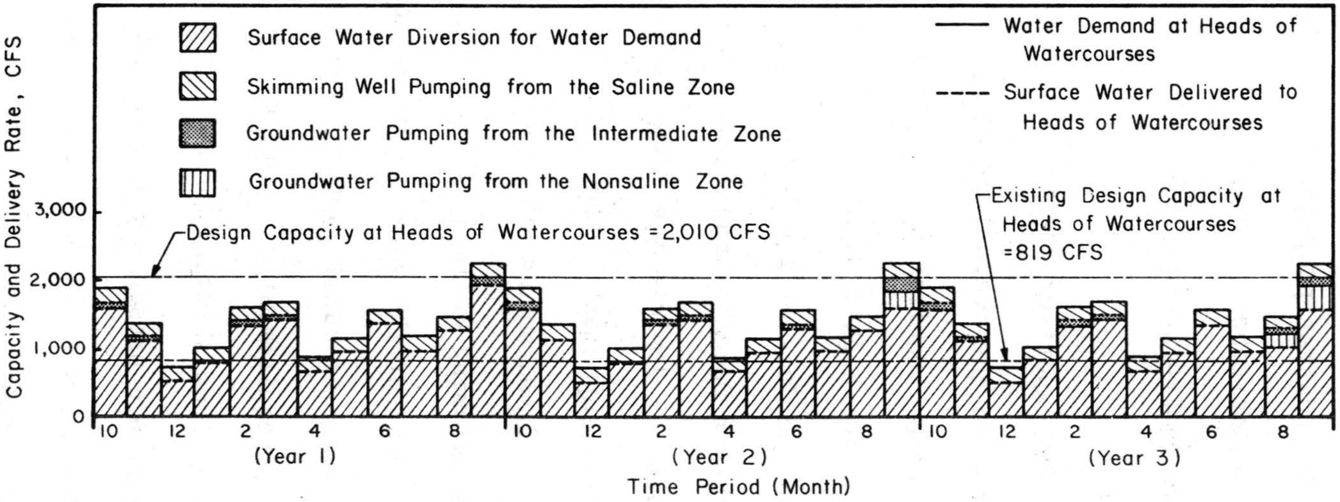


Figure 6.5. Allocation of available surface water and ground water to the saline zone (150% cropping intensity in all three zones).

1. The required expansion of the existing canal capacity in this area is small. Only a 12% increase of the existing capacity is needed.

2. Groundwater pumping is concentrated in the months of September, October, November, February and March of each year. The major part of the pumped groundwater is obtained from the aquifer beneath the area.

3. The export of groundwater from this area to the saline area is limited to the amount required to satisfy the salt balance constraints.

4. Water requirements can be satisfied by surface water diversions during the high flow months such as July and August of the first two years and July of the third year and during some of the low flow months of December and January because of lower demands.

Figure 6.5 depicts the amount of water allocated from surface and ground water supplies to the saline area. It is summarized as follows:

1. A fixed amount of groundwater (300 cfs) is pumped constantly from skimming wells in this area.

2. Most of the water requirements are supplied from surface water. The existing canal capacity was increased to the maximum monthly water requirement of September after excluding the amount of water pumped from skimming wells.

Figure 6.6 presents the fluctuations of the water table for all three areas during the three year period. The initial depth to water table was assumed to be 15 , 16.5 and 18 feet respectively for the nonsaline, intermediate and saline areas. Analysis of the water table data can be summarized as follows:

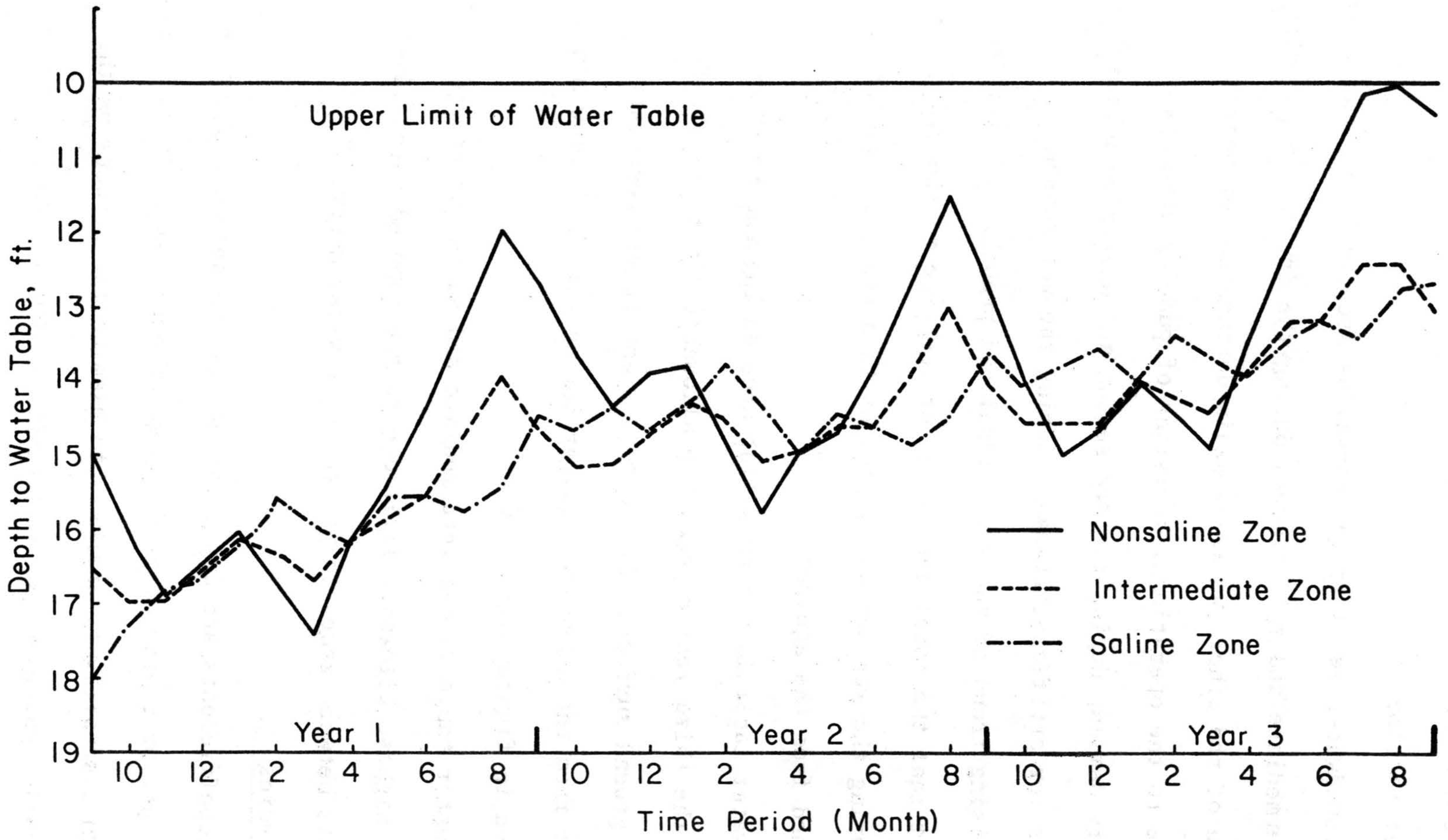


Figure 6.6. Fluctuation of the depth to water table with time (150% cropping intensity in all three zones).

1. In general, the groundwater tables in the three areas rise with respect to time.

2. The groundwater level in the nonsaline area is lower than those of the intermediate and saline areas during the dry season and rises above those of the other two areas again during the wet season. This is due to the operational decisions of pumping groundwater to help satisfy demands during the dry season and diversion of surplus river flows for artificial recharge during the wet season.

3. The rising trend of the water level can probably be explained by the fact that the total net amount of recharge to the groundwater aquifer during the wet season is greater than the total amount of water pumped from the aquifer.

4. The water table in the nonsaline area during the month of August of the third year reached the allowable limit of 10 feet below the ground surface. This explains why it was necessary to pump water from the nonsaline aquifer during this high flow month as depicted in figures 6.2 and 6.3.

5. The water table in the intermediate and saline areas did not reach the highest allowable limit due to the imposed water level constraints between zones to prevent salt water movement.

Sensitivity Studies.

The cost coefficients and input parameters are seldom known with complete certainty or to the desired degree of precision. It is necessary to perform a sensitivity analysis to examine the effects on the optimal solution by changing values for certain coefficients or parameters. If sensitivity studies show that the optimal solution is sensitive to some of the coefficients or parameters, special care should be taken in estimating these values. A special effort must be concentrated on collecting and analyzing data for estimating those coefficients and parameters to which the optimal solution is most sensitive.

The optimal solution obtained when the cropping intensity in the saline area was reduced from 150 to 100 percent (see Appendix E) are similar to those discussed on figure 6.2 to 6.6. The design capacity at watercourse outlets of the nonsaline area increases from 5,156 to 6,050 cfs while tubewell installed capacity decreases from 6,058 to 5,109 cfs. In the intermediate area design capacity at heads of watercourses of the saline area reduces from 2,010 to 1,050 cfs due to the reduced cropping intensity and the corresponding decrease in demand. Drainage tubewell capacity in the saline area reduces from 1,045 to 705 cfs due to the reduced surface water recharge to this area. The overall cost reduces to RS 144 million compared to RS 166 million for 150% cropping intensity.

When the storage coefficient changes from 0.16 to 0.25 , there are also some significant changes. The design capacity at heads of watercourses of the nonsaline area is 6,198 cfs, an increase of about 20% , and the tubewell installed capacity reduced from 6,058 to 5,428 cfs. There is no change on the design capacity at heads of watercourses of the intermediate area, while tubewell installed capacity in the same area increases from 1,650 to 2,259 cfs. Drainage tubewell installed capacity in the saline area increases to 1,342 crs, an increase of about 30% . Total cost of this case increases to RS 175 million .

Discussion of Results.

The existing canal capacities in each of the three zones need to be enlarged in order to divert more surface water to satisfy water demands.

This is different from suggestions made by some of the consultants to Pakistan such as Revell's group (1964) and T&K (1967). They suggested only the expansion of the canal network in the saline zone to divert more surface water to the area due to the unfeasibility of using its underlain saline groundwater. Remodeling ratios at the head of the main canal and at heads of watercourses are greater than 1.6 . This suggests that a second new canal built along the existing one is more feasible according to the cost curve shown in figure 4.4.

The optimal operational decisions allocating the available surface and ground water to the three areas shows that water requirements can always be satisfied. Whenever the available surface water is less than the total water requirements, the full amount of available surface water is diverted and the deficiency is supplied by groundwater.

There is no water shortage for a cropping intensity of 150% in all three groundwater quality zones in this selected study. There is a possibility of increasing cropping intensity above 150% in the model area.

Artificial recharge is necessary to conserve some of the surplus surface water during some of the wet season months when available surface water is greater than water demands. Because of limited canal capacity in the nonsaline area and the waterlogging constraints, it is not always feasible to recharge surplus surface water to the aquifer.

In general, groundwater pumped in each of the three areas will be utilized in its own area. The transfer of groundwater from the relative fresh water area to the more saline area is limited to the amount required to satisfy the salt balance constraint. It will be more feasible to transfer surface water than pumped water to the more saline groundwater areas for satisfying its water requirements.

There is a trend for water levels to rise under the optimal conjunctive use policy for the selected study as shown on figure 6.6. A longer period of operational study must be evaluated to assess this trend. It is expected that the water levels will continuously increase until the upper limit imposed by the waterlogging constraint is reached and then possibly fluctuates below this upper limit.

Water demands in the saline zone will be supplied from surface water except for the amount of water pumped from skimming wells and that part transferred from the pumped groundwater in the nonsaline and intermediate zones.

The result of reducing the cropping intensity in the saline area from 150 to 100 percent indicates that the capacities of the canal distribution system in the nonsaline and intermediate zones can be increased while tubewell installation capacities can be decreased. This is because more surface water can be allocated to the nonsaline and intermediate areas due to the reduced water requirement in the saline area. The total cost of capital and operation in this case as about 13% less than that for 150% cropping intensity in the saline zone. Economic studies should be conducted to determine which one is more feasible.

The results from different surface water availability studies indicate that in periods of low river flows, higher capacities of the canal distribution system and tubewell installation in the nonsaline area were desirable. This probably is due to the need to pump more groundwater to meet the demands. On the other hand, a larger canal capacity would be required to divert the water for artificial recharge to conserve the surplus water whenever it is available.

The value of the storage coefficients have a great effect on the optimal design of the conjunctive use system for both ground and surface water. Careful evaluation of the storage coefficient by further investigation is needed.

The results presented here are example results only. There was little contact with Pakistan experts and reliable results must await the review of persons more closely associated with the situation in Pakistan.

CHAPTER VII

CONCLUSIONS AND SUGGESTIONS FOR FUTURE STUDIES

Conclusions

In this research study, the irrigation system of the Indus Basin, Pakistan and its problems were outlined. The complexity of the whole Basin irrigation system required decomposing the system into several subsystems which are essentially the canal commanded areas. A physical model for the canal subsystem was defined and a mathematical model was developed for the model area. The objective was to minimize the total cost of construction, operation and maintenance of the canal subsystem under the conjunctive use of groundwater and surface water delivered in an optimal pattern. Following are the major conclusions drawn from this study.

1. It was shown that under the conjunctive use policy, water requirements for a cropping intensity of 150 percent for the total model area can be met even under low flow situations. This would enable the present water deficient irrigation system to become, to a large extent, an "on demand" system, permitting higher cropping intensity of at least 150 percent on all the existing canal commanded areas.
2. In general, it is necessary to enlarge the existing design canal capacities in three different groundwater quality areas in order to divert more water supplies from river flows as available.
3. The increased canal capacity in the nonsaline area can be used during surplus surface water periods for artificial recharge for storage in the groundwater aquifer. However, the amount of recharge will be limited by the waterlogging constraint.

4. The minimum cost conjunctive use policy indicates that surface water should be transferred to the more saline groundwater zones rather than import relatively fresh groundwater from the non-saline or intermediate zones. Generally, the groundwater that is pumped would be used within the same zone except for that amount which must be pumped and exported to maintain the salt balance requirements.

5. The results showed that the design of the conjunctive use system is most critical during years of low flows of river water and thus must be used in the optimal design of the irrigation system.

6. The aquifer storage coefficient (or drainage yield coefficient) is a significant factor in designing the optimal system due to the sensitivity of pumping costs, crop response to groundwater level and the great fluctuation of groundwater level when the storage coefficient is small.

7. Skimming well pumping is a promising measure to utilize the relatively fresh water situated on top of very saline water. This ground water is essentially recharged from surface water delivered through the canal system. In addition to the increase of irrigation water supply, skimming well pumping helps to lower the water table in the saline zone and reduce the hazard of lateral salt-water movement and contamination of non-saline areas.

8. In solving a problem of a complex water resources system, engineering judgement and mathematical manipulation are equally important. Intuitive engineering judgements are necessary in order to simplify a complicated system to a manageable system. Mathematical manipulation enables one to resolve an otherwise complex

problem into subproblems with smaller dimensions which are more amendable to solution.

9. The mathematical model for the Lower Jhelum canal command in this study can be applied to other canal commanded areas in the Indus Basin, Pakistan since most of them have more or less similar properties. And it is hoped that through the coordination of all subsystems, an overall system optimization of the Indus Basin under the conjunctive use of groundwater and surface water can be achieved.

10. The mathematical model developed in this study can also be applied to other areas in the world with similar groundwater salinity problems.

Suggestions for Future Studies

1. The mathematical model developed in this study only includes the conjunctive use system of groundwater and surface water for the separate canal command areas. Further efforts are necessary to combine all the canal commanded subsystems and surface reservoir subsystems for overall system optimization.

2. The surplus surface water in a canal commanded area can be exported to other areas that require more surface water. This also calls for the need of overall Basin system optimization.

3. The water level tends to rise up to the limit of water logging constraints under the optimal conjunctive use policy from the results of a three year study. A longer period of operational study should be evaluated to assess this trend.

4. It was assumed in this study that on the average, a four cfs well in the nonsaline area with fresh water thickness greater than 500 feet will be safe without salt-water coning contamination.

In the intermediate zone, the four cfs well can also be used, but the pumped water must be mixed with surface water before it is applied for irrigation. In the saline area, the 0.25 cfs skimming well can be used with relative fresh water layer of 100 feet thickness. Details of the descriptions and assumptions were presented in Appendix C. Further investigations on the groundwater quality situations and experimental studies will be necessary to assess the adoption of these well sizes.

5. The proposed measure for preventing lateral salt water contamination in this research is to control the relative water levels in three respective water quality areas within some limits by adjusting pumping in three different areas, diversion for recharge in the nonsaline zone whenever surplus water is available and pumping for drainage in the saline area. From the long term salt balance and groundwater utilization view points, this policy might be suitable. The water quality in the saline area could possibly be improved gradually through drainage of more saline water and eventually eliminating the hazard of salt-water contamination due to lateral movement. Experimental and numerical studies will be necessary to determine these limits of allowable difference of water level in three respective areas.

6. Other possible alternatives such as construction of pumping troughs, recharge ridges and lining of canal distribution systems in the saline area should be evaluated and compared to the alternative adopted in this study.

7. Certain kinds of composite wells might be adopted to prevent salt water coning contamination. The composite well will include

a deep well portion which will pump the relatively saline water for export to other areas or drainage to the river, and a shallow well portion which will pump essentially the upper relatively fresh water for irrigation use. Experimental studies should be carried out to assess their feasibility. Several methods of artificial recharge are available which include the selection of some possible sites for percolation basins, induced recharge by locating more pumping wells near the river flood plains and installation of recharge wells. Studies are needed to determine the most feasible and economic technique to recharge the aquifer.

8. Although considerable information is available on groundwater quality in Pakistan, further information is needed to define the areal and vertical groundwater quality distributions.

9. The water quality criteria for irrigation and mixing criteria of different quality have been tentatively adopted, but much more detailed research is needed in order to determine the optimum mixtures so the groundwater can be most efficiently utilized.

10. Extensive field investigations on aquifer characteristics, particularly storage coefficients which affect water level fluctuation, should be carried out.

11. The proposed cropping pattern and cropping intensity in this study was assumed optimally at a level of 150 percent. Another study for determining optimal cropping pattern and intensity should be conducted in which the objective is to maximize the net return of the agricultural output, subject to limitations on the available water and lands.

12. The changes of sediment transport and flow characteristics due to canal enlargement need to be studied to explore control measures for preventing sediment deposition during low flow seasons. The extra cost involved in sediment control should be included in the cost of canal remodeling.

13. Power generation from surface reservoirs should also be included in terms of providing part or all of the power needed for tubewell pumping so that the power needed from other sources can be reduced. Flood control aspect of aquifer storage should also be included in the study.

14. The economic aspect of the model can also be extended to add sets of constraints on the availability of fertilizer, labor, capital, and so forth during each operating period.

15. It was assumed in this study that 10 percent of the pumped groundwater must be exported out of the area to preserve the long term salt balance. Further studies should be made to assess this value.

16. There are other alternate ways of exporting the saline water out of the area such as diversion to an evaporation lake, etc.. Studies should be made on all the possible salt water exporting alternatives and their feasibilities.

17. Numerical models and physical models should be used to study the complicated nature of lateral salt water movement, salt water coning under various depths of fresh water and functions of pumping trough and recharge barrier.

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APPENDIX A

A. LIST OF SYMBOLS

B. GLOSSARY AND ABBREVIATIONS

A. LIST OF SYMBOLS

Symbol	Explanation
AR1(k)	Delivery rate at the head of the main canal for artificial recharge to the nonsaline area in period k (cfs).
b_ℓ	Vector representing existing capacities of the design capacity variables.
b_u	Vector of the upper limits of the design capacity variables.
CAR	Unit cost of operation and maintenance for artificial recharge (RS/cfs).
CD	Row vector of cost coefficients for the design variables.
CLFD	Seepage loss factor from the head of the main canal to heads of distributaries.
CLHW	Seepage loss factor from the head of the main canal to heads of watercourses.
CO	Row vector of cost coefficients for operational variables over the chosen time period (k=1,2,...n).
COS	Unit cost of operation and maintenance for the canal distribution system.
CST	Unit cost of water shortage (RS/AF/year), RS is short for Pakistani money; Rupees.
CTI1,CTI2,CTI3	Unit cost of energy for pumping for area 1, 2, 3.
$CW_i(k)$	Delivery rate of surface water at heads of watercourses for area i (i=1,2,3) during period k (cfs).
DCH	Design capacity at the head of the main canal (cfs).
DCW_i	The lumped capacity at heads of watercourses for area i (i=1,2,3) (cfs).
$DGW_i(k)$	Depth to groundwater table from the ground surface in area i (i=1,2,3) during period k (feet).
DI_i	Initial depth to groundwater table from the ground surface in area i (i=1,2,3) (feet).
DIP_i	Total tubewell installation capacities in area i (i=1,2,3) (cfs).
DISK	Skimming well installation capacity in area 3 (cfs).
DMA_i	Maximum allowable depth to groundwater table from the ground surface in area i (i=1,2,3) (feet).
$DMCW_i$	Maximum design capacity at heads of watercourses for area i (i=1,2,3) (cfs).
DMI_i	Minimum allowable depth to groundwater table from the ground surface in area i (i=1,2,3) (feet).
$DMIP_i$	Maximum design capacity of tubewells in area i (i=1,2,3) (cfs).
$DT_i(k)$	Net change of water table in area i (i=1,2,3) during period k (feet).
$EDCW_i$	Existing design capacity at heads of watercourses in area i (i=1,2,3) for surface water delivery (cfs).
$EDIP_i$	Existing tubewell installed capacity in area i (i=1,2,3) (cfs).
H_1,H_2,H_3	Dynamic heads including drawdown and head losses for tubewell pumping for area 1, 2, 3 .
n	Number of subperiods.
$PDC_i(k)$	Decline of water table due to pumping during period k for area i (i=1,2,3).
PER1, PER2	Fractions of water pumped from area 1 and 2 that need to be exported.
$P_{ii}(k)$	Rates of tubewell pumping in area i delivered to heads of watercourses in area i during period k (i=1,2) (cfs).
$P_{ij}(k)$	Rate of tubewell pumping in area i delivered to heads of distributaries in area j during period k (i=2,3;j=i+1,3)

Symbol	Explanation
	(cfs).
PD3(k)	Rate of pumping for drainage in area 3 during period k (cfs).
P33(k)	Rate of skimming well pumping in area 3 during period k (cfs).
$RECH_i(k)$	Rise of water table due to recharge during period k for area i (i=1,2,3).
RIN(k)	River flow allocated to model area during period k (cfs).
$RMIX_i$	Mixing ratio of the pumped groundwater in area i (i=2,3) with respect to the total surface and groundwater from area 1.
$RT_{12}(k), RT_{23}(k)$	Relative water table constraints between areas 1 and 2, and areas 2 and 3 (feet).
$SHRT_i(k)$	Water shortage in area i during period k (i=1,2,3) (cfs).
$SLMIT_i(k)$	Limit of water shortage in area i (i=1,2,3) during period k.
U	Unit charge of power for pumping per acre foot of water per foot of lift.
VP_k	Total volume of water pumped during period k (AF).
$WR_i(k)$	Water requirements at heads of watercourses for area i during period k (i=1,2,3) (cfs).
WRM_i	Maximum water requirement during a subperiod for area i (i=1,2,3) (cfs).
x	Column vector of operational variables.
y	Column vector of design capacity variables.

B. GLOSSARY AND ABBREVIATIONS

AF

Acre feet.

Barani

The agricultural practice which relies upon rainfall alone for crop water requirements.

Barrage

A low dam or weir equipped with a series of gates to regulate the water surface level upstream from the weir.

Branch

A large irrigation channel with a capacity generally in the range of 3,000 to 6,000 cusecs taking off from a main canal.

Canal (or Main Canal)

A channel for conveyance of water, generally in Pakistan referring to a large channel which delivers water from a river to branches, lesser channels, and having a capacity of 5,000 to 15,000 cusecs or more.

Consumptive use

The amount of water lost from a given area during a specified time by transpiration from vegetation and by evaporation from water and plant surfaces and from the adjacent soil.

Crop water requirement

The total quantity of water required by a crop for normal growth under field conditions.

Cropping intensity

The cropped area expressed as a percentage of the CCA.

Cropping pattern

The sequence of crops grown in any given area during a single year and the proportion of cropland devoted to each crop during the year.

Culturable area (CA)

That portion of the gross area which is cultivable.

Culturable commanded area or canal commanded area (CCA)

The culturable area beneath a canal system which can be irrigated by gravity flow from the canal system.

Delta

The depth of irrigation water applied to cropped land.

Distributary

An irrigation channel of intermediate size, generally with a capacity in the range of 100 to 1,000 cusecs, and usually taking off from a branch or main canal.

Doab

The land between two river tributaries.

Gross area (GA)

The entire area within the irrigation project boundaries.

HARZA

Harza Engineering Company International of USA, General Consultants to WAPDA.

Headworks

The structures provided at the intake of a main canal for controlling the flow of water into the canal.

HUNTINGS

Hunting Technical Services Limited of United Kingdom Consultants to WAPDA for Lower Indus Basin area.

IACA

Irrigation and Agricultural Consultants Association, Consultants to International Bank for Reconstruction and Development - World Bank for the Indus Special Study.

Inundation canal

A canal which is dependent upon the level of water in the river for its supply.

Irrigation water requirement

The quantity of water required for normal crop growth and leaching minus effective precipitation. The irrigation water requirement includes losses from the point of reference to the crop.

Kharif

The summer irrigation season; the six months from April 15 to October 15. Also used to denote summer crops and cropping season.

Kharif:Rabi ratio

The ratio of the total areas cropped in the two cropping seasons.

Leaching requirement

The fraction of the water entering the soil that must pass through the root zone in order to prevent soil salinity from exceeding a specified value under long term average or steady state conditions.

LJC

Lower Jhelum Canal.

MAF

Million acre feet.

Minor

A small irrigation channel, generally with a capacity of 10 to 300 cusecs, taking off from a distributary.

Monsoon

The rainy season associated with the southwest monsoon.

Nonperennial Canal

Irrigation channel which normally flows during the summer (Kharif) period but may carry intermittent supplies during other periods.

Perennial canal

An irrigation channel which normally carries water throughout the year.

Persian wheel

A dug well equipped with an endless chain of buckets or runs for lifting the water to the surface; usually powered by bullocks or camels.

Rabi

The winter irrigation season; the six months from October 15 to April 15. Also used to denote winter crops and cropping seasons.

Residual-sodium-carbonate (RSC)

A term used to denote the amount of carbonate plus bicarbonate anions remaining in an irrigation water after deduction of an amount equivalent to the concentrations of calcium and magnesium.

RS

Pakistani money; Rupees.

SCARP

Salinity Control and Reclamation Program.

Sodium-adsorption-ratio (SAR)

A ratio used to express the alkali hazard of irrigation waters and soil solutions; also a measure of the relative activity of sodium ions in soil solutions:

$$SAR = \frac{Na^{+}}{\sqrt{\frac{Ca^{++}+Mg^{++}}{2}}} \quad .$$

Specific yield

The ratio of the volume of water that will drain under gravity from saturated rock or soil mass to the volume of the mass.

Storage coefficient

The volume of water that an aquifer releases from or takes into storage per unit surface area of aquifer per unit change in head normal to that surface. For an unconfined aquifer the storage coefficient is identical to the specific yield.

T&K

Tipton and Kalmbach, Inc., WAPDA consultants for the Indus Basin North Zone.

Total dissolved solids (TDS)

The concentration of dissolved minerals in ppm obtained by evaporating to dryness a filtered sample of water. Commonly referred to as "dissolved solids."

Tubewell

A drilled well, cased and screened. SCARP project tubewells are gravel-packed.

USAID

United States Agency for International Development.

USICA

United States International Cooperational Administration.

WAPDA

Water and Power Development Authority, Pakistan, Lahore.

WASID

Water and Soil Investigation Division, Pakistan.

Watercourse

An irrigation channel taking off from a distributary, minor or sub-minor; used to carry water to farm fields.

APPENDIX B

REVIEW OF FLEXIBLE TOLERANCE METHOD

The flexible tolerance method by Paviani and Himmelblau (1969) is based on the unconstrained flexible polygon search technique of Nelder and Mead (1964), and combines all constraints into a single tolerence in the process of search. New points are searched iteratively to improve the objective function and satisfy the tolerance criterion.

1. The Original Problem.

The original problem can be stated in general as follows:

$$\text{Minimize} \quad y = f(x) \qquad x \in E^n \tag{B-1}$$

Subject to

$$h_i(x) = 0 \quad , \quad i = 1,2,\ldots,m \tag{B-2}$$

$$g_i(x) \geq 0 \quad , \quad i = m+1,\ldots,p \quad . \tag{B-3}$$

where $f(x)$, $h_i(x)$ and $g_i(x)$ may be linear and/or nonlinear function, and x is the vector of decision variables.

2. The Modified Problem.

All the violated constraints of equations B-2 and B-3 are combined into one gross inequality. A certain tolerance criterion is set up to limit the value of this gross inequality. The problem at any stage k of the search becomes as follows:

$$\text{Minimize} \quad y = f(x) \qquad x \in E^n$$

Subject to

$$\Phi^{(k)} - T(x) \geq 0 \quad . \tag{B-4}$$

and

$$T(x) = \left[\sum_{i=1}^{n} h_i^2(x) + \sum_{i=m+1}^{P} u_i g_i^2(x) \right]^{\frac{1}{2}} \tag{B-5}$$

where $u_i = 0 \quad , \quad \text{if } g_i(x) \geq 0$

$$u_i = 1 \quad , \quad \text{if } g_i(x) \leq 0 \qquad \text{(B-6)}$$

and $\Phi^{(k)}$ is the tolerance criterion at stage k which is selected as a positive decreasing function of the vertices of the flexible polyhedron in E^n, i.e.,

$$\Phi^{(k)} = \Phi^{(k)} [x_1^{(k)}, x_2^{(k)}, \ldots, x_n^{(k)}] \quad , \qquad \text{(B-7)}$$

and
$$\Phi^{(k)} \leq \Phi^{(k-1)} \leq \ldots \leq \Phi^{(1)} \leq \Phi^{(0)} \quad . \qquad \text{(B-8)}$$

3. The Tolerance Criterion.

The tolerance criterion in this algorithm is defined as follows:

$$\Phi^{(k)} = \min. [\Phi^{(k-1)}, \frac{m+1}{r+1} \sum_{i=1}^{r+1} || x_i^{(k)} - x_{r+2}^{(k)} ||]$$

and

$$\Phi^{(0)} = 2(m+1)t \qquad \text{(B-9)}$$

where t is the size of initial polyhedron; $x_i^{(k)}$ is the ith vertex of polyhedron in E^n; $r = n-m$ is the number of degrees of freedom; and, $x_{r+2}^{(k)}$ is the centroid of the polyhedron.

The second term in the bracket of equation B-9 represents the average distance from each $x_i^{(k)}$, $i=1,\ldots,r+1$, to the centroid $x_{r+2}^{(k)}$ of the polyhedron in E^n. The term,

$$\sum_{i=1}^{r+1} || x_i^{(k)} - x_{r+2}^{(k)} || = [\sum_{i=1}^{r+1} \sum_{j=1}^{n} (x_{ij}^{(k)} - x_{r+2,j}^{(k)})^2]^{\frac{1}{2}} \quad ,$$

is always positive. So $\Phi^{(k)}$ is a strictly positive decreasing function of $x_i^{(k)}$, $(i=1,2,\ldots,r+2)$ as presented in equation B-8.

After any change in $x_i^{(k)}$, equation B-4 combined with B-6 is checked regardless of any possible improvement in the value of the objective function. If it is not satisfied, the sum of the squared values of all the violated constraints is minimized by an unconstrained minimization procedure of Nelder and Mead (1964) until the square root of this sum is less than or equal to $\Phi^{(k)}$. Then the value of the objective function is computed to determine whether the new point is improved or not.

4. <u>The Unconstrained Flexible Polygon Search.</u>

The unconstrained flexible polygon search of Nelder and Mead (1964) minimizes a function of n independent variables using $n+1$ vertices of a flexible polyhedron in E^n . The vertex that yields the highest value of the objective function, $f(x)$, is projected through the center of gravity of the remaining vectors and is successively replaced by better points in the process of search until the minimum of $f(x)$ is found.

Let $x_i^{(k)} = [\, x_{i1}^{(k)} \,, \ldots , \, x_{in}^{(k)} \,]$, $i=1,2,\ldots,n+1$ be the point (vertex) for which the value of the objective function is $f(x^{(k)})$ at kth stage. And define

$$f(x_h^{(k)}) = \max\ [f(x_i^{(k)}) \,, \ldots , \, f(x_n^{(k)})] \text{ for which } x_i^{(k)} = x_h^{(k)}$$
$$f(x_\ell^{(k)}) = \min\ [f(x_i^{(k)}) \,, \ldots , \, f(x_n^{(k)})] \text{ for which } x_i^{(k)} = x_\ell^{(k)} \;.$$

Let $x_{n+2}^{(k)}$ be the centroid of all the vertices with $i \neq h$, the coordinates of which are given by

$$x_{n+2}^{(k)} = (1/n) \cdot [\, (\sum_{i=1}^{n+1} x_{ij}^{(k)}) - x_{hj}^{(k)} \,] \;, \text{ for } j=1,2,\ldots,n \quad . \tag{B-10}$$

Given the initial x vector for $n+1$ vertices, the search procedure involves four operations within stage k .

a. Reflection of $x_h^{(k)}$ through the centroid,

$$x_{n+3}^{(k)} = x_{n+2}^{(k)} + \alpha(x_{n+2}^{(k)} - x_h^{(k)}) \tag{B-11}$$

where α is the reflection coefficient of a positive value.

b. Expansion of $(x_{n+3}^{(k)} - x_{n+2}^{(k)})$, if reflection has produced a new minimum (i.e., if $f(x_{n+3}^{(k)}) \leq f(x_{\ell}^{(k)})$.

$$x_{n+4}^{(k)} = x_{n+2}^{(k)} + \gamma(x_{n+3}^{(k)} - x_{n+2}^{(k)}) \tag{B-12}$$

where γ is the expansion coefficient of a value greater than unity.

c. Contraction of $(x_h^{(k)} - x_{n+2}^{(k)})$, if reflection results in $f(x_{n+3}^{(k)}) > f(x_j^{(k)})$, for all $i=h$,

$$x_{n+5}^{(k)} = x_{n+2}^{(k)} + \beta(x_h^{(k)} - x_{n+2}^{(k)}) \tag{B-13}$$

where β is the contraction coefficient of a value between 0 and 1 .

d. Reduction or overall contraction of $x_i^{(k)}$ on $x_{\ell}^{(k)}$ if contraction failed to produce a better point than $x_h^{(k)}$

$$x_i^{(k)} = x_{\ell}^{(k)} + 0.5(x_i^{(k)} - x_{\ell}^{(k)}) , \quad i=1,2,\ldots,n+1 \quad . \tag{B-14}$$

The values of $\alpha=1$, $\beta=0.5$, and $\gamma=2$ have been recommended for the unconstrained minimization problem.

The search is terminated if

$$\left\{ \frac{1}{n+1} \sum_{i=1}^{n+1} [f(x_i^{(k)}) - f(x_{n+2}^{(k)})]^2 \right\}^{\frac{1}{2}} \leq \varepsilon \tag{B-15}$$

where ε is an arbitrarily small value.

5. <u>Initiation of Search.</u>

The (n+1) vertices of the initial polyhedron in E^n are found from

$$x_i^{(0)} = x_o^{(0)} + D_i \quad , \quad i=1,\ldots,n+1 \tag{B-18}$$

where x_o is the starting vector and the elements of D_i are the elements of the ith row of a $(n+1) \cdot n$ matrix. The row of this matrix determines the n coordinates of each of the sought (n+1) vectors:

$$D = \begin{bmatrix} 0 & 0 & 0 & \cdots & 0 \\ u & v & v & \cdots & v \\ v & u & v & \cdots & v \\ v & v & u & \cdots & v \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ v & v & v & \cdots & u \end{bmatrix} \tag{B-17}$$

where

$$u = (t/n\sqrt{2})(\sqrt{n+1} + n-1) \tag{B-18}$$

$$v = (t/n\sqrt{2})(\sqrt{n+1} - 1) \tag{B-19}$$

and t is the distance between two vertices. Generally t is chosen according to the following equation:

$$t = \min \{ [(0.2/n) \cdot \sum_{i=1}^{n} L_i],\ L_1, L_2, \ldots, L_n \} \tag{B-20}$$

where L_i is the difference between the upper and lower bounds of the independent variable x_i. If the upper and lower bounds of x are not known, any reasonable guess for t is acceptable.

6. <u>Termination of Search.</u>

The vertices of the flexible polyhedron are drawn near and near to that of the minimum of $f(x)$ in the course of search, and in the final

$$\lim_{x \to x^*} \Phi^{(k)} = 0 \quad . \tag{B-21}$$

For practical purposes, it is sufficient to carry on the search until $\Phi^{(k)}$ becomes smaller than a selected small value ε, and the following inequalities are satisfied:

$$f(x) \leq f(x^* \pm \varepsilon) \tag{B-22}$$

$$T(x) \leq \varepsilon \quad . \tag{B-23}$$

7. Summary of the Flexible Tolerance Search Procedures:

1. Assume x_o and find t described in item 5.

2. Check the sum of the squares of the values of the violated constraints, if any; if the square root of this sum is greater than the current $\Phi^{(k)}$, minimize the sum by means of the unconstrained minimization algorithm until a vector x is found.

3. Compute $\Phi^{(k)}$ by equation B-9.

4. Use the unconstrained flexible polygon search described in item 4 to find a new point with better objective function within the tolerance criterion $\Phi^{(k)}$.

5. Check convergence criterion.

If $\Phi^{(k)} \leq \varepsilon$, the search is terminated.

If $\Phi^{(k)} > \varepsilon$, return to step 3 to start the $(k+1)$th stage.

Figure B-1 is the flow chart describing the above procedures which is adopted from Himmelblau (1972).

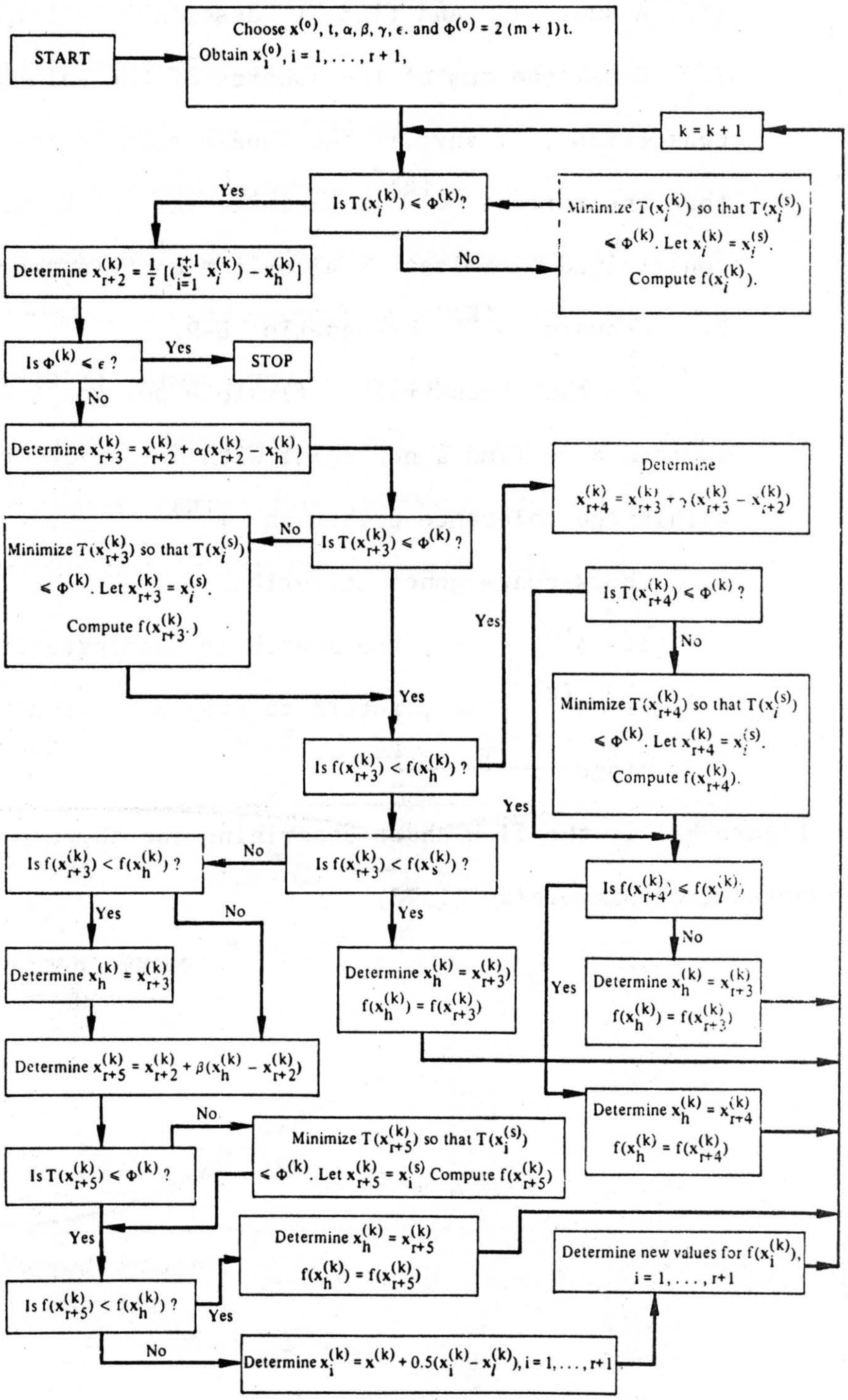

Figure B-1. Flow diagram of the flexible tolerance algorithm (Adopted from Himmelblau, 1972).

APPENDIX C

DEVELOPMENT AND ANALYSIS PERFORMED TO FORMULATE THE PROBLEM

Salt Water Coning Beneath Fresh Water Wells in Pakistan.

The Indus Basin of Pakistan consists of a vast alluvial plain underlain by an unconfined aquifer in which relatively fresh water lies above more saline groundwater. When a well, partially penetrating the upper fresh water layer, is pumped, there is a tendency for an upward movement of salt water called upconing. At some critical point depending on the amount of discharge and length of pumping period, the salt water begins to move into the well and the discharge water becomes a mixture of the fresh and salt water, which might not be acceptable for irrigation.

Review of the Theory on Salt Water Upconing.

A. Steady State Salt Water Upconing.

For the sake of simplicity, the following assumptions had been made in estimating the height of salt water upconing beneath a well:

1. The fresh water and salt water are separated by an abrupt interface and have distinct and uniform densities on both sides.
2. The aquifer is homogeneous and isotropic and there is no pressure discontinuity across the interface.
3. The flow is defined by Darcy's law.

The potential within each separated fluid can then be defined as

$$\Phi_f = \frac{P_f}{\rho_f g} + z \qquad \text{(C-1)}$$

$$\Phi_s = \frac{P_s}{\rho_s g} + z \quad . \qquad \text{(C-2)}$$

Where Φ is the potential, P is the pressure, ρ is the density, subscripts f and s denote the fresh water and salt water and z is the elevation at the point of interest measured above some chosen datum. The problem can further be reduced to solve the linear Laplace equation in the fresh water layer subject to the boundary conditions after combining with the continuity equation. Detailed derivation of the equation can be obtained from other references such as Muskat (1965) and will not be shown here. One which must be considered is the fluid pressures at the interface. Since it is assumed that no pressure discontinuity exists across the interface, the pressure at all points on the interface must be the same in both fluids at that point.

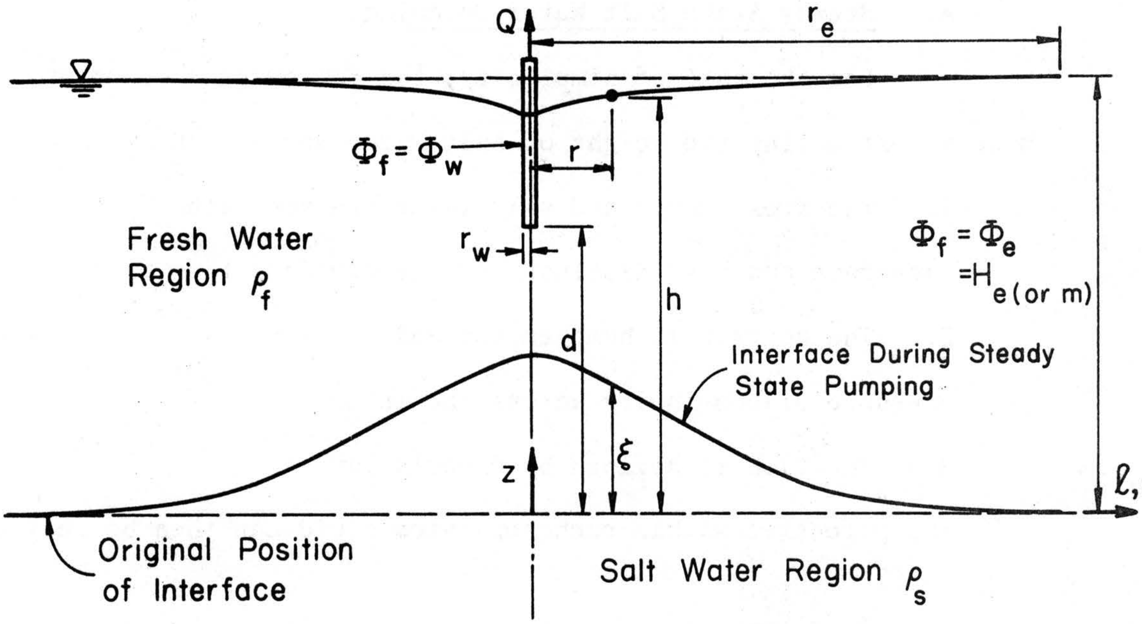

Figure C-1. Salt water coning below a fresh water well.

Based on equations C-1 and C-2, the interface can be derived as:

$$\xi = \frac{\rho_s}{\Delta\rho} \cdot \Phi_s^i - \frac{\rho_f}{\Delta\rho} \cdot \Phi_f^i \tag{C-3}$$

where ξ is the vertical coordinate of the interface at any point, $\Delta\rho$ is the difference between ρ_s and ρ_f and superscript i denotes the value at interface. After combining the above equation with flux defined by Darcy's law, then the slope of the interface is

$$\text{Sin } \theta = \frac{\partial\xi}{\partial\ell} = \frac{\rho_f}{\Delta\rho} \cdot \frac{q_f^i}{k_f} - \frac{\rho_s}{\Delta\rho} \cdot \frac{q_s^i}{k_s} \tag{C-4}$$

where q_f^i , q_s^i are the velocities tangent to the interface, k_f , k_s are horizontal permeability of the aquifer in the region of each fluid, and θ is the angle the interface makes with the horizontal.

For the case when there is only flow in the fresh water region, the potential is constant throughout the salt water zone. Equation C-4 reduces to

$$\text{Sin } \theta = \frac{\rho_f}{\Delta\rho} \cdot \frac{q_f^i}{k_f} \quad . \tag{C-5}$$

For a partially penetrating well when only fresh water is pumped from the well under steady state conditions as illustrated in figure C-1 with the thickness of fresh water equal to H_e, and taking the original interface as a datum, the elevation of the interface at a distance r from the well center is

$$\xi_r = \frac{\rho_f}{\Delta\rho} [\Phi_e - \Phi^i_{\text{at } r}] \tag{C-6}$$

where Φ_e is the potential along any vertical line at a distance greater than the well influence radius r_e . At $r=0$, the elevation of the apex of the salt water cone is

$$\xi_{r=0} = \frac{\rho_f}{\Delta\rho} [\Phi_e - \Phi_f^i \text{ (at } r=0)] \quad . \tag{C-7}$$

The classical Ghyben-Herzberg relation stated that

$$\xi_w = \frac{\rho_f}{\Delta\rho} [\Phi_e - \Phi_w] \quad . \tag{C-8}$$

Equation C-7 will be identical to equation C-8 provided that the potential of the interface below the well is equal to the potential in the well. Such conditions exist only if there is no vertical flow; i.e. all the flow is horizontal. But actually the potential at the interface directly beneath the well must be greater than the potential in the well since there is flow into the well from below. Therefore, the use of Ghyben-Herzberg relation essentially neglects the vertical component of the flow and overestimates the height of salt water cone for the same drawdown.

Muskat and Wyckoff (1935), Wang (1965) and Dagan and Bear (1968), et al. had tried to solve the upconing problem based on the above theory with various approaches and assumptions. Recently, McWhorter (1972) also tried to solve the salt water upconing problem based on the above theory and assuming that the vertical component of flow is small. Then the velocity q_f is not a function of vertical coordinate z, and for sufficient flat slopes of the interfaces and a static salt water region equation C-4 can be approximated as follows without superscript i .

$$\frac{d\xi}{dr} = \frac{\rho_f}{\Delta\rho} \cdot \frac{q_f}{k_f} \quad . \tag{C-9}$$

The well discharge then is

$$Q = q_f A = A \frac{\Delta\rho}{\rho_f} k_f \frac{d\xi}{dr} \tag{C-10}$$

where $A = 2\pi(h - \xi)\, r$

r = radius from the center of the well

h = height of drawdown surface above the original interface at distance r from the center of the well.

Based on the Ghyben-Herzberg relation and the assumption that the vertical component of flow is small

$$A = 2\pi \{m - (\frac{\Delta\rho}{\rho_f} + 1)\cdot\xi\}\cdot r \quad . \qquad \text{(C-11)}$$

By separation of variables, integration and the combination of equation C-10 and C-11,

$$\frac{\xi}{d} = \{ (\frac{1}{1 + \frac{\Delta\rho}{\rho_f}})^2 + \frac{Q\rho_f}{xm^2\Delta\rho k_f} \cdot \frac{\ell n(\frac{r_e}{r})}{(1 + \frac{\Delta\rho}{\rho_f})} \}^{\frac{1}{2}} - \frac{1}{(1 + \frac{\Delta\rho}{\rho_f})} \qquad \text{(C-12)}$$

where d is the height of well bottom above the original interface. The criterion to restrict the value of ξ so that the interface will remain stable can be stated as

$$\xi_{max} \leq f(m-d) \qquad \text{(C-13)}$$

and the value of f must be determined. A value around 0.5 might be reasonable and is assumed for this study. Substituting equation C-13 into equation C-12, the maximum allowable well discharge can be estimated accordingly.

B. Nonsteady State Upconing Beneath Wells.

McWhorter (1972) derived the following approximate differential equation:

$$\frac{\partial^2\psi}{\partial r^2} + \frac{1}{r}\frac{\partial\psi}{\partial r} = \alpha \frac{\partial\psi}{\partial t} \qquad \text{(C-14)}$$

$$\psi = \left(\frac{m}{1 + \frac{\Delta\rho}{\rho_f}} - \xi \right)^2 \tag{C-15}$$

$$\alpha = \rho_f \cdot \frac{s}{(\Delta\rho k_f \sqrt{\overline{\psi}})} \tag{C-16}$$

s is the storage coefficient and $\overline{\psi}$ is an estimated weighted average of ψ .

The above equation is based on the following assumptions:

1. The aquifer is homogeneous and isotropic.
2. The effect of flow in the saline zone on the distribution of head on the interface is not accounted for.
3. The Ghyben-Herzberg relation applies.
4. Taking an estimated weighted average of ψ makes equation C-14 linear.

The boundary and initial conditions are:

Limit
$$r \frac{\partial\psi}{\partial r} = \frac{Q}{\pi \frac{\Delta\rho}{\rho_f} \left(1 + \frac{\Delta\rho}{\rho_f}\right) k_f}$$

$$\psi(\infty,t) = \psi_\infty = \{m/(1 + \Delta\rho/\rho_f)\}^2$$

$$\psi(r,0) = \psi_0 = \{m/(1 + \Delta\rho/\rho_f)\}^2 \quad . \tag{C-17}$$

The solution is

$$\psi = \psi_\infty - [Q/\{2\pi(\Delta\rho/o_f)(1 + \Delta\rho/\rho_f)k_f\}] \cdot w(u)$$

$$w(u) = \int_{\frac{r^2}{4\alpha t}}^{\infty} \frac{e^{-u}}{u} du \quad . \tag{C-18}$$

By superposition, the above equation can be extended to any number of time steps with different pumping rates so that the salt water cone will not exceed the allowable limit.

C. Electric Analog Model for Studying Upconing.

Bennett, et al. (1967) applied the graphical procedure developed by Muskat (1935) to study the salt water coning beneath a steady state fresh water well supplied by uniform areal recharge. They obtained the required potential distribution through an analog model made up of a network of electrical resistances. The model, in addition, was equipped with a system of switches which were used to adjust the lower boundary of the network to simulate the truncation of the fresh water zone by the salt water cone. The highest possible stable position of the salt water cone for different conditions was obtained by trial and error using the adjusting procedure.

D. Physical and Numerical Models.

Sahni (1972) used both physical and numerical models to study the design and phenomenon of coning below a partially penetrating well in the upper fresh water layer. Several graphs were presented in dimensionless coordinates for different well penetrations, and different ratios of the radius of well influence and fresh water thickness. These graphs can be used for rough estimation of maximum allowable well discharge and critical drawdown at a well without causing salt water contamination.

Application of Upconing Theory to Pakistan.

McWhorter's equation for steady upconing beneath a fresh water well was used to calculate the maximum allowable well capacity for different fresh water thicknesses and the necessary fresh water thickness to sustain certain pumping rates. The maximum allowable upconing in applying McWhorter's equation is assumed to be half the distance between well bottom and original interface. The results are shown on tables C.1, C.2 and C.3.

Table C.1. Maximum allowable well capacity (cfs) for different fresh water thickness (horizontal permeability = 0.003 cfs/ft^2 , radius of influence = 0.5 mile , well radius = 1 foot).

Fresh Water Thickness (ft)	Depth of Well = 200 feet Density Difference ($\Delta\rho$) 0.01	0.015	0.02	Depth of Well = 250 feet Density Difference ($\Delta\rho$) 0.01	0.015	0.02
500	2.1	3.1	4.1	1.68	2.5	3.4
600	3.4	5.0	6.7	2.88	4.3	5.8
700	5.0	7.4	9.9	4.38	6.6	8.8
800	6.8	10.3	13.7	6.18	9.1	12.4
900	9.0	13.5	18.1	8.27	12.4	16.6

Table C.2. Minimum freshwater thickness for well. Capacity at 4 cfs . (Horizontal permeability = 0.003 cfs/ft^2, depth of well = 200 feet , radius of influence = 2500 feet , well radius = 1 foot.)

Density Difference $\Delta\rho$ (gm/cm^3)	Freshwater Thickness (ft)
0.01	640
0.015	550
0.02	500
0.025	455

Table C.3. Minimum freshwater thickness for skimming well (horizontal permeability = 0.003 cfs/ft^2 , depth of well = 30 feet, radius of influence - 1500 feet , radius of well = 1 foot).

Density Difference $\Delta\rho$ (gm/cm^3)	Q=0.5 cfs Freshwater Thickness (ft)	Q=0.25 cfs Freshwater Thickness (ft)
0.015	162	107
0.020	143	119
0.025	130	97

Sahni's graphs (1972) developed from his numerical model were used to check the above calculations, and it was found that his maximum allowable discharges were almost double the above figures. The use of the above figures will be conservative and on the safe side.

The electric analog model developed by Bennett, et al. (1967) had also been applied to conditions in the Punjab region of Pakistan. The results indicate that there are good prospects for the development of wells capable of discharging fresh water above a static cone in the underlying salt water. They concluded that in areas where the original thickness of fresh water is appreciable, say 500 feet or more, there whould be little danger of serious contamination in reclamation projects of the type presently under development in the Punjab. Where the fresh water thickness is thin, the concept of skimming wells appear to be reasonable.

McWhorter's unsteady flow equation was also used to check the time at which unstable upconing for a certain pumping rate is reached. Table C.4. shows the results.

Observing from the above calculations, it can be concluded that on the average, a 4 cfs well in the nonsaline zone with freshwater thickness of more than 500 feet will be safe without salt water upconing contamination. In the intermediate zone, a 4 cfs well can also be used, but the pumped water must be mixed with surface water before it is used for irrigation. In the saline zone, a 0.25 cfs skimming well will be used assuming that a relative fresh water layer of 100 to 150 feet exists. This pumped water is assumed to be mixed with surface water before applying it for irrigation.

Table C.4. Time for salt water cone to become unstable (horizontal permeability = 0.003 cfs/ft^2).

Pumping Rate (cfs)	Freshwater Thickness (ft)	Depth of Well (ft)	Density Difference $\Delta\rho(gm/cm^3)$	Days to Reach Unstable Upconing
4	500	200	0.025	110
		250	0.025	13
	600	200	0.015	60
			0.020	7300
		250	0.020	7300
		300	0.020	90
	700	200	0.010	45
			0.015	300
		250	0.010	15
			0.015	300
		300	0.010	4
			0.015	300
	800	250	0.010	300
0.5	200	40	0.015	75
			0.020	180
0.25	100	20	0.025	7
	150	25	0.015	180
		30	0.015	180
	200	40	0.015	180

Alternative Measures for Preventing Lateral Salt Water Movement.

Despite the feasibility and inherent advantage of tubewell development in the Indus Basin, Pakistan, the tubewell pumping will undoubtedly disturb the existing environmental equilibrium and introduce new problems that require solutions. One potential hazard which must be considered in the design and management of the system is that excessive pumping in the fresh water area might produce a reverse gradient from the saline area to the fresh water area and the fresh water will be contaminated gradually.

A. General Measures for Preventing Salt Water Movement.

Many kinds of measures are available to control the salt water lateral movement. The most widely used measures applicable to the Indus Basin, Pakistan are as follows:

1. Modification of pumping - This is done by rearranging the pumping pattern to pump more water near the source line such as the river to induce recharge and pump less near the saline zone. It can also be done by reducing pumping in the nonsaline zone and pump for drainage in the saline zone to maintain a gradient of ground water movement from the nonsaline zone to the saline zone.

Either reduction or rearrangement of pumping, of course, might not allow the full development and utilization of the available ground water storage capacity. Pumping for drainage in the saline zone will allow more extraction of fresh groundwater in the nonsaline zone, but will result in an increased drainage cost in the saline zone to prevent salt water movement and contamination.

2. Artificial recharge in the nonsaline zone - Whenever there is excess surface water available during the summer season, the

additional water can be diverted to the nonsaline zone for use as artificial recharge to the aquifer. In this case the aquifer in the nonsaline zone is functioning as a storage reservoir. It has the advantage of storing and controlling the excess river flow for later withdrawal by wells as needed. For an unconfined aquifer, recharge by spreading is relatively inexpensive and technically feasible. The possible limitations of the artificial recharge will depend on the surplus water available, design capacity of the conveyance system in the nonsaline zone and the available aquifer space to store the additional river flow.

3. Pumping Trough - If a line of wells were constructed adjacent to and along the boundary of the relatively fresh and salt water zones, pumping from these wells would form a trough in the ground water table. These control wells must be pumped at rates which will intercept all the salt water moving from the relatively salt water area toward the fresh water area. The necessary pumping rate will depend on the spacing of the wells, gradient created due to overdraft in the relatively fresh water area, the density difference and the permeability of the aquifer.

The water pumped from the control wells is a mixture of salt water and fresh water. The ratio of this mixture will depend on the differential head between the two zones. There is no question that part of the useful groundwater will be wasted.

4. Pressure Ridge - To create a pressure ridge adjacent to and along the boundary of the relatively fresh water and salt water zones is the exact opposite of the pumping trough control measure. In an unconfined aquifer, surface spreading could create a water

ridge to suppress the salt water movement instead of using recharge well. The amount of fresh water required to create the necessary pressure to repel the salt water will depend on the gradient of salt water movement. To maintain a dynamic balance, a small amount of the recharged water would flow to the saline zone.

This method of control has the advantage of not restricting the usable ground water storage capacity in the nonsaline zone but the supplemental water to create the pressure ridge must be available. Due to the fact that surface water is already insufficient for irrigation water requirements, it will not be possible to supply the amount of water necessary to create such a pressure ridge.

B. Proposed Procedures for Preventing Salt Water Movement.

In view of the complicated nature of salt water lateral movement, the lack of complete information concerning salt water and fresh water distribution, the costly aspects of pumping through schemes, and the insufficiency of the available surface water for creating a pressure ridge, a simplified measure is suggested for this research. The reasoning and criteria are described below:

1. Artificial recharge will always be feasible in the nonsaline zone, and it has the advantage of raising the water table and preventing salt water moving into this zone. The amount of artificial recharge delivery will depend on the extra canal capacity available, the extra aquifer space available for storing recharge water, and the available surplus surface water.

2. Due to the limit of canal capacity or the available surplus water especially during the dry season, artificial recharge might not be enough, or even not available at all, to keep the water table

in the nonsaline zone high enough to prevent salt water contamination. In this case, it might be necessary to pump some of the saline water and allow it to drain away thus lowering the water table in the saline area. Deep drainage tubewells will be provided to pump the saline water for drainage. For the full development of ground water resource and providing more water for agriculture development in the future, pumping of water from the saline zone to supplement the irrigation water requirement might be necessary. Pumping for drainage from the saline zone and recharge of the more fresh water from surface and other fresh water zone will gradually improve the water quality in this zone and in the long run eliminate the hazard of salt water contamination due to lateral movement. Drainage of this saline water at the present stage is a problem that must be solved. Several measures have been suggested including direct diversion to the sea, diversion to the river during the high flow period and diversion to evaporation pans on some of the uncultivated waste land. Each measure will have its own advantages and disadvantages and involved different costs. The costs involved for different measures are not available at present. In this study, it is assumed that all drainage water will be diverted to the river. How much water is allowed to drain to the river can be regulated, however, the effect of this drainage on the river will not be considered.

3. Another advantage of deep well drainage is that pumping from the deep salt water zone has the effect of lowering the interface between the upper fresh water and saline water. If the well screen is well below the interface, the relative fresh water in the upper

thin layer discharged into the deep well can be small and the skimming wells of depth 30 to 50 feet with well capacity around 0.25 to 0.5 cfs can be provided for skimming the upper fresh water. A multiple purpose well is being tested in Pakistan which combines the deep drainage well and shallow skimming well together. The upper part of the well will skim the upper fresh water, while the lower part of the well will pump the saline water out for drainage. The relative positions of the well screens on both parts must be adjusted so that the loss of fresh water discharged from the deep well for drainage will be minimized. In this study, separated deep wells and skimming wells are considered and the loss of fresh water to the deep well is assumed small and negligible.

4. The analysis of the lateral movement of salt water due to different amounts of pumping and recharge to the three different zones is quite complicated. In order to simplify the problem, the Ghyben-Herzberg relation was used to estimate the change of the interface between the salt and fresh water. The Ghyben-Herzberg relation states that supposing the head at the salt water zone is constant, then a change of water table at the fresh water zone will cause the rise or fall of the interface according to the following formula:

$$\Delta h_i = \frac{\rho_f}{\rho_s - \rho_f} \cdot \Delta h_f \qquad \text{(C-19)}$$

where

ρ_f = density of fresh water

ρ_s = density of salt water

Δh_f = change of water table in fresh water area

Δh_i = change of interface.

A rise of the interface in the fresh water area corresponds to the lateral movement of salt water from the saline area toward the fresh water area. The above estimate is always conservative. Assuming the change of water table in each zone will be proportional to the storage coefficient, amount of water recharged and amount of water pumped in each respective zone, then the possible change of the interface can be estimated according to the relative change of the water table.

5. As mentioned at the beginning of this appendix, it is relatively safe to have a 4 cfs well in the fresh water area with a thickness of at least 500 feet . With approximately 700 feet thickness of fresh water in the nonsaline zone, the relative changes in water table for a density difference of 0.025 will be about 5 feet without salt water contamination. Limits on the relative differences of depths to water table in the three zones are set up and the constraints are simplified as follows:

$DGW_1(k) - DGW_2(k) \leq$ some limit within the range of 5 feet,

$DGW_2(k) - DGW_3(k) \leq$ some limit within the range of 5 feet,

where $DGW_1(k)$, $DGW_2(k)$, $DGW_3(k)$ = depth to water table in the nonsaline, intermediate and saline zones for period k .

Due to the change of the available surface water, the interface in the nonsaline zone will be fluctuating up and down within a certain range. During the dry season, the water table is lowered from tubewell pumping and causes the rise of interface. During the wet season the water table in the nonsaline zone can be recovered through artificial recharge and thus suppress further lateral movement of the interface. Within this range the water is contaminated, but as long as this range is restricted within some limit by the above constraints, there will be no salt water contamination from the discharging wells.

Calculation of Recharge Coefficients for Water Delivery.

The surface water diverted from the head of the main canal through branches, distributaries and watercourses to the irrigation field contributes recharge to the groundwater aquifer. The pumped groundwater distributed through heads of watercourses or heads of distributaries also contributes part of the recharge. The amount of recharge from the main canal and branches to the three different ground water quality areas is assumed proportional to the total length covered in each area. This proportion is estimated for the nonsaline, intermediate and saline zones.

A. Recharge to the Nonsaline Zone.

A.1. Watercourses.

Total water delivered to heads of watercourses

$= P_{11}(k) + CW_1(k) + CLHW \cdot AR1(k)$.

Seepage loss = $CSWC \cdot [P_{11}(k) + CW_1(k) + CLHW \cdot AR1(k)]$.

Recharge = CRWC $\cdot$ (seepage loss)

where CSWC = seepage loss coefficient on watercourses,

CRWC = recharge coefficient as fraction of CSWC,

CLHW = delivery efficiency from the head of the main canal to heads of watercourses.

A.2. Irrigation Field.

Seepage loss = $CSIF \cdot (1-CSWC) \cdot [P_{11}(k) + CW_1(k)]$

Recharge = CRIF $\cdot$ (seepage loss)

where CSIF = seepage loss coefficient on irrigation field,

CRIF = recharge coefficient as fraction of CSIF.

A.3. Distributaries.

Flow at distributary heads = $[CW_1(k)+CLHW \cdot AR1(k)]/CLFD$,

Seepage loss = $(CSDH/CLFD) \cdot [CW_1(k)+CLHW \cdot AR1(k)]$,

Recharge = $CRDH \cdot$ (seepage loss)

where CLFD = delivery efficiency from the head of the main canal to heads of distributaries,

CSDH = seepage loss coefficient within distributaries,

CRDH = recharge coefficient as fraction of CSDH.

A.4. The Head of the Main Canal to Distributary Heads.

Flow at the head of the main canal

$$= \sum_{i=1}^{3} [CW_i(k)/CLHW] + AR1(k)$$

Seepage loss from the head of the main canal to heads of watercourses for overall area

$$= CSHW \cdot [\sum_{i=1}^{3} CW_i(k)/CLHW + AR1(k)]$$

Recharge down to heads of watercourses for overall area

$= CRHW \cdot$ (seepage loss)

Recharge down to distributary heads for the nonsaline area

$$= 5/8\ (CRHW \cdot CSHW - CRDH \cdot CSDH \cdot CLHW/CLFD) \cdot [AR1(k) + \sum_{i=1}^{3} CW_i(k)/CLHW]$$

where CSHW = seepage loss coefficient from the head of the main canal to heads of watercourses.

CRHW = recharge coefficient as fraction of CSHW .

A.5. Artificial Recharge = $CPAR \cdot CLHW \cdot AR1(k)$,

where CPAR = portion of artificial diversion which contributes to artificial recharge.

B. Recharge to the Intermediate Zone.

B.1. Watercourses.

Total water delivered to heads of watercourses

$= CW_2(k) + P_{22}(k) + CLFD \cdot P_{12}(k)$

Recharge $= CRWC \cdot CSWC \cdot [CW_2(k)+P_{22}(k)+CLFD \cdot P_{12}(k)]$.

B.2. Irrigation Field.

Recharge $= CRIF \cdot CSIF \cdot (1-CSWC) \cdot [CW_2(k)+P_{22}(k)+CLFD \cdot P_{12}(k)]$

B.3. Distributaries.

Flow at distributary heads $= CW_2(k)/CLFD+P_{12}(k)$,

Seepage loss $= (CSDH/CLFD) \cdot CW_2(k)+CSDH \cdot P_{12}(k)$,

Recharge $= (CRDH \cdot CSDH/CLFD) \cdot CW_2(k)+CRDH \cdot CSDH \cdot P_{12}(k)$.

B.4. Canal Head Down to Distributary Heads.

$$\text{Recharge} = 1/8\ (CRHW \cdot CSHW-CRDH \cdot CSDH \cdot CLHW/CLFD) \cdot [AR1(k) + \sum_{i=1}^{3} CW_i(k)/CLHW] \quad .$$

C. Recharge to the Saline Zone.

C.1. Watercourses.

Total water delivered to heads of watercourses

$= CW_3(k)+CLFD \cdot P_{13}(k)+CLFD \cdot P_{23}(k)+P_{33}(k)$,

Recharge $= CRWC \cdot CSWC \cdot [CW_3(k)+CLFD \cdot P_{13}(k)+CLFD \cdot P_{23}(k)+P_{33}(k)]$

C.2. Irrigation Field.

$$\text{Recharge} = CRIF \cdot CSIF \cdot (1-CSWC) \cdot [CW_3(k)+CLFD \cdot P_{13}(k)+CLFD \cdot P_{23}(k) +P_{33}(k)] \quad .$$

C.3. Distributaries.

Flow at distributary heads $= CW_3(k)/CLFD+P_{13}(k)+P_{23}(k)$,

Seepage loss $= (CSDH/CLFD) \cdot CW_3(k)+CSDH \cdot [P_{13}(k)+P_{23}(k)]$

Recharge $= (CRDH \cdot CSDH/CLFD) \cdot CW_3(k)+CRDH \cdot CSDH \cdot [P_{13}(k)+P_{23}(k)]$

C.4. Canal Head to Distributary Heads.

$$\text{Recharge} = 2/8\ (\text{CRHW}\cdot\text{CSHW}-\text{CRDH}\cdot\text{CSDH}\cdot\text{CLHW}/\text{CLFD})$$
$$\cdot[\text{AR1}(k)+\sum_{i=1}^{3} \text{CW}_i(k)/\text{CLHW}] \quad .$$

Derivation of the Power Cost.

Due to different recharge and pumping rates during each operational period, the power cost of pumping will vary from time to time. A general formula for calculating power cost for any period k is as follows:

1. Rise of water table in area i during period k is

$$\text{RECH}_i(k) = \text{REQ}_i(k)/(\text{SC}_i\cdot\text{GA}_i)$$

where $\text{RECH}_i(k)$ = rise of water table from recharge in area i during period k.

SC_i = storage coefficient of the aquifer in area i

GA_i = gross area of the aquifer in area i

$\text{REQ}_i(k)$ = total amount of recharge in area i during period k.

2. Decline of water table due to pumping

$$\text{PDC}_i(k) = \text{VP}_i(k)/(\text{SC}_i\cdot\text{GA}_i)$$

where $\text{PDC}_i(k)$ = decline of water table due to pumping in area i during period k

$\text{VP}_i(k)$ = total amount of water pumped in area i during period k.

3. There are other sources of recharge to the aquifer such as rainfall throughput. Assume rise of water table due to the recharge from these sources is $\text{ER}_i(k)$.

4. Total pumping lift in any period k is the sum of the initial depth to ground water table; the dynamic head including discharge lift, initial drawdown and hydraulic loss; and, the accumulated

water table decline from the first period to period k. A pumping drawdown of 6 feet per cfs of discharge and a hydraulic loss of 1 foot per cfs of discharge is adopted in this study according to T&K (1967).

Let DI_i be the initial depth to the ground water table and QW_i be the average size of the well in area i, then the total pumping lift in period k can be expressed as follows:

$$H_i(k) = DI_i + (6+1) \cdot QW_i + \sum_{k=1}^{k} \cdot [PDC_i(k) - RECH_i(k) - ER_i(k)] \quad .$$

5. At a cost of 0.09 RS per Kwh (T&K, 1967) and a wire to water efficiency at 60%, the power cost per acre foot per foot of lift is equal to

$$U = 1.025\left(\frac{0.09}{0.06} \right) = 0.184 \text{ (RS/AF/ft)}$$

where 1.025 is a conversion factor from RS/Kwh to RS/AF/ft.

Pumping cost at any period k then is

$$C_k = 0.184 \cdot VP_i(k) \cdot H_i(k) \quad .$$

If k is the monthly period, total annual cost of tubewell pumping in area i is

$$CP_i = \sum_{k=1}^{12} 0.184 \cdot VP_i(k) \cdot H_i(k) \quad .$$

PROGRAM FLEXIPLEX

```
      PROGRAM FLEXI (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C
C     * * * * * * * P R O G R A M   F L E X I P L E X * * * * * * *
C     THIS PROGRAM SOLVE THE DESIGN AND OPERATIONAL PROBLEM FOR LOWER
C     JHELUM CANAL COMMANDED AREA UNDER THE CONJUNCTIVE USE OF SURFACE
C     AND GROUND WATER
C     FLEXIBLE TOLERANCE METHOD IS USED TO SEARCH THE OPTIMAL DESIGN
C     ALTERNATIVE SO THAT THE OVERALL COST OF CONSTRUCTION AND OPERATION
C     AND MAINTENANCE COST WILL BE MINIMIZED
C
C     NOTATIONS*****
C     AGA1,AGA2,AGA3=GROSS AREA OF AREA 1, 2 AND 3 (ACRES)
C     AR1(I)=RATE OF ARTIFICIAL RECHARGE DELIVERY TO THE NONSALINE AREA
C     DURING PERIOD I (CFS)
C     AA(I),AB(I)=RELATIVE WATER LEVEL CONSTRAINTS BETWEEN AREA 1 AND 2,
C     AND AREA 2 AND 3 WHENEVER JOINT OPERATIONAL DECISIONS ARE NEEDED
C     AC(I),AE(I)=RELATIVE WATER LEVEL CONSTRAINTS BETWEEN AREA 1 AND 2,
C     AND AREA 2 AND 3 WHENEVER DIRECT RIVER DIVERSION IS FEASIBLE
C     AD1,AD2,AD3=COEFFICIENTS TAKING INTO ACCOUNT THE POSSIBLE INCREASE
C     OF COST DUE TO THE INCREASE OF FRICTION HEAD THROUGH THE DISCHARGE
C     LINE
C     BF1(I)=BASE FLOW TO RIVER DURING PERIOD I FOR AREA 1 (CFS)
C     CA1,CA2=COST OF CANAL REMODELING UP TO DISTRIBUTARY HEAD WHEN
C     THE REMODELING RATIO LESS THAN AND GREATER THAN 60 PERCENT
C     CD=COST OF EXTRA DRAINAGE WORKS PER CFS FOR AREA 3
C     CST=COST OF SHORTAGE PER ACRE FEET
C     CT1,CT2,CT3=COST OF TUBEWELL INSTALLATION PER CFS FOR AREA 1, 2
C     AND 3
C     CTSK=COST OF SKIMMING WELL INSTALLATION PER CFS FOR AREA 3
C     CCH1,CCH2=COST OF CANAL REMODELING FROM DISTRIBUTARY HEAD UP TO
C     CANAL HEAD
C     CLFD=DELIVERY EFFICIENCY FROM THE HEAD OF THE MAIN CANAL DOWN TO
C     THE HEAD OF THE DISTRIBUTARY
C     CLHW=DELIVERY EFFICIENCY FROM THE HEAD OF THE MAIN CANAL DOWN TO
C     THE HEAD OF WATERCOURSE
C     CSWC,CRWC=SEEPAGE LOSS COEFFICIENT ON WATERCOURSE AND RECHARGE
C     COEFFICIENT AS FRACTION OF SEEPAGE LOSS
C     CSIF,CRIF=SEEPAGE LOSS COEFFICIENT ON IRRIGATION FIELD AND
C     RECHARGE COEFFICIENT AS FRACTION OF SEEPAGE LOSS
C     CSDH,CRDH=SEEPAGE LOSS COEFFICIENT FROM THE HEAD OF CANAL DOWN TO
C     THE HEADS OF DISTRIBUTARIES AND RECHARGE COEFFICIENT AS FRACTION
C     OF SEEPAGE LOSS
C     CSHW,CRHW=SEEPAGE LOSS COEFFICIENT FROM THE HEAD OF THE MAIN CANAL
C     DOWN TO THE HEADS OF WATERCOURSES AND RECHARGE COEFFICIENT AS
C     FRACTION OF SEEPAGE LOSS
C     CW1(I),CW2(I),CW3(I)=DELIVERY RATES OF SURFACE WATER SUPPLY AT
C     WATERCOURSE HEADS FOR AREA 1, 2 AND 3 DURING PERIOD I (CFS)
C     CL11,CL21,CL31=RECHARGE COEFFICIENT AS PORTION OF CW1(I),CW2(I),
C     CW3(I) ATTRIBUTED TO RECHARGE FOR AREA 1
C     CL12,CL22,CL32=RECHARGE COEFFICIENT AS PORTION OF CW1(I),CW2(I),
C     CW3(I) ATTRIBUTED TO RECHARGE FOR AREA 2
C     CL13,CL23,CL33=COEFFICIENT AS PORTION OF CW1(I),CW2(I),CW3(I),
C     ATTRIBUTED TO RECHARGE FOR AREA 3
C     CLAR1,CLAR2,CLAR3=RECHARGE COEFFICIENT AS PORTION OF AR1(I)
C     ATTRIBUTED TO RECHARGE FOR AREA 1, 2 AND 3
C     CLP[illegible] CLP12,CLP13=RECHARGE COEFFICIENT AS PORTION OF P11(I),P12(I)
C     P13([illegible] ATTRIBUTED TO RECHARGE FOR AREA 1, 2 AND 3 RESPECTIVELY
C     CLP22,CLP23=RECHARGE COEFFICIENT AS PORTION OF P22(I),P23(I)
C     ATTRIBUTED TO RECHARGE FOR AREA 2 AND 3
C     CLP33=RECHARGE COEFFICIENT AS PORTION OF P33(I) ATTRIBUTED TO RECH
C     CLP33=RECHARGE COEFFICIENT AS PORTION OF P33(I) ATTRIBUTED TO
C     RECHARGE FOR AREA 3
C     DCH,DCW1,DCW2,DCW3=DESIGN CAPACITIES AT THE HEAD OF THE MAIN CANAL
C     THE HEADS OF WATERCOURSES IN AREA 1, 2 AND 3 (CFS)
C     DIP1,DIP2=DESIGN CAPACITIES OF IRRIGATION TUBEWELL IN AREA 1 AND 2
C     (CFS)
C     DIP3,DISK=DESIGN DRAINAGE TUBEWELL CAPACITY AND SKIMMING WELL
C     CAPACITY IN AREA 3 (CFS)
C     DGW1(I),DGW2(I),DGW3(I)=DEPTH TO WATER TABLE FROM THE GROUND
C     SURFACE IN AREA 1, 2 AND 3(FT)
C     DI1,DI2,DI3=INITIAL DEPTH TO WATER TABLE IN AREA 1, 2 AND 3 (FT)
C     DMA1,DMA2,DMA3=MAXIMUM ALLOWABLE DEPTH TO WATER TABLE (FT)
C     DMI1,DMI2,DMI3=MINIMUM ALLOWABLE DEPTH TO WATER TABLE (FT)
C     EDCH,EDCW1,EDCW2,EDCW3=EXISTING CAPACITIES AT THE HEAD OF THE MAIN
C     CANAL AND THE HEADS OF WATERCOURSES IN AREA 1, 2 AND 3  (CFS)
C     EDIP1,EDIP2=EXISTING IRRIGATION TUBEWELL INSTALLATION CAPACITIES
C     IN AREA 1 AND 2 (CFS)
C     EDIP3,EDISK=EXISTING DRAINAGE TUBEWELL AND SKIMMING WELL
C     INSTALLATION CAPACITIES IN AREA 3 (CFS)
C     EFAR=PERCENTAGE OF RECHARGE FOR ARTIFICIAL RECHARGE DIVERSION
C     ENR=COST OF ENERGY PER ACRE FOOT PER FOOT OF LIFT
C     ER1(I),ER2(I),ER3(I)=NATURAL RECHARGE FROM RIVER AND RAINFALL
C     DURING PERIOD I FOR AREA 1, 2 AND 3 (FT)
C     MS=NUMBER OF SUBPERIODS
C     NY=NUMBER OF YEARS
C     OA=COST OF ARTIFICIAL RECHARGE DIVERSION PER CFS
C     OS=COST OF CANAL OPERATION PER CFS
C     PER1,PER2=PERCENTAGE OF THE AMOUNT OF PUMPING IN THE NONSALINE
C     AND INTERMEDIATE ZONES THAT NEED TO BE EXPORTED
C     PI1,PI2,PI3,PIK=FACTOR(GREATER THAN 1) ACCOUNT FOR THE NEED TO
C     INCREASE THE CAPACITY OF TUBEWELL DUE TO POSSIBLE FAILURE OF
C     SOME OF THE TUBEWELLS IN AREA 1, 2 AND 3 AND SKIMMING WELL IN
C     AREA 3
C     P11(I),P12(I),P13(I)=RATE OF PUMPING IN AREA 1 TO BE DELIVERED TO
C     AREA 1, 2 AND 3 DURING PERIOD I (CFS)
C     P22(I),P23(I)=RATE OF PUMPING IN AREA 2 TO BE DELIVERED TO AREA 2
C     AND 3 DURING PERIOD I (CFS)
C     PD3(I)=RATE OF PUMPING FOR DRAINAGE IN AREA 3 DURING PERIOD I(CFS)
C     RATE OF SKIMMING WELL PUMPING FOR AREA 3 DURING PERIOD I (CFS)
C     PRO1,PRO2,PRO3=PROPORTION OF THE LENGTH OF THE MAIN CANAL AND
C     BRANCHES IN AREA 1, 2 AND 3
C     RIN(I)=AVAILABLE RIVER FLOW AT THE HEAD OF THE MAIN CANAL DURING
C     PERIOD I (CFS)
C     RMIX2,RMIX3=MIXING RATIO OF GROUND WATER AND SURFACE WATER IN AREA
C     2 AND 3
C     SC1,SC2,SC3=STORAGE COEFFICIENT IN AREA 1, 2 AND 3
C     SHRT1(I),SHRT2(I),SHRT3(I)=WATER SHORTAGE IN AREA 1, 2 AND 3
C     DURING PERIOD I (CFS)
C     WR1(I),WR2(I),WR3(I)=WATER REQUIREMENT DURING PERIOD I FOR AREA 1,
C     2 AND 3 (CFS)
C     NX      TOTAL NUMBER OF INDEPENDENT VARIABLES
C     NC      TOTAL NUMBER OF EQUALITY CONSTRAINTS
C     NIC     TOTAL NUMBER OF INEQUALITY CONSTRAINTS
C     SIZE    EDGE LENGTH OF THE INITIAL POLYHEDRON
C     CONVER  CONVERGENCE CRITERION FOR TERMINATION OF THE SEARCH
C     BETA    THE CONTRACTION COEFFICIENT
C     ALFA    THE REFLECTION COEFFICIENT
C     GAMA    THE EXPANSION COEFFICIENT
C     X(I)    THE ASSUMED VECTOR TO INITIATE THE SEARCH
C     FDIFER  THE TOLERANCE CRITERION FOR CONSTRAINT VIOLATION
C     ICONT   A COUNTER TO RECORD STAGE COMPUTATIONS
C     NCONT   A COUNTER TO PRINT INFORMATION EVERY (NX+1) STAGE
C     LOW     AN INDEX TO IDENTIFY INFORMATION RELATED TO THE LOWEST
C             VALUE OF OBJ. FUNCTION IN MOST RECENT POLYHEDRON
C     LHIGH   AN INDEX TO IDENTIFY INFORMATION RELATED TO LARGEST VALUE
C             OF OBJ. FUNCTION IN MOST RECENT POLYHEDRON
C     LSEC    AN INDEX TO IDENTIFY INFORMATION RELATED TO THE SECOND
C     LARGEST VALUE OF OBJ. FUNCTION IN MOST RECENT POLYHEDRON
C
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      DIMENSION X(10), X1(10,10), X2(10,10), R(20), SUM(10), F(10), SP(1
     10), ROLD(20)
      COMMON /1/ NX,NC,NIC,STEP,ALFA,BETA,GAMA,IN,INF,EDIFFR,SEQL,K1,K2,
     1K3,K4,K5,K6,K7,K8,K9,X,X1,X2,R,SUM,F,SP,ROLD,SCALE,FOLD,SIZE
      COMMON /2/ LEEAS,L5,L6,L7,L8,L9,R1A,R2A,R3A
      COMMON /PRN/ EDCW1,EDCW2,EDCW3,EDCH,EDIP1,EDIP2,EDIP3,EDISK,DCH,DC
     1W1,DCW2,DCW3,DIP1,DIP2,DIP3,DISK,CLHW
      COMMON /DPP/ NY,MS,CLFD,CL11,CL21,CL31,CL12,CL22,CL32,CL13,CL23,CL
     133,CLAR1,CLAR2,CLAR3,CLP11,CLP12,CLP13,CLP22,CLP23,CLP33,TA1,TA2,T
     2A3,DI1,DI2,DI3,DMI1,DMI2,DMI3,OS,OA,ENR,CST,AD1,AD2,AD3,CON,PWR,AA
     3(12),AB(12),WR1(12),WR2(12),WR3(12),RF1(12),FR1(12),FR2(12),FR3(12
     4),RIN(75),PERD(12),IDENT,RMIX2,RMIX3,SSHT1,SSHT2,SSHT3,AC(12),AE(1
     52),IFAL
      COMMON /COS/ CA1,CA2,CCH1,CCH2,PI1,PI2,PI3,PIK,CT1,CT2,CT3,CD,CA33
     1,CTSK,RA1,RA2,RA3,RA4,PER1,PER2,DMCW1,DMCW2,DMIP1,DMIP2,WRM1,WRM2,
     2TWR,ZO,TOTAL
      COMMON /AY/ DGW1(75),DGW2(75),DGW3(75),CW1(75),CW2(75),CW3(75),AR1
     1(75),P11(75),P12(75),P13(75),P22(75),P23(75),P33(75),P34(75),C(75)
     2,SHRT1(75),SHRT2(75),SHRT3(75)
      DIMENSION TITLE(10)
C
C     READ INPUT DATA
C
      READ (5,188) TITLE
      READ (5,181) NX,NC,NIC,SIZE,CONVER
      ALFA=1.
      BETA=0.5
      GAMA=2.
      STEP=SIZE
      IFAL=0
      READ 159, NY,MS
      READ 160, AGA1,AGA2,AGA3,C1,C2,C3,CS,STIN
      READ 162, DMA1,DMA2,DMA3,DMI1,DMI2,DMI3,DI1,DI2,DI3
      READ 160, CA1,CCH1,CA2,CCH2,CD,CT1,CT2,CT3
      READ 160, CSWC,CRWC,CSIF,CRIF,CSDH,CRDH,CSHW,CRHW
      READ 162, PI1,PI2,PI3,PRO1,PRO2,PRO3,SC1,SC2,SC3
      READ 160, PIK,CST,CTSK,OA,OS,ENR,CP1,CP2
      READ 161, (WR1(I),I=1,MS)
      READ 161, (WR2(I),I=1,MS)
      READ 161, (WR3(I),I=1,MS)
      READ 161, (ER1(I),I=1,MS)
      READ 161, (FR2(I),I=1,MS)
      READ 161, (FR3(I),I=1,MS)
      READ 161, (RF1(I),I=1,MS)
      READ 161, (AA(I),I=1,MS)
      READ 161, (AB(I),I=1,MS)
      READ 161, (AC(I),I=1,MS)
      READ 161, (AE(I),I=1,MS)
      TWR=0.
      DO 102 K1=1,MS
         SWR=WR1(K1)+WR2(K1)+WR3(K1)
         IF (SWR-TWR) 102,102,101
  101    TWR=SWR
  102 CONTINUE
      NTP=NY*MS
      READ 161, (RIN(I),I=1,NTP)
      READ 160, EDCH,EDCW1,EDCW2,EDCW3,EDIP1,EDIP2,EDIP3,EDISK
      READ 160, PER1,PER2,EFAR,AD1,AD2,AD3,RMIX2,RMIX3
      READ 163, OW,PWR,DISK
      AAW 0.0
      ABW=0.0
C
C     CALCULATE MAX WATER REQUIREMENT AND DESIGN CAPACITY AT HEADS OF
C     WATER COURSES IN THE SALINE ZONE
C
      WRM3=0.0
      DO 104 I=1,MS
         SWR3=WR3(I)
         IF (SWR3-WRM3) 104,104,103
  103    WRM3=SWR3
  104 CONTINUE
      DCW3=WRM3-DISK
      RMIXD2=RMIX2*CLFD
      RMIXD3=RMIX3*CLFD
C
C     WRITE INPUT DATA
C
      PRINT 164, NY,MS
      PRINT 165, (RIN(I),I=1,NTP)
      IA=1
      PRINT 166, IA,(WR1(J),J=1,MS)
      IA=IA+1
      PRINT 166, IA,(WR2(J),J=1,MS)
      IA=IA+1
      PRINT 166, IA,(WR3(J),J=1,MS)
      PRINT 167, AGA1,AGA2,AGA3,DMA1,DMA2,DMA3,DMI1,DMI2,DMI3
C
C     CALCULATE RECHARGE COEFFICIENTS
C
      CLHW=1.0-CSHW
      CLFD=1.0-CSDH
      RE11=CRWC*CSWC
      RE12=CRIF*CSIF*(1.0-CSWC)
      RE13=(CSDH/CLFD)*CRDH
      RF14=(CSHW/CLHW)*CRHW-RE13
C
C     RECHARGE COFFF FOR CW1,CW2,CW3,AR1,P11 IN NONSALINE ZONE
C
      CL11=RF11+RF12+RE13+PRO1*RF14
      CL21=PRO1*RF14
      CL31=CL21
      CLAR1=RE11*CLHW+EFAR*CLHW+RF13*CLHW+PRO1*(CRHW*CSHW-RE13*CLHW)
      CLP11=RF11+RF12
C
C     RECHARGE COFFF FOR CW1,CW2,CW3,AR1,P12,P22  INTERMEDIATE ZONE
C
      CL12=PRO2*RF14
      CL22=RE11+RF12+RE13+PRO2*RF14
      CL32=CL12
      CLAR2=PRO2*(CRHW*CSHW-RE13*CLHW)
      CLP12=CLFD*RE11+CLFD*RF12+CSDH*CRDH
      CLP22=RF11+RE12
C
C     RECHARGE COFFF FOR CW1,CW2,CW3,AR1,P13,P23   SALINE ZONE
C
      CL13=PRO3*RF14
      CL23=CL13
      CL33=RF11+RE12+RE13+PRO3*RF14
      CLAR3=PRO3*(CRHW*CSHW-RF13*CLHW)
      CLP13=CLFD*RF11+CLFD*RE12+CSDH*CRDH
      CLP23=CLP13
      CLP33=RF11+RE12
      CON=(365.0*24.0*60.0*60.0/43560.0)/MS
      TA1=CON/(AGA1*SC1)
      TA2=CON/(AGA2*SC2)
      TA3=CON/(AGA3*SC3)
      PRINT 168, TA1,TA2,TA3
      WRM1=0.0
      WRM2=0.0
      DO 106 I=1,MS
         SWR1=WR1(I)
         IF (SWR1-WRM1) 106,106,105
  105    WRM1=SWR1
  106 CONTINUE
      DO 108 I=1,MS
         SWR2=WR2(I)
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      IF (SWR2-WRM2) 108,108,107
107   WRM2=SWR2
108 CONTINUE
    PRINT 169, WRM1,WRM2,WRM3
    READ 170, DMCW1,DMCW2,DMIP1,DMIP2
109 READ 170, DCW1,DCW2,DIP1,DIP2
    X(1)=DCW1
    X(2)=DCW2
    X(3)=DIP1
    X(4)=DIP2
    IF (X(1).EQ.0.) GO TO 158
    DCH=(DCW1+DCW2+DCW3)/CLHW
    PRINT 171, (X(J),J=1,NX)
    WRITE (6,195) TITLE
    WRITE (6,185) NX,NC,NIC,SIZE,CONVER
    K1=NX+1
    K2=NX+2
    K3=NX+3
    K4=NX+4
    K5=NX+5
    K6=NC+NIC
    K7=NC+1
    K8=NC+NIC
    K9=K8+1
    N=NX-NC
    N1=N+1
    IF (N1.GE.3) GO TO 110
    N1=3
    N=2
110 N2=N+2
    N3=N+3
    N4=N+4
    N5=N+5
    N6=N+6
    N7=N+7
    N8=N+8
    XN=N
    XNX=NX
    XN1=N1
    R1A=0.5*(SQRT(5.)-1.)
    R2A=R1A*R1A
    R3A=R2A*R1A
    L5=NX+5
    L6=NX+6
    L7=NX+7
    L8=NX+8
    L9=NX+9
    ICONT=1
    NCONT=1
    WRITE (6,183)
    WRITE (6,184) (X(J),J=1,NX)
    FDIFER=2.*(NC+1)*STEP
    FOLD=FDIFER
    IN=N1
    CALL SUMR
    SR(N1)=SQRT(SEQL)
    WRITE (6,192) FDIFER,SR(N1)
    IF (SR(N1).LT.FDIFER) GO TO 111
    CALL WRITEX
    WRITE (6,186)
    INF=1
    STEP=0.05*FDIFER
    CALL FEASBL
    WRITE (6,193)
    WRITE (6,184) (X2(INF,J),J=1,NX)
    WRITE (6,194) SR(INF)
    IF (FOLD.LT.1.0) GO TO 157
111 WRITE (6,182)
    WRITE (6,187) ICONT,FDIFER
    CALL WRITEX
    FTER=R(K9)
C
C   COMPUTE CENTROID OF ALL VERTICES OF INITIAL POLYHEDRON
C
    STEP1=STEP*(SQRT(XNX+1.)+XNX-1.)/(XNX*SQRT(2.))
    STEP2=STEP*(SQRT(XNX+1.)-1.)/(XNX*SQRT(2.))
    ETA=(STEP1+(XNX-1.)*STEP2)/(XNX+1.)
    DO 112 J=1,NX
      X(J)=X(J)-ETA
112 CONTINUE
    CALL START
    DO 113 I=1,N1
    DO 113 J=1,NX
      X2(I,J)=X1(I,J)
113 CONTINUE
    DO 116 I=1,N1
      IN=I
      DO 114 J=1,NX
114   X(J)=X2(I,J)
      CALL SUMR
      SR(I)=SQRT(SEQL)
      IF (SR(I).LT.FDIFER) GO TO 115
      CALL FEASBL
      IF (FOLD.LT.1.0) GO TO 157
115   CALL PROBLEM (3)
      F(I)=R(K9)
116 CONTINUE
117 STEP=0.05*FDIFER
    ICONT=ICONT+1
C
C   SELECT LARGEST VALUE OF OBJECTIVE FUNCTION FROM POLYHEDRON VERTICE
C
    FH=F(1)
    LHIGH=1
    DO 118 I=2,N1
      IF (F(I).LT.FH) GO TO 118
      FH=F(I)
      LHIGH=I
118 CONTINUE
C
C   SELECT MINIMUM VALUE OF OBJECTIVE FUNCTION FROM POLYHEDRON VERTICE
C
119 FL=F(1)
    LOW=1
    DO 120 I=2,N1
      IF (FL.LT.F(I)) GO TO 120
      FL=F(I)
      LOW=I
120 CONTINUE
    DO 121 J=1,NX
121 X(J)=X2(LOW,J)
    IN=LOW
    CALL SUMR
    SR(LOW)=SQRT(SEQL)
    IF (SR(LOW).LT.FDIFER) GO TO 122
    INF=LOW
    CALL FEASBL
    IF (FOLD.LT.1.0) GO TO 157
    CALL PROBLEM (3)
    F(LOW)=R(K9)
    GO TO 119
122 CONTINUE
C
C   FIND CENTROID OF POINTS WITH I DIFFERENT THAN LHIGH
C
    DO 124 J=1,NX
```

```
      SUM2=0.
      DO 123 I=1,N1
123   SUM2=SUM2+X2(I,J)
124 X2(N2,J)=1./XN*(SUM2-X2(LHIGH,J))
    SUM2=0.
    DO 125 I=1,N1
    DO 125 J=1,NX
      SUM2=SUM2+(X2(I,J)-X2(N2,J))**2
125 CONTINUE
    FDIFER=(NC+1)/XN1*SQRT(SUM2)
    IF (FDIFER.LT.FOLD) GO TO 126
    FDIFER=FOLD
    GO TO 127
126 FOLD=FDIFER
127 CONTINUE
    FTEP=F(LOW)
    NCONT=NCONT+1
    IF (NCONT.LT.2*N1) GO TO 131
    IF (ICONT.LT.1500) GO TO 128
    FOLD=0.5*FOLD
128 NCONT=0
    WRITE (6,182)
    WRITE (6,187) ICONT,FDIFER
    CALL WRITEX
    DO 130 IY=1,NY
      DO 129 J=1,MS
        L=MS*(IY-1)+J
        PRINT 176, IY,J,L,RIN(L),WR1(J),WR2(J),WR3(J)
        PRINT 177, CW1(L),CW2(L),CW3(L),AK1(L),P11(L),P12(L),P13(L),
   1    P22(L),P23(L),P33(L),P34(L)
        L1=L+1
        PRINT 178, DGW1(L1),DGW2(L1),DGW3(L1)
        PRINT 179, C(L)
129   CONTINUE
130 CONTINUE
    PRINT 180, Z0,TOTAL,R(11)
131 IF (FDIFER.LT.CONVER) GO TO 154
C
C     SELECT SECOND LARGEST VALUE OF OBJECTIVE FUNCTION
C
    IF (LHIGH.EQ.1) GO TO 132
    FS=F(1)
    LSEC=1
    GO TO 133
132 FS=F(2)
    LSEC=2
133 DO 134 I=1,N1
      IF (LHIGH.EQ.I) GO TO 134
      IF (F(I).LT.FS) GO TO 134
      FS=F(I)
      LSEC=I
134 CONTINUE
C
C     REFLECT HIGH POINT THROUGH CENTROID
C
    DO 135 J=1,NX
      X2(N3,J)=X2(N2,J)+ALFA*(X2(N2,J)-X2(LHIGH,J))
      X(J)=X2(N3,J)
135 CONTINUE
    IN=N3
    CALL SUMR
    SR(N3)=SQRT(SFOL)
    IF (SR(N3).LT.FDIFER) GO TO 136
    INF=N3
    CALL FEASBL
    IF (FOLD.LT.1.0) GO TO 157
136 CALL PROBLEM (3)
    F(N3)=R(K9)
    IF (F(N3).LT.F(LOW)) GO TO 139
    IF (F(N3).LT.F(LSEC)) GO TO 137
    GO TO 143
137 DO 138 J=1,NX
138 X2(LHIGH,J)=X2(N3,J)
    SR(LHIGH)=SR(N3)
    F(LHIGH)=F(N3)
    GO TO 117
C
C     EXPAND VECTOR OF SEARCH ALONG DIRECTION THROUGH CENTROID AND
C     REFLECTED VECTOR
C
139 DO 140 J=1,NX
      X2(N4,J)=X2(N3,J)+GAMA*(X2(N3,J)-X2(N2,J))
      X(J)=X2(N4,J)
140 CONTINUE
    IN=N4
    CALL SUMR
    SR(N4)=SQRT(SFOL)
    IF (SR(N4).LT.FDIFER) GO TO 141
    INF=N4
    CALL FEASBL
    IF (FOLD.LT.1.0) GO TO 157
141 CALL PROBLEM (3)
    F(N4)=R(K9)
    IF (F(LOW).LT.F(N4)) GO TO 137
    DO 142 J=1,NX
142 X2(LHIGH,J)=X2(N4,J)
    F(LHIGH)=F(N4)
    SR(LHIGH)=SR(N4)
    GO TO 117
143 IF (F(N3).GT.F(LHIGH)) GO TO 145
    DO 144 J=1,NX
144 X2(LHIGH,J)=X2(N3,J)
145 DO 146 J=1,NX
      X2(N4,J)=BETA*X2(LHIGH,J)+(1.-BETA)*X2(N2,J)
      X(J)=X2(N4,J)
146 CONTINUE
    IN=N4
    CALL SUMR
    SR(N4)=SQRT(SFOL)
    IF (SR(N4).LT.FDIFER) GO TO 147
    INF=N4
    CALL FEASBL
    IF (FOLD.LT.1.0) GO TO 157
147 CALL PROBLEM (3)
    F(N4)=R(K9)
    IF (F(LHIGH).GT.F(N4)) GO TO 152
    DO 148 J=1,NX
    DO 148 I=1,N1
148 X2(I,J)=0.5*(X2(I,J)+X2(LOW,J))
    DO 151 I=1,N1
      DO 149 J=1,NX
        X(J)=X2(I,J)
149   CONTINUE
      IN=I
      CALL SUMR
      SR(I)=SQRT(SFOL)
      IF (SR(I).LT.FDIFER) GO TO 150
      INF=I
      CALL FEASBL
      IF (FOLD.LT.1.0) GO TO 157
150   CALL PROBLEM (3)
151 F(I)=R(K9)
    GO TO 117
152 DO 153 J=1,NX
153 X2(LHIGH,J)=X2(N4,J)
    SR(LHIGH)=SR(N4)
```

```
      F(LHIGH)=F(N4)
      GO TO 117
  154 WRITE (6,189) ICONT,FDIFFR
      IFAL=1
      CALL WRITEX
C
C     OUTPUT OF FINAL SOLUTION
C
      WRITE (6,190)
      DCW1=X(1)
      DCW2=X(2)
      DIP1=X(3)
      DIP2=X(4)
      DCH=(DCW1+DCW2+DCW3)/CLHW
      PRINT 172, DCH,DCW1,DCW2,DCW3,DIP1,DIP2,DIP3,DISK
      PRINT 173, RA4,RA1,RA2,RA3
      PRINT 174, DI1,DI2,DI3
      PRINT 175
      DO 156 IY=1,NY
         DO 155 J=1,MS
            N=MS*(IY-1)+J
            PRINT 176, IY,J,N,RIN(N),WR1(J),WR2(J),WR3(J)
            PRINT 177, CW1(N),CW2(N),CW3(N),AR1(N),P11(N),P12(N),P13(N),
     1      P22(N),P23(N),P33(N),P34(N)
            N1=N+1
            PRINT 178, DGW1(N1),DGW2(N1),DGW3(N1)
            PRINT 179, C(N)
  155    CONTINUE
  156 CONTINUE
      PRINT 180, ZO,TOTAL,R(11)
      GO TO 109
  157 WRITE (6,189) ICONT,FDIFFR
      CALL WRITEX
      WRITE (6,191)
      GO TO 109
  158 STOP
C
  159 FORMAT (2I5)
  160 FORMAT (8F10.0)
  161 FORMAT (8F10.0)
  162 FORMAT (9F8.0,8X)
  163 FORMAT (3F10.0)
  164 FORMAT ( 11H NO OF YEAR,I3, 22H NO OF PERIOD PER YEAR,I3)
  165 FORMAT ( 27H RIVER INFLOW AT CANAL HEAD,/2X,12F10.2)
  166 FORMAT ( 27H WATER REQUIREMENT FOR AREA,I2/2X,12F10.2)
  167 FORMAT ( 34H GROSS AREA(ACRES) FOR THREE ZONES,3F12.2/, 55H MAXIMU
     1M ALLOWABLE DEPTH TO WATER TABLE FOR THREE ZONES,3F10.2/, 55H MINI
     2MUM ALLOWABLE DEPTH TO WATER TABLE FOR THREE ZONES,3F10.2)
  168 FORMAT (  5H TA1=,F20.9,  5H TA2=,F20.9,  5H TA3=,F20.9)
  169 FORMAT ( 17H MAX WATER DEMAND,3F10.2)
  170 FORMAT (4F10.0)
  171 FORMAT (/10X, 13HINITIAL GUESS,10F10.2/)
  172 FORMAT (1H1, 15H FINAL SOLUTION,//, 34H LOWER JHELUM CANAL COMMAND
     1ED AREA,//, 30H DESIGN CAPACITY OF THE SYSTEM,/, 11H CANAL HEAD,F1
     20.2, 25H WATERCOURS HEADS- AREA 1,F10.2,  7H AREA 2,F10.2,  7H ARE
     3A 3,F10.2/, 26H TUBEWELL CAPACITY- AREA 1,F10.2,  7H AREA 2,F10.2,
     4 21H AREA 3-DRAINAGE WELL,F10.2, 14H SKIMMING WELL,F10.2)
  173 FORMAT ( 17H REMODELING RATIO,/, 11H CANAL HEAD,F10.3, 26H WATERCO
     1URSE HEADS, AREA 1,F10.3,  7H AREA 2,F10.3,  7H AREA 3,F10.3/)
  174 FORMAT ( 37H INITIAL DEPTH TO WATER TABLE, AREA 1,F10.2,  7H AREA
     12,F10.2,  7H AREA 3,F10.2/)
  175 FORMAT (//, 20H OPERATION DECISIONS,/)
  176 FORMAT (  5H YEAR,I3,  7H PERIOD,I3,  6H STAGE,I3, 21H AVAILABLE R
     1IVER FLOW,F10.2, 16H DEMANDS, AREA 1,F10.2,  7H AREA 2,F10.2,  7H
     2AREA 3,F10.2)
  177 FORMAT ( 36H FLOW AT HEAD OF WATERCOURSE, AREA 1,F10.2,  7H AREA 2
     1,F10.2,  7H AREA 3,F10.2, 34H ARTIFICIAL RECHARGE AT CANAL HEAD,F1
     20.2/, 20H PUMPING AREA 1, P11,F10.2,  4H P12,F10.2,  4H P13,F10.2/
     39X, 11H AREA 2 P22,F10.2,  4H P23,F10.2/9X, 26H AREA 3 SKIMMING PU
     4MPING P33,F10.2, 21H DRAINAGE PUMPING P34,F10.2)
  178 FORMAT ( 46H DEPTH TO WATER TABLE AT END OF PERIOD, AREA 1,F10.2,
     1 7H AREA 2,F10.2,  7H AREA 3,F10.2)
  179 FORMAT ( 15H OPERATION COST,F20.2/)
  180 FORMAT (/, 12H DESIGN COST,F20.1, 15H OPERATION COST,F20.1, 11H TO
     1TAL COST,F25.1)
  181 FORMAT (3I5,2F10.0)
  182 FORMAT (/,40X,48H * * * * * * * * * * * * * * * * * * * * * * * *)
  183 FORMAT (//,41H THE STARTING VECTOR SELECTED BY USER IS )
  184 FORMAT (8F16.6)
  185 FORMAT (//,10X,40H NUMBER OF INDEPENDENT VARIABLES          I5,/,10X
     1,40H NUMBER OF EQUALITY CONSTRAINTS            I5,/,10X,40H NUMBER OF
     2 INEQUALITY CONSTRAINTS          I5,/,10X,40H SIZE OF INITIAL POLYHED
     3RON                 F12.5,/,10X,40H THE DESIRED CONVERGENCE IS
     4      E12.5)
  186 FORMAT (//,71H THE INITIAL X VECTOR DOES NOT SATISFY THE INITIAL T
     1OLERANCE CRITERION )
  187 FORMAT (/,10X,27H STAGE CALCULATION NUMBER =I5,20X,27H THE TOLERAN
     1CE CRITERION = F14.6)
  188 FORMAT (10A8)
  189 FORMAT (//,39H TOTAL NUMBER OF STAGES CALCULATIONS = I5,10X,25H TH
     1E CONVERGENCE LIMIT = F14.6)
  190 FORMAT (//,50X,25H THESE ARE FINAL ANSWERS )
  191 FORMAT (//,50X,29H THESE ARE NOT FINAL ANSWERS )
  192 FORMAT (//,10X,40H THE INITIAL TOLERANCE CRITERION IS      F12.5,/,
     110X,40H THE SUM OF VIOLATED CONSTRAINTS IS      E12.5)
  193 FORMAT (//,70H THE VECTOR FOUND BY PROGRAM WHICH SATISFIES THE INI
     1TIAL TOLERANCE IS )
  194 FORMAT (/,31H SUM OF VIOLATED CONSTRAINTS = E17.7)
  195 FORMAT (1H1,30X,10A8)
C
      END
```

SUBROUTINE FEASBL

```
      SUBROUTINE FEASBL
C
C*****SUBROUTINE FEASBL MINIMIZES THE SUM OF THE SQUARE VALUES OF THE
C     VIOLATED CONSTRAINTS.  IT IS CALLED EVERY TIME THE COMBINED VALUE
C     OF THE VIOLATED CONSTRAINTS EXCEEDS THE VALUE OF THE TOLERANCE
C     CRITERION FOR THE CURRENT STAGE
C
      DIMENSION X(10), X1(10,10), X2(10,10), R(20), SUM(10), F(10), SR(1
     10), ROLD(20), R1(20), R3(20), FLG(10), H(10)
      COMMON /1/ NX,NC,NIC,STEP,DUM1,DUM2,DUM3,IN,INF,FDIFFR,SEQL,K1,K2,
     1K3,K4,K5,K6,K7,K8,K9,X,X1,X2,R,SUM,F,SR,ROLD,SCALE,FOLD,SIZE
      COMMON /2/ LFEAS,L5,L6,L7,L8,L9,R1A,R2A,R3A
      COMMON /PRN/ EDCW1,EDCW2,EDCW3,EDCH,EDIP1,EDIP2,EDIP3,EDISK,DCH,DC
     1W1,DCW2,DCW3,DIP1,DIP2,DIP3,DISK,CLHW
      COMMON /COS/ CA1,CA2,CCH1,CCH2,PI1,PI2,PI3,PIK,CT1,CT2,CT3,CD,CA33
     1,CTSK,RA1,RA2,RA3,RA4,PER1,PER2,DMCW1,DMCW2,DMIP1,DMIP2,WRM1,WRM2,
     2TWR,ZO,TOTAL
      COMMON /DPP/ NY,MS,CLED,CL11,CL21,CL31,CL12,CL22,CL32,CL13,CL23,CL
     133,CLAR1,CLAR2,CLAR3,CLP11,CLP12,CLP13,CLP22,CLP23,CLP33,TA1,TA2,T
     2A3,DI1,DI2,DI3,DMI1,DMI2,DMI3,OS,OA,ENE,CST,AD1,AD2,AD3,CON,PWR,AA
     3(12),AB(12),WR1(12),WR2(12),WR3(12),RF1(12),FR1(12),FR2(12),FR3(12
     4),RIN(75),PERD(12),IDENT,RMIX2,RMIX3,SSHT1,SSHT2,SSHT3,AC(12),AF(1
     52),IFAL
      COMMON /AY/ DGW1(75),DGW2(75),DGW3(75),CW1(75),CW2(75),CW3(75),AR1
     1(75),P11(75),P12(75),P13(75),P22(75),P23(75),P33(75),P34(75),C(75)
     2,SHRT1(75),SHRT2(75),SHRT3(75)
```

```
      ALFA=1.
      BETA=0.5
      GAMA=2.
      XNX=NX
      ICONT=0
      LCHEK=0
      ICHEK=0
  101 CALL START
      DO 103 I=1,K1
         DO 102 J=1,NX
  102    X(J)=X1(I,J)
         IN=I
         CALL SUMR
  103 CONTINUE
C
C     SELECT LARGEST VALUE OF SUM(I) IN SIMPLEX
C
  104 SUMH=SUM(1)
      INDEX=1
      DO 105 I=2,K1
         IF (SUM(I).LE.SUMH) GO TO 105
         SUMH=SUM(I)
         INDEX=I
  105 CONTINUE
C
C     SELECT MINIMUM VALUE OF SUM(I) IN SIMPLEX
C
      SUML=SUM(1)
      KOUNT=1
      DO 106 I=2,K1
         IF (SUML.LE.SUM(I)) GO TO 106
         SUML=SUM(I)
         KOUNT=I
  106 CONTINUE
C
C     FIND CENTROID OF POINTS WITH I DIFFERENT THAN INDEX
C
      DO 108 J=1,NX
         SUM2=0.
         DO 107 I=1,K1
  107    SUM2=SUM2+X1(I,J)
         X1(K2,J)=1./XNX*(SUM2-X1(INDEX,J))
C
C     FIND REFLECTION OF HIGH POINT THROUGH CENTROID
C
         X1(K3,J)=2.*X1(K2,J)-X1(INDEX,J)
  108 X(J)=X1(K3,J)
      IN=K3
      CALL SUMR
      IF (SUM(K3).LT.SUML) GO TO 112
C
C     SELECT SECOND LARGEST VALUE IN SIMPLEX
C
      IF (INDEX.EQ.1) GO TO 109
      SUMS=SUM(1)
      GO TO 110
  109 SUMS=SUM(2)
  110 DO 111 I=1,K1
         IF ((INDEX-I).EQ.0) GO TO 111
         IF (SUM(I).LE.SUMS) GO TO 111
         SUMS=SUM(I)
  111 CONTINUE
      IF (SUM(K3).GT.SUMS) GO TO 114
      GO TO 128
C
C     FORM EXPANSION OF NEW MINIMUM IF REFLECTION HAS PRODUCED ONE MINIM
C
  112 DO 113 J=1,NX
         X1(K4,J)=X1(K2,J)+2.*(X1(K3,J)-X1(K2,J))
  113 X(J)=X1(K4,J)
      IN=K4
      CALL SUMR
      IF (SUM(K4).LT.SUML) GO TO 126
      GO TO 128
  114 IF (SUM(K3).GT.SUMH) GO TO 116
      DO 115 J=1,NX
  115 X1(INDEX,J)=X1(K3,J)
  116 DO 117 J=1,NX
         X1(K4,J)=0.5*X1(INDEX,J)+0.5*X1(K2,J)
  117 X(J)=X1(K4,J)
      IN=K4
      CALL SUMR
      IF (SUMH.GT.SUM(K4)) GO TO 124
C
C     REDUCE SIMPLEX BY HALF IF REFLECTION HAPPENS TO PRODUCE A LARGER V
C     UE THAN THE MAXIMUM
C
      DO 118 J=1,NX
      DO 118 I=1,K1
  118 X1(I,J)=0.5*(X1(I,J)+X1(KOUNT,J))
      DO 120 I=1,K1
         DO 119 J=1,NX
  119    X(J)=X1(I,J)
         IN=I
         CALL SUMR
  120 CONTINUE
  121 SUML=SUM(1)
      KOUNT=1
      DO 122 I=2,K1
         IF (SUML.LT.SUM(I)) GO TO 122
         SUML=SUM(I)
         KOUNT=I
  122 CONTINUE
      SR(INF)=SQRT(SUM(KOUNT))
      DO 123 J=1,NX
  123 X(J)=X1(KOUNT,J)
      GO TO 130
  124 DO 125 J=1,NX
  125 X1(INDEX,J)=X1(K4,J)
      SUM(INDEX)=SUM(K4)
      GO TO 121
  126 DO 127 J=1,NX
         X1(INDEX,J)=X1(K4,J)
  127 X(J)=X1(INDEX,J)
      SUM(INDEX)=SUM(K4)
      SR(INF)=SQRT(SUM(K4))
      GO TO 130
  128 DO 129 J=1,NX
         X1(INDEX,J)=X1(K3,J)
  129 X(J)=X1(INDEX,J)
      SUM(INDEX)=SUM(K3)
      SR(INF)=SQRT(SUM(K3))
  130 ICONT=ICONT+1
      DO 131 J=1,NX
  131 X2(INF,J)=X(J)
      IF (ICONT.LT.2*K1) GO TO 146
      ICONT=0
      DO 132 J=1,NX
  132 X(J)=X1(K2,J)
      IN=K2
      CALL SUMR
      DIFFR=0.
      DO 133 I=1,K1
  133 DIFFR=DIFFR+(SUM(I)-SUM(K2))**2
      DIFFR=1./(K7*XNX)*SQRT(DIFFR)
      IF (DIFFR.GT.1.0E-14) GO TO 146
```

```
C
C     IF FLEXIBLE SIMPLEX METHOD FAILED TO SATISFY THE CONSTRAINTS WITHI
C     THE TOLERANCE CRITERION FOR THE CURRENT STAGE. THE SEARCH IS
C     PERTURBED FROM THE POSITION WHERE THE X VECTOR IS STUCK AND THEN
C     FEASBL IS REPEATED ONCE MORE FROM THE BEGINNING.
C
      IN=K1
      STEP=20.*FDIFER
      CALL SUMR
      SR(INF)=SQRT(SFOL)
      DO 134 J=1,NX
  134 X1(K1,J)=X(J)
      DO 143 J=1,NX
         FACTOR=1.
         X(J)=X1(K1,J)+FACTOR*STEP
         X1(L9,J)=X(J)
         IN=L9
         CALL SUMR
         X(J)=X1(K1,J)-FACTOR*STEP
         X1(L5,J)=X(J)
         IN=L5
         CALL SUMR
  135    IF (SUM(L9).LT.SUM(K1)) GO TO 136
         IF (SUM(L5).LT.SUM(K1)) GO TO 137
         GO TO 138
  136    X1(L5,J)=X1(K1,J)
         SUM(L5)=SUM(K1)
         X1(K1,J)=X1(L9,J)
         SUM(K1)=SUM(L9)
         FACTOR=FACTOR+1.
         X(J)=X1(K1,J)+FACTOR*STEP
         IN=L9
         CALL SUMR
         GO TO 135
  137    X1(L9,J)=X1(K1,J)
         SUM(L9)=SUM(K1)
         X1(K1,J)=X1(L5,J)
         SUM(K1)=SUM(L5)
         FACTOR=FACTOR+1.
         X(J)=X1(K1,J)-FACTOR*STEP
         IN=L5
         CALL SUMR
         GO TO 135
C
C     ONE DIMENSIONAL SEARCH BY GOLDEN SECTION ALONG EACH COORDINATE
C
  138    H(J)=X1(L9,J)-X1(L5,J)
         X1(L6,J)=X1(L5,J)+H(J)*R1A
         X(J)=X1(L6,J)
         IN=L6
         CALL SUMR
         X1(L7,J)=X1(L5,J)+H(J)*R2A
         X(J)=X1(L7,J)
         IN=L7
         CALL SUMR
         IF (SUM(L6).GT.SUM(L7)) GO TO 140
         X1(L8,J)=X1(L5,J)+(1.-R3A)*H(J)
         X1(L5,J)=X1(L7,J)
         X(J)=X1(L8,J)
         IN=L8
         CALL SUMR
         IF (SUM(L8).GT.SUM(L6)) GO TO 139
         X1(L5,J)=X1(L6,J)
         SUM(L5)=SUM(L6)
         GO TO 142
  139    X1(L9,J)=X1(L8,J)
         SUM(L9)=SUM(L8)
         GO TO 142
  140    X1(L9,J)=X1(L6,J)
         X1(L8,J)=X1(L5,J)+R3A*H(J)
         X(J)=X1(L8,J)
         IN=L8
         CALL SUMR
         STEP=SIZE
         SUM(L9)=SUM(L6)
         IF (SUM(L7).GT.SUM(L8)) GO TO 141
         X1(L5,J)=X1(L8,J)
         SUM(L5)=SUM(L8)
         GO TO 142
  141    X1(L9,J)=X1(L7,J)
         SUM(L9)=SUM(L7)
  142    IF (ABS(X1(L9,J)-X1(L5,J)).GT.0.01*FDIFER) GO TO 138
         X1(K1,J)=X1(L7,J)
         X(J)=X1(L7,J)
         SUM(K1)=SUM(L5)
         SR(INF)=SQRT(SUM(K1))
         IF (SR(INF).LT.FDIFER) GO TO 144
  143 CONTINUE
      ICHEK=ICHEK+1
      STEP=FDIFER
      IF (ICHEK.LE.2) GO TO 101
      FOLD=1.0E-12
      WRITE (6,164)
      WRITE (6,161)
      WRITE (6,162) (X(J),J=1,NX)
      WRITE (6,163) FDIFER,SR(INF)
      GO TO 157
  144 DO 145 J=1,NX
         X2(INF,J)=X1(K1,J)
  145 X(J)=X1(K1,J)
  146 IF (SR(INF).GT.FDIFER) GO TO 104
C
C     MODIFIED LAGRANGE INTERPOLATION FOR TIGHT INEQUALITIES
C
      IF (SR(INF).GT.0.) GO TO 159
      CALL PROBLEM (3)
      FINT=R(K9)
      DO 147 J=1,NX
  147 X(J)=X2(INF,J)
      CALL PROBLEM (2)
      DO 148 J=K7,K8
  148 R1(J)=R(J)
      DO 149 J=1,NX
  149 X(J)=X1(KOUNT,J)
      CALL PROBLEM (2)
      DO 150 J=K7,K8
  150 R3(J)=R(J)
      DO 151 J=1,NX
         H(J)=X1(KOUNT,J)-X2(INF,J)
  151 X(J)=X2(INF,J)+0.5*H(J)
      CALL PROBLEM (2)
      FLG(1)=0.
      FLG(2)=0.
      FLG(3)=0.
      DO 152 J=K7,K8
         IF (R3(J).GE.0.) GO TO 152
         FLG(1)=FLG(1)+R1(J)*R1(J)
         FLG(2)=FLG(2)+R(J)*R(J)
         FLG(3)=FLG(3)+R3(J)*R3(J)
  152 CONTINUE
      SR(INF)=SQRT(FLG(1))
      IF (SR(INF).LT.FDIFER) GO TO 159
      ALFA1=FLG(1)-2.*FLG(2)+FLG(3)
      BETA1=3.*FLG(1)-4.*FLG(2)+FLG(3)
      RATIO=BETA1/(4.*ALFA1)
      DO 153 J=1,NX
```

```
153 X(J)=X2(INF,J)+H(J)*RATIO
    IN=INF
    CALL SUMR
    SR(INF)=SQRT(SEQL)
    IF (SR(INF).LT.FDIFER) GO TO 156
    DO 155 I=1,20
       DO 154 J=1,NX
154    X(J)=X(J)-0.05*H(J)
       CALL SUMR
       SR(INF)=SQRT(SEQL)
       IF (SR(INF).LT.FDIFER) GO TO 156
155 CONTINUE
156 CALL PROBLEM (3)
    IF (FINT.GT.R(K9)) GO TO 157
    SR(INF)=0.
    GO TO 159
157 DO 158 J=1,NX
158 X2(INF,J)=X(J)
159 CONTINUE
    DO 160 J=1,NX
160 X(J)=X2(INF,J)
    RETURN
C
161 FORMAT (//108H IT IS NOT POSSIBLE TO SATISFY THE VIOLATED CONSTRAT
   1NT SET FROM THIS VECTOR. THE SEARCH WILL BE TERMINATED. /68H PLEAS
   2E CHOOSE A NEW STARTING VECTOR AND REPEAT SOLUTION AGAIN        )
162 FORMAT (//,63H THE VECTOR FOR WHICH THE CONSTRAINTS COULD NOT BE S
   1ATISFIED IS/,(8E16.6))
163 FORMAT (//,27H THE TOLERANCE CRITERION = E14.6,20X,49H THE SQUARE
   1ROOT OF THE CONSTRAINTS SQUARED IS = E16.6)
164 FORMAT (//,81H * * * * * * * SUBROUTINE  FEASBL  FAILS TO FIND A FEA
   1SIBLE POINT * * * * * * * *   )
C
    END
```

SUBROUTINE WRITEX

```
      SUBROUTINE WRITEX
C
C     THIS SUBROUTINE WRITE THE OBJECTIVE FUNCTION VALUE, DESIGN
C     CAPACITY VARIABLES AND THE EQUALITY AND INEQUALITY CONSTRAINT
C     VALUES AT THE DESIRED STAGE OF CALCULATION
C
      DIMENSION X(10), X1(10,10), X2(10,10), R(20), SUM(10), F(10), SR(1
     10), ROLD(20)
      COMMON /1/ NX,NC,NIC,STEP,ALFA,BETA,GAMA,IN,INF,FDIFER,SEQL,K1,K2,
     1K3,K4,K5,K6,K7,K8,K9,X,X1,X2,R,SUM,F,SR,ROLD,SCALE,FOLD,SIZE
      COMMON /2/ LFEAS,L5,L6,L7,L8,L9,R1A,R2A,R3A
      COMMON /PRN/ FDCW1,FDCW2,FDCW3,FDCH,FDIP1,FDIP2,FDIP3,FDISK,DCH,DC
     1W1,DCW2,DCW3,DIP1,DIP2,DIP3,DISK,CLHW
      COMMON /COS/ CA1,CA2,CCH1,CCH2,PI1,PI2,PI3,PIK,CT1,CT2,CT3,CD,CA33
     1,CTSK,RA1,RA2,RA3,RA4,PER1,PER2,DMCW1,DMCW2,DMIP1,DMIP2,WRM1,WRM2,
     2TWR,ZO,TOTAL
      COMMON /DPP/ NY,MS,CLFD,CL11,CL21,CL31,CL12,CL22,CL32,CL13,CL23,CL
     133,  AR1,CLAR2,CLAR3,CLP11,CLP12,CLP13,CLP22,CLP23,CLP33,TA1,TA2,T
     2A3,DI1,DI2,DI3,DMI1,DMI2,DMI3,OS,OA,ENK,CST,AD1,AD2,AD3,CON,PWR,AA
     3(12),AB(12),WR1(12),WR2(12),WR3(12),HF1(12),FR1(12),FR2(12),FR3(12
     4),RIN(75),PERD(12),IDENT,RMIX2,RMIX3,SSHT1,SSHT2,SSHT3,AC(12),AF(1
     52),IFAL
      COMMON /AY/ DGW1(75),DGW2(75),DGW3(75),CW1(75),CW2(75),CW3(75),AR1
     1(75),P11(75),P12(75),P13(75),P22(75),P23(75),P33(75),P34(75),C(75)
     2,SHRT1(75),SHRT2(75),SHRT3(75)
      CALL PROBLEM (3)
      WRITE (6,103) R(K9)
      WRITE (6,104) (X(J),J=1,NX)
      IF (NC.EQ.0) GO TO 101
      CALL PROBLEM (1)
      WRITE (6,105) (R(J),J=1,NC)
  101 IF (NIC.EQ.0) GO TO 102
      CALL PROBLEM (2)
      WRITE (6,106) (R(J),J=K7,K6)
  102 RETURN
C
  103 FORMAT (/,28H OBJECTIVE FUNCTION VALUE = E17.7)
  104 FORMAT (/,29H THE INDEPENDENT VECTORS ARE /(6E17.7))
  105 FORMAT (/,36H THE EQUALITY CONSTRAINT VALUES ARE /(6E17.7))
  106 FORMAT (/,34H THE INEQUALITY CONSTRAINT VALUES /(6E17.7))
C
      END
```

SUBROUTINE SUMR

```
      SUBROUTINE SUMR
C
C*****THIS SUBROUTINE COMPUTES THE SUM OF THE SQUARE VALUES OF THE
C     VIOLATED CONSTRAINTS IN ORDER TO BE COMPARED WITH THE TOLERANCE
C     CRITERION
C
      DIMENSION X(10), X1(10,10), X2(10,10), R(20), SUM(10), F(10), SR(1
     10), ROLD(20)
      COMMON /1/ NX,NC,NIC,STEP,ALFA,BETA,GAMA,IN,INF,FDIFER,SEQL,K1,K2,
     1K3,K4,K5,K6,K7,K8,K9,X,X1,X2,R,SUM,F,SR,ROLD,SCALE,FOLD,SIZE
      COMMON /2/ LFEAS,L5,L6,L7,L8,L9,R1A,R2A,R3A
      COMMON /PRN/ FDCW1,FDCW2,FDCW3,FDCH,FDIP1,EDIP2,EDIP3,EDISK,DCH,DC
     1W1,DCW2,DCW3,DIP1,DIP2,DIP3,DISK,CLHW
      COMMON /COS/ CA1,CA2,CCH1,CCH2,PI1,PI2,PI3,PIK,CT1,CT2,CT3,CD,CA33
     1,CTSK,RA1,RA2,RA3,RA4,PER1,PER2,DMCW1,DMCW2,DMIP1,DMIP2,WRM1,WRM2,
     2TWR,70,TOTAL
      SUM(IN)=0.
      CALL PROBLEM (2)
      SEQL=0.
      IF (NIC.EQ.0) GO TO 102
      DO 101 J=K7,K8
         IF (R(J).GE.0.) GO TO 101
         SEQL=SEQL+R(J)*R(J)
  101 CONTINUE
  102 IF (NC.EQ.0) GO TO 104
      CALL PROBLEM (1)
      DO 103 J=1,NC
  103 SEQL=SEQL+R(J)*R(J)
  104 SUM(IN)=SEQL
      RETURN
C
      END
```

SUBROUTINE PROBLEM (INQ)

```
      SUBROUTINE PROBLEM (INQ)
C
C     THIS SUBROUTINE EVALUATE OBJECTIVE FUNCTION AND CONSTRAINTS
C
      DIMENSION X(10), X1(10,10), X2(10,10), R(20), SUM(10), F(10), SR(1
     10), ROLD(20)
      COMMON /1/ NX,NC,NIC,STEP,ALFA,BETA,GAMA,IN,INF,FDIFER,SEQL,K1,K2,
     1K3,K4,K5,K6,K7,K8,K9,X,X1,X2,R,SUM,F,SR,ROLD,SCALE,FOLD,SIZE
      COMMON /PRN/ EDCW1,EDCW2,EDCW3,EDCH,EDIP1,EDIP2,EDIP3,EDISK,DCH,DC
     1W1,DCW2,DCW3,DIP1,DIP2,DIP3,DISK,CLHW
      COMMON /COS/ CA1,CA2,CCH1,CCH2,PI1,PI2,PI3,PIK,CT1,CT2,CT3,CD,CA33
     1,CTSK,RA1,RA2,RA3,RA4,PER1,PER2,DMCW1,DMCW2,DMIP1,DMIP2,WRM1,WRM2,
     2TWR,ZO,TOTAL
      COMMON /DPP/ NY,MS,CLED,CL11,CL21,CL31,CL12,CL22,CL32,CL13,CL23,CL
     133,CLAR1,CLAR2,CLAR3,CLP11,CLP12,CLP13,CLP22,CLP23,CLP33,TA1,TA2,T
     2A3,DI1,DI2,DI3,DMI1,DMI2,DMI3,OS,OA,ENR,CST,AD1,AD2,AD3,CON,PWR,AA
     3(12),AR(12),WR1(12),WR2(12),WR3(12),RF1(12),ER1(12),ER2(12),ER3(12
     4),RIN(75),PERD(12),IDENT,RMIX2,RMIX3,SSHT1,SSHT2,SSHT3,AC(12),AF(1
     52),IFAL
      COMMON /AY/ DGW1(75),DGW2(75),DGW3(75),CW1(75),CW2(75),CW3(75),AR1
     1(75),P11(75),P12(75),P13(75),P22(75),P23(75),P33(75),P34(75),C(75)
     2,SHRT1(75),SHRT2(75),SHRT3(75)
      GO TO (101,102,103), INQ
C
C     EQUALITY CONSTRAINTS
C
  101 CONTINUE
      GO TO 119
C
C     INEQUALITY CONSTRAINTS
C
  102 CONTINUE
      R(1)=X(1)-EDCW1
      R(2)=DMCW1-X(1)
      R(3)=X(2)-EDCW2
      R(4)=DMCW2-X(2)
      R(5)=X(3)-EDIP1
      R(6)=DMIP1-X(3)
      R(7)=X(4)-EDIP2
      R(8)=DMIP2-X(4)
      R(9)=X(1)+(1.0-PER1)*X(3)-WRM1
      R(10)=X(2)+(1.0-PER2)*X(4)-WRM2
      GO TO 119
C
C     OBJECTIVE FUNCTION
C
  103 CONTINUE
      RA1=(X(1)-EDCW1)/EDCW1
      RA2=(X(2)-EDCW2)/EDCW2
      RA3=(DCW3-EDCW3)/EDCW3
      DCW1=X(1)
      DCW2=X(2)
      DIP1=X(3)
      DIP2=X(4)
      DCH=(DCW1+DCW2+DCW3)/CLHW
      RA4=(DCH-EDCH)/EDCH
      CA[illegible]DYP
      IF (TOTAL-10.0E16) 105,104,105
  104 ZO=10.0E16
      GO TO 118
  105 IF (RA1-0.6) 106,106,107
  106 CA11=CA1
      GO TO 108
  107 CA11=CA2
  108 IF (RA2-0.6) 109,109,110
  109 CA22=CA1
      GO TO 111
  110 CA22=CA2
  111 IF (RA4-0.6) 112,112,113
  112 CH33=CCH1
      GO TO 114
  113 CH33=CCH2
  114 IF (RA3-0.6) 115,115,116
  115 CA33=CA1
      GO TO 117
  116 CA33=CA2
  117 ZO=CA11*(DCW1-EDCW1)+CA22*(DCW2-EDCW2)+CA33*(DCW3-EDCW3)+CH33*(DCH
     1-EDCH)+PI1*CT1*(DIP1-EDIP1)+PI2*CT2*(DIP2-EDIP2)+(PI3*CT3+CD)*(DIP
     23-EDIP3)+PIK*CTSK*(DISK-EDISK)
      ZO=ZO*FLOAT(NY)
  118 R(11)=ZO+TOTAL
      PRINT 120, TOTAL,ZO,R(11)
  119 RETURN
C
  120 FORMAT (10X, 6H TOTAL,E20.7, 3H ZO,E20.7, 11H TOTAL COST,E20.8)
C
C
      END
```

SUBROUTINE START

```
      SUBROUTINE START
C     THIS SUBROUTINE CALCULATE THE (N+1) VERTICES OF THE NEW POLYHEDRON
C     DURING THE SEARCH
C
      DIMENSION T(10,10)
      DIMENSION X(10), X1(10,10), X2(10,10), R(20), SUM(10), F(10), SR(1
     10), ROLD(20)
      COMMON /1/ NX,NC,NIC,STEP,ALFA,BETA,GAMA,IN,INF,FDIFER,SEQL,K1,K2,
     1K3,K4,K5,K6,K7,K8,K9,X,X1,X2,R,SUM,F,SR,ROLD,SCALE,FOLD,SIZE
      COMMON /2/ LFEAS,L5,L6,L7,L8,L9,R1A,R2A,R3A
      VN=NX
      STEP1=STEP/(VN*SQRT(2.))*(SQRT(VN+1.)+VN-1.)
      STEP2=STEP/(VN*SQRT(2.))*(SQRT(VN+1.)-1.)
      DO 101 J=1,NX
  101 T(1,J)=0.
      DO 103 I=2,K1
         DO 102 J=1,NX
  102    T(I,J)=STEP2
         L=I-1
         T(I,L)=STEP1
  103 CONTINUE
      DO 104 I=1,K1
      DO 104 J=1,NX
  104 X1(I,J)=X(J)+T(I,J)
      RETURN
C
      END
```

SUBROUTINE DYP

```
      SUBROUTINE DYP
C
C     THIS SUBROUTINE DOES THE OPERATIONAL STUDY FOR OPTIMAL ALLOCATION
C     OF WATER FROM SURFACE AND AQUIFER TO THREE DIFFERENT GROUNDWATER
C     SALINITY AREAS
C
      COMMON /PRN/ FDCW1,FDCW2,FDCW3,FDCH,FDIP1,FDIP2,FDIP3,FDISK,DCH,DC
     1W1,DCW2,DCW3,DIP1,DIP2,DIP3,DISK,CLHW
      COMMON /DPP/ NY,MS,CLFD,CL11,CL21,CL31,CL12,CL22,CL32,CL13,CL23,CL
     133,CLAR1,CLAR2,CLAR3,CLP11,CLP12,CLP13,CLP22,CLP23,CLP33,TA1,TA2,T
     2A3,DI1,DI2,DI3,DMI1,DMI2,DMI3,OS,OA,FNR,CST,AD1,AD2,AD3,CON,PWR,AA
     3(12),AR(12),WR1(12),WR2(12),WR3(12),RF1(12),FR1(12),FR2(12),FR3(12
     4),RIN(75),PERD(12),IDENT,RMIX2,RMIX3,SSHT1,SSHT2,SSHT3,AC(12),AE(1
     52),IFAL
      COMMON /COS/ CA1,CA2,CCH1,CCH2,PI1,PI2,PI3,PIK,CT1,CT2,CT3,CD,CA33
     1,CTSK,RA1,RA2,RA3,RA4,PER1,PER2,DMCW1,DMCW2,DMIP1,DMIP2,WRM1,WRM2,
     2TWR,70,TOTAL
      COMMON /AY/ DGW1(75),DGW2(75),DGW3(75),CW1(75),CW2(75),CW3(75),AR1
     1(75),P11(75),P12(75),P13(75),P22(75),P23(75),P33(75),P34(75),C(75)
     2,SHRT1(75),SHRT2(75),SHRT3(75)
      COMMON /SPX/ A(20,30),B(20),Z(30),XP(30),NO,NM,NM1,NM2,NM3,NM4,COS
     1T,IPHASE,IOP
      PRINT 176, DCW1,DCW2,DIP1,DIP2
      FRECH=0.25
      IDENT=0
      IOP=0
C
C     START LOOPS OF YEAR
C
      TOTAL=0.
      DGW1(1)=DI1
      DGW2(1)=DI2
      DGW3(1)=DI3
      DO 167 IY=1,NY
C
C     START LOOPS OF SUBPERIOD DURING A YEAR
C
         DO 166 J=1,MS
C
C     CONVERT YEAR,SUBPERIOD TO A SEQUENCE OF STAGES
C
            N=MS*(IY-1)+J
            SSHT1=0.0
            SSHT2=0.0
            SSHT3=0.0
            DO 101 I1=1,20
              B(I1)=0.0
            DO 101 J1=1,30
              A(I1,J1)=0.0
  101       CONTINUE
            DO 102 I1=1,30
              Z(I1)=0.0
  102       XP(I1)=0.0
            P33(N)=DISK
C
C     CHECK FEASIBILITY OF DIRECT RIVER DIVERSION
C     DIRECT RIVER DIVERSION IS THE OPTIMAL DECISION WHEN CANAL CAPACITY
C     AND SURFACE INFLOW CAN MEET THE DEMANDS AS LONG AS GROUND WATER
C     TABLE CONSTRAINTS NOT VIOLATED. ONCE GROUND WATER TABLE CONSTRAINT
C     ARE VIOLATED JOINT DECISION ON HOW MUCH WATER FROM EACH SOURCE TO
C     EACH AREA NEED TO BE DECIDED AND IN THIS CASE PUMPING WILL ALWAYS
C     BE NECESSARY. AND SINCE GROUND WATER TABLE IS MUCH MORE NEAR THE
C     SURFACE NOW THAT IT CAN REDUCE THE PUMPING COST
C
            DF1=RIN(N)-DCH
            IF (DF1) 103,103,104
  103       DPD=RIN(N)
            GO TO 105
  104       DPD=DCH
            DF2=DCH-(WR1(J)+WR2(J)+WR3(J)-DISK)/CLHW
            IF (DF2) 147,147,107
  105       DF3=RIN(N)-(WR1(J)+WR2(J)+WR3(J)-DISK)/CLHW
            IF (DF3) 106,106,107
  106       DPD=RIN(N)
            GO TO 147
  107       DPD=(WR1(J)+WR2(J)+WR3(J)-DISK)/CLHW
            P11(N)=0.
            P12(N)=0.
            P13(N)=0.
            P22(N)=0.
            P23(N)=0.
            P34(N)=0.
            SHRT1(N)=0.
            SHRT2(N)=0.
            SHRT3(N)=0.
C
C     CHECK DCW2 AND WR2(J)-PUMPING IS NECESSARY IF DCW2 IS LESS THAN
C     WR2(J)
C
            IF (DCW2-WR2(J)) 108,109,109
  108       CW2(N)=DCW2
            P22(N)=WR2(J)-DCW2
            GO TO 110
  109       CW2(N)=WR2(J)
            P22(N)=0.0
  110       IF (DCW1-WR1(J)) 111,112,112
  111       CW1(N)=DCW1
            P11(N)=WR1(J)-DCW1
            GO TO 113
  112       CW1(N)=WR1(J)
            P11(N)=0.0
  113       CW3(N)=WR3(J)-DISK
C
C     CHECK DEPTH TO WATER TABLES
C
            DC1=DGW1(N)-(CL11*CW1(N)+CL21*CW2(N)+CL31*CW3(N)-RF1(J)-(CLP
     1      11-1.0)*P11(N))*TA1-FR1(J)-DMI1
            DC2=DGW2(N)-(CL12*CW1(N)+CL22*CW2(N)+CL32*CW3(N)-(CLP22-1.0)
     1      *P22(N))*TA2-FR2(J)-DMI2
            DC3=DGW3(N)-(CL13*CW1(N)+CL23*CW2(N)+CL33*CW3(N))*TA3-FR3(J)
     1      -DMI3+(1.0-CLP33)*DISK*TA3
            IF (DC1) 147,147,114
C
C     CALCULATE ARTIFICIAL RECHARGE DIVERSION
C     IF WATER LOGGING CONSTRAINT VIOLATED, JOINT DECISION IS NECESSARY
C     IF WATERLOGGING CONSTRAINT NOT VIOLATED, ARTIFICIAL RECHARGE MUST
C     BE THE LEAST OF THE THREE BELOW
C     COMPARE AVAILABLE EXTRA DISCHARGE CAPACITY AT HEAD OF CANAL,
C     AVAILABLE EXTRA SURFACE WATER AND STORAGE SPACE IN THE AQUIFER
C     EXTRA CANAL CAPACITY FOR ARTIFICIAL RECHARGE
C
  114       UAR1=(DCW1-CW1(N))/CLHW
C
C     EXTRA RIVER FLOW FOR ARTIFICIAL RECHARGE
C
            UAR2=RIN(N)-(CW1(N)+CW2(N)+CW3(N))/CLHW
C
C     EXTRA AQUIFER SPACE FOR ARTIFICIAL RECHARGE
C
            UAR3=DC1/(TA1*CLHW)/(1.+CLAR1/CLHW)
            IF (UAR1) 121,121,115
  115       DU1=UAR1-UAR2
```

```
      IF (DU1) 118,118,116
116   DU2=UAR2-UAR3
      IF (DU2) 117,117,120
117   AR1(N)=UAR2
      GO TO 122
118   DU3=UAR1-UAR3
      IF (DU3) 119,119,120
119   AR1(N)=UAR1
      GO TO 122
120   AR1(N)=UAR3
      GO TO 122
121   AR1(N)=0.0
C
C     IF DIRECT RIVER DIVERSION FEASIBLE, NO CHECK ON WATER QUALITY NEED
C     CHECK ON RELATIVE WATER LEVEL CONSTRAINT BETWEEN AREA 1 AND 2
C
122   DT1=DC1+DMI1-CLAR1*AR1(N)*TA1
      DT2=DC2+DMI2-CLAR2*AR1(N)*TA2
      DT12=DT1-DT2
      IF (DT12) 125,125,123
123   IF (AR1(N)-UAR2) 124,128,124
124   P22(N)=(AC(J)+(DGW1(N)-(CL11*CW1(N)+CL31*CW3(N)-BF1(J))*TA1-
     1ER1(J))-(DGW2(N)-(CL12*CW1(N)+CL32*CW3(N))*TA2-ER2(J))+(CL22
     2*TA2-CL21*TA1)*WR2(J))/((1.0-CLP22)*TA2+CL22*TA2-CL21*TA1)
      IF (P22(N).LT.0.0) GO TO 126
      GO TO 139
125   DTA12=(DT2-DT1)-AC(J)
      IF (DTA12) 127,126,126
126   P22(N)=0.0
      GO TO 139
127   IF (AR1(N)-UAR2) 124,128,124
128   NO=5
      NM=6
      NM1=2
      NM2=1
      NM3=3
      NM4=0
      IPHASE=0
      A(1,1)=1.0
      A(2,1)=1.0
      A(2,3)=1.0/CLHW
      A(3,4)=-1.0
      A(3,5)=1.0
      A(4,2)=1.0
      A(4,3)=1.0
      A(5,1)=CLAR1
      A(5,3)=CL21
      A(5,4)=1.0/TA1
      A(6,1)=CLAR2
      A(6,2)=CLP22-1.0
      A(6,3)=CL22
      A(6,5)=1.0/TA2
      DDB=UAR1-UAR3
      IF (DDB) 129,129,130
129   B(1)=UAR1
      GO TO 131
130   B(1)=UAR3
131   B(2)=RIN(N)-(CW1(N)+CW3(N))/CLHW
      B(3)=AC(J)
      B(4)=WR2(J)
      B(5)=DGW1(N)/TA1-(CL11*CW1(N)+CL31*CW3(N)-BF1(J))-ER1(J)/TA1
      B(6)=DGW2(N)/TA2-(CL12*CW1(N)+CL32*CW3(N))-ER2(J)/TA2
      Z(1)=OA
      Z(2)=ENR*(DGW2(N)+28.0)
      Z(3)=OS
      CALL SIMPLEX
      IF (IPHASE-1) 138,132,138
132   P22(N)=WR2(J)
      CW2(N)=0.0
      UAR2=RIN(N)-(CW1(N)+CW2(N)+CW3(N))/CLHW
      DBB=UAR1-UAR3
      IF (DBB) 133,133,134
133   UARP=UAR1
      GO TO 135
134   UARP=UAR3
135   DUP=UAR2-UARP
      IF (DUP) 136,136,137
136   AR1(N)=UAR2
      GO TO 140
137   AR1(N)=UARP
      GO TO 140
138   AR1(N)=XP(1)
      P22(N)=XP(2)
      CW2(N)=XP(3)
      DT1=XP(4)
      DT2=XP(5)
      GO TO 141
139   CW2(N)=WR2(J)-P22(N)
140   DT1=DGW1(N)-(CL11*CW1(N)+CL21*CW2(N)+CL31*CW3(N)-BF1(J)+AR1(
     1N)*CLAR1)*TA1-ER1(J)
      DT2=DGW2(N)-(CL12*CW1(N)+CL22*CW2(N)+CL32*CW3(N)+CLAR2*AR1(N
     1)+(CLP22-1.0)*P22(N))*TA2-ER2(J)
141   DT3=DGW3(N)-(CL13*CW1(N)+CL23*CW2(N)+CL33*CW3(N)+(CLP33-1.0)
     1*DTSK+CLAR3*AR1(N))*TA3-ER3(J)
C
C     CHECK RELATIVE WATER LEVEL CONSTRAINT BETWEEN AREAS 2 AND 3
C
      DT23=DT2-DT3
      IF (DT23) 142,142,145
142   DTB23=(DT3-DT2)-AE(J)
      IF (DTB23) 144,143,143
C
C     CALCULATE DRAINAGE PUMPING RATE
C
143   P34(N)=0.0
      PDRA1=FRECH*(CL13*CW1(N)+CL23*CW2(N)+CL33*CW3(N)+(CLP33-1.0)
     1*DTSK+CLAR3*AR1(N)+ER3(J)/TA3)
      P34(N)=PDRA1
      GO TO 146
144   P34(N)=(AE(J)-(DT3-DT2))/TA3
      GO TO 146
145   P34(N)=(DT23+AE(J))/TA3
146   DT3=DT3+P34(N)*TA3
      C(N)=(OS*(CW1(N)+CW2(N)+CW3(N))+OA*AR1(N)+ENR*((DGW2(N)+DT2)
     1*0.5+28.0)*P22(N)+ENR*((DGW3(N)+DT3)*0.5+42.0)*P34(N))*CON
      GO TO 164
C
C     WHEN JOINT DECISION IS NECESSARY, INPUT COEFFICIENTS FOR PERFORM
C     THE SUBROUTINE SIMPLEX
C
147   NO=15
      NM=16
      NM1=10
      NM2=0
      NM3=6
      NM4=0
      MPP=NM+1
      NOM=NO+NM1
      IF (IFAL.EQ.1) IOP=1
      IPHASE=0
      A(1,1)=1.0
      B(1)=DCW1
      A(2,2)=1.0
      A(2,5)=CLFD
      B(2)=DCW2
      A(3,3)=1.0
```

```
      A(3,6)=CLED
      A(3,8)=CLED
      B(3)=DCW3
      A(4,1)=1.0
      A(4,2)=1.0
      A(4,3)=1.0
      DBB=DCH-RIN(N)
      IF (DBB) 148,148,149
148   B(4)=DCH*CLHW
      GO TO 150
149   B(4)=RIN(N)*CLHW
150   A(5,4)=1.0
      A(5,5)=1.0
      A(5,6)=1.0
      B(5)=DIP1
      A(6,7)=1.0
      A(6,8)=1.0
      B(6)=DIP2
      A(7,4)=PER1
      A(7,5)=-1.0
      A(7,6)=-1.0
      A(8,7)=PER2
      A(8,8)=-1.0
      A(9,13)=1.0
      A(9,14)=-1.0
      B(9)=AA(J)
      A(10,14)=1.0
      A(10,15)=-1.0
      B(10)=AB(J)
      A(11,1)=1.0
      A(11,4)=1.0
      A(11,10)=1.0
      B(11)=WR1(J)
      A(12,2)=1.0
      A(12,5)=CLED
      A(12,7)=1.0
      A(12,11)=1.0
      B(12)=WR2(J)
      A(13,3)=1.0
      A(13,6)=CLED
      A(13,8)=CLED
      A(13,12)=1.0
      B(13)=WR3(J)-P33(N)
      B(14)=(DGW1(N)-DMI1-FR1(J))/TA1+RF1(J)
      IF (B(14)) 152,151,151
151   A(14,1)=CL11
      A(14,2)=CL21
      A(14,3)=CL31
      A(14,4)=CLP11-1.0
      A(14,5)=-1.0
      A(14,6)=-1.0
      A(14,13)=1.0/TA1
      GO TO 153
152   B(14)=-B(14)
      A(14,1)=-CL11
      A(14,2)=-CL21
      A(14,3)=-CL31
      A(14,4)=1.0-CLP11
      A(14,5)=1.0
      A(14,6)=1.0
      A(14,13)=-1.0/TA1
153   B(15)=(DGW2(N)-DMI2-FR2(J))/TA2
      IF (B(15)) 155,154,154
154   A(15,1)=CL12
      A(15,2)=CL22
      A(15,3)=CL32
      A(15,5)=CLP12
      A(15,7)=CLP22-1.0
      A(15,8)=-1.0
      A(15,14)=1.0/TA2
      GO TO 156
155   B(15)=-B(15)
      A(15,1)=-CL12
      A(15,2)=-CL22
      A(15,3)=-CL32
      A(15,5)=-CLP12
      A(15,7)=1.0-CLP22
      A(15,8)=1.0
      A(15,14)=-1.0/TA2
156   B(16)=(DGW3(N)-DMI3-FR3(J))/TA3+(1.0-CLP33)*DISK
      IF (B(16)) 158,157,157
157   A(16,1)=CL13
      A(16,2)=CL23
      A(16,3)=CL33
      A(16,6)=CLP13
      A(16,8)=CLP23
      A(16,9)=-1.0
      A(16,15)=1.0/TA3
      GO TO 159
158   B(16)=-B(16)
      A(16,1)=-CL13
      A(16,2)=-CL23
      A(16,3)=-CL33
      A(16,6)=-CLP13
      A(16,8)=-CLP23
      A(16,9)=1.0
      A(16,15)=-1.0/TA3
159   DO 160 I=1,3
160   Z(I)=OS
      CON1=ENR*(DGW1(N)+28.0)
      Z(4)=CON1
      Z(5)=AD1*CON1
      Z(6)=AD2*CON1
      CON2=ENR*(DGW2(N)+28.0)
      Z(7)=CON2
      Z(8)=AD3*CON2
      Z(9)=ENR*(DGW3(N)+42.0)
      Z(10)=CST
      Z(11)=CST
      Z(12)=CST
      CALL SIMPLEX
      IF (IPHASE-1) 161,170,161
C
C     SOLUTION THROUGH JOINT DECISION BY SIMPLEX METHOD
C
161   C(N)=COST*CON
      CW1(N)=XP(1)
      CW2(N)=XP(2)
      CW3(N)=XP(3)
      P11(N)=XP(4)
      P12(N)=XP(5)
      P13(N)=XP(6)
      P22(N)=XP(7)
      P23(N)=XP(8)
      P34(N)=XP(9)
      SHRT1(N)=XP(10)
      SHRT2(N)=XP(11)
      SHRT3(N)=XP(12)
      AP1(N)=0.
      N1=N+1
      DGW1(N1)=XP(13)+DMI1
      DGW2(N1)=XP(14)+DMI2
      DGW3(N1)=XP(15)+DMI3
C
C     CHECK WATER SHORTAGES
C
```

```
          IF (SHRT1(N)-PWR*WR1(J)) 162,162,170
  162     IF (SHRT2(N)-PWR*WR2(J)) 163,163,172
  163     IF (SHRT3(N)-PWR*WR3(J)) 165,165,173
C
C     STATES TRANSFORMATION
C
  164     N1=N+1
          IFAL=0
          DGW1(N1)=DT1
          DGW2(N1)=DT2
          DGW3(N1)=DT3
C
C     ACCUMULATED TOTAL COST UP TP PERIOD N
C
  165     TOTAL=TOTAL+C(N)
  166   CONTINUE
  167 CONTINUE
C
C     CALCULATE DESIGN CAPACITY OF DRAINAGE TUBEWELL
C
      DIP3=0.
      NTP=NY*MS
      DO 169 I=1,NTP
        IF (DIP3-P34(I)) 168,169,169
  168   DIP3=P34(I)
  169 CONTINUE
      GO TO 175
  170 SSHT1=SHRT1(N)
C
C     IF SHORTAGE OF WATER REQUIREMENT GREATER THAN ALLOWABLE LIMIT,
C     ASSIGN A VERY LARGE COST TO ELIMINATE THIS DESIGN ALTERNATIVE
C
      IF (SHRT2(N)-PWR*WR2(J)) 171,171,172
  171 IF (SHRT3(N)-PWR*WR3(J)) 174,174,173
  172 SSHT2=SHRT2(N)
      IF (SHRT3(N)-PWR*WR3(J)) 174,174,173
  173 SSHT3=SHRT3(N)
  174 PRINT 177
      TOTAL=10.0E16
      IDENT=1
  175 RETURN
C
  176 FORMAT ( 16H DESIGN VARIABLE,4F12.2)
  177 FORMAT ( 28H CURRENT DESIGN NOT FEASIBLE)
C
      END
```

SUBROUTINE SIMPLEX

```
      SUBROUTINE SIMPLEX
C
C     THIS SUBROUTINE MINIMIZE THE COST OF OPERATION THROUGH THE JOINT
C     DECISIONS OF ALL THE OPERATIONAL VARIABLES BY USING THE STANDARD
C     SIMPLEX METHOD
C
C     VARIABLE LIST*****
C     1. M = NUMBER OF ORIGINAL EQUATIONS
C     2. NO= NUMBER OF ORIGINAL VARIABLES
C     3. A = VARIABLE COEFFICIENT MATRIX. DIMENSION FOR M+2 X NOVAR
C        ELEMENTS.
C     4. B = CONSTANT COLUMN VECTOR. DIMENSION FOR M+2 ELEMENTS.
C     5. M1 = NUMBER OF LESS-THAN-OR-EQUAL INEQUALITIES.
C     6. M2 = NUMBER OF GREATER-THAN-OR-EQUAL INEQUALITIES.
C     7. M3 = NUMBER OF EQUALITY STATEMENTS WITH POSITIVE B(I).
C     8. M4 = NUMBER OF EQUALITY STATEMENTS WITH NEGATIVE B(I).
C     9. NOVAR = TOTAL NUMBER OF VARIABLES IN REVISED STANDARD
C        CANNONICAL FORM.
C     10. BASIS = ARRAY CONTAINING J INDICES OF BASIS FOR ANY ITERATION.
C          DIMENSION FOR M ELEMENTS. ELEMENTS ARE INTEGERS.
C     11. R = PIVOT ROW NUMBER FOR ANY ITERATION. MUST BE AN INTEGER.
C     12. S = PIVOT COLUMN NUMBER FOR ANY ITERATION. MUST BE AN INTEGER.
C
      COMMON /SPX/ A(20,30),B(20),Z(30),XP(30),NO,M,M1,M2,M3,M4,COST,IPH
     1ASE,IOP
      DIMENSION BASIS(20)
      INTEGER R,S,BASIS
      M1234=M1+M2+M3+M4
      IF (M1234-M) 160,101,160
  101 NOVAR=NO+M1+M2
      MPLUS2=M+2
      M11=M1+1
      DO 102 I=1,20
        BASIS(I)=0
  102 CONTINUE
C
C     CONVERSION OF MATRIX A TO STANDARD CANNONICAL FORM
C
      NPLUS1=NO+1
      IF (M1) 103,107,103
C
C     FIRST, ADDITION OF SLACK VARIABLES TO M1 SUBMATRIX
C
  103 DO 106 I=1,M1
        K=I
        NPLUSK=NO+K
        A(I,NPLUSK)=1.0
        DO 105 J=NPLUS1,NOVAR
          IF (J-NPLUSK) 104,105,104
  104     A(I,J)=0.0
  105   CONTINUE
        IF (B(I)) 158,106,106
  106 CONTINUE
C
C     NEXT, CONVERT M2 SUBMATRIX
C
  107 IF (M2) 108,112,108
  108 M1PLUS1=M1+1
      M1M2=M1+M2
      DO 111 I=M1PLUS1,M1M2
        K=I
        NPLUSK=NO+K
        A(I,NPLUSK)=-1.0
        DO 110 J=NPLUS1,NOVAR
          IF (J-NPLUSK) 109,110,109
  109     A(I,J)=0.0
  110   CONTINUE
        IF (B(I)) 158,111,111
  111 CONTINUE
C
C     SUBMATRIX M3 NEEDS NO CONVERSION OR ARTIFICIAL VARIABLES
C
  112 IF (M4) 113,117,113
C
C     FINALLY, CONVERT M4
C
  113 M123P1=M1+M2+M3+1
      DO 116 I=M123P1,M
        DO 114 J=1,NO
  114   A(I,J)=-A(I,J)
```

```
      IF (B(I)) 115,115,159
115   B(I)=-B(I)
116   CONTINUE
C
C     CONSTRUCTION OF INFEASIBILITY EQUATION
C
117   MPLUS2=M+2
      DO 118 J=1,NOVAR
         A(MPLUS2,J)=0.0
      DO 118 I=M11,M
         A(MPLUS2,J)=-A(I,J)+A(MPLUS2,J)
118   CONTINUE
      B(MPLUS2)=0.0
      DO 119 I=M11,M
         B(MPLUS2)=-B(I)+B(MPLUS2)
119   CONTINUE
C
C     INSERTION OF COST FUNCTION COEFFICIENTS INTO MATRIX A
C
      MPLUS1=M+1
      DO 120 J=1,NO
         A(MPLUS1,J)=Z(J)
120   CONTINUE
      DO 121 J=NPLUS1,NOVAR
         A(MPLUS1,J)=0.0
121   CONTINUE
      B(MPLUS1)=0.0
C
C     PROBLEM SHOULD NOW BE IN THE STANDARD INFEASIBILITY FORM AND READY
C     FOR PHASE ONE OPERATIONS
C
      K=1
      K1=1
      DO 124 I=1,M
         IF (I-M1) 122,122,123
122      BASIS(I)=NO+K
         K=K+1
         GO TO 124
123      BASIS(I)=NOVAR+K1
         K1=K1+1
124   CONTINUE
C
C     BEGIN MAIN ITERATION PROCEDURE *** PHASE ONE ***
C     LOCATION OF MOST NEGATIVE REALTIVE PRICE.
C
      MP=M+2
      NO=NOVAR
      KOUNT=1
125   TEST=0.0
      DO 127 J=1,NO
         IF (A(MP,J)-TEST) 126,127,127
126      ABVALA=ABS(A(MP,J))
         IF (ABVALA.LE.0.001) GO TO 127
         TRAP=ABS(A(MP,J))-ABS(TEST)
         ABTRAP=ABS(TRAP)
         IF (ABTRAP.LE.0.00001) GO TO 127
         TEST=A(MP,J)
         S=J
C
C     S     THE PIVOT COLUMN NUMBER
C
127   CONTINUE
      IF (TEST) 128,149,128
C
C     DETERMINATION OF PIVOT ROW NUMBER
C     R IS THE PIVOT ROW NUMBER
C
128   DENOM=10.0E8
      DUM=0.0
      DO 132 I=1,M
         IF (A(I,S)) 132,132,129
129      EPSILON=ABS(B(I))
         IF (EPSILON.LT.0.00001) GO TO 130
         TEST=B(I)/A(I,S)
         IF (TEST.GE.DENOM) GO TO 132
         DENOM=TEST
         K=I
         GO TO 132
130      IF (A(I,S)-DUM) 132,132,131
131      DUM=A(I,S)
         KK=I
132   CONTINUE
      IF (DUM) 133,134,133
133   ABDUM=ABS(DUM)
      EPSILON=0.0001
      IF (ABDUM.LT.EPSILON) GO TO 134
      R=KK
      GO TO 136
134   IF (DENOM-10.0E8) 135,161,135
135   R=K
136   PIVOT=A(R,S)
      IF (PIVOT-1.0000) 137,140,137
137   DO 139 J=1,NOVAR
         IF (A(R,J)) 138,139,138
138      A(R,J)=A(R,J)/PIVOT
139   CONTINUE
      B(R)=B(R)/PIVOT
140   BASIS(R)=S
      DO 146 I=1,MP
         IF (I-R) 141,146,141
141      IF (A(I,S)) 142,146,142
142      DO 145 J=1,NO
            IF (J-S) 143,145,143
143         IF (A(R,J)) 144,145,144
144         A(I,J)=A(I,J)-A(I,S)*A(R,J)
145      CONTINUE
         B(I)=B(I)-A(I,S)*B(R)
146   CONTINUE
      DO 148 I=1,MP
         IF (I-R) 147,148,147
147      A(I,S)=0.0
148   CONTINUE
      A(R,S)=1.0
      K=0
      KK=0
      KOUNT=KOUNT+1
      GO TO 125
149   IF (MP-MPLUS2) 150,151,150
150   GO TO 155
151   IF (B(MP)) 152,153,153
152   TOL=0.001
      B(MP)=ABS(B(MP))
      IF (B(MP)-TOL) 153,153,154
C
C     BEGIN PHASE TWO
C
153   MP=M+1
      GO TO 125
154   PRINT 165
      GO TO 163
155   COST=-B(MP)
      DO 156 J=1,NO
156   XP(J)=0.0
      DO 157 I=1,M
         J=BASIS(I)
157   XP(J)=B(I)
```

```
      GO TO 164
  158 PRINT 167, I
      GO TO 163
  159 PRINT 168, I
      GO TO 162
  160 PRINT 169
      GO TO 162
  161 PRINT 166
  162 STOP
  163 IPHASE=1
  164 RETURN
C
  165 FORMAT (3X,42HTHERE IS NO FEASIBLE SOLUTION TO PHASE ONE)
  166 FORMAT (3X,31HOBJECTIVE FUNCTION IS UNBOUNDED)
  167 FORMAT (3X,39HIMPROPER PROBLEM FORMULATION. EQUATION ,I3,26HHAS NE
     1GATIVE B COEFFICIENT)
  168 FORMAT (3X,39HIMPROPER PROBLEM FORMULATION. EQUATION ,I3,26HHAS PO
     1SITIVE B COEFFICIENT)
  169 FORMAT (3X,60HIMPROPER PROBLEM FORMULATION. M1 +M2 +M3 +M4 NOT EQU
     1AL TO M.)
C
      END
&00                                                              01/07/74
&17                                                              01/07/74
```

APPENDIX E

RESULTS FROM DIFFERENT COMPUTER RUNS FOR LOWER JHELUM CANAL COMMANDED AREA

A. Input Data

1. General Data.

Items	Nonsaline area	Intermediate area	Saline area
Existing capacity at heads of watercourses (cfs)	2,154	819	819
Aquifer gross area (acres)	1,077,100	330,100	330,100
Size of tubewell (cfs)	4	4	6 (drainage well) 0.25 (skimming well)
Storage coefficient	0.16	0.16	0.16
Initial depth to water table (feet)	15	16.5	18
Minimum allowable depth to water table (feet)	10	10	10
Maximum allowable depth to water table (feet)	90	90	90
Dynamic head of pumping (feet)	28	28	42 (drainage well) 2 (skimming well)
Delivery efficiency from canal head to heads of watercourses	0.70	0.70	0.70
Delivery efficiency from canal head to heads of distributaries	0.85	0.85	0.85
Annual cost of canal from distributary heads to heads of watercourses (RS/cfs)			
a. remodeling ratio ≤ 0.6	2,119.82	2,119.82	2,119.82
b. remodeling ratio > 0.6	917.33	917.33	917.33
Annual cost of canal from canal head to heads of distributaries (RS/cfs)			
a. remodeling ratio ≤ 0.6	741.95	714.95	714.95
b. remodeling ratio > 0.6	321.44	321.44	321.44
Annual tubewell installation operational and maintenance cost (RS/cfs)	3,042	3,042	2,208
Annual cost of extra drainage works for salt water effluents (RS/cfs)	0	0	4,600
Annual cost of shortage (RS/AF)	177	177	177
Annual cost of canal operation and maintenance (RS/cfs)	0	0	0
Cost of energy for pumping (RS/AF/ft)	0.184	0.184	0.184

2. Monthly Water Requirements (cfs).

Month	Area 1 with 150% cropping intensity	Area 2 or 3 with 150% cropping intensity	Area 3 with 100% cropping intensity
Jan.	6,490	1,870	1,090
Feb.	4,670	1,340	930
Mar.	2,430	700	510
Apr.	3,510	1,000	760
May	5,540	1,590	1,220
Jun.	5,820	1,670	1,250
Jul.	2,980	850	610
Aug.	3,920	1,120	700
Sep.	5,360	1,540	960
Oct.	4,160	1,170	720
Nov.	5,220	1,450	860
Dec.	7,720	2,210	1,230

3. Limits on relative differences of water levels in areas 1 and 2 and areas 2 and 3, and natural recharge from rainfall and river in feet.

Month	Limit on relative water level difference between area 1&2, area 2&3	Natural recharge area 1	Natural recharge area 2 or 3
Oct.	0.50	0.003	0.001
Nov.	0.75	0.003	0.001
Dec.	1.00	0.003	0.001
Jan.	1.25	0.003	0.001
Feb.	1.50	0.003	0.001
Mar.	0.75	0.140	0.050
Apr.	0.50	0.030	0.010
May	0.25	0.050	0.020
Jun.	0.50	0.120	0.040
Jul.	1.00	0.100	0.030
Aug.	1.50	0.080	0.003
Sep.	0.00	0.010	0.003

4. River flows allocated to the model area - low flow condition.

Year	Month	Allocated river flows (cfs)	Year	Month	Allocated river flows (cfs)
1	Oct.	5,420	2	Apr.	7,080
	Nov.	3,990		May	6,380
	Dec.	5,040		Jun.	10,190
	Jan.	6,830		Jul.	11,150
	Feb.	5,530		Aug.	12,450
	Mar.	5,650		Sep.	6,230
	Apr.	8,910	3	Oct.	4,600
	May	8,560		Nov.	3,370
	Jun.	13,060		Dec.	4,350
	Jul.	14,070		Jan.	7,690
	Aug.	15,490		Feb.	6,310
	Sep.	7,950		Mar.	6,410
2	Oct.	5,850		Apr.	10,160
	Nov.	4,310		May	10,050
	Dec.	5,410		Jun.	15,030
	Jan.	5,560		Jul.	16,070
	Feb.	4,400		Aug.	17,570
	Mar.	4,540		Sep.	9,140

B. Results

1. RUN No. E.1 - Low river flow with 150% cropping intensity in three zones.

Total cost including fixed and operational cost for three year period = RS 166 millions.

a. Design capacity of the system.

Item	Zones		
	Nonsaline	Intermediate	Saline
Canal capacity at heads of water courses (cfs)	5,156	915	2,010
Remodeling ratio	2.39	1.12	2.45
Tubewell installed capacity (cfs)	6,058	1,650	1,045 (drainage well)
			300 (skimming well)

Total design capacity at head of the main canal = 11,544 cfs ; remodeling ratio = 2.13 .

b. Operational decisions.

Year	Month	Stage	CW1(k)	CW2(k)	CW3(k)	AR1(k)	P11(k)	P12(k)	P13(k)	P22(k)	P23(k)	P33(k)	P34(k)	DGW1(k)	DGW2(k)	DGW3(k)
1	Oct.	1	1,697	508	1,589	0	4,793	479	0	955	95	200	0	16.1	17.0	17.3
	Nov.	2	1,079	610	1,104	0	3,591	359	0	425	42	200	0	16.9	16.9	16.8
	Dec.	3	2,336	692	500	0	94	9	0	0	0	200	0	16.5	16.6	16.6
	Jan.	4	3,107	881	793	0	403	40	0	85	8	200	0	16.0	16.1	16.2
	Feb.	5	1,930	608	1,333	0	3,610	361	0	675	67	200	0	16.7	16.3	15.5
	Mar.	6	2,046	511	1,399	0	3,774	377	0	838	84	200	930	17.5	16.7	16.0
	Apr.	7	2,980	850	650	2,510	0	0	0	0	0	200	586	16.2	16.2	16.2
	May	8	3,960	1,073	920	113	0	0	0	47	0	200	195	15.3	15.6	15.8
	Jun.	9	5,156	898	1,287	0	204	20	0	625	62	200	514	14.3	15.5	15.5
	Jul.	10	4,160	1,170	970	1,422	0	0	0	0	0	200	831	13.0	14.8	15.8
	Aug.	11	5,156	1,450	1,250	0	64	0	0	0	0	200	408	11.9	13.9	15.4
	Sep.	12	3,139	526	1,900	0	4,581	458	0	1,295	129	200	0	12.7	14.7	14.4
2	Oct.	13	1,975	531	1,589	0	4,515	452	0	955	95	200	829	13.7	15.1	14.6
	Nov.	14	1,286	628	1,104	0	3,384	338	0	425	42	200	156	14.4	15.1	14.4
	Dec.	15	2,430	700	500	224	0	0	0	0	0	200	524	13.9	14.7	14.7
	Jan.	16	2,288	811	793	0	1,222	122	0	85	8	200	0	13.8	14.3	14.3
	Feb.	17	1,201	546	1,332	0	4,339	434	0	675	67	200	0	14.8	14.5	13.8
	Mar.	18	1,448	342	1,389	0	4,372	437	0	957	96	200	1,045	15.8	15.1	14.3
	Apr.	19	2,980	570	650	1,080	0	0	0	280	0	200	872	15.0	15.0	15.0
	May	20	2,748	816	903	0	1,172	117	0	205	20	200	0	14.7	14.6	14.5
	Jun.	21	4,965	882	1,287	0	395	40	0	625	62	200	866	13.8	14.6	14.6
	Jul.	22	4,160	1,170	970	1,422	0	0	0	0	0	200	831	12.5	13.8	14.8
	Aug.	23	5,156	1,450	1,250	0	64	0	0	0	0	200	208	11.4	13.0	14.5
	Sep.	24	2,212	264	1,884	0	5,507	550	0	1,478	148	200	0	12.6	14.0	13.6
3	Oct.	25	1,168	463	1,589	0	5,322	532	0	955	95	200	1,003	14.0	14.5	14.0
	Nov.	26	679	576	1,104	0	3,991	399	0	425	42	200	140	15.0	14.5	13.8
	Dec.	27	2,189	381	475	0	240	24	0	298	30	200	0	14.7	14.6	13.6
	Jan.	28	3,510	1,000	800	225	0	0	0	0	0	200	785	13.9	14.0	14.0
	Feb.	29	2,483	651	1,333	0	3,107	311	0	675	67	200	0	14.4	14.2	13.3
	Mar.	30	2,452	629	1,406	0	3,368	337	0	755	75	200	886	14.9	14.4	13.7
	Apr.	31	2,980	850	650	3,108	0	0	0	0	0	200	596	13.5	13.9	13.9
	May	32	3,920	1,120	920	1,765	0	0	0	0	0	200	172	12.2	13.2	13.4
	Jun.	33	5,156	898	1,287	0	204	20	0	625	62	200	514	11.1	13.1	13.1
	Jul.	34	4,160	1,170	970	472	0	0	0	0	0	200	814	10.1	12.4	13.4
	Aug.	35	3,236	915	1,036	0	1,984	0	198	535	53	200	0	10.0	12.4	12.7
	Sep.	36	3,907	915	1,576	0	3,813	0	381	1,295	129	200	794	10.4	13	12.6

2. RUN No. E.2 - Low river flow with 150% cropping intensity in the nonsaline and intermediate areas and 100% in the saline area.

Total cost including fixed and operational cost for the three year period - RS 144 millions.

a. Design capacity of the system.

Item	Zones		
	Nonsaline	Intermediate	Saline
Canal capacity at heads of watercourses (cfs)	6,050	1,241	1,050
Remodeling ratio	2.81	1.52	1.28
Tubewell installed capacity (cfs)	5,109	1,643	705 (drainage well)
			300 (skimming well)

Total design capacity at head of the main canal = 11,917 cfs ; remodeling ratio = 2.20

b. Operational decisions.

Year	Month	Stage	CW1(k)	CW2(k)	CW3(k)	AR1(k)	P11(k)	P12(k)	P13(k)	P22(k)	P23(k)	P33(k)	P34(k)	DGW1(k)	DGW2(k)	DGW3(k)
1	Oct.	1	2,090	867	837	0	4,400	440	0	629	63	200	0	15.9	16.5	17.6
	Nov.	2	1,130	940	722	0	3,539	354	0	99	10	200	0	16.8	16.0	17.3
	Dec.	3	2,430	323	310	664	0	0	0	377	0	200	34	16.1	16.1	17.2
	Jan.	4	3,243	977	560	0	266	27	0	0	0	200	0	15.6	15.6	16.9
	Feb.	5	3,090	0	781	0	2,450	113	132	1,494	149	200	0	15.8	16.9	16.4
	Mar.	6	2,023	919	1,014	0	3,797	380	0	429	43	200	52	16.5	16.7	16.0
	Apr.	7	2,980	850	410	2,853	0	0	0	0	0	200	499	15.1	16.2	16.2
	May	8	3,920	1,120	500	646	0	0	0	0	0	200	85	14.2	15.5	15.9
	Jun.	9	5,360	1,540	760	987	0	0	0	0	0	200	152	12.8	14.6	15.4
	Jul.	10	4,160	1,170	520	2,700	0	0	0	0	0	200	123	11.1	13.8	15.0
	Aug.	11	5,220	1,450	660	163	0	0	0	0	0	200	51	10.0	13.0	14.5
	Sep.	12	3,716	1,241	607	0	4,004	0	400	969	97	200	0	10.5	13.3	13.9
2	Oct.	13	2,368	891	837	0	4,122	412	0	629	63	200	0	11.3	13.3	13.5
	Nov.	14	1,337	958	722	0	3,333	333	0	99	10	200	0	12.1	12.8	13.2
	Dec.	15	2,430	700	310	496	0	0	0	0	0	200	37	11.5	12.4	13.0
	Jan.	16	2,424	908	560	0	1,086	109	0	0	0	200	0	11.3	11.8	12.8
	Feb.	17	1,216	874	990	0	4,324	432	0	349	35	200	0	12.4	11.6	12.4
	Mar.	18	2,179	59	939	0	3,641	364	0	1,301	130	200	0	13.1	12.7	11.9
	Apr.	19	2,980	836	410	1,043	0	0	0	14	0	200	483	12.2	12.2	12.2
	May	20	2,954	1,014	498	0	966	97	0	24	2	200	0	11.9	11.6	11.9
	Jun.	21	5,272	1,142	719	0	88	0	9	398	40	200	0	10.8	11.3	11.3
	Jul.	22	3,946	1,152	520	0	214	21	0	0	0	200	0	10.0	10.5	10.8
	Aug.	23	3,041	1,056	642	0	2,179	218	0	209	21	200	0	10.0	10.1	10.4
	Sep.	24	3,075	378	908	0	4,645	464	0	1,437	144	200	565	10.8	11.1	10.6
3	Oct.	25	1,845	562	812	0	4,644	464	0	913	91	200	705	11.9	11.5	11.0
	Nov.	26	1,356	335	669	0	3,314	331	0	723	72	200	363	12.6	11.9	11.1
	Dec.	27	2,419	346	280	0	11	1	0	353	35	200	0	12.2	12.0	11.0
	Jan.	28	3,510	1,000	560	447	0	0	0	0	0	200	696	11.4	11.5	11.5
	Feb.	29	2,448	979	990	0	3,092	309	0	349	35	200	0	11.9	11.2	11.0
	Mar.	30	2,865	639	984	0	2,955	296	0	780	78	200	228	12.2	11.5	10.7
	Apr.	31	2,980	850	410	3,096	0	0	0	0	0	200	503	10.8	10.9	10.9
	May	32	3,858	1,115	500	0	61.7	6	0	0	0	200	0	10.0	10.3	10.6
	Jun.	33	3,051	936	725	0	2,309	231	0	408	41	200	0	10.0	10.0	10.1
	Jul.	34	2,377	705	329	0	1,783	178	0	465	47	200	220	10.0	10.0	10.0
	Aug.	35	3,073	919	432	0	2,147	0	215	531	53	200	331	10.0	10.0	10.0
	Sep.	36	4,642	1,083	673	0	3,078	0	308	1,127	113	200	541	10.1	10.5	10.0

3. RUN No. E.3 - Low river flow with 150% cropping intensity in all three areas with storage coefficients at 0.25.

Total cost including fixed and operational cost for the three year period = RS 175 millions.

a. Design capacity of the system.

Item	Zones		
	Nonsaline	Intermediate	Saline
Canal capacity at heads of watercourses (cfs)	6,198	921	2,010
Remodeling ratio	2.88	1.12	2.45
Tubewell installed capacity (cfs)	5,429	2,259	1,342 (drainage well)
			300 (skimming well)

Total design capacity at head of the main canal = 13,040 cfs ; remodeling ratio = 2.41 .

b. Operational decisions.

Year	Month	Stage	CW1(k)	CW2(k)	CW3(k)	AR1(k)	P11(k)	P12(k)	P13(k)	P22(k)	P23(k)	P33(k)	P34(k)	DGW1(k)	DGW2(k)	DGW3(k)
1	Oct.	1	1,692	513	1,589	0	4,798	480	0	949	95	200	0	15.7	16.8	17.5
	Nov.	2	1,074	615	1,104	0	3,596	360	0	419	42	200	0	16.2	16.8	17.2
	Dec.	3	2,336	692	500	0	94	9	0	0	0	200	0	16.0	16.5	17.1
	Jan.	4	3,102	886	793	0	408	41	0	79	8	200	0	15.7	16.2	16.8
	Feb.	5	1,924	613	1,333	0	3,615	362	0	669	67	200	0	16.1	16.4	16.4
	Mar.	6	1,956	592	1,406	0	3,864	386	0	749	75	200	0	16.6	16.5	16.0
	Apr.	7	2,980	850	650	2,510	0	0	0	0	0	200	640	15.8	16.2	16.2
	May	8	3,920	1,120	920	330	0	0	0	0	0	200	270	15.2	15.8	16.0
	Jun.	9	5,360	1,540	1,340	1,197	0	0	0	0	0	200	343	14.2	15.1	15.6
	Jul.	10	4,160	1,170	970	2,911	0	0	0	0	0	200	666	13.0	14.6	15.6
	Aug.	11	5,220	1,450	1,250	1,397	0	0	0	0	0	200	680	12.1	14.0	15.5
	Sep.	12	3,134	531	1,900	0	4,586	459	0	1,289	129	200	0	12.6	14.5	14.9
2	Oct.	13	1,969	536	1,589	0	4,521	452	0	949	95	200	0	13.2	14.8	14.4
	Nov.	14	1,280	632	1,104	0	3,390	339	0	419	42	200	0	13.7	14.7	14.1
	Dec.	15	2,430	700	500	224	0	0	0	0	0	200	750	13.3	14.5	14.5
	Jan.	16	2,282	816	793	0	1,228	123	0	79	8	200	0	13.3	14.2	14.2
	Feb.	17	1,196	551	1,333	0	4,344	434	0	669	67	200	0	13.9	14.3	13.9
	Mar.	18	1,240	531	1,406	0	4,580	458	0	749	75	200	405	14.6	14.5	13.8
	Apr.	19	2,980	850	650	880	0	0	0	0	0	200	922	14.1	14.2	14.2
	May	20	2,742	821	903	0	1,178	118	0	199	20	200	0	14.0	14.0	13.9
	Jun.	21	4,959	887	1,287	0	400	40	0	619	62	200	852	13.3	13.9	13.9
	Jul.	22	4,160	1,170	970	2,506	0	0	0	0	0	200	1,342	12.3	13.4	14.4
	Aug.	23	5,220	1,213	1,250	1,397	0	0	0	237	0	200	944	11.3	13.0	14.5
	Sep.	24	2,785	0	1,576	0	4,935	188	305	2,050	205	200	0	11.9	14.2	13.9
3	Oct.	25	1,555	109	1,556	0	4,935	493	0	1,341	134	200	1,208	12.7	14.9	14.4
	Nov.	26	674	581	1,104	0	3,996	400	0	419	42	200	10	13.3	14.9	14.1
	Dec.	27	1,891	654	500	0	539	54	0	0	0	200	0	13.2	14.6	14.0
	Jan.	28	3,510	1,000	800	218	0	0	0	0	0	200	784	12.7	14.3	14.3
	Feb.	29	2,428	656	1,333	0	3,112	311	0	670	67	200	0	13.0	14.4	13.8
	Mar.	30	2,447	634	1,406	0	3,373	337	0	749	75	200	503	13.3	14.5	13.8
	Apr.	31	2,980	850	650	3,760	0	0	0	0	0	200	976	12.3	14.2	14.2
	May	32	3,920	1,120	920	1,821	0	0	0	0	0	200	296	11.4	13.7	14.0
	Jun.	33	5,360	1,540	1,340	1,197	0	0	0	0	0	200	343	10.4	13.1	13.6
	Jul.	34	3,594	872	949	0	566	57	0	249	25	200	0	10.0	12.8	13.1
	Aug.	35	2,925	921	1,010	0	2,294	0	229	529	53	200	85	10.0	12.8	12.8
	Sep.	36	3,902	921	1,576	0	3,818	0	382	1,290	129	200	850	10.3	13.2	12.7

4. RUN No. E.4 - High river flow with 150% cropping intensity for all three areas.

Total cost including fixed and operational cost for the three year period = RS 136 millions.

a. Design capacity of the system.

Item	Zones		
	Nonsaline	Intermediate	Saline
Canal capacity at heads of watercourses (cfs)	4,407	1,532	2,010
Remodeling ratio	2.04	1.87	2.45
Tubewell installed capacity (cfs)	4,198	1,374	981 (drainage well)
			300 (skimming well)

Total design capacity at head of the main canal = 11,356 cfs ; remodeling ratio = 2.20 .

b. Operational decisions.

Year	Month	Stage	CW1(k)	CW2(k)	CW3(k)	AR1(k)	P11(k)	P12(k)	P13(k)	P22(k)	P23(k)	P33(k)	P34(k)	DGW1(k)	DGW2(k)	DGW3(k)
1	Oct.	1	2,674	776	1,605	0	3,816	382	0	770	77	200	0	15.6	16.7	17.2
	Nov.	2	1,525	1,073	1,140	0	3,145	314	0	0	0	200	0	16.2	16.0	16.7
	Dec.	3	2,430	700	500	1,374	0	0	0	0	0	200	64	15.4	15.6	16.5
	Jan.	4	3,510	1,000	800	344	0	0	0	0	0	200	103	14.6	15.0	16.1
	Feb.	5	1,953	1,226	1,385	0	3,588	359	0	59	6	200	0	15.3	14.3	15.5
	Mar.	6	3,082	188	1,364	0	2,738	274	0	1,249	125	200	0	15.5	15.3	14.8
	Apr.	7	2,980	850	650	2,038	0	0	0	0	0	200	360	14.3	14.7	14.7
	May	8	3,920	1,120	920	695	0	0	0	0	0	200	153	13.3	14.1	14.3
	Jun.	9	4,407	1,450	1,339	0	953	95	0	9	0	200	0	12.6	13.2	13.5
	Jul.	10	4,160	925	970	352	0	0	0	245	0	200	830	11.7	12.7	13.7
	Aug.	11	4,406	1,223	1,250	0	813	0	0	227	0	200	642	10.7	12.2	13.7
	Sep.	12	3,904	800	1,918	0	3,816	382	0	1,085	108	200	0	11.1	12.7	12.7
2	Oct.	13	2,674	595	1,589	0	3,816	382	0	951	95	200	609	11.7	13.1	12.6
	Nov.	14	1,390	1,061	1,140	0	3,280	328	0	0	0	200	0	12.4	12.4	12.1
	Dec.	15	2,430	700	500	1,144	0	0	0	0	0	200	184	11.6	12.0	12.0
	Jan.	16	3,510	1,000	800	1,281	0	0	0	0	0	200	111	10.6	11.4	11.6
	Feb.	17	3,088	1,323	1,385	0	2,452	245	0	59	6	200	0	10.7	10.7	10.9
	Mar.	18	3,693	740	1,406	0	2,128	213	0	749	75	200	0	10.6	10.9	10.1
	Apr.	19	2,980	850	650	19	0	0	0	0	0	200	542	10.0	10.4	10.4
	May	20	2,270	846	909	0	1,650	165	0	134	13	200	40	10.0	10.0	10.0
	Jun.	21	3,018	934	1,089	0	2,342	0	234	606	61	200	651	10.0	10.0	10.0
	Jul.	22	2,337	697	775	0	1,823	0	182	473	47	200	455	10.0	10.0	10.0
	Aug.	23	3,020	908	1,017	0	2,200	0	220	542	54	200	569	10.0	10.0	10.0
	Sep.	24	4,407	1,414	1,661	0	3,313	0	331	796	80	200	949	10.0	10.0	10.0
3	Oct.	25	4,003	848	1,372	0	2,487	0	249	1,022	102	200	759	10.1	10.5	10.0
	Nov.	26	2,462	1,022	1,129	0	2,208	221	0	130	13	200	488	10.3	10.0	10.0
	Dec.	27	2,056	405	478	0	374	37	0	263	26	200	178	10.0	10.0	10.0
	Jan.	28	2,137	640	653	0	1,373	0	137	360	36	200	305	10.0	10.0	10.0
	Feb.	29	3,072	341	1,074	0	2,468	0	247	1,249	125	200	571	10.2	11.0	10.0
	Mar.	30	2,004	1,428	1,125	0	3,816	0	382	242	24	200	606	10.9	10.5	10.0
	Apr.	31	2,980	649	650	539	0	0	0	201	0	200	564	10.1	10.3	10.3
	May	32	2,562	894	785	0	1,358	0	136	226	23	200	160	10.0	10.0	10.0
	Jun.	33	3,017	934	1,089	0	2,342	0	234	606	61	200	651	10.0	10.0	10.0
	Jul.	34	2,337	697	775	0	1,823	0	182	473	47	200	455	10.0	10.0	10.0
	Aug.	35	3,020	908	1,017	0	2,200	0	220	542	54	200	569	10.0	10.0	10.0
	Sep.	36	3,904	1,015	1,584	0	3,816	0	383	1,195	120	200	981	10.4	10.[illegible]	10.1