EVALUATION OF SEEPAGE LOSSES IN UNLINED CANALS OF BURA IRRIGATION SCHEME

BY

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A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Land and Water Management of the University of Nairobi.

September 1991

DECLARATION

I, hereby, declare that this thesis is my original work and has not been submitted for a degree in any other university.

---------------------- 22/8/91
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This thesis has been submitted with our approval as university supervisors.

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DEDICATION

This work is dedicated to my mother Evangelina Magwe Nyagah and father Salvatore Nyagah for their enormous contributions in educating me and for invaluable encouragement and concern to date.

ACKNOWLEDGEMENT

I am grateful to Dr. F. N. Gichuki and Mr. K. J. Lenselink for their invaluable encouragement, guidance, critical comments and for enthusiastically devoting their valuable time at all stages of this study. I am grateful to the Small Scale Irrigation Development Project (SSIDP) for the scholarship accorded me through the Ministry of Agriculture. I am greatly indebted to Dr. W. Scheltema and Mr. H. K Mwathe for providing me with transport which was intergral to the data collection in the field. Without their assistance and cooperation, this study would not have been possible.

I express my gratitude to the management of Bura Irrigation Settlement Project (BISP) for the provision of facilities that were necessary for this study. I am indebted to the technical and subordinate staff of BISP for their invaluable assistance in one way or the other. Special gratitude goes to Messrs P. N. Macharia and J. Mutisya. My gratitude go to the technical staff of the department of Agricultural Engineering (Kabete Campus) for their assistance. In particular, I am grateful to Messrs D.N. Kabuthi and Z. G. Wanyoike for their assistance in fabricating and testing the seepage meter.

I express my gratitude to Irrigation and Drainage Branch Secretaries B. C. Koske and M. K. Kinai for the diligence and devotion in typing this report.

I am very grateful to my wife Teresa and our children, Anne and David for their patience, tolerance, encouragement and prayers during the time of study when I was away from them. I lack words to express my gratitude to them for the great sacrifice, perseverance and support they accorded me, without which I could not' have been able to complete this study. Finally, of all those who helped me in one way or another, their assistance was highly appreciated and to them I say a profound "Thank You"

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ABSTRACT

Evaluation of seepage losses in unlined canals in Bura Scheme.

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The research project addresses itself to the evaluation of seepage losses in unlined canals in the scheme. The objectives of the study, were; to evaluate seepage losses in the conveyance and distribution system, investigate and analyse the factors influencing seepage losses; and develop management and operational strategies for improvement of conveyance efficiency in the supply and distribution systems of the scheme.

Seepage evaluation was done in the major conveyance and distribution systems of the scheme, using the inflow-outflow, seepage meter and ponding methods.

Seepage losses in the supply canal obtained by inflow-outflow and seepage meter methods are 22.6 l/s/km (1.1 %/km) and 9.49 l/s/km (0.38%/km), respectively. These results represent 39.8% and 14.0% loss of the inflow discharge, respectively, in the supply canal, which is about 36.8 km long. Results reported from studies on seepage, elsewhere, indicate that seepage losses in the supply canal range from 3 to 86% of the inflow discharge.

Evaluation in the night storage reservoirs indicate that seepage losses are 0.39 and 0.28 cm/day on average. Results obtained from other studies on general soils, report seepage to range from 20 to 58 cm/day.

Assessment of effects of seepage on ground water rise indicate that the rise range from 0.03 to 0.01 m/year. Thus, it would take the ground water table (estimated at less or equal to 30 m from ground surface) about 1,000 to 3,000 yars to reach the ground surface. The only adverse effect to be caused by rising ground water table, in the short run, would be expected to occur only in those sections overlying perched water table.

Lining considerations on the conveyance and distribution systems and the consequent unit cost of lining were assessed. Seepage losses were evaluated to be low. Lining option could be undertaken if cost of lining is less than the unit cost of water lost through seepage, reported, as Kshs. 2.05 per metre square.

Improvement on water use efficiency could be acheived in Bura Irrigation Settlement Scheme through better operation and maintenance programme in the supply and distribution system.

LIST OF SYMBOLS

ASAL - Arid and Semi-Arid Lands

BISP - Bura Irrigation and Settlement Projectb

cumccs - Cubic metres per second

Eo - Open water cvaporation

ETo - Referencc crop evaporation

GoK - Government of Kenya

GIR - Gross irrigation requirements

ha - Hectares

ILACO - International Land Reclamation Company

l/s - Litres per second

L - Length

l/s/km - Litres per second per kilometre

K - Hydraulic conductivity proportional factor (m/day)

m/day - Metres per day

NIB - National Irrigation Board

NSPU - Night Storage Pumwani Branch

NSR - Night storage reservoir

ppm - Parts per million

v - Velocity

T - Time

%/km - Percentage per kilometre

1. INTRODUCTION

1.1 Background information

Water is an important resource for agricultural development. The demand for this vital resource continues to rise with the increasing agricultural production required to meet the food supply for the rising population. Increase in agricultural land in a country such as Kenya is possible through development of the Arid and Semi-Arid Lands (ASAL) which comprise about 80 percent of the total area of the country. Significant development of the ASAL areas for agricultural production is possible through irrigated agriculture. Since water supply is limited, available water for irrigation needs to be used efficiently.

Water supply for irrigation in Bura irrigation scheme is by pumping from an intake pond fed directly by the Tana River. Irrigation water is then conveyed by a supply canal that stretches for about 40 kilometres downstream from the intake. The supply canal feeds a number of branch canals. Water is further distributed to block feeders, which obtain the additional water from night storage reservoirs. The block feeders supply water to unit feeders, from where the water is siphoned to the furrows which apply water to the root zone of the crop. In Bura irrigation scheme, the conveyance, storage and distribution network comprise of 40 kilometers of main canal, 29 kilometers of branch canals, 19 night storage reservoirs and 78 kilometers of feeder canals. With such an enormous total wetted perimeter the potential for seepage losses is high.

1.2 Importance of the study

Conservation of water supplies is becoming increasingly important as the demand for this vital resource continues to rise rapidly and new sources of supply become scarcer and more expensive to develop.

In irrigation development, adoption of the most efficient water conveyance and application systems is therefore receiving more attention.

Since water conveyance in Bura scheme is by unlined channels, it is inevitable that water losses are incurred through seepage. Seepage losses are directly proportional to the wetted perimeter of the conveying channel. In Bura Irrigation Scheme, the operation and maintenance cost for pumping and desilting is estimated to be close to one million Kenya Shillings per month (Sang, 1989). With such enormous expenses, it is imperative that the water is used efficiently. It is therefore, necessary to compare actual losses with design values used in design assumptions of conveyance losses in the scheme. This will enable a statement to be made if changes need to be incorporated in the water delivery schedules.

Seepage losses from the canals recharge the soil above any impermeable layer. This has the effect of raising the water table. Where the impermeable layer is near the ground surface, this water table rise, soon results in waterlogging the area traversed by the canal. If the impermeable layer is very deep seepage losses continue to recharge this layer without evidence of waterlogging being noticed on the surface initially. Though no waterlogging is evident on the areas traversed by the supply and branch canals in the scheme, understanding of the level of seepage losses is important so that improvement to minimize seepage could be instituted to prevent waterlogging of the land in later years of scheme operation. During project investigation stage, water table was not reached after soil augering at 30 m depth from ground surface.

The information generated by the study will give an indication on the efficiency with which the water resource is being utilized. The information on seepage losses furnished by the study will offer the basis on which recommendations for the improvements can be done if levels of seepage are found to be high.

1.3 Objectives

The objectives of the research project are:

1. Evaluation of seepage losses in the supply canal and the major distribution system;

2. Investigations and analysis of the factors influencing seepage losses in the supply canal and major distribution system; and

3. Developing management and operational strategies for the improvement of conveyance efficiency of the water supply and distribution system of the scheme.

1.4 Scope

As the scheme area is 2500 ha, an intensive investigation of seepage that covers the whole conveyance and distribution system, and addressing in detail the various factors influencing seepage, was not feasible. This was partly due to transport problems and time constraints, and partly to limitations in the methods used in seepage determination.

This study addresses itself to the seepage evaluation at five flow measuring points along the supply canal, one branch canal, and two night storage reservoirs. Seepage evaluation was done using the inflow-outflow method and seepage meter for the conveyance system and by ponding method for the night storage reservoirs.

2. LITERATURE REVIEW

2.1 General view

Different phenomena cause different types of channel losses. While most losses will occur when the channels are flowing steadily, a portion are attributable to transient phenomena.

Channel losses can be divided into various broad categories as shown in Table 1.

Table 1: categories of water losses

I.	Steady state losses
A)	Seepage into the bed and banks
i)	Normal infiltration into bed and bank soils
ii)	Excess seepage into bank holes and cracks
B)	Visible leakage through and over the banks
i)	Overtopping
ii)	Leakage through the banks
iii)	Leakage through closed outlets
C)	Evaporation from the water surface
II.	Transient Losses
A)	Initial seeapge into dry banks in excess of normal long-term seepage rates.
B)	Dead storage
C)	Short term leakage
i)	Bank washouts and breaches
ii)	Outlet breaks.
III.	Wastage
A)	Purposeful wastage due to lack of need
B)	Malicious wastage

Source: Trout and Kemper, 1980.

It has been estimated that 1/4 to 1/3 of all the water diverted for irrigation purposes is lost in conveyance. US Bureau of Reclamation record from 46 irrigation projects show that losses range from 3 to 86 % (Lauritzen and Terrell, 1960) as mentioned by Hagan et al. (1967). It is argued that if only 1/5 of the total water diverted for irrigation purposes in USA is lost to the user, the quantity seeping from canals would be 27.2 billion m^3/year. If the intended users could retain this water, they would be able to irrigate 2.2 million additional hectares, using 1220 mm per year. Experience from other parts of the world have documented the problems posed by seepage on the availability of water for increased agricultural production. Thus, seepage is a serious economic loss when water lost is not recoverable for irrigation or other uses (Hansen et al. 1979). In Kenya, about 55.0% of the irrigated area is served by open channels of which less than 1% of the total length of the channels is lined.

2.2 Theoretical aspects of seepage

Seepage refers to the process of water movement from a canal into and through the bed and wall material (Kraatz, 1977). Knowledge of seepage rates is required for economic evaluation and design of conveyance and distribution channels in irrigation schemes. Evaluation and redress to seepage losses in irrigation schemes ensures efficient use of the limiting water resources necessary for the development of part of the arid and semi-arid lands of this country. The theoretical aspects of seepage are thus elucidated below.

2.2.1 Soil water potential

Soil water potential is defined as the work required to transfer a unit quantity of water from a standard reference state, where the potential is taken zero, to the situation where the potential has the defined value (Groenevelt and Kijne, 1971). Potential can be taken to have energy status connotation and thus indicates the availability of soil water. The lower the potential, the lower is the availability of the water.

Total potential energy of water, Yt, is made up of the matric potential, Ym,under unsaturated flow conditions, and pressure potential Yp under-saturated conditions the osmotic potential Yo and the gravitational potential Yg. But for normal groundwater flow, related to seepage, the osmotic potential can be ignored. Thus

$$Yt = Yp + Yg \qquad (1a)$$

$$Yt = Ym + Yg \qquad (1b)$$

Equations 1a and 1b are for saturated and unsaturated conditions, respectively.

2.2.2 Equation of flow

For flow of water in the soil to occur between two places a driving force is necessary as expressed in the equation of flow.

The difference in total potential energy of water between two locations in the soil is the driving forces from the point of high to the low potential location. The basic general equations describing this flow is given as:

$$q = - k*dh/ds \qquad (2)$$

where q is the flow under unsaturated flow (cm/sec), k is the hydraulic conductivity (cm/sec), dh is difference in hydraulic potential between two points separated by a distance ds where s is measured along the direction of flow (Hillel, 1973).

2.2.3 Seepage process

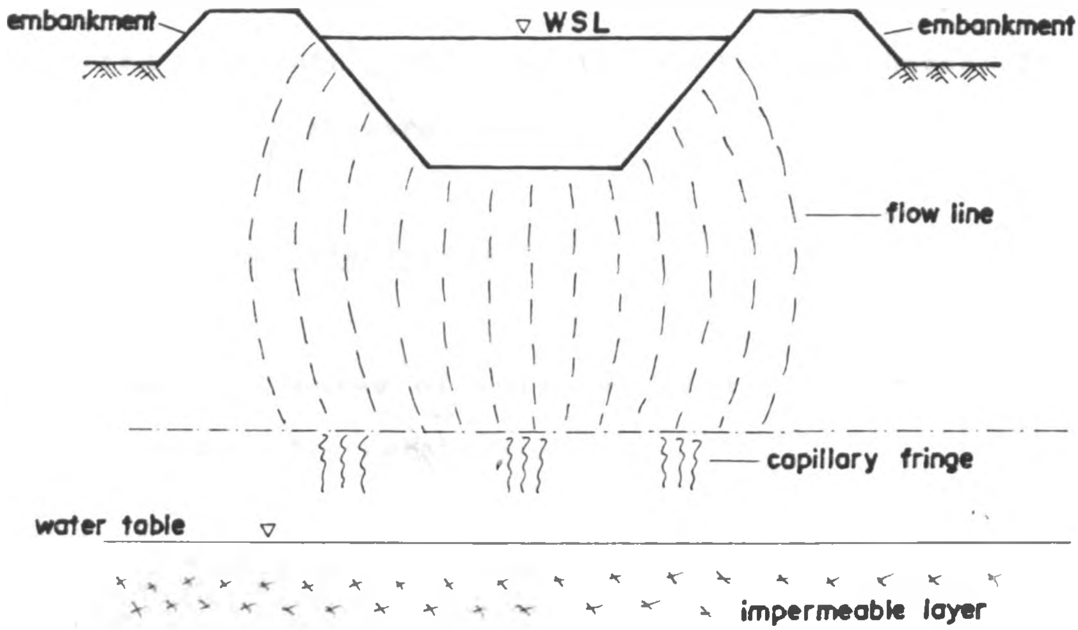
Seepage loss in irrigation water conveyance systems form the major portion of loss in canal (Sharma, 1984). The loss of water from the canal by seepage is illustrated in Figure 1. The movement of water within the soil may occur by the filling up of the pores within the limits of the water bearing horizon, often referred to as percolation; or it may occur by downward motion through the aeration zone, often referred to as infiltration [Denisov as reported by Ali (1986)].

There are many terms used to express amount of seepage; the following are commonly used, namely;

1) Volume per unit area of wetted perimeter per day (m^3/m^2 /day). This can be related to soil type and results extrapolated for similar soil conditions.
2) Volume per unit length of canal/day (m^3/m/day)
3) Percentage of total flow per km of canal.

The first term is most applicable in seepage determination by ponding method on a canal section with regular dimensions.

The second and the third terms are used in seepage expression on an operational canal system. The latter expressions allows comparisons on seepage for various sections of the canal system for purposes of seepage control measures.



source Wesseling (1972)

Figure 1: Loss of water due to seepage

2.3 Factors influencing seepage

The survey of the soil profile along the canal has been reported by Kraatz (1977) as the most important single technical step in the investigation of seepage more so during the pre-construction stage of a project. This survey is made with the objective of determining the location, extent and physical characteristics of the various underlying soil layers. The sequence of permeable and impermeable strata in the area traversed by the canal and the capability of these strata to transmit water mainly determine the magnitude of water lost by seepage. Factors influencing seepage have been reported by Kraatz (1977), Gupta (1983), and Sharma (1984) to include those discussed below.

2.3.1 Soil characteristics

Two attributes of soil physical properties are soil texture and soil permeability:

Soil texture refers to the relative proportion of sand, silt and clay on the soil. The soil texture forms the basic matrix and the geometry of voids created in this soil matrix is dependent of the class of soil texture. The soil texture therefore influences considerably the phases (water and air) contained in the spaces in the soil matrix. A coarse textured red soil will allow the flow of water more readily than a fine textured soil. Soil textural characteristics thus influence seepage from the canal.

Soil permeability and hydraulic conductivity:

Permeability and conductivity are frequently used interchangeably (Hansen et al., 1979). Permeability in a quantitative term is the characteristic of a pervious medium relating to the readiness with which it transmits fluids (Michael, 1978). Hydraulic conductivity is the proportionality factor k in Equation (2). The values of k depend on the properties of the fluid as well as those of the soil, and they reflect any interactions of the fluid with the porous medium, such as the swelling of a soil. A soil that has a high porosity and a coarse open texture has a high saturated hydraulic conductivity value. For two soils of the same total porosity, the soil with the smaller pores has the lower conductivity because of the resistance to flow in small pores. Thus, the soil characteristics which affect k are the total porosity, the distribution of pore size, and the tortuosity (the pore geometry of the soil). The fluid attributes which affect the conductivity are fluid density and viscosity (Hillel, 1973). If the soil surrounding the canal contains different layers of differing permeability, the flow (seepage) is governed by the least permeable layer.

2.3.2 Design and Operation

The loss of water by seepage depends on the length of the water course, its wetted perimeter and the intrinsic permeability of the strata through which the channel passes. Kruse et al. edited by Jensen (1983) have emphasised that unlined channels should be well designed and maintained to minimize water loss by seepage and to allow efficient irrigation. Aspects of design influencing seepage in the canal are discussed below.

Discharge influences seepage in the canal through effects of wetted perimeter of the section and the water depth in the canal.

Velocity: The slope of the canal influences flow velocity and must be low enough so that velocity does not cause scouring. The maximum velocity is determined by the erosivity of the canal bed material while the minimum velocity is determined by the need to discourage siltation and weed growth. Weed growth increases the flow resistance and increased flow depths and wetted perimeter, hence increasc the seepage loss of the channel.

Wetted perimeter: The cross-section of a channel which carries large flows should be such that it will give the largest possible hydraulic radius under the natural surface conditions. Theoretically, the most efficient cross-section for an open channel is a semi-circle, as the wetted perimeter would be minimum and its hydraulic radius maximum. However, trapezoidal shape are much more common for large flows, thus d/b ratio increases hence raising wetted perimeter.

Depth of water: Bouwer (1962) noted that water depth affects seepage not only through its effects on pressure head, but also through the wetted perimeter. Bouwer (1965), as reported by Ali (1986), demonstrated that the efficiency of a canal with uniform flow for conveying water increases with increasing water depth. Bouwer (1962) mentioned that the hydraulic gradients under which prolonged seepage flow takes place are made up ,of pressure (water depth) and elevation differences between the canal bottom and the groundwater or some other surface of lower potential at a certain discharge under the canal. When the difference is five times or more the surface width of the canal, seepage losses reach the upper limit or the infinity condition. Trout and Kemper (1980) have remarked that if bed and bank soil are fairly homogeneous, the loss rate should increase as flow depth increases, only because of larger wetted perimeter length and pressure head (wetted depth).

Flow duration: When canal water infiltrates into the soil and percolates to join the groundwater, the seepage water accumulates in the soil raising the groundwater table. As the time during which the canal is in operation increases the initial water content of the soil below the canal increases resulting in a reduced infiltration rate. Hence, the longer the period the canal is in operation the smaller the initial seepage rate when the water flows in the canal.

2.3.3 State of water

The condition of water conveyed influences the seepage rate of the canal. Three conditions of water have an attribute to seepage. These are sediment load, temperature and groundwater level.

Sediment load: Material suspended in canal water is carried by seepage water into the pores of the soil in which the canal is constructed. If the water contains considerable amounts of suspended material, the seepage rate may be reduced in a relatively short time. Even small amounts of sediment will have sealing effects over a period of time. If the velocity is reduced, the sediment carrying capacity of the water decreases, resulting in settlement of part of the suspended material. This forms a thin, slowly permeable layer along the wetted perimeter of the canal, which decreases the seepage. Examples of this effects are:

(i) For the Inter-State canal, Nebraska, it was estimated that a slight mud content of a maximum of 1000 ppm would reduce the overall seepage by 20 percent (Kraatz, 1977).

(ii) It is recorded by Kraatz (1977) that in 17 km long canal of Donzere-Mondragon, France, seepage losses immediately after construction amounted to 16 m^3/s but were reduced to 3 m^3/s (18.8%) within five years of operation by natural sealing effect of the silt-laden water of the Rhone.

Temperature affects seepage due to its effect on soil and water with the resultant change in viscosity. Bouwer and Rohwer (1959) as reported by Ali (1986) have suggested that changes in vapour pressure with temperature may affect the seepage rate, since vapour pressure changes rapidly with temperature, air bubbles would be expected to expand and contract especially within soil material hence changing the effective porosity. Bouwer and Rohwer (1959), as reported by Ali (1986), have further stated that, although seepage tends to vary inversely with temperature, seepage should not be corrected for viscosity changes due to temperature for the purpose of comparison with other data.

Groundwater level: Seepage rates in canals are affected by the groundwater level in the soil formation in which the canal is constructed. When the water table is above the bed of the canal the percolation of water from canal is reduced. When the groundwater level is above that flow line the ground water flow from the perimeter into the canal, resulting in seepage gain. When the water table is shallow, the seepage from the sides is greater than from the bed, and the reverse is true with a deep water table. In all cases the maximum seepage losses occur at the toe of the slope, i.e at the junction of the bed and sides of the canal. The significant depth within which the nature of the soil affects seepage losses has been found to be five times the bed width of canal. The effect of seepage losses on the original water table is insignificant (Bouwer, 1962). With a deep groundwater table, the seepage losses will be larger than with a high water table in the soil, but the higher water table will have a much more severe effect on the top soil and the crops (Nugteren, 1971).

2.4 Determination of seepage losses

The seepage losses can be determined either by direct measurement or by indirect (estimation) from the relevant hydraulic properties of the soil and the boundary condition, such as depth to groundwater, canal cross-section and water depth.

Direct methods are mainly employed for post-construction seepage measurements. The objectives are;

(a) to determine seepage losses from unlined canals and to locate reaches with excess seepage as a basis for lining considerations.

(b) to check seepage losses in completed reaches of a canal system under construction with aim of predicting seepage rates in the uncompleted parts of the system and adapting the design to the findings.

(c) to record seepage rates on lined or unlined canals as comparative data for the planning and design of other irrigation projects.

(d) to determine the exact amount of water conveyed in the canal system in order to operate the system properly.

Indirect methods are normally used in pre-construction period and provide a rough estimate of the possible extent of seepage losses for evaluation of economic benefits of lining the canal.

2.4.1 <u>Direct measurement</u>

Various methods are used for direct measurement of seepage in canals. These methods are reviewed below.

2.4.1.1 Inflow-outflow method

The inflow-outflow method consists of measuring the water flowing into and out of a section of canal being tested as in figure 2.

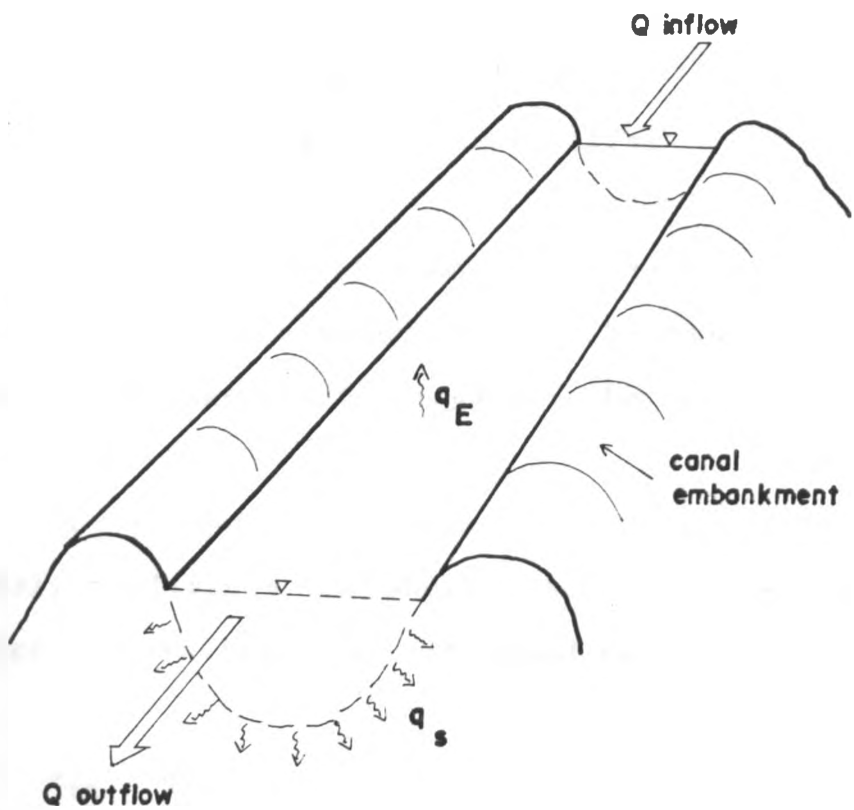


Figure 2: Parameters for inflow-outflow method.

The parameters for a water balance for reach under study are shown in figure 2. Water balance equation for the reach is,

$$q_{in} - q_{out} = q_E + q_s + \Delta s \qquad (3)$$

where;

q_{in} = inflow discharge (l/s)

q_{out} = outflow discharge (l/s)

q_E = Evaporation rate (l/s)

q_s = Seepage rate (l/s)

Δs = change in storage (l/s)

For all practical purposes, steady, and uniform flow conditions are maintained during the study period, thus, change in storage can be assumed to be zero.

For the considered duration of measurement, q_E, is normally negligible for short reaches of the canal. Hence, seepage loss in l/s/Km is estimated as;

$$S = \frac{q_{in} - q_{out}}{L} \qquad (4)$$

where,

S = seepage rate (l/s/km)

L = length of canal reach (km).

During measurement the water level in the canal is kept constant to eliminate the effect of canal and bank storage. Webber (1971) observed that in the majority of cases, where relatively long straight channels (of constant cross-section and bed slope) are involved, the flow for the most part is so nearly uniform that the assumption of uniform flow condition is reasonable. All diversions and leaks within the canal section being tested must be taken into account. The difference between the quantities of water flowing into and out of the canal reach is attributed to seepage. The advantage of this method is that the canal is not removed from service, but the accuracy is less than for ponding method as recorded by French (1985), as reported by Ali (1986).

2.4.1.1.1 Flow measurement devices

Several devices are available for flow measurement in open channels. These include circular or rectangular submerged orifices, V-notch, Cipoletti and rectangular weirs, Parshall, trapezoidal and cutthroat flumes and propeller meters. Each device has certain inherent advantages and disadvantages under field conditions (Trout, 1982).

A current meter measures the water velocity in the channel directly and consists of a propeller which is caused to revolve by the force of current when immersed in water. An electrical gadget is used to record the number of revolutions of the propeller (Punmia and Lal, 1986).

The main advantage of current meters is that they create negligible head loss and thus do not disturb the normal flow. Good quality small propeller meters give velocity measurements quickly and easily. Comparing current meters with the devices listed above, current meters are less accurate especially in determining the velocity distribution in small, irregularly shaped channels. Current meters are generally too inaccurate for loss measurements in small channels.

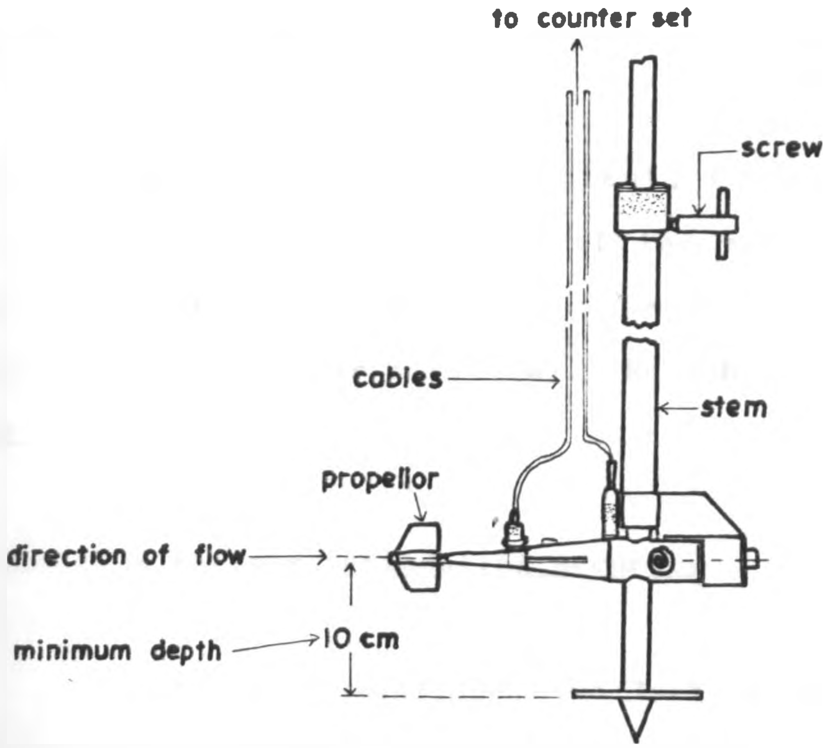


Figure 3: Current meter. Source: Kraatz (1977)

<u>Weirs</u> operate under a wide range of flows, but are accurate only under free flow conditions and thus require a relatively large head loss. Weirs are commonly used for permanent installations when head loss is not a problem (Trout, 1982). A Romijn weir is an outflow structure with horizontal crest above which the deviation from a hydrostatic pressure distribution because of centripetal acceleration may be neglected (Bos, 1978). In other words, the streamlines are practically straight and parallel. The discharge is related to the upstream water level over the crest in the following relationship:

$$Q = C_d C_v b h^{1.5} \qquad (5)$$

where C_d = discharge coefficient, C_v = coefficient for neglecting the velocity head of the approach channel; b = width of the weir (m) and h = head measured at a point upstream of the weir crest (m). Romijn weirs are present in Bura.

2.4.1.1.2 <u>Factors influencing accuracy</u>

Trout (1982) mentioned two factors that influence the accuracy of inflow-outflow conveyance loss measurements, i.e: 1. the accuracy of the measuring device and 2. the influence of the devices on flow conditions.

Trout (1982) remarked that flow measuring devices are accurate to only within 3 to 8 percent under field conditions. However, this level of accuracy is optimistic and is only achievable under very good installation conditions. It is recommended that long enough channel sections are measured so that the measured loss is large compared to the measurement inaccuracy.

Measurement devices used in channels such as flumes, weirs, or orifices create a backwater effect. This backwater effect causes the depth of water upstream of the device to increase and the increase extends upstream for several hundred meters depending on the flow and channel characteristics. Consequently, water loss for an earthen channel will increase as the flow depth increases.

2.4.1.2 Ponding method

The method involves ponding water in a canal section to approximate operating depth and then recording or periodically measuring the drop in the water surface with time. This is the most accurate method, but large canals must be taken out of operation for 2 weeks to make the measurements (Worstell, 1976). To avoid this withdrawal of canals from operation, measuring could be done on main canals either before or after the irrigation season. However, it is feared that seepage rates probably differ from the seasonal average. For long canals (as in the case of Bura), if the ponded section is long, the average seepage rate measured will not identify any localized high seepage zones within the ponded section.

To eliminate the effect of wind, the rate of drop should be measured at each end of the pool and averaged. Staff or hook gauges attached to existing structures or stakes driven into the canal bed should be used as shown in Fig. 4.

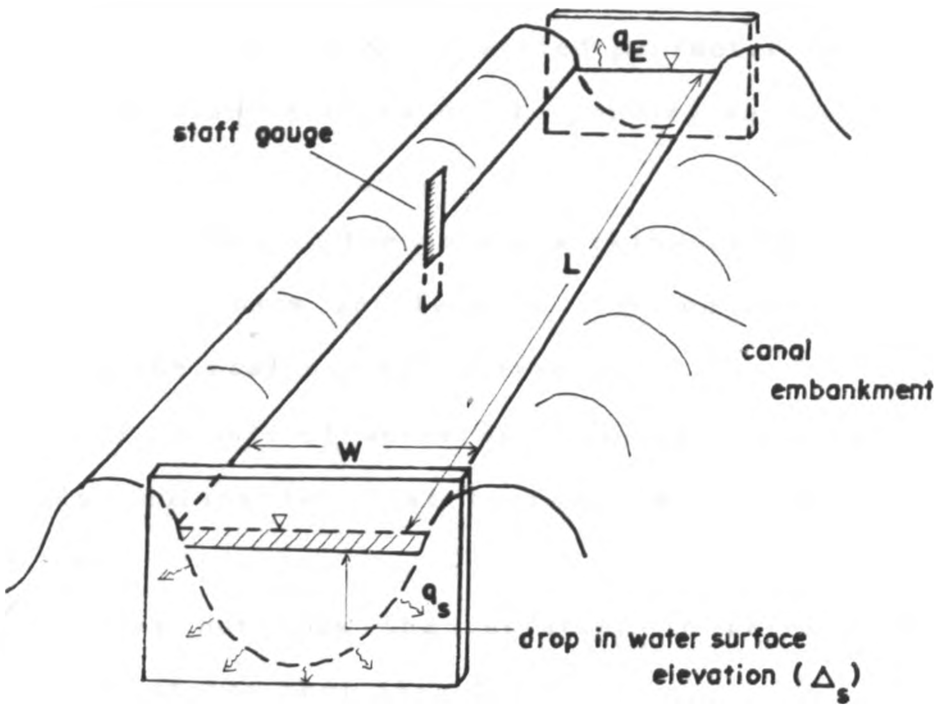


Figure 4: Ponded water in a canal

All structural leaks should be carefully measured and since the testing may take considerable time, evaporation and rainfall should be recorded so that the drop in water surface can be corrected accordingly. The following formula is suggested for computing the rate of seepage (Kraatz, 1977).

$$S = \frac{W\,(d_1 - d_2)}{P} \quad \text{m/day} \qquad (6)$$

where S = average seepage in m^3/m^2/day; W = average width of water surface of the ponded reach(m); d_1 = depth of water at beginning of measurement (m) d_2 = depth of water after 24 hours (m); and P = average wetted perimeter (m).

Some disadvantages of the ponding method are;

1. Interruption of the normal working of the canal.
2. Heavy expenses involved in the construction of dams to form the pool and their removal.
3. Large amounts of water are required to fill the pool and also during the test to compensate the drop in water level.
4. It does not show the variation in rates from different parts of the pool (Kraatz, 1977).

Trout and Kemper (1980) have recommended that the length of section measured will depend on the slope of the channel. The variation in the ponded water level from the operational level should not be more than 2 cm. If channel slopes are small (less than 0.0005), sections as long as 100 meters should be measured. If slopes are steep (greater than 0.001), section lengths should be less than 30 meters. Section lengths should be as long as possible within these limits so that a larger and more representative sample is measured. The end effects of seepage into or around the dam are then minimized.

2.4.1.3 <u>Seepage meter method</u>

A seepage meter is a modified version of a constant head permeameter developed for use under water. Seepage meters are, in principle suitable for measuring local seepage rates in canals or ponds. They are particularly useful for locating sections of the canal with excessive seepage. As a precaution, installation of seepage meters should be done with least disturbance.

Seepage meters should not be used in very gravelly soil due to difficulty of forcing the bell into the bed of the canal. Sandy soils are unsuitable for seepage meters use since there is danger of the seepage meter being washed away by the current. Various types of seepage meter have been developed. The most important are:

1. Constant head seepage meter; (Fig. 5) and
2. Falling head seepage meter (Fig. 6)

The constant head seepage meter with a floating flexible bag is perhaps the simplest and cheapest device as regards construction and operation. It consist of a water tight seepage cup connected by a hose to a flexible (plastic) water bag floating on the water surface (See Figure 5). The floating flexible bag seepage meter was developed by the Salinity Laboratory of the Department of Agriculture -USA.

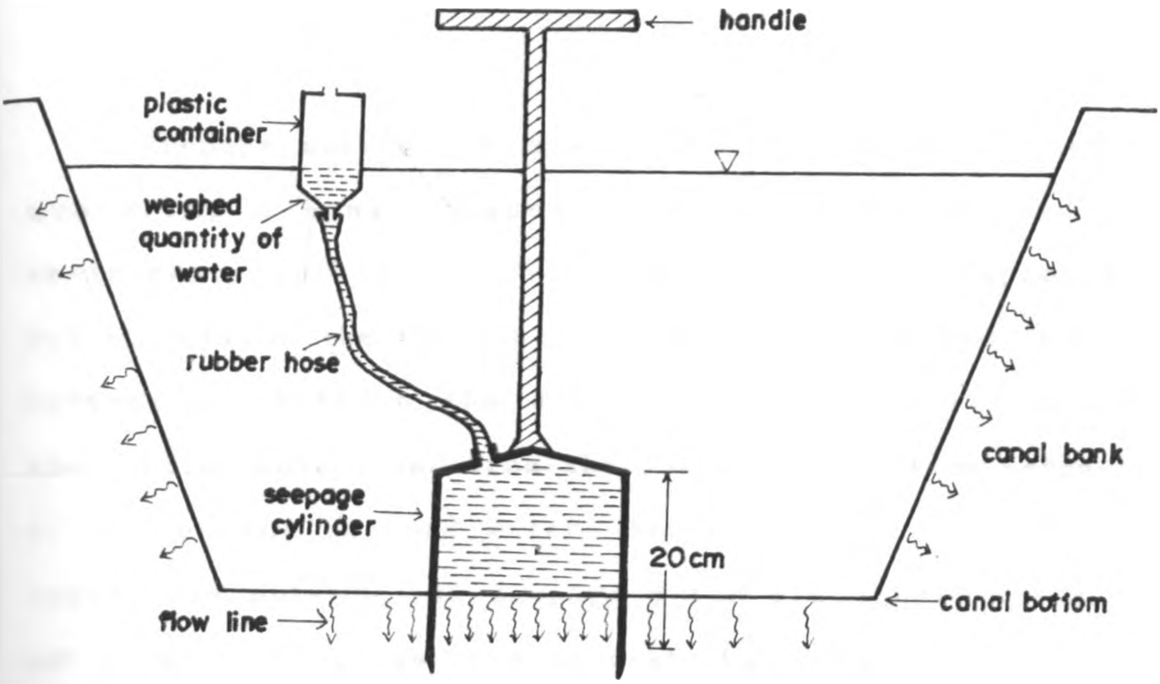


Figure 5. Seepage meter with submerged plastic bag.

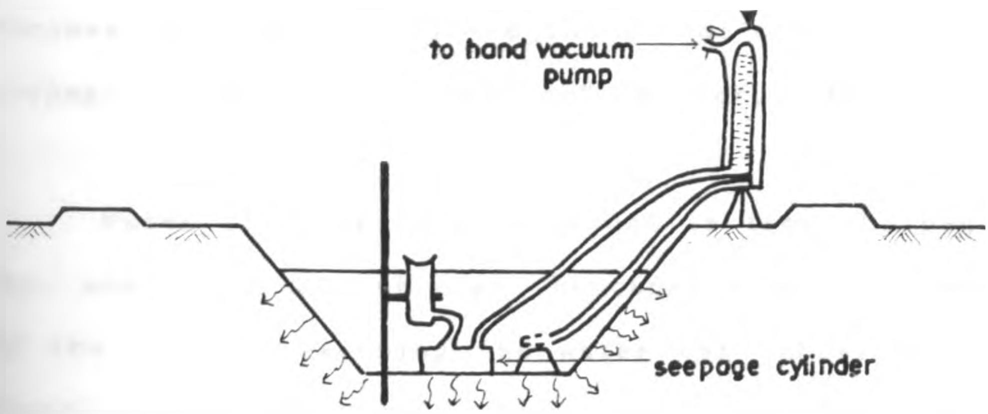


Figure 6. Falling head seepage meter. Source: Kraatz (1977).

Seepage meters are simple and convenient devices for measuring seepage losses, but it is recognized that refinements in the techniques are needed to improve the reliability of the results. Seepage is determined from head differences between the seepage meter interior and the surrounding water, and from the corresponding flow rates into or out of the seepage meter (Bouwer, 1962). If the head inside and outside the seepage meter are equal, leakage is not present, and, assuming no disturbance by installation of the meter, the flow is that of the original seepage flow prior to installation of the meter. If the head inside the meter is lowered, leakage flow begins to occur and the net flow leaving the meter through the bottom area enclosed by it becomes the resultant of the individual components Qs, due to seepage and Ql, due to "leakage" (Bouwer, 1962).

Water flow during measurement is from the bag into the cup, where it seeps through the canal subgrade area isolated by the cup. By keeping the water bag submerged, it adapts itself to the shrinking volume so that the heads on the areas within and outside the cup are equal. Seepage rate is computed from the weight of water lost in a known period of time and the area covered by the meter (Kraatz, 1977).

Bouwer (1962) experienced that repeating the seepage measurements every 1/2 hour or so gives consistent results. Therefore, one or two measurements per seepage meter location should be sufficient. He cautions that if a series of measurements is to be performed in a flowing canal, the operator should begin downstream and work in the upstream direction.

For the falling head technique, the seepage meter is connected to a falling level reservoir (See Figure 6). In measuring seepage using this technique, the reservoir is raised approximately 2.5 cm above the water surface in the canal. The subsequent rate of fall of the water level in the reservoir is measured by taking the time and water level readings. The latter reading is expressed in terms of the distance between the water level in the reservoir and the free water surface in the canal, and the seepage is then calculated from the water level versus time measurements (Bouwer and Rice, 1963).

Seepage rate, qs can be estimated using the following formula,

$$qs = H \frac{R^2{}_v}{Re^2} \qquad (7)$$

Where,

H = rate of water level drop in supply (cm/min);

Rv = radius of reservoir (cm);

Re = radius of cylinder (cm).

2.4.2 Indirect measurements

A quantitative evaluation of the seepage loss in a canal can be obtained by calculation using various methods. This may be done by empirical, analytical or electrical analogue methods. These methods are mainly employed in estimation of seepage losses for the economic evaluation of the benefits of a proposed canal. It is noted that empirical methods give only rough estimates while analytical methods give reasonably accurate results if they are applied to conditions for which they are developed (Bouwer 1969), as reported by Kraatz (1977).

2.4.2.1 Empirical formulae

Davis and Wilson, quoted by Dhillon (1967) and reported by Kraatz (1977), have suggested a relation for estimation of seepage losses in unlined canals. After obtaining the results of surveys on eight different canals systems, the United States Bureau of Reclamation derived the following relationship known as the Moritz formula (1967) and adapted by the Alberta Department of Agriculture Irrigation Division (1979) as reported by Manz (1985) as in equation (8).

Seepage rates are estimated using the following formula:

$$qs = 1.16 \times 10^{-5} \; C \; A^{0.5} \qquad (8)$$

Where:

qs = seepage rates ($m^3/s/m$)

A = cross-sectional area of canal (m^2)

C = Coefficient, a function of the material used to construct the canal (Appendix vii)

In India, the following formula has been used by International Commision on Irrigation and Drainage (1968):

$$qs = 1.923 \times 10^{-6} \; . \; Q^{0.0652} \; . \; P \qquad (9)$$

Where;

qs = seepage rate ($m^3/s/m$)

Q = discharge in the canal (m^3/s)

P = wetted perimeter of the canal (m)

Observations made on some of the important canals in the Punjab showed that C ranged from 1.1 to 1.8.

Offengenden proposed the following equation for estimating seepage losses (FAO/UNESCO, 1967) for earthen canals.

$$S = \frac{a}{Q^{m}} . Q. \frac{L}{100} \qquad (10)$$

where S = water loss per km in percent, Q = discharge, (m^3/s 1 = length of canal; a and m are empirical constants depending on soil permeability (See Appendix vii).

2.4.2.2 <u>Analytical methods</u>

Analytical methods of estimating seepage give high accurate results when applied to conditions for which they were developed. French (1985) as reported by Ali (1986) gives an example of the analytical method to obtain solution to the relevant porous media equation for an appropriate set of boundary conditions. The investigation involves some assumptions; namely: That the channel lining be impervious and the thickness negligible; the porous media under the channel is to be isotropic, homogenous and of infinite depth; that the capillary action is absent. The investigation were done, first for channel in which the sides were lined but the bottom was unlined.

The second situation involved a channel in which the sides were unlined and the bottom lined. The solutions obtained from the investigations were summarized in graphs. The total seepage loss q per unit length of material underlying the channel is thus obtained. The parameters of the open channel that are required in order to obtain the solution are the angle (0) made by the sides of the channel with the horizontal in radians, water depth (y) in the channel and the bottom width (b) of the channel, [Subramanya et al. (1973) as reported by Ali (1986)].

2.4.2.3 Simulation system models

To solve problems of water flow in soil a system is derived to simulate the operation of the prototype system. Such models are differential equations that govern groundwater flow, hence, simulation of groundwater flow systems is done by application of differential equations. Examples of models that are applicable for this analysis are the physical and analogue models (real simulation systems) and mathematical models which are abstract simulation systems.

Physical models consist of one dimensional flow in soil columns and two or three dimensional flow in sand tanks. The porous medium is usually homogenous, isotropic and consists of artificial or natural granular material. The actual physical shape of the medium is modelled and the boundary conditions are simulated as heads of water or as drains. Measurement is done either by means of piezometer or by using dye to trace the streamlines. This method gives an indication of the flow pattern but has a disadvantage in that the difficulty of simulating the true permeability of the media is encountered. For detailed discussion of the simulation models, reference is made to work done by Prickett (1975) and De Laat (1980), as reported by Ali (1986).

2.5 Consequences of seepage

Seepage losses in irrigation canals may lead to serious economic problems not only due to loss of water but due to other physical effects. These effects are discussed below.

Water losses from seepage on pervious soils can reduce the quantity available for irrigation to inadequate amounts (Booher, 1974). A review and summary of 765 seepage tests made in the Western United States showed that an average unit seepage loss rate range from 0.03 - 0.6 m/day (Worstell, 1976).

A study on seepage losses in canals passing through black cotton soil (Montmorillonitic clay) in Mwea, Kenya, reported the range to be 0.65 - 1.44 m^3/m^2/day (Mulwa, 1981). A similar test done in unlined canals traversing through red soils (Nitosols) gave seepage losses 3 to 4 times higher than those obtained in black cotton soils. Kwenjanga (1984), in a study on canal seepage in Lower Tigondo (Kajiado), Kenya, obtained average seepage losses in clay loam soils of 2.49 l/s/km of canal section. Seepage rates for Bura Irrigation Settlement Scheme's supply/main canal were determined by Ali (1986), using both analytical methods and empirical formulae as shown in Table 2.

Table 2. Seepage rates in the main canal BISP

Method of analysis	Seepage rate (m^3/s/million m^2)
Empirical formulae	
Davis and Storenson (1969)	2.89
USBR (Dhillon, 1967)	7.26
ICID (1968)	3.00
Offengenden (FAO/UNESCO, 1967)	6.12
Analytical methods	
Subramanya et al. (1973)	7.27
Vendernikov (ILRI, 1979)	15.52

Source: Ali (1986)

Average seepage obtained in Table 2 converted to comparable units is 60.11 cm per day. This is much higher than results obtained through direct measurement (see table 13).

The cost of water losses from irrigation conveyance systems depend upon where the lost water eventually ends up, whether it can be beneficially reused, or whether it creates additional problems and costs. The other important aspect to consider is the value of the lost water for agricultural production (Trout and Kemper, 1980).

When evaluating the cost of conveyance losses, Trout and Kemper (1980) recommended that reuse values which must be subtracted from the cost could include:

1. the value of beneficial plants growing near the channels which derive their water from channel losses;
2. the value of water pumped from the groundwater or diverted from return flow to the river minus the cost of repumping and any water quality decrease.

The costs which must be added to the sacrificed value of the lost water could include:

1. the decreased value of crops near the channels due to leakage; and
2. the cost of land degradation and depressed crop yields caused by salinity and waterlogging attributed to channel deep percolation.

Damage to adjacent land: Seepage loss adds to the groundwater leading to a rise in groundwater table. When the groundwater table reaches the ground surface this leads to waterlogging of the adjacent areas to the canal rendering them unsuitable for farming. This may be accompanied by salinity problem. According to Kraatz (1977), in one project in the Western United States some 8500 ha of cultivated land had to be abandoned due to waterlogging by seepage from canals. In the Punjab area, India considerable decrease in crop yields and determination in the general health of the people caused by malaria and other water borne diseases were attributed to seepage loss induced water logging. In addition, the effects of water-logging can result in decreased yield. Effects of water-logging by seepage from canals have been observed along the main canal in New Mutaro Irrigation Project, Laikipia, Kenya (Personal Observations, 1987).

Increased dimensions of canal: Worstell (1976) reported that the designer of the irrigation canal need to provide sufficient capacity in the canals to allow for the seepage losses. Consequently, the unlined canal dimensions are increased to cater for seepage losses incurred during the conveyance of water.

<u>Increased maintenance cost</u>: Seepage from irrigation canals results in weed growth along the banks and adjacent to the canal. This increases the cost of the maintenance of the canal. The banks of the canal may also be subjected to slips and sloughing which result in canal breaches.

2.6 Lining Options

The type of lining options can be grouped into six broad categories, namely;

(1) Hard - surface lining

(2) Exposed membranes

(3) Buried membranes

(4) Earth lining

(5) Soil Sealants and

(6) Flumes and pipes.

2.6.1 Hard - Surface Lining

For example, Precast cement concrete, unreinforced with a thickness of 5 cm acts as a hard-surface lining. This type of lining has an estimated life of 50 years. Seepage losses ($m^3/m^2/24hr$) varies between 0.03 to 0.15 depending on how well construction was done and subsequent level of maintenance. This type of lining is suitable for all sizes of canals, topographical, climatical and operational conditions (Kraatz, 1977).

The other example is Precast concrete block of 7 cm width. The durability is similar to that of Portland cement concrete. The range of seepage is in the order of 0.03 - 0.06 $m^3/m^2/24hr$. However, construction requires special equipment.

2.6.2 Exposed Membranes

An example of these is the Polyvinyl with a thickness of 0.19 mm. The life of the membrane takes only a few irrigation seasons. The effectiveness of the membranes vary widely depending on weed penetration and other mechanical damage as well as weathering. The membranes are suitable only as temporary lining for seepage control.

2.6.3 Buried Membranes

The example here is sprayed-in-place asphat. The durability depends largely on erosion resistance of cover material, maintenance, and operation. The effectiveness range between 0.03 to 0.06 $m^3/m^2/24hr$. The membranes have the advantage of being easily transported.

2.6.4 Earth Lining

The lining is in three classes based on thickness. These are thick compacted (90 cm), thin compacted (30 cm) and loosely placed earth such as loam and clay. Suitable soil from canal excavation or nearby borrow pit area is essential for economy. The measured seepage losses through the material range from 0.08 to 0.02 $m^3/m^2/24hr$.

2.6.5 Soil Sealants

The examples of the sealants are waterborne bentonite, sodium carbonate and resinuous polymers. The life of the soil sealants is relatively short, taking one or two irrigation seasons. The sealants offer a means of temporary control of seepage in unlined canals.

2.6.6 Flumes and Pipes

The examples of these are concrete flumes, pipes and lay-flat tubing. There life is relatively long lasting for about 50 years. Flumes and pipes are particularly suitable for areas with irregular or rolling topography and intensive cultivation.

2.7 Pre-Construction Seepage Studies

Soil mechanical investigations in the supply, main canal and in the command areas were done during the planning and construction stages of the project. The soil mechanical investigations of relevance to this study are the soil texture and the permeability tests.

2.7.1 Soil texture

Soil textural analysis was done at the planning stage of the project. Soil textural analysis was done for soils in the sites where soil permeability tests were done. Profile pits up to three metres below the ground surface were dug. Soil textural composition per metre of profile pit was analyzed and recorded. Relative proportions of clay, silt, sand and gravel were then analyzed and recorded. (Appendix 11.)

2.7.2 Soil permeability tests

For the determination of permeability, auger holes were made along the designed supply and main canal during the project implementation stage. The auger hole tests were done at an intensity of about 1.5 km throughout the entire section of the canal. The auger holes were then filled with water and replenished until the replenishment became more or less constant. This procedure used by ILACO (1975) is that recommended by United States Bureau of Reclamation. The amount of water used was then measured. The permeability of the soil is calculated using formula:

$$k = Q/(Cu*r*H) \qquad (11)$$

where k = permeability coefficicnt of Darcy (cm/s); Q = measured water supply in (cm^3/s); r = radius of auger holes (cm); H = water depth in auger hole (cm); and Cu = unsaturated conductivity coefficient depending on H:r, in this case 84; CurH = 105,000 cm^2.

The measured permeability coefficient are presented on Fig. 7.

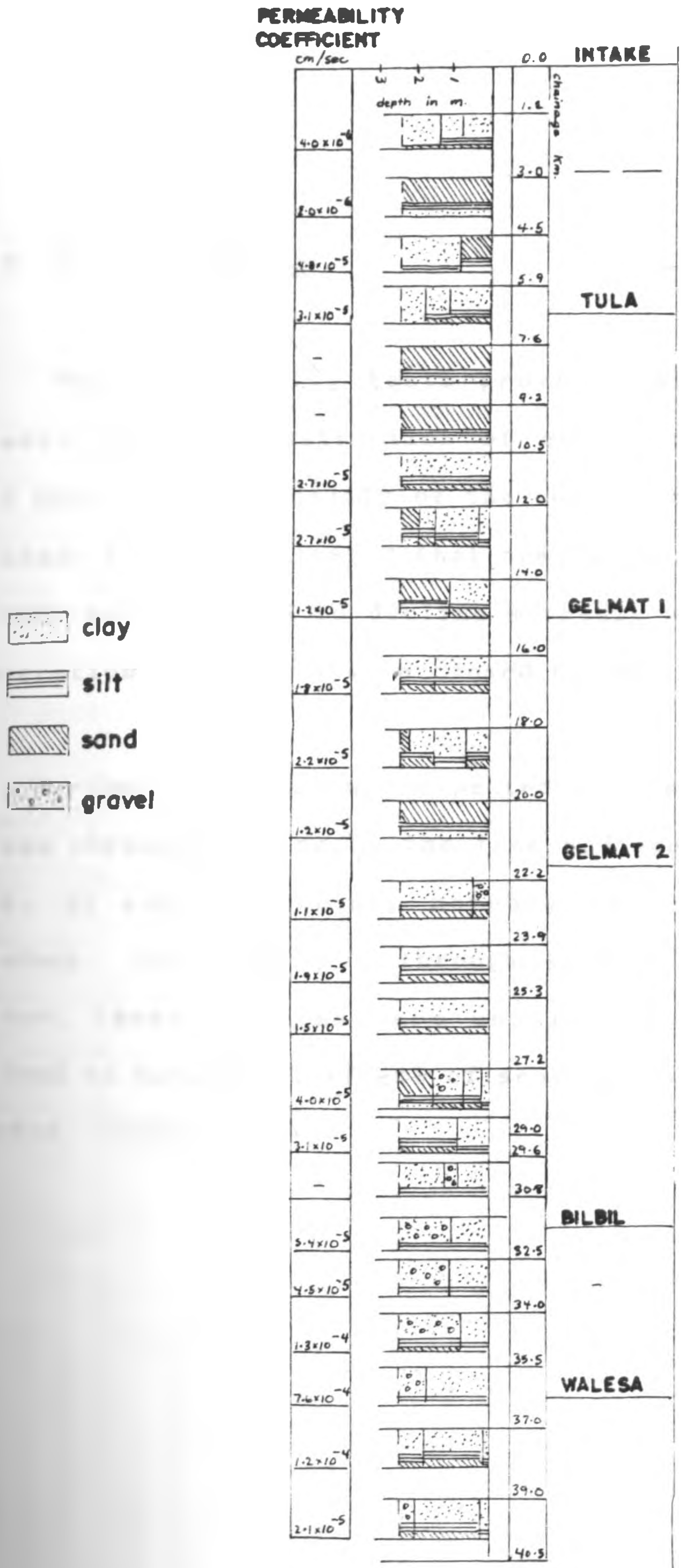


Figure 7: Permeability coefficients in main canal

2.7.3 Recommendations

The permeability tests proved that the subsoil is suitable for the construction of an unlined canal. It was found that the permeability of the soil is low (ILACO, 1975). The study further indicated that some seepage loss may occur, but this was expected to decline markedly when the canal was in operation and cracks were closed by swelling.

Further reduction was expected to take place in course of time through sealing by the fine sediments carried by the water. At some locations, notably at or near the laga crossings, the canal cut through permeable, sandy layers. However, these sections were considered to be short and combined to beneficial effect of sealing, no much seepage was expected (ILACO, 1975).

3. METHODOLOGY

3.1 Description of the study area

Bura Irrigation Settlement Scheme was developed to open up new areas for crop production through irrigation and to settle landless people recruited from all parts of the country

3.1.1 Location

Bura Irrigation Settlement Scheme is situated administratively, in Tana River district in Coast Province of Kenya. The scheme area is situated on Latitude 1^{o} 9' S and Longitude 39^{o} 52' E. The scheme is about 300 km east of Nairobi and 350 km north of Mombasa. The study area, Bura West, abbreviated BW, on Figure 8 is situated on the west bank of the Tana River.

3.1.2 Climate

Rainfall in Bura is low (300-400 mm per annum) and erratic. Rainfall comes mainly in the two rainy seasons, namely March to May and October to December. The latter season is wetter than the first on average. Temperatures are high all year round with little seasonal variation.

Mean maximum temperatures do not fall below 31^{o} C. Average minimum temperatures are above 20^{o} C. The hottest months are February and March (Muchena, 1987). The mean measured annual evaporation using US Weather Bureau Class A evaporation pans for Garissa and Hola is 2,712 and 2,490 mm respectively. Bura irrigation settlement scheme is situated between these two meteorological stations. The estimated open water evaporation (Eo) according to Penman, for Garissa and Hola is 2,374 and 2,293 mm per annum respectively (Woodhead, 1968). This means an average of about 6.4 mm per day.

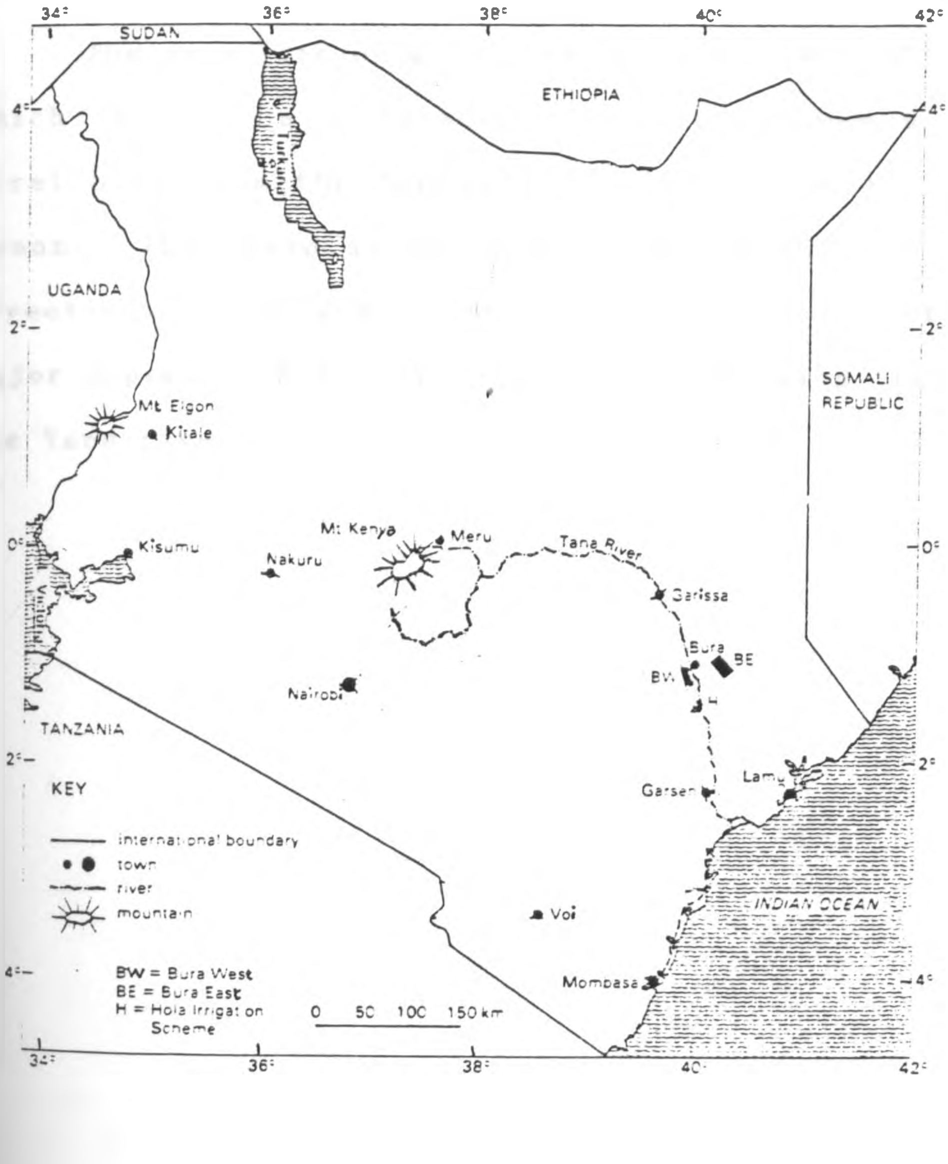


Figure 8: Location map of the study area

3.1.3 Topography

The area lies on an old alluvial terrace of Tana river, which flows in a north-south direction across a vast plain stretching from the foothills of Mt. Kenya to the Indian Ocean. The plain slopes gently and evenly in west-east direction in the scheme area. This area is cut by a few major depressions formed by seasonally flowing tributaries of the Tana river (lagas) as shown in Figure 9.

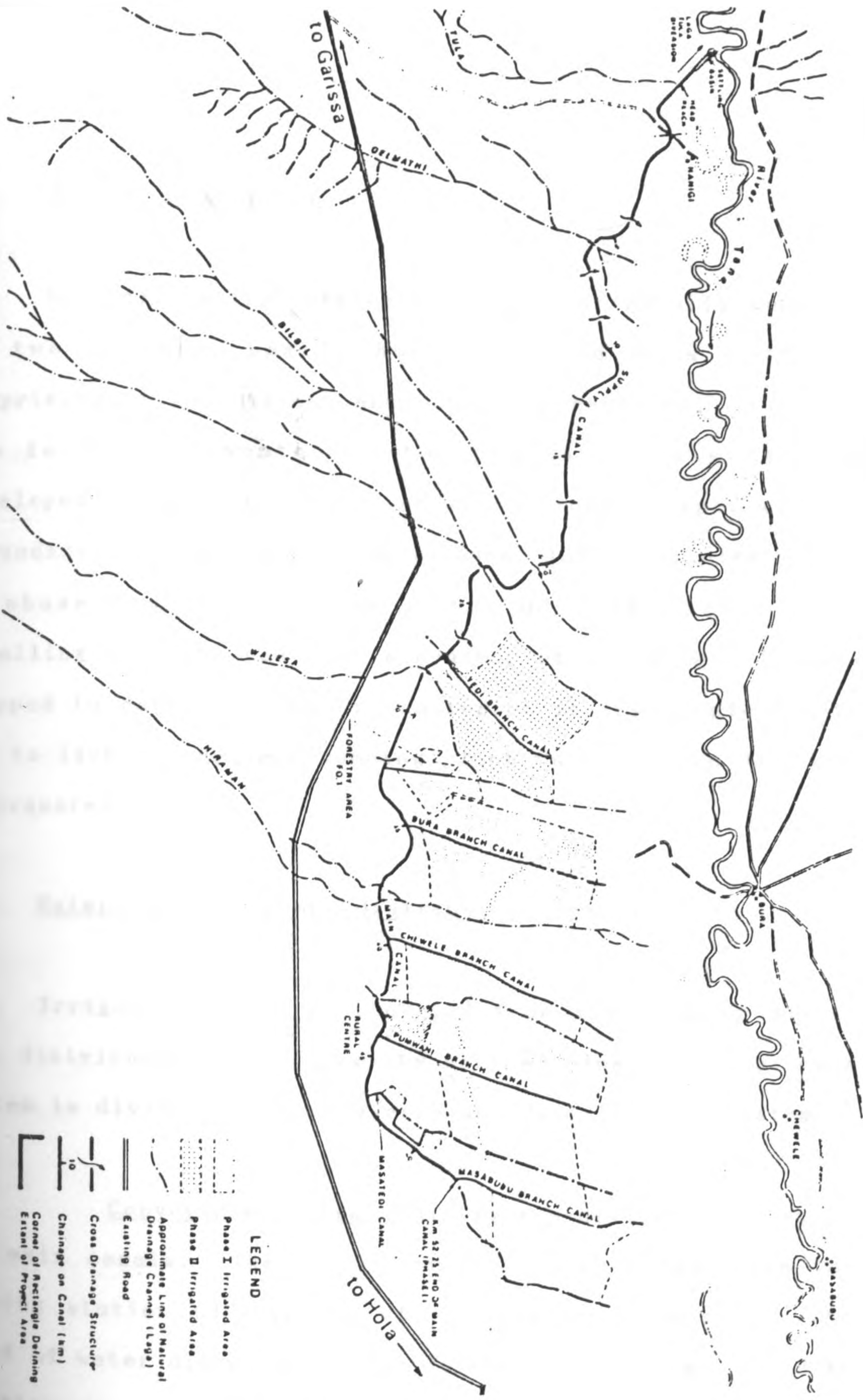


Figure 9: Bura irrigation project layout
Source : Sir M. Macdonald (1979)

3.1.4 Scheme description

Bura irrigation settlement project originally consisted of two schemes, namely Bura west and Bura east each comprising of a total net area of 6,700 hectares. Bura west was to be implemented in two phases. The scheme was developed for growing cotton and food crops (maize, cowpeas, groundnuts) by irrigation (World Bank, 1977). Implementation of phase 1 started in 1979, but by 1986 only an area totalling 3,900 ha had been prepared, of which only 2,500 is cropped to date. The major constraint is insufficient water due to lack of adequate pumping plant to supply to the 3,900 ha prepared.

3.1.4.1 Water conveyance and distribution system

Irrigation water is conveyed from the pumping station and distributed to the farms by earthen canals. The canal system is divided into conveyance and distribution system.

<u>Conveyance system</u>: This is sub-divided into supply and main canals. The supply canal conveys water from the pumping station (downstream of the settling basin) to the point of water diversion to the forest branch canal (36.8 km downstream). The main canal conveys water from the point of first water diversion (Forest branch canal) to the last

command area (Masabubu). The main canal supplies water to Bura, Chewele and Pumwani command areas as shown in Figure 9.

<u>Distribution system</u>: The distribution system consists of the branch canals which distribute water to the command areas, block feeders and the unit feeders. Branch canals obtain their flow from main canal. Block feeders start from the night storage reservoir offtake gates and distribute the flow to a number of unit feeders. As an exception,there are some direct off-take block feeders. The direct off-take block feeders obtain their flows directly from the main canal through the regulating Romijn gates at the head of the block feeder. Unit feeders take water from block feeders to the unit during daytime irrigation period only. Each unit consist of 30.24 ha net area. Water is siphoned from the unit feeders into the furrow, which apply water to the root zone of the crop (Standard for furrow is 280 m long and 0.9 m spaced) as shown in figure 10.

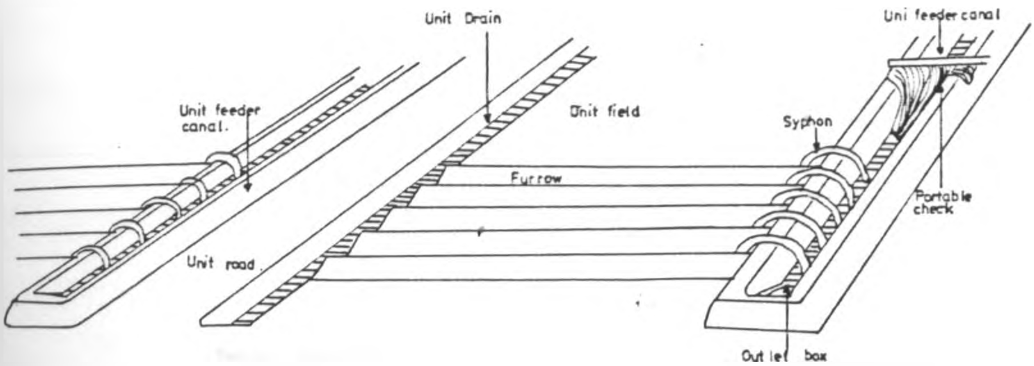


Figure 10: Unit feeder canal

Source : NIB (1981)

<u>Night storage reservoirs</u>: Night storage reservoirs (NSR) are constructed at the head of the block feeder canals. The reservoirs are supplied by the branch canal through an inlet (Romijn weir) gate during night time. Several night storage reservoirs are constructed on either side of the branch canal. The storage capacity of the reservoir is sufficient to contain the night water supply from the branch canal for a water level range of 75 cm (ILACO, 1975). The daytime flow from the branch canal passes directly into the block feeder. The off-take gate at the reservoir is closed

during night time and open during the day, thus supplementing the direct inflow to the block feeder during day time.

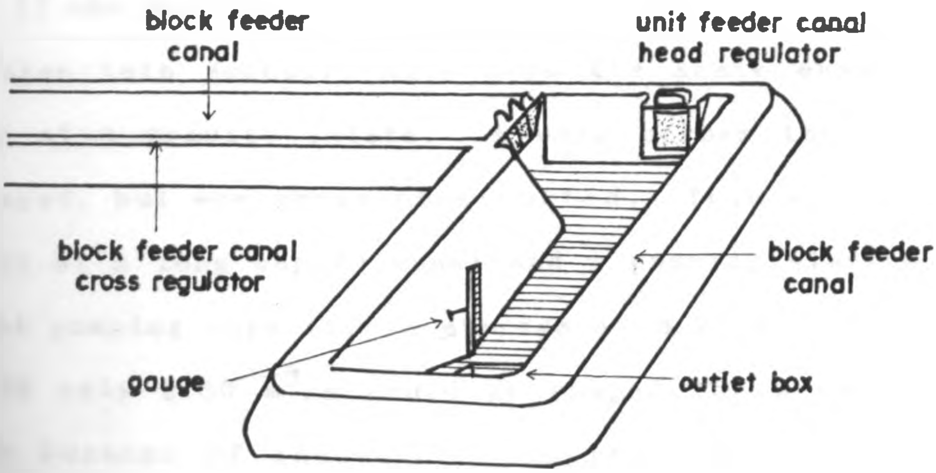


Figure 11: Night storage and block feeder canal

Source : NIB (1981)

3.1.4.2 Scheme intake requirements

The possibility of water abstraction was studied and documented by ILACO (1975). A comparison of the annual cost showed that the saving in investment costs (resulting from the fact that the supply canal could be 23 km shorter) are negated by the higher operation costs (fuel and spares parts). It was concluded that a gravity intake would be the most economical solution to the water need of the scheme.

If the future irrigation developments on the east bank are taken into account, this tips the scale even more in favour of a gravity intake. A weir across the river was envisaged, but was never constructed. This explains the anomaly of a long supply canal and a pumping station. The current pumping capacity on average is 3.25 m^3/s. However, in 1988 only 2.50 m^3/s could be pumped for a period of 9 months because of inadequate pumping capacity with the subsequent scheme operation below optimum.

Crop water requirements for the scheme during the time of this study have been calculated. The cropped area used in the irrigation water requirements is that of 1988. Comparisons have been made between the irrigation requirements computed for 1988, the revised irrigation requirement in Table 3 computed at the design stage of the project and the current pumping capacity of the scheme.

The monthly irrigation requirements for the scheme was calculated to form part of the basis of the design criteria for irrigation and drainage works (MacDonald, 1979). The crops considered were cotton, groundnuts, maize and cowpeas. The resulting irrigation water requirements distribution throughout the year is indicated in Table 3. The values in Table 3 have been obtained after adjusting the irrigation requirements for the initially planned area (6700 ha) to the presently cropped area (2500 ha).

Table 3. Monthly irrigation requirement schedule for 2500 ha.

Month	Total water $m^3 * 10^3$	Reqd. m^3/s	Gardens	A	B	C	D	Canal m^3/s
Jan	309	0.12	0.03	0.15	0.22	0.37	0.05	0.64
Feb	2261	0.94	0.03	0.97	1.39	0.54	0.05	1.98
Mar	3478	1.30	0.03	1.13	1.91	0.60	0.05	2.56
Apr	1987	0.77	0.03	0.80	1.15	0.50	0.05	1.70
May	4124	1.54	0.03	1.57	2.26	0.62	0.05	2.93
Jun	3925	1.51	0.03	1.54	2.21	0.56	0.05	2.82
Jul	2832	1.06	0.03	1.09	1.57	0.56	0.05	2.18
Aug	1068	0.04	0.04	0.44	0.63	0.56	0.05	1.15
Sep	1550	0.05	0.05	0.65	0.93	0.47	0.05	1.46
Oct	2245	0.05	0.05	0.75	1.08	0.48	0.05	1.61
Nov	1406	0.04	0.04	0.58	0.83	0.50	0.05	1.38
Dec	987	0.03	0.03	0.40	0.58	0.48	0.05	1.11

Notes:

Column (A) is obtained by adding column (3) and (4).

Column (B) is obtained by dividing column A by 0.8 (field-efficiency) and multiplying by 1.15 (branch and block feeder losses).

Column C is estimated losses in the supply and main canal only

Column D gives the water requirements for wildlife and rural c

Canal head discharge = Column B + C + D

Source: MacDonald and Partners, 1979.

3.2 Seepage measurements in the supply canal

Seepage measurements in the supply canal were done by inflow-outflow and seepage meter methods.

3.2.1 Inflow-outflow measurement

This method was considered suitable for this study because there are no additional inflows in the section of the canal selected for seepage analysis. The cross-section of the supply canal is large (width of water surface at design depth for discharge of 8.6 m^3/s at canal head is 12.6, and 8.0 m at average flow depth 1.0 m for a discharge of 2.5 m^3/s at the time of the study) and as such ponding method would have been cumbersome and expensive. The supply canal conveys water all the time and hence putting the canal out of operation for 2 weeks (necessary if ponding is done) would not be feasible.

In the inflow-outflow method, measurement involved determining the flow rate at various sections of the supply canal. The measuring points were selected at equal intervals along the supply canal.

For the purposes of this study, sections of 10 km were selected. Due to the remoteness of the area, together with the vastness of the distance to be covered (40.15 km), 10 km reaches were considered representative for the entire canal section. Location of the measuring points was facilitated by the already existing chainage works for every 1 km in the supply and the main canal.

Four measuring points were selected for the water flow rate determination as shown in Figure 12.

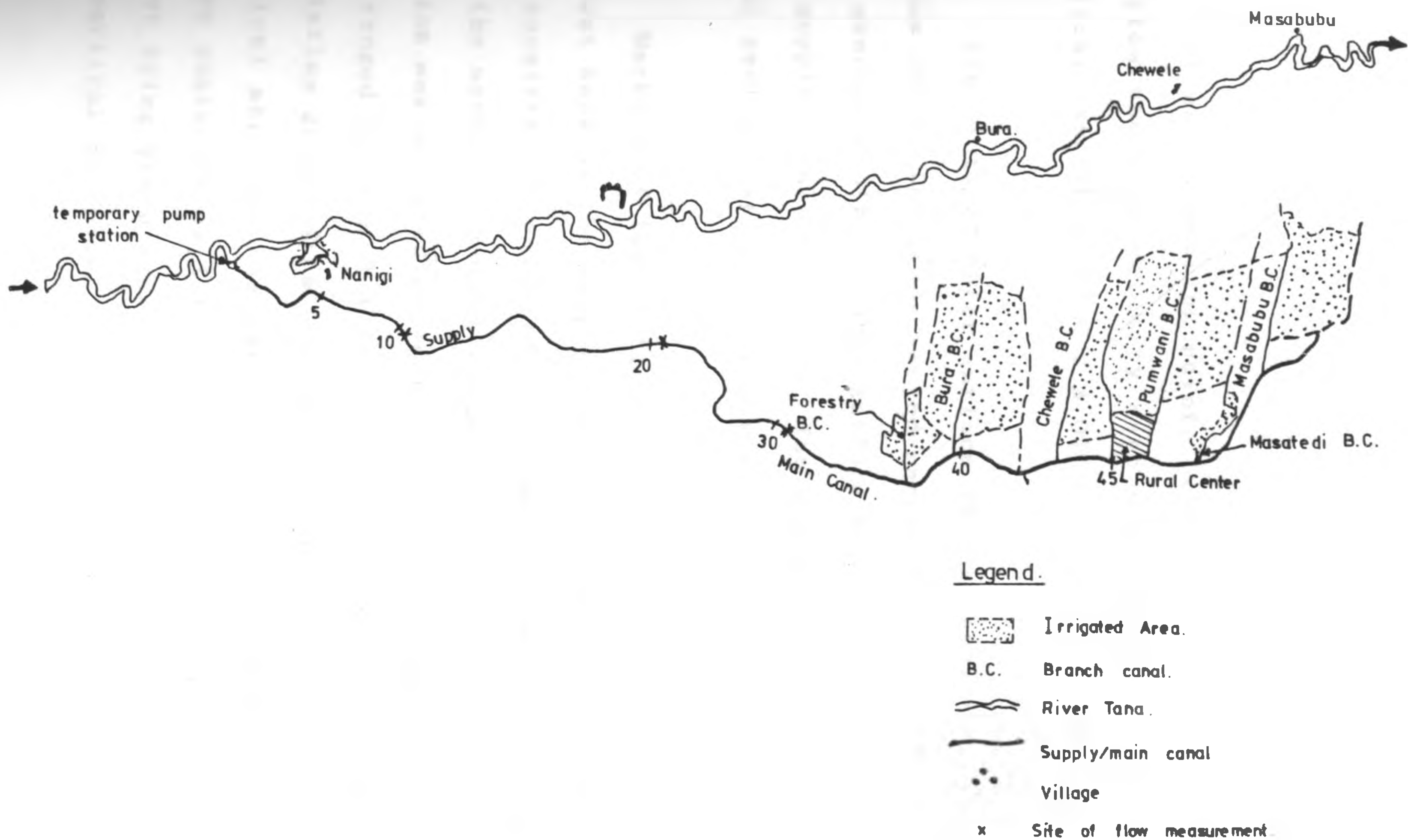


Figure 12: Points of flow measurements in the supply canal

Source: Sir M Macdonald (1979)

At each measuring point, both the cross-sectional area of flow and flow velocity were determined in a number of vertical sections. A C-2 A.OTT current meter was used.

The width of water surface in the canal at various flow depths during the study period ranged from 6.0 to 8.4 meters. The average width of water surface for all measurements along the supply canal was 7.6 m. The width was divided into vertical sections selected at 1 m intervals.

Marking of the vertical sections was started from east to west bank of the supply canal. The first vertical section was consistently selected 0.5 m away from the east bank in all the measuring points along the canal. The last vertical section was selected at variable interval from the west bank, but ranged between 0.3 to 0.5 m depending on the extent of vegetation growth from the bank to water surface. Marking of vertical sections was facilitated by tying a rope to two wooden posts across the canal as shown in Figure 13. Felt-pen or tying green grass stems on the rope were used to mark the vertical sections.

Figure 13: Current meter and rope.

At each vertical section, flow depth $_{(D)}$ was measured. Computation of 0.2D and 0.8D at each vertical section was hence done. Velocity at 0.2D and 0.8D was measured using a current meter. This two-point method enables average velocity to be obtained i.e $0.5(V_{0.2} + V_{0.8})$ which is more reliable than velocity obtained at one point (0.6D per vertical section. Multiplying the cross-sectional area of flow with velocity gives the discharge in the vertical section. These discharges are then added to give canal discharge. Procedure for current meter computations are shown in Appendix V.

A C-2 A.OTT small current meter was used in the determination of the velocity in the vertical sections. The current meter was equipped with 6 propellers of varying sizes. Only propeller number 1 was found appropriate for velocity measurement on basis of consistency. Other propellers would work either at certain vertical sections and not in others at the same flow measuring point.

The number of revolutions per 30 second were then recorded in the data sheet. Velocity (V) in m/s was computed using the equation (12).

$$V = 0.0577n + 0.006 \tag{12}$$

where n = revolutions per second. Equation 12 was obtained after calibration using a calibrated propellor.

Propellor number 3 was was used for calibration, whose equation is shown as,

$$V = 0.252n + 0.006 \tag{13}$$

After flow measurements at subsequent points in the canal, seepage was computed by getting the difference between the inflow and outflow from the successive points in the canal (Eqn. 4). The difference between the two flows is attributed to seepage.

Seepage losses in canals are expressed in three ways namely; in l/s/km; %/km; and m/day. In this study seepage losses are expressed in l/s/km and m/day for the data obtained from inflow-outflow and seepage meter and ponding methods. Seepage losses expressed in m/day only, was adopted for ponding method used in storage reservoirs. Due to variations in wetted area during operation stage of the canal as a result of sedimentation and vegetation growth, $m^3/s/m^2$ of wetted area was not used as a unit of seepage for this study.

Expressing seepage in l/s/km and m/day was found desirable as it allows comparison to be drawn between the results of the study and those documented during pre-construction stage of the project on seepage and permeability coefficients of the soils.

3.2.2 Seepage meter measurements

A seepage meter was used for seepage loss evaluation in the supply canal. Representative measuring points were chosen along the canal. The chosen points were chainage 11, 21, 31 and 38.6 along the supply canal. This method was recommended for use in order to provide a set of comparative data on seepage in the canal with that collected using inflow-outflow method.

A constant head seepage meter was used. Seepage meter consist of a water tight cylinder (30 cm diameter and 20 cm height). The top is closed with a conical shaped cover on which an iron rod (1.25 cm diameter) is connected. Conical shape is to ensure there is no air entrapment in the cylinder and to allow more stable flow over and around the seepage meter in case of a fast flowing canal. A hose pipe is connected from the top of cylinder to a container with measured volume of water. The handle and hose pipe vary with local conditions (1.0 - 1.5 m). The cylinder of 30 cm diameter was pushed only a small distance into canal subgrade in order to avoid, as far as possible, disturbance of the existing soil structure. Penetration of approximately 2.5 cm is recommended as adequate for good seal. While submerging and pushing the cylinder into the soil, the hose was kept open at its upper end to allow air and excess water from the cylinder to escape. When equilibrium was achieved, the hose was then connected to the container with a measured volume of water.

This container was then released to settle on the water surface in the canal. The experiment was allowed to continue for one hour, after which the volume of water remaining in the container was measured. The difference between the original volume and the remaining volume of water in the container was obtained after a known period of time. Seepage rate was computed from the volume of water that seeped per hour through the canal wetted area covered by the seepage meter. In order to make seepage comparable to that obtained from canal section, the rate was converted to seepage in 1/s/km.

3.3 Seepage measurement in the Bura branch canal

Bura branch canal was selected for discharge measurement. Selection was based on the following criteria:

1. Comparison between soil types traversed by Bura and Pumwani branch canals show a close similarity. Profile analysis for the two dominant soil units (pf 2.1 and 2.2) showed that they are chromic-calcaric CAMBISOLS. The textural analysis for depths 0-155 cm in both soil units gave similar results (Muchena, 1987). Similar soil units are also found in Chewele command area.

   However, unlike in Bura and Pumwani command areas where dominant units are clearly delineated, in Chewele command, the soil units appear in a jig-saw pattern.

2. At the time of data collection, irrigation was mainly going on in Bura command. This hence, fitted in the plan of the actiyities intended for the study.

3. Pumwani branch canal was found unsuitable due to clogging by weeds and excessive accumulation of sediment deposits.

Flow rate measurements in the branch canal were taken at 1.5 km interval along the canal. Flow rate measurements at the head of the canal were done by use of Romijn weir. For other successive measuring points, the current meter was used. Due to the shallow water depths in the branch canal, two point measurements per vertical section was not feasible. The propeller movement would glide to a halt at 0.2D. Hence flow velocity determination was done at 0.6D only. The flow vertical sections were selected at 0.5m intervals across the branch canal. Inflow-outflow method was then used for seepage rate computation. However, problems were encountered since intake to the reservoirs were open in various sections hence invalidating collected data. Only seepage measurements in the first reach (1.5 km) was used in evaluation of seepage.

3.4 Seepage measurement in the night storage reservoirs

The reservoirs selected for seepage analysis are in Pumwani branch canal. In Pumwani branch canal 9 storage reservoirs are operational. Two reservoirs were selected for seepage evaluation. In the project design documents the reservoirs selected for seepage evaluation are referred to according to their systematic nomenclatures namely NSPU4 and NSPU6 i.e. night storage reservoir in Pumwani canal numbers 4 and 6 respectively.

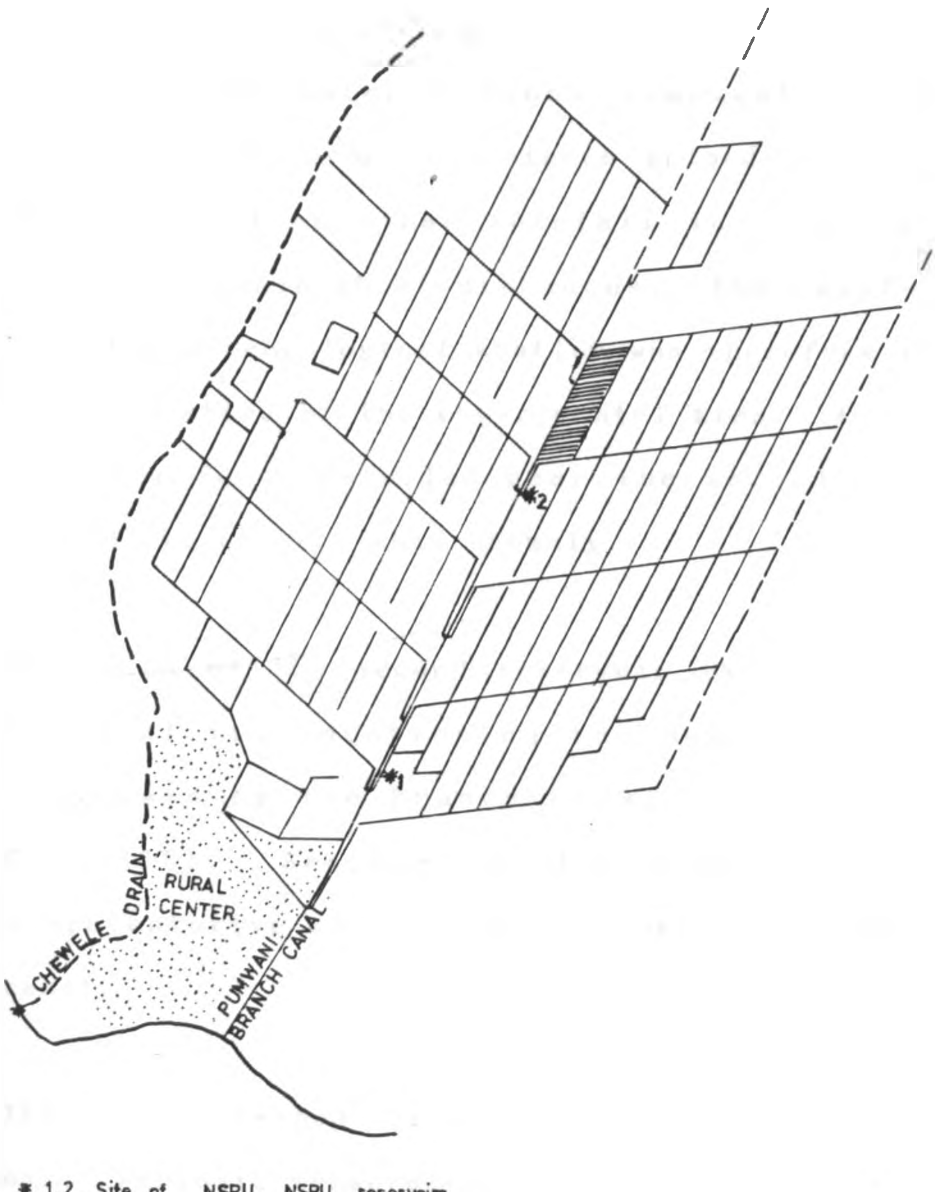


Figure 14: Pumwani branch canal.

Selection of the two reservoirs was based on the following criteria, namely;

1. The reservoir NSPU4 is located about 500 m from the meteorological station. The station was envisaged to provide rainfall data (at the time of experiment) to be used in the water balance computations for the reservoirs. However, experience gained at the time of the study showed that rainfall in Bura is highly variable within short distances. The rainfall data from the meteorological station was therefore found not representative of the experimental area. A rain-gauge was therefore installed near the second reservoir (NSPU6) to correct this anomaly.

2. Selection of the second reservoir (NSPU6) was on the basis of its location on the dominant soil unit traversed by the branch canal (Chromic-calcaric CAMBISOL). Seepage results would therefore be representative of the area traversed by the branch canal.

The ponding method was used in seepage analysis in the reservoir. Periodic measurement of the drop in water surface with time was done. A staff gauge at reduced levels above sea level (already installed) was used for measuring the subsidence of the water surface over 24-hr period. Both intake and outlet gates were closed water-tight to ensure that no water was added to or removed from the reservoir during the duration of the experiment i.e. within 14 days. Use of gunny bags filled with soil was used to ensure that no leakage from the gates to the reservoir.

A class A evaporation pan was installed at the reservoir embankment. A rain gauge was also installed to record any rain that would fall during the duration of the experiment. Stray domestic and wild animals, and curious local people were kept away from interfering with the experiment by engaging a day and night watchman.

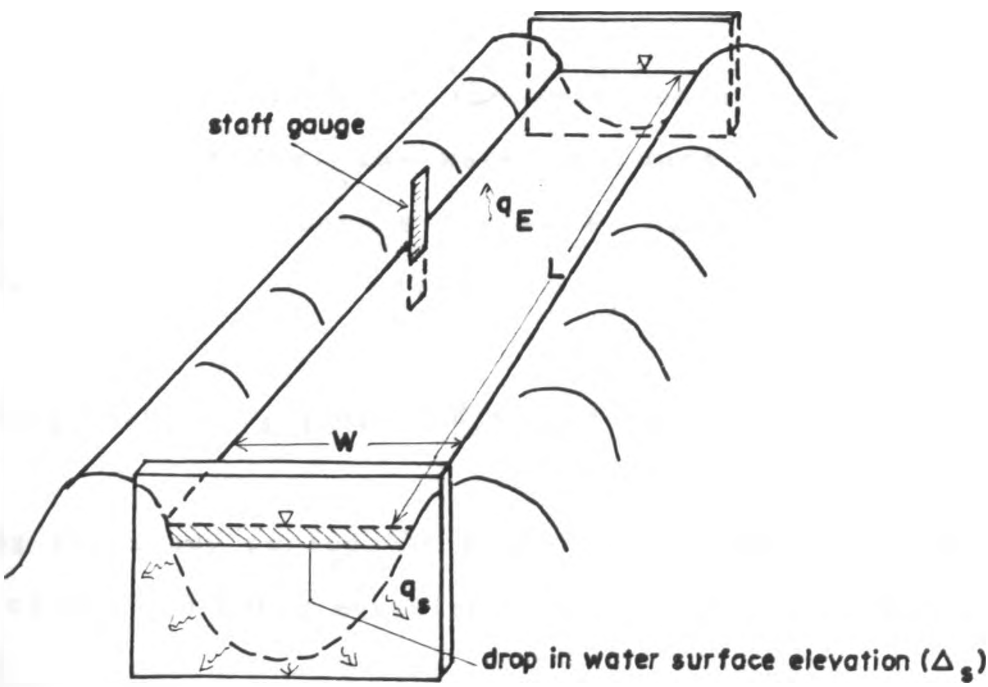


Figure 15: Water balance for reservoir

Readings on surface water level and evaporation was done at 9.00 am. consistently during the duration of the experiments. Rain gauge readings were also done at the same time in case of reception of any rain.

Average water depth in the reservoir was determined by use of levelling staff, lowered by two men from a boat. To facilitate depth measurements, the length of the reservoir was subdivided in equal intervals. The same was done on the width of the reservoir. The results on depth of water were read directly by use of levelling staff and the readings recorded.

The study period for the water balance equation: was approximately 1 day. The water balance is presented in Equation (14). Parameters for water balance are shown in figure (2).

$$Inflow_i + Rain_i = (d_{i+1} - d_i) + E_i + Sp_i + Outflow \qquad (14)$$

During the study period the reservoir inflow - outflow gates were closed and hence equation (14) reduces to equation (15);

$$Sp_i = Rain_i - E_i + (d_i - d_{i+1}) \qquad (15)$$

where;

Sp_i = Seepage depth (mm/day) for day i

$Rain_i$ = Rainfall during day i (mm/day)

E_i = Evaporation during day i (mm/day)

d_i = Water level during day i (mm)

d_{i+1} = Water level during d_{i+1} day

3.5. Crop water requirements

Crop water requirements were computed for the purpoise of estimating irrigation water requirements and hence compare the planned irrigation requirement at the pre-construction stage of the project.

The data for weather parameters necessary in computation of crop water requirements were obtained from Bura Meteorological Station. Meteorological data from this station covered the data recorded from 1983 to 1988. The weather parameters recorded in the station are; daily rainfall, maximum and minimum temperature, relative humidity, evaporation, sunshine hours and wind run values.

Penman method (1948) was adopted for crop water requirements. Penman method is suggested for areas where measured data on temperature, humidity, wind and sunshine or radiation is available (Doorenbos and Pruitt, 1977).

From this data reference crop evapotranspiration in mm/day is computed by the equation:

$$ETo = c[W.Rn + (1-W)f(u)(ea - ed)] \qquad (14)$$

where ETo = reference crop evapotranspiration in mm/day; W = temperature-related weighting factor; Rn = net radiation in equivalent evaporation (mm/day); f(u) = wind related function; (ea - ed) = difference between the saturation vapour pressure at mean air temperature and the mean actual vapour pressure of the air, both in mbar; c = adjustment factor to compensate for the effect of day and night weather conditions.

The description of variables and their method of calculation is elaborated by Doorenbos and Pruitt (1977) and presented in Appendix 111.

4. RESULTS AND DISCUSSIONS

4.1 Supply Canal

Seepage evaluation in the supply canal was done using inflow-outflow and seepage meter methods. The results obtained from the two methods are presented in the subsequent sections.

4.1.1 Inflow-outflow results in supply canal

The details apertaining to the inflow-outflow measurements are shown in Appendix VI. The details include the date of measurement, width of channel, depth of water in the channel at the four measuring points. The results on seepage in the supply canal are presented in Table 4.

Table 4: Inflow-outflow results

		Seepage rate					
Date	Inflow	1-11km		11-21km		21-31km	
	m^3/s	L/S/km	mm/day	l/s/km	mm/day	l/s/km	mm/day
15/11/88	1.083	39.3	9.3	26.6	340.4	2.0	21.9
23/11/88	2.447	39.6	382.2	32.1	410.8	12.6	138.7
25/11/88	2.002	15.7	151.2	36.3	464.6	28.0	308.1
10/12/88	2.157	13.5	130.2	17.5	223.9	36.6	402.8
12/12/88	2.397	24.9	240.3	5.4	69.1	25.1	276.2
14/12/88	2.361	16.2	156.3	12.2	156.1	22.2	244.3
Average	2.075	24.9	178.3	21.7	277.5	21.1	232.0

As can be deduced from Table 4 average seepage rates for the three reaches under consideration show that seepage rates are almost a constant (24.9, 21.7 and 21.1 l/s/km). Seepage rates show a slight wider variation for the same reaches as expressed in mm per day (178.3, 277.5 and 232.0 mm per day). Seepage rates equal to or higher than 50 l/s/km are considered high. Figure 16 represents a scatter diagram for seepage readings.

As can be observed from figure 16, the distribution of seepage is equally spread around the mean seepage. Variations on seepage measured in the same day could be attributed to lack of attainance of steady flow conditions in the canal and errors in taking readings.

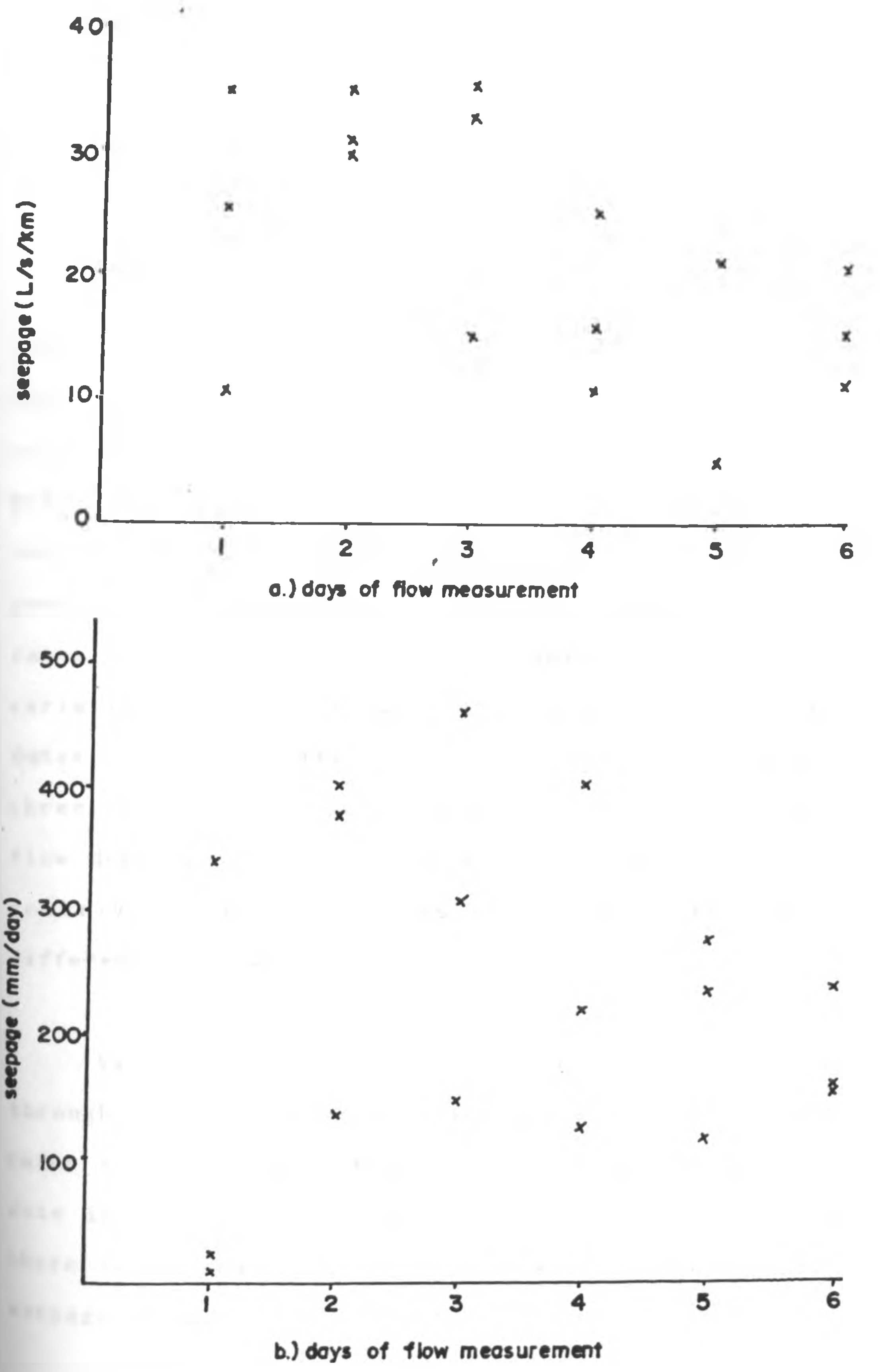


Figure 16: Distribution of seepage in the supply canal.

The results indicate that there is a significant variations between seepage rates taken on different dates in the same reach. In the first reach (1 - 11km) seepage rates range from 13.5 to 39.6 l/s/km. In the second reach (11 - 21km) the range is from 5.4 to 36.3 l/s/km while the third reach (21 - 31km) seepage ranges from 2.0 to 36.6 l/s/km. The temporal variations are not directly related to the inflow rates as can be observed in the table. Though there is some variation between seepage rates in a reach for different dates, there is little variation between the means for the three reaches. Thus variations in flow conditions such as flow depth or due to errors in taking readings could have contributed to variations observed in the reach for different readings.

Variations of inflow rates influence seepage rate through the influence of wetted perimeter. Increased inflow rates increase the wetted perimeter thus increasing seepage rate in the canal. However, as can be observed in Table 4 there is no consistent direct relation between inflow and seepage rates.

Representative wetted perimeter of the canal is shown in Figure 17.

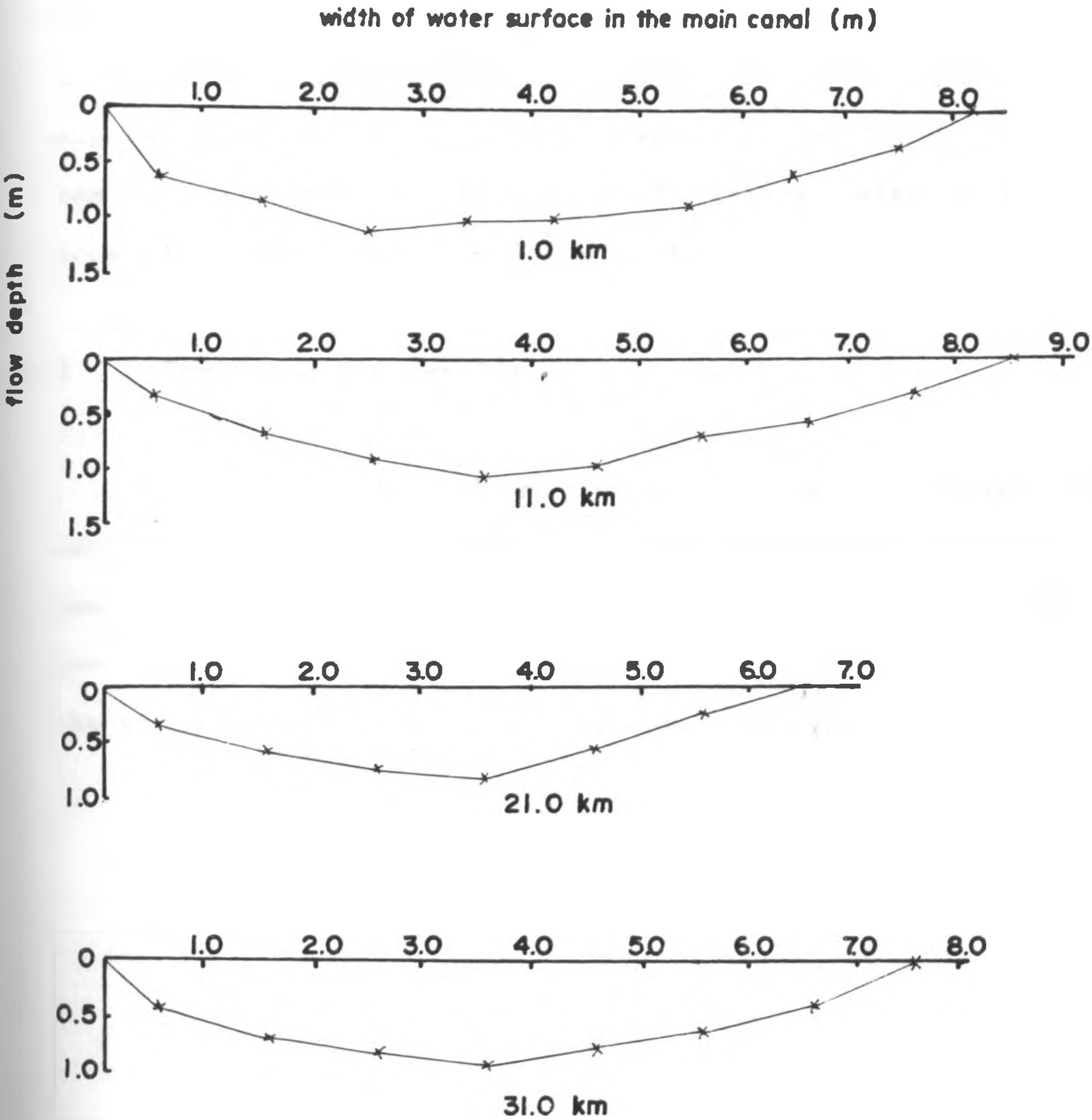


Figure 17: Bed shape of main canal at the flow measuring points.

The differences between lengths for wetted perimeter of the canal as measured from figure 17 show that the difference among the four plotted sections is not significant. The results of lengths of wetted perimeter expressed as ratios was obtained as 1.3: 1.3: 1.2:1. Thus it can be concluded that influence of seepage rates by wetted area of the main canal is not significant.

4.1.2 Seepage meter results

Seepage meter readings for evaluation of seepage rate were recorded in millilitres per hour. The readings were then converted from millilitres per hour into litres per second per kilometre (l/s/km). A sample calculation to illustrate the procedure of conversion is shown in Appendix VIII.

The results of seepage obtained by seepage meter method is presented in Table 5.

Table 5: Seepage meter results

	Seepage in l/s/km at chainage (km)			
Replicate	11.0	21.0	31.0	41.0
1	9.44	9.48	9.50	9.70
2	9.30	9.29	9.58	9.65
3	9.56	9.15	9.73	9.55
Average	9.43	9.31	9.60	9.63

Seepage results obtained at four points along the supply canal using a seepage meter gave a consistent level of seepage in the supply canal (See Table 5). The level of seepage is almost constant as can be deduced from the four averages, namely, 9.43, 9.31, 9.60 and 9.63 l/s/km.

Taking an average inflow of 2.525 m^3/s obtained from six consistent inflow rates measured with the current meter, seepage rates obtained by seepage meter are expressed as a percentage of the inflow rate. These give 0.37, 0.37, 0.38

per cent per kilometre of the inflow respectively. Total length of supply canal is 36.8 km. Therefore total water lost in the supply canal is 14.0 per cent of the discharge of the head of supply canal. The results of seepage rate in the supply canal are indicative of low levels of seepage.

4.2 Bura Branch Canal

Seepage losses in Bura Branch Canal using inflow-outflow method have been computed for the first canal reach (0 - 1.5 km) downstream of the canal off-take. Flow measurements at the other two points downstream were affected by abstraction of water to the night reservoirs. Closure of the reservoirs intake gates, during time of study was not feasible since supply for irrigation water was in great demand in the scheme.

Table 6: Seepage losses in l/s/km in the Bura branch canal

Replicate	Inflow m^3/s 0.0km	Outflow m^3/s 1.5 km	Seepage l/s km
1	0.750	0.661	59.3
2	0.653	0.579	49.3
3	0.586	0.486	66.7
4	0.591	0.589	1.3
5	0.574	0.520	36.0
6	0.556	0.435	80.7
7	0.677	0.560	78.0
8	0.322	0.295	18.0
Average	0.589	0.516	55.43

As deduced from Table 6 seepage rates are high in the measured reach of the Bura branch canal. The average wetted perimeter for the reach is 4.23 m. Seepage losses expressed as $l/s/m^2$ are shown in Table 8.

Table 7: Wetted area of Bura branch canal reach (0 - 1.5 km)

Replicate	wetted Perimeter m	wetted area m^2
1	4.25	6375
2	4.20	6300
3	4.25	637
Average	4.23	6350

Table 8: Seepage losses in $l/s/m^2$ of Bura branch canal

Replicate	seepage l/s/km	losses $l/s/m^2$
1	59.3	14.0×10^{-3}
2	49.3	12.0×10^{-3}
3	66.7	16.0×10^{-3}
4	36.0	8.5×10^{-3}
5	80.7	19.0×10^{-3}
6	78.0	18.0×10^{-3}
7	18.0	4.3×10^{-3}
Average	55.4	13.0×10^{-3}

Comparing seepage rates in Table 8 with seepage rates from the general soils surrounding the canal in Table 11-4, shows that seepage losses in the Bura branch canal are on the higher level. The average seepage loss 55.0 l/s/km for the average inflow of 0.589 m^3/s correspond to 9.41 per cent water loss per kilometre. The values of seepage are high as compared to soil permeability coefficients for soils traversed by the branch canal (0.028 m/day) which is equivalent to 3.24×10^{-4} $l/s/m^2$

Using the empirical formula (Eqn 10), seepage losses in the branch canal are in the order of 0.03 per cent per kilometre as compared to an average of 9.41 per cent per km obtainted from exeperimental results in this study. Seepage losses from the branch canal are bound to be lower than obtained from the inflow-outflow method. This discrepancy could be attributed to errors during time of measurements. Low discharges recorded at the downstream measuring station would be attributed to the low number of revolutions made by the current meter as read from the counter due to greatly reduced water velocity at 0.8D due to increased friction of water with the canal bed at shallow water depths.

4.3 Night storage reservoir

Two night storage reservoirs were analysed for seepage losses. These are $NSPU_4$ and $NSPU_6$ (see section 3.4). The results on seepage are presented in Table 9.

Table 9. Reservior seepage losses

Date	Water Level Datum A.S.L.	$\triangle WL$ mm/day	E_{pan} mm/day	E^* mm/day	S mm/day
	NSPU4				
2.12.88	484				
3.12.88	475	9.0	6.5	5.4	3.6
4.12.88	465	10.0	6.5	5.4	4.6
5.12.88	492				
6.12.88	484	8.0	4.5	3.7	4.3
7.12.88	460				
8.12.88	450	10.0	7.5	6.2	3.8
9.12.88	442	8.0	5.8	4.8	3.2
10.12.88	432	10.0	6.6	5.5	4.5
11.12.88	424	8.0	5.5	4.6	3.4
Average					3.9
	$NSPU_6$				
1.2.89	889				
2.2.89	891	8.0	7.0	5.8	2.2
3.2.89	882	9.0	6.8	5.6	3.4
4.2.89	973	9.0	6.5	5.4	3.6
5.2.89	863	8.0	6.3	5.2	2.8
6.2.89	859	6.0	5.5	4.6	1.4
7.2.89	850	9.0	5.0	4.2	4.8
8.2.89	844	6.0	5.5	4.6	1.4
9.2.89	830	7.0	5.3	4.4	2.6
Average					2.8

NB:

1. Datum above sea level (A.S.L); $NSPU_4$ = 101.0 m, $NSPU_6$ = 97.0 m.
2. E^* = E_{pan} readings (mm/day) x 0.83.
3. C.F. = 0.83 for pan evaporation (correction factor)
4. P = Precipitation (mm/day) is not shown in the table 9.

The column on precipitation (p) was ommitted because only very few days experienced rain. Secondly, run-off from the reservior, embankment had the effect of raising water level in the reservior. Control of run-off inflow to the reservoir was not feasible and hence this caused discrepancy in the computation of seepage rates from water balance equation.

The water level change in the reservior for 24 hours period was obtained by reading the graduations on the already installed staff gauge at the reservior. Corrections for evaporation and rainfall within the similar period were also taken into consideration. The readings for evaporation losses and rainfall are recorded in mm/day. Since evaporation rate in the evaporation pan is higher than the pond (evaporation pan effect), evaporation loss readings from the evaporation pan are adjusted by multipying readings with a correction of 0.83.

Seepage losses in the reservior were computed from equation (17).

$$S = \Delta W + P - E \qquad (17)$$

S = Seepage loss mm/day

ΔW = Change of water level in mm/day

P = Precipitation in mm

E = Evaporation rate in mm/day

As observed in Table 9 seepage losses in NSPU4 range from 3.2 to 4.6 mm per day. In $NSPU_6$ seepage range from 1.4 to 4.8 mm/day. The average seepage rates for $NSPU_4$ and $NSPU_6$ are 3.9 and 2.8 mm per day respectively. This represents a low rate of seepage in the reserviors.

The wetted perimeters of the two reserviors, namely $NSPU_4$ and $NSPU_6$ were determined at the time of this study. The results obtained are shown in Table 10 and Table 11.

Table 10: Water depth (m) in $NSPU_4$

Measuring points breadth of reservoir	Measuring points at 100m interval on reservior length				
	A	B	C	D	E
1	0.62	0.67	0.67	0.69	0.60
2	1.60	1.57	1.64	1.52	1.50
3	1.53	2.17	1.84	1.72	1.47
4	1.10	1.10	1.10	0.80	0.60
Average	1.21	1.38	1.31	1.18	1.04

Data was collected on 5th December 1988.

The average water depth in the reservior at the time of measurements was obtained as 1.22m. Water surface area was (450 x 50) 22500 m^2. Thus, the volume of stored water was 27450 m^3.

The volume of water lost through seepage per day is (0.0039 x 22500) $87m^3$. This represents 0.52 percent loss per day of the stored volume.

Table 11: Water depth (m) in $NSPU_6$

Measuring points on the breadth at 12.0m interval	Measuring points at 60m interval on the length of reservoir							
	A	B	C	D	E	F	G	H
1	1.00	1.51	1.60	1.64	1.55	1.49	1.60	1.20
2	1.60	1.53	1.90	1.95	1.80	1.50	1.86	1.26
3	1.50	1.86	2.10	1.97	1.90	2.25	2.00	1.69
4	1.50	1.94	1.95	1.82	1.70	1.96	1.58	1.59
5	1.18	1.70	1.62	1.70	1.55	1.60	1.13	0.90
Average	1.36	1.71	1.83	1.82	1.70	1.76	1.63	1.33

Data collected on 3rd February 1989.

The average water depth in the reservoir at the time of measurement was 1.64m. Water surface area (455 x 55m) was 25025 m^2. Average seepage loss is 0.0028 m/day.This represents a volume of 70.1 m^3/day. Taking the average water depth in the reservoir as 1.64m, volume of water stored at the time of measurement was 41041 m^3. Comparing the volume of stored water and seepage loss per day, this represents 0.17 per cent of the stored volume of water.

The level of seepage in the two reservoirs is low. Looking at the soil unit in which $NSPU^4$ is located (pf 2.2)the permeability coefficient for this unit as given in the reference is 5 cm/day which is classified as low. Taking into consideration the sediment deposition in the reservoir wetted perimeter, seepage rates are lower than corresponding soil permeability coefficient due to this effect. The location of $NSPU_6$ in relation to soil types is soil unit 2.1 which is reported in the reference to have a permeability coefficient of 15 cm/day. The soil texture in this unit is sandy clay to clay. The effect of sediment deposition on the wetted perimeter has the effect of the reduction of seepage losses.

4.4 Comparisions of seepage rates from methods used

For effective comparisons seepage rates have been presented in mm/day in Table 12.

Table 12: Seepage Rates Comparisons (mm/day)

Section/ Method	Seepage per reach (mm/day) 0-11 km	11-21 km	21-31 km
Main canal:			
Inflow-outflow	222	89.9	297.2
Seepage meter	91.0	119.0	105.6
Mean soil permeability coefficients	18.5	19.4	18.5
Branch canal:	(0.0 - 1.5km):		
Inflow-outflow		1132.0	
Mean soil permeability		48.0	
Reserviors ($NSPU_4$ & 6):			
Ponding method	3.9	2.8	
Mean soil permeability (pf 2.2, 2.1)	100.0	100.0	

As can be deduced from Table 12, seepage rates measured by inflow-outflow are higher in the main canal as compared to results obtained from corresponding reaches measured by seepage meter. The rates of seepage from seepage meter show consistency, ranging from 91.0 to 119 mm/day. The average rate is 105.2 mm/day. Seepage rates for both inflow-outflow and seepage meter methods are much higher than the average soil permeability rates. This variation could be attributed to the effects of water depth in the canal in operation. Water depth has a direct relationship with seepage rate in the unlined canal.

The same trend can be observed for the branch canal i.e, seepage rates measured by inflow-outflow method are higher (1132.0 mm/day) than average soil permeability coefficients (48 mm/day).

In the case of the reserviors, seepage rates are much lower than the mean soil permeability coefficients. This is contrary to the deductions made for the main and branch canals. This variation could be attributed to sealing of pores in the wetted area of the reservior by sediment deposition. While the same could be said for both main and branch canal, in the case of the reserviors, the rate of sediment deposition is highest since water movement is minimal as compared to the canals.

Reasons for high variations between inflow-outflow and seepage meter methods as compared to permeability coefficients in the corresponding reach are discussed below.

4.4.1 Inflow-outflow (current meter) method

In the process of taking flow measurements by use of current meter, errors in flow measurement are attributed to;

(a) The propellor could be tilted either vertically or angled. This would result in underestimation of velocity readings.

(b) Interference of flow by the data collector. For optimal data collection, the collector should not obstruct the flow movement downstream of the propellor position.

(c) Obstructions and entanglement of the propellor by weeds.

(d) Changes in canal water level. For optimal conditions, steady state conditions may not have been met adequately in this study.

4.4.2 Seepage meter

(a) The reasons for low values can be attributed to lack of optimal loçation of meter on the canal wetted area.

(b) Errors could have been introduced during water level measurements by the collector.

4.5 Computation of annual seepage and evaporation losses

Computations were made in order to provide information on relative proportions in which water is lost in the supply and the distribution systems of the scheme. This information is vital if mitigation is to be proposed aimed at minimizing the water losses in the scheme.

4.5.1 Water losses in the reservoirs

Water losses in the reservoir are attributed to evaporation and seepage losses. Average pan evaporation rate during the time of the experiment (December 1988 and February 1989) was 4.7 mm/day. It is worthy noting, however, that the short rains of 1988 were exceptionally wet and longer than is usual in Bura (semi-arid) area. The average annual open water evaporation for Bura obtained over a large number of years is 6.4 mm/day (section 3.1.2). The latter evaporation rate was used to compute the amount of water lost annually through evaporation in both canals and the reservoirs. A standard reservoir dimension was adopted in the computation, namely, 450 m by 50 m, which is the dimensions of most of the reservoirs in the scheme.

The total number of operational reservoirs is nineteen. Annual seepage and evaporation losses are tabulated in Table 14.

Table 13: Distribution of NSR_s in Various Soil Units.

Soil unit	Branch canal	No of reservoirs	Measured seepage cm/day	Soil permeability coefficient cm/day
pf 2.1	Pumwani	6	0.28	15
pf 2.2	Bura, Chewele, Pumwani	11	0.42	5
pf 3.2	Chewele	2	0.42	0.4

Table 14: Annual seepage and evaporation losses in NSRs

Branch	No. of reser-voir	seepage rate m/day	seepage $m^3/yr.$ *1000	volume Evap. losses $m^3/yr.$ *1000	Total loss $m^3/yr.$ *1000
Pumwani	6	0.0028	138	315	453
Bura, Pumwani	11	0.0042	380	578	958
Chewele Chewele	2	0.0042	69	105	174
Percentage			37%	63%	100%

4.5.2 Computation of water loss from supply canal

Seepage meter results in Table 5 have been used in the computation of the water losses by seepage in the supply canal. The data obtained from seepage meter were adopted on the basis of its consistency. Calculation of water loss by evaporation from the supply canal follow subsequently.

Water lost by seepage:

Length of supply canal is 36.8 km.

Average wetted area of the supply canal (Appendix. IX) is 78500 m^2. Average seepage rate in the supply canal is 9.49 l/s/km which is equivalent to 0.0012 $l/s/m^2$. Total wetted area of supply canal (36.8 km) is (78500 x 36.8/10) 1288,880 m^2.

The rate of seepage is 37.8 $m^3/m^2/yr$ in the supply canal. Volume of water lost in the supply canal is 10,920,000 m^3/year.

Water loss by evaporation:

Average width of water surface is 7.1m at the time of experiment (November/December 1988). Average open water evaporation in Bura is 0.0064 m/day. Volume of water lost per year is 610,350 m^3/year.

4.5.3 Water losses in the branch canals

Water lost by seepage:

Seepage rate adopted for computation of water loss in Bura branch canal is 0.03 per cent of inflow as calculated using empirical formula (Eq.10). Average inflow rate at the time of the experiment was 0.589 m^3/s (Table 6). This gave seepage rate as 0.118 l/s/km. Average wetted area per km derived from Table 7 is 4233 m^2/km. Therefore water loss from the branch canals per year from seepage loss is 2.8×10^{-5} $l/s/m^2$. This is equivalent to 373 m^3/yr/km.

The total length of three operational branch canals to date is 29 km. Water loss is 108,373 m^3/year.

Water losses by evaporation:

Average width of water surface (at the time of experiment) was 2.5m.

Average open water evaporation for Bura is 0.0064 m/day. Therefore volume of water lost per year is 113,216 m^3/yr.

4.5.4 Crop and irrigation water requirements

Crop water requirements in the scheme were calculated during the period of the study (1988). The maximum cropped area for 1988 was in May (2374 ha).

Table 15:. Crop water requirements (1988)

Month	ETO mm/day	Crop factor kc	ETcrop mm/day
Jan	7.84		
Feb	6.56	0.20	1.31
Mar	7.01	0.34	2.38
Apr	6.91	0.65	4.49
May	8.44	1.01	8.53
Jun	6.66	1.09	7.26
Jul	7.18	1.09	7.82
Aug	7.04	1.03	7.26
Sept	7.40	1.03	7.62
Oct	8.11		
Nov	5.71		
Dec	4.92		

Note: The crop factor used is for cotton only. The crop factor for cotton was computed for Hola by Lenselink K.J. (1985). The dash refers to months when cotton is not grown in the scheme.

Table 16: Irrigation water requirements (1988)

Month	Cropped area ha	ETc mm/day	Net water req./month m^3
Feb	643	1.31	235,822
Mar	1286	2.38	984,810
Apr	1830	4.49	2,465,010
May	2374	8.53	6,277,568
Jun	2137	7.26	4,654.386
Jul	1781	7.82	4,317,500
Aug	1187	7.26	267,146
Sep	119	7.62	9,068

The crop coefficient used in the calculation of ETcrop is for cotton. Cotton grows in the scheme from February to September. Other crops grown in the scheme are maize and groundnuts. Using the ETcrop of cotton, peak water requirements in the scheme are then computed. Hence, considerations are made for months of April, May, June and July when crop water requirements are highest and then compared with pumped volume in the same year (1988).

Table 17:. Comparison of gross irrigation requirements with pumped volume (1988)

Month	GIR (2500 ha) $m^3 x 10^3$	Volume pumped $m^3 x 10^3$
Apr	1,987	1,832
May	4,124	5,263
Jun	3,925	4,288
Jul	2,832	4,896

From Table 17 it can be observed that gross irrigation requirements and the pumped water volume in 1988, the values are close to each other. However, the peak gross irrigation requirements, in April are higher than the pumped water volume for the same month.

The pumped water volume in the other months indicate adequate water supply to the scheme as determined at the pumping station.

The total irrigated area under cotton in 1988 was 2374 ha. Cotton is the major crop during the first growing season in the scheme.

4.5.5 Implications of seepage losses to the scheme

4.5.5.1 Extent of water loss

Seepage and evaporation losses in the various sections of the scheme are expressed as a percentage of the total seepage and evaporation in the scheme respectively. The results are presented in Table 18.

Table 18: Comparison of water loss in the sections of the scheme

Section	Seepage loss m^2/yr $* 10^3$	per cent loss	Evaporation m^2/yr $* 10^3$	per cent loss
NSR	587	5.1	999	56.3
Branch Canals	108	0.9	113	6.4
Supply Canal	10,919	94.0	610	37.4
Total	11,614	100.0	1,722	100.0

From Table 18 it can be deduced that seepage losses are highest in the supply canal (94.0%) followed by NSRs (5.1%) and the least is from branch canals (0.9%). Evaporation losses are highest in night storage reservoirs (56.0%)

followed by supply canal contributing (37.4%) and branch canals with the least (6.4 %) of the total evaporation loss in the scheme. Variation in percentage losses are attributed to corresponding variations in length of wetted perimeter (seepage) and corresponding size variations of water surfaces in case of evaporation losses for various sections of the scheme.

4.5.5.2 Effects of seepage losses to groundwater table

Groundwater recharge was computed from supply and branch canals/night storage reservoirs. In the case of branch canals, effects from block and unit feeders from furrows have been neglected for the sake of simplifying the calculations since they do not carry water all the time. Since no information was available on groundwater aquifer, hypothetical values were adopted for computation of seepage effect to groundwater rise.

Soil investigations during planning stage of the project showed that water table was far below 30 meters of augering along the area traversed by the supply canal. For computation, groundwater table was assumed to be at least 30 meters or deeper from the ground surface.

The rate of groundwater rise was calculated as shown below;

Rate of groundwater rise

$$= \frac{\text{Volume of seepage water (canals and reservoirs)}}{\text{Area of aquifer} \times \text{Drainable porosity.}}$$

The effective porosity ranges for different soils was chosen using the table of classification developed by Johnson (1966). The effective porosity percentage chosen for corresponding soils in Bura was 3 - 12% (sandy clay soils) with a mean porosity of 7.0 percent (See appendix XI). Volume of seepage water is obtained from Table 18. Results of computation of effects of seepage on groundwater rise are summarised in Table 19.

Table 19: Rate of groundwater rise

Hypothetical area of aquifer (km^2)	Rate of rise m/yr	No. of years for water to reach surface
50	0.03	1000
80	0.02	1500
100	0.02	1500
120	0.01	3000

From Table 19 it can be concluded that the effects of seepage losses on the groundwater rise are negligible. Observations on the surrounding areas traversed by the supply canals showed no evidence of rising water table.

Since the land slopes gently towards Tana river, (Section 3.1.3.) there may also be drainage towards the river which would further remove any effects of seepage on the groundwater rise.

4.5.5.3 Implications of seepage losses for cost of pumping

Cost of pumping includes both fixed and operating costs. Due to unavailability of the pump details at installation period, only fuel costs have been estimated in this study.

For diesel or petrol engines, power required is obtained from equation 18.

$$P = \frac{Q \times H \times S_f \times A_f \times T_f}{102 \times E_p \times E_t} \qquad (18)$$

Where;

P = Power required (kw)

Q = Flow rate (l/s)

H = Total pumping head (m)

E_p = Pump Efficiency

E_t = Transmission Efficiency

S_f = Safetly factor (normally taken as 1.20)

A_f = Altitude derating factor (1% reduction for energy 100 m above sea level)

T_f = Temperature derating factor (2% reduction for every 5 deg C above 30 deg.)

Average flow rate Q for the 3 4 pumps used in Bura is 2.13 m^3/s. Total pumping head is 5.0m. Water is pumped into a siltation basin from where it flows by gravity to the head of the supply canal and subsequently to the distribution system.

Mean temperature is less than 30°C since mean maximum temperature is 31°C. Altitude at the pumping station is about 120m above sea level. Transmission is by 'V'-belt drives. Pump efficiency was taken to be 0.75.

$$P = \frac{2130 \times 5 \times 1.2 \times 1.01 \times 1}{102 \times 0.75 \times 0.95}$$

$$= 177.6 \text{ Kw}$$

But 1 Kw = 1.34 Hp

pump power = 238 Hp

Fuel consumption by pump is estimated as 0.23 l/BHPh (Michael, 1978).

BHPh = Brake horse power-hour.

The total number of hours the 3-4 pumps were operational, calculated from the pump records of the pump attendant is 6549 hours.

Price of fuel (diesel) was Ksh 6.00 per litre in 1988. Rate of fuel consumption was thus calculated as;

0.23 l/BHh x 238 = 55 l/hour.

Fuel consumption in 6549 hours = 55 x 6549

= 360195 l

Cost of fuel for pumping only = 360195 x 6

= 2161170

= 2.2 million Ksh/yr.

From Table 18 seepage losses per year for various sections of the scheme are summarized. Total seepage loss per year is 1.16×10^7 m^3. Total pumped water volume for the same period is $4.39 \times 10^7 m^3$. Seepage loss represents 26.5 per cent of the total pumped water. Fuel costs for various sections of the scheme are summarised in Table 20.

Table 20: Seepage losses per year for sections of the scheme.

Section	Volume seepage (m^3) $*10^3$	Volume of fuel Ksh 6/l	Cost of fuel (1988)	Maintenance cost (10% of fuel cost	Total cost Ksh
NSR	587	4,791	28746	2875	31621
Branch Canal	108	900	5400	540	5940
Supply Canal	10,919	89,688	538128	53813	591941
Grand Total					629502

It is thus, necessary to compute unit cost of seepage in terms of fuel in order to make a decision whether or not lining the canals should be done. The unit cost is used to determine the break-even point for making a decision on lining or not to line. For the computation of the unit cost of seepage surface area of the channel is necessary. The wetted area of the supply, branch canals and night storage reservoirs were obtained as 288 880, 122 757 and 450 680 m^2 respectively.

$$\text{Unit cost of seepage} = \frac{\text{Cost due to seepage loss}}{\text{surface area}}$$

The unit cost of seepage for various sections of the conveyance and distribution systems are summarized in Table 21.

Table 21: Unit cost of seepage for conveyance and distribution systems

Section	Seepage area m^2	Cost due to seepage Ksh	Unit cost Ksh/m^2
NSRs	450 680	31621	0.07
Branch Canals	122 757	5940	0.05
Supply Canal	288 880	591 941	2.05

It is therefore concluded that lining could be undertaken if the lining costs were less than the unit cost of water lost by seepage as shown in table 21.

5. SUMMARY AND CONCLUSIONS

Summary

1 After evaluation of seepage losses in the supply and distribution systems of the scheme, seepage losses in the supply canal were found to be on average 14.0 percent of the inflow. This represents an average seepage loss of 0.38 per cent per kilometre.

2 Seepage losses in the branch canal are in the order of 0.03 per cent per kilometre.

3 Seepage losses in the night storage reserviors were found to be on average 0.39 and 0.28 cm/day in the two storage reservoirs analysed. This represents a low seepage rate in the reserviors.

4 Water losses by seepage in $NSPU_4$ and $NSPU_6$ storage reservoirs represent 0.52 and 0.17 per cent per day loss of the maximum stored volume respectively.

5 Comparisons between seepage and evaporation losses in the reservoirs showed, that, out of the total water lost in the reservoirs, seepage losses contribute 37% while evaporation losses constitute to 63 per cent. This

evaporation to seepage losses in the storage reservoirs are (1.7:1), that is, about twice as high evaporation than seepage losses from the reservoirs.

6 Comparing seepage losses in the supply and distribution systems of the scheme, the results showed that 94% of the seepage losses occur in the supply canal while 5.1 and 0.9 per cent occur in the night ,storage reserviors and branch canals respectively. For evaporation losses from water distribution system, however, results indicate that 56.3, 37.4 and 6.4 per cent occur in the night storage reservoirs, supply canal and branch canals, respectively.

7 Computing the effects of seepage losses to groundwater water table, results showed that the effect of seepage losses to groundwater rise was negligible. Considering hypothetical aquifer areas, ranging from 50 to 120 km^2, the rate of rise of the groundwater table ranged from 0.03 to 0.01 metres per year, respectively. This is extremely low rate and would take 3000 - 1000 years for the groundwater table to reach the ground level.

8 Implications of seepage losses on cost of pumping were evaluated in this study by computing unit cost of seepage in Ksh/m^2 for the night storage reservoirs, branch and supply canals. The highest unit cost of seepage (Ksh 2.05/m^2) was obtained for the supply canal. Considering the current cost of lining 1 m^2 of the wetted area, costs of lining are bound to be much higher than Ksh 2.0 per metre square.

9 Lining as an option to reduce seepage losses is not attractive since the rates of seepage are low. It would not be economical to line the canals considering the amount of water to be saved. This concurs with the design assumptions of the scheme that seepage losses were anticipated to be low and thus no need for lining.

5.2 CONCLUSIONS

From the afore mentioned, conclusions are drawn that seepage losses in the supply, branch canals and night storage reserviors are low. Improvements on water use efficiency can be attained if the supply and the distribution systems are well operated and maintained.

Lining should not be considered as an option to improve conveyance and distribution efficiencies in the scheme. It is more beneficial to expend on maintenance of the system than to line the conveyance and distribution systems.

6. RECOMMENDATIONS

6.1 Detailed study on conveyance and distribution efficiencies of the scheme should be carried inorder to give an elaborate indication of efficiencies in various sections of the scheme. These would be precedent to working out a water management programme necessary for effective operation of the scheme.

6.2 The present pumping capacity is not adequate. A bigger and more reliable water supply is necessary to meet the irrigation water requirements.

6.3 Night storage reservoirs are efficient in water distribution as the losses are very low. They are recommended for further use in water distribution system when the scheme expansion is done.

6.4 A water management programme needs to be prepared with an aim of improving water use efficiency in the scheme.

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8. APPENDICES

Appendix 1. Soil Description

Table 1 -1 Soil textural results

Chainage (km)	Soil type
1.2	sandy clay loam loamy sand
3.0	clay soil
4.5	clay soil loamy sand sandy loam
7.6	clay soil
9.2	clay soil
10.5	sandy loam
12.0	loamy sand, sandy loam clay loam
14.0	sandy laom, clay soil
16	sandy clay loam
18	sandy clay loam loamy sand
20.0	clay soil
22.2	sandy loam, loam gravel clay
23.9	sandy clay loam
25.3	sandy clay loam
27.2	sandy loam clay, gravel/sand
29.0	loamy sand sandy clay loam

Table 1 -1 Cont'd

29.6	loamy sand
30.8	loamy sand gravel/sand
32.5	loamy sand
34.0	sandy loam gravel/sand
37.0	loamy sand sandy clay loam
39.0	loamy sand sandy clay loam gravel/sand
40.5	sandy clay loam

Source: ILACO (1975).

<u>Table 1 - 2:</u> Soil texture (Bura and Pumwani Canals).

Bura branch canal		Pumwani branch canal	
Chainage (km) Unit	Soil unit	Chainage (km)	Soil unit
0	Pf 1.2	0	Pf 1.2
1.5	pf 2.2	1.5	Pf 2.4
3.0	pf 2.2 pf 2.3	3.0	Pf 2.2 Pf 2.2
4.5	pf 2.3	4.5	Pf 2.1
6.0		6.0	Pf 2.1 Pf 2.4
		7.5	Pf 2.5
		9.0	Pf 2.5

Source: ILACO (1975)

Table 1 - 3 <u>Legend</u>

Soil Unit	Soil type	FAO-Revised classification
Pf 1.2	Moderately well drained to imperfectly drained, very deep, dark reddish brown to brown, firm sandy clay to clay	Calcaric-haplic SOLENETZ Salic phase
Pf 2.1	Well drained, very deep dark reddish brown to dark brown, friable sandy clay to clay	Chromic calcaric CAMBISOLS Partly sodic phase
Pf 2.2	Well drained, very deep dark red to dark reddish brown, friable to firm, sandy clay to clay	Chromic calcaric CAMBISOLS Salic-sodic phase
Pf 2.3	as in Pf 2.2	
Pf 2.4	Well drained to moderately well drained, very deep dark reddish, brown firm clay	Calcaric-haplic SOLONETZ Salic phase
Pf 2.5	as in Pf 2.4	
Pf 3.2	Moderately well drained to imperfectly drained very deep, dark reddish brown, firm to very firm cracking clay	Chromic-calic and chromic haplic VERTISOLS salic sodic phase

Source: ILACO (1975)

Appendix II Soil permeability data

Table II - I Soil permeability coefficients along the supply canal.

Chainage (km)	Permeability (m/day)	Chainage (km)	Permeability (m/day)
1.2	0.004	22.2	0.010
3.0	0.002	23.9	0.010
4.5	0.041	25.3	0.016
5.9	0.027	27.2	0.013
7.6	-	29.0	0.035
9.2	-	29.6	0.027
10.5	-	30.8	-
12.0	0.023	32.0	0.047
14.0	0.023	34.0	0.110
16.0	0.016	37.0	0.660
18.0	0.016	39.0	-
20.0	0.019	40.5	0.018

Table II - 2: Soil permeability coefficients along the branch canal.

Mapping unit	Permeability coefficients (m/day)
Pf 2.1	0.150
Pf 2.2	0.050
Pf 2.3	0.030
Pf 3.1	0.005
Pf 3.2	0.004

Source: Muchena (1987)

Table II - 3: Seepage rates of general soil groups.

General Soil Group	Average rate (m/day)
Clay	0.20
Silty	0.17
Loamy	0.26
Sandy	0.58
Unspecified	0.35

Source: Worstell (1976)

Table II- 4 Seepage losses per m^2 of wet canal perimeter for various soils.

Surrounding soils	$l/s/m^2$
Clay	$1.0* \times 10^{-3}$
Loamy clay	$2.1* \times 10^{-3}$
Sandy clay	$2.3* \times 10^{-3}$ $-4.6* \times 10^{-3}$
Sand	$5.8* \times 10^{-3}$
Sand-gravel	$8.7* \times 10^{-3}$
Gravel	$1.2* \times 10^{-2}$ - $2.1* \times 10^{-3}$

Source: Nugteren J. (1977).

Appendix III. Computation of crop water requirements.

Penman method.

Penman method (1948) is suggested for aeas where measured data on temperature, humidity, wind and sunshine duration or radiation are available. Compared to other methods for crop water requirements computations, it is likely to provide the most satisfactory results (Doorenbos and Pruitt, 1977). This is obtained by the formula;

$ETo=c[W.Rn + (1-W).f(u). (ea-ed)]$ (see Eqn 14) where:

ET_o = reference crop evapotranspiration in mm/day

W = temperature related weighting factor

Rn = net radiation in equivalent evaporation in mm/day

f(u) = wind - related function

(ea-ed) = difference between the satuvation vapour pressure at mean air temperature and the mean actual vapour pressure of the air, both in mbar.

c = adjustment factor to compensate for the effect of day and night weather conditions.

<u>Vapour pressure (ea - ed)</u>

This is obtained from the table after the following weather parameters have been measured, namely, T max. T min, RH max and RH min.

<u>Wind function f(u).</u>

The effect of wind on ETo has been studies for different climates resulting in a revised wind function and defined as:

$f(u)=0.27\ (1 + U/100)$

where;

U=24 - hr wind run in km/day at 2m height.

Where wind data are not collected at 2m height, the appropriate corrections for wind measurements taken at different height are read from a table (Doorenbos and Pruitt, 1977).

Weighting factor (1 - w)

(1 - w) is a weighting factor for the effect of wind and humudity on ETo. Values of (1 - w) as related to temperature and altitude and altitude are given in a table.

Weighting factor (W)

W is the weighting factor for the effect of radiation on ETo. Values of W as related to temperature and altitude are given in prepared tables.

Net radiation (Rn)

Net radiation (Rn) is the difference between all incoming and outcoming radiation. It can be measured, but such data are seldom available. Rn can be calculated from solar radiation or sunshine hours (or degree of cloud cover), temperature and humidity data.

Total nct radiation (Rn) is equal to the difference Rns and Rnl i.e. Rn=Rns - Rnl.

<u>Adjustment factor (c)</u>

The Penam equation (eqn 14) assures the most common conditions where radiation is medium to high, maximum relative humidity is medium to high and moderated daytime wind about double thc night time wind. However, there conditions are not always net.

Table III - 1 Weather parameters

1983			TEMP ^{0}C		R/HUMIDITY		EVAP.	RAD.	S/SHINE	W/RUN
Month	R/fall	raindays	max	min	9.00hrs	1500hrs	mm	mJ/m^2	hrs	m/s
Jan.	-	-	-	-	-	-	-	-	-	-
Feb.	-	-	-	-	-	-	-	-	-	-
Mar.	-	-	-	-	-	-	-	-	-	-
Apr.	-	-	-	-	-	-	-	-	-	-
May	28.00	10	33.40	26.70	75	57	7.4	-	-	3.4
Jun.	7.90	2	32.40	22.40	69	54	6.9	-	-	4.2
Jul.	0.0	0	31.90	21.30	69	48	7.7	-	-	4.3
Aug.	2.70	1	31.70	21.20	64	44	8.5	-	7.4	4.1
Sept.	20.10	3	31.10	20.80	65	42	8.2	-	7.3	4.0
Oct.	0.00	0	34.30	23.00	65	38	9.1	-	6.8	3.4
Nov.	21.60	3	35.20	23.80	70	47	7.7	-	6.9	2.4
Dec.	58.00	6	34.20	25.80	75	48	6.7	7.8	7.2	1.7

Table III-I Cont'

1984											
Jan	0	0	35.3	22.4	75	40	8.1	22.17	7.7	2.0	
Feb	0	0	35.5	22.1	71	44	8.3	22.14	6.8	4.7	
Mar	0	0	36.9	23.9	68	37	8.2	23.4	17.9	2.6	
Apr	11.1	5	36.4	25.6	68	44	9.6	20.6	67.9	3.7	
May	5.4	2	33.8	22.8	66	51	9.1	20.1	97.3	4.1	
Jun	14.7	3	30.9	21.2	66	49	7.4	19.3	76.9	4.7	
Jul	13.0	4	30.5	19.5	70	48	7.3	16.90	6.4	4.2	
Aug	0.0	0	30.9	20.0	68	45	8.4	17.50	6.0	4.3	
Sep	7.0	13	2.0	20.4	68	40	9.4	21.68	7.1	4.0	
Oct	10.0	2	33.0	22.3	66	45	8.4	21.39	7.1	3.5	
Nov	84.5	11	33.7	23.5	68	45	7.0	22.03	7.5	2.0	
Dec	56.0	4	33.1	22.1	73	51	5.8	21.27	6.5	1.3	
1985											
Jan	14.7	3	34.7	22.4	73.5	44.2	6.2	21.89	6.6		1.5
Feb	7.1	2	35.8	23.0	70.0	40.0	7.5	23.59	7.7		1.6
Mar	12.5	4	35.9	23.5	70.0	43.0	8.2	22.22	7.2		2.4
Apr	30.0	7	35.1	23.6	71.4	47.0	7.8	19.65	7.6		3.1
May	23.2	3	32.7	22.0	71.4	54.5	7.3	18.83	6.2		3.5
Jun	2.4	1	31.7	20.3	70.5	50.4	7.7	17.17	6.3		3.9
Jul	7.0	4	30.8	19.6	71.8	52.6	7.5	17.60	6.6		4.2
Aug	9.9	4	31.2	20.1	72.4	48.5	8.0	16.69	6.4		4.6
Sep	7.5	5	33.1	20.7	71.0	43.4	8.4	20.02	7.8		4.2
Oct	17.9	5	33.1	21.2	77.4	49.5	8.4	19.90	7.3		3.8
Nov	29.0	6	34.7	23.6	75.0	50.7	7.8	22.00	7.8		2.6
Dec	8.1	1	34.4	24.3	72.0	44.5	9.0	21.71	6.8		1.9

Table III-1 Cont'

1986										
Jan	0.0	0	35.0	23.17	0.0	39.0	8.5	22.46	7.0	1.6
Feb	0.0	0	36.0	23.16	6.0	41.0	7.8	22.46	7.6	2.2
Mar	22.3	5	35.6	24.17	1.0	47.0	8.4	22.18	7.4	2.4
Apr	150.3	9	34.4	23.97	4.0	49.0	8.4	21.00	7.0	3.0
May	66.2	11	31.9	22.8	76.0	55.0	6.3	17.60	6.0	3.6
Jun	3.0	13	1.7	21.3	70.0	47.0	7.5	18.06	6.6	4.0
Jul	3.6	1	30.8	20.0	72.0	45.0	7.7	17.33	6.3	4.3
Aug	11.3	6	31.2	19.7	73.0	47.0	7.5	18.37	7.4	3.8
Sep	4.9	2	32.9	21.0	68.0	43.0	8.0	19.53	6.7	4.0
Oct	4.1	2	35.2	23.3	65.0	43.0	8.8	22.17	8.0	3.3
Nov	52.1	9	34.7	23.7	70.0	54.0	7.1	19.60	7.0	2.2
Dec	112.3	9	32.5	22.9	77.0	54.0	5.3	18.90	6.7	1.0
1987										
Jan	0.0	0	38.8	20.8	72	40	6.6	21.89	7.0	1.1
Feb	0.0	0	35.7	23.1	70	40	7.6	22.09	7.6	1.3
Mar	0.0	0	36.5	24.2	69	43	8.7	22.36	7.5	1.6
Apr	64.9	6	35.2	24.67	74	57	7.8	16.24	7.8	2.1
May	15.0	2	32.7	23.6	76	55	8.1	18.57	6.9	3.4
Jun	0.0	0	31.7	24.5	67	54	7.0	20.68	6.9	3.8
Jul	10.3	3	31.0	26.9	75	46	6.6	22.07	8.3	4.7
Aug	55.9	7	31.2	20.0	76	50	5.8	18.95	6.6	6.0
Sep	0.0	0	32.2	21.6	68	40	8.2	17.34	6.6	5.0
Oct	0.0	0	34.1	23.1	67	41	8.9	20.71	7.9	3.5
Nov	52.2	8	34.7	23.7	70	66	7.2	19.60	7.1	2.2
Dec	112.3	9	32.5	22.9	77	54	5.3	18.90	6.7	1.0

Table III-1 Cont'

1988

Jan	13.7	3	36.0	23.9	86	63	6.0	19.22	6.9	4.56
Feb	0.0	0	35.0	24.4	73	42	8.2	21.27	7.6	1.34
Mar	43.4	2	36.0	24.6	65	45	6.3	20.59	8.0	1.60
Apr	13.2	1	34.8	24.8	74	51	7.2	18.61	4.8	3.02
May	0.0	0	32.3	23.4	69	46	8.5	17.77	8.0	4.07
Jun	25.5	9	31.7	21.8	74	54	5.3	14.43	5.7	3.41
Jul	2.1	1	30.8	21.2	69	46	6.7	15.84	7.0	3.46
Aug	10.2	4	31.2	21.3	72	45	6.4	16.88	7.8	3.12
Sep	11.3	4	32.0	20.9	69	43	6.6	17.64	7.1	3.06
Oct	32.6	9	34.7	22.7	65	37	7.0	20.29	8.1	2.80
Nov	204.2	9	33.4	23.1	74	49	2.1	20.18	7.1	1.10
Dec	191.6	16	32.7	22.3	77	56	0.2	18.34	7.6	0.45

NB

HEIGHT OF ANENOMETER 2M

Table III - 2 RATIO OF n/N LATITUDE 5º

			Year			
Month	1983	1984	1985	1986	1987	1988
Jan		0.65	0.56	0.59	0.59	0.58
Feb		0.57	0.65	0.64	0.64	0.64
Mar		0.66	0.60	0.62	0.63	0.63
Apr		0.65	0.62	0.57	0.64	0.39
May		0.59	0.50	0.49	0.56	0.65
Jun		0.56	0.51	0.53	0.56	0.46
Jul		0.52	0.54	0.51	0.67	0.57
Aug	0.60	0.40	0.52	0.60	0.54	0.543
Sep	0.60	0.59	0.64	0.55	0.55	0.59
Oct	0.57	0.59	0.61	0.67	0.66	0.68
Nov	0.58	0.63	0.66	0.59	0.60	0.61
Dec	0.61	0.55	0.58	0.57	0.57	0.58

Table III - 3 Crop water requirements computation per month.

Weather parameters

Tmax	Tmin	Tmean	Rhmax	Rhmin	Rhmean	ea	ed
36.00	23.90	29.95	0.86	0.63	0.75	42.40	31.59
35.80	24.40	30.10	0.73	0.42	0.58	42.40	24.60
36.00	24.60	30.30	0.65	0.45	0.55	42.40	23.30
34.80	24.80	29.80	0.74	0.51	0.63	42.40	26.70
32.30	23.40	27.85	0.69	0.46	0.58	37.80	21.90
31.70	21.80	26.75	0.74	0.54	0.64	35.70	22.80
30.80	21.20	26.00	0.69	0.46	0.58	33.60	19.50
31.20	21.30	26.15	0.72	0.45	0.59	33.60	19.80
32.00	20.90	26.45	0.69	0.43	0.56	33.60	18.80
34.70	22.70	28.70	0.65	0.37	0.51	37.80	19.30
33.40	23.10	28.25	0.74	0.49	0.62	37.80	23.40
32.70	22.30	27.50	0.77	0.56	0.67	35.70	23.90

Table III - 3 Cont'

ea-ed	U24hrs	f(u)	W	1-W	n/N	Ra	Rns
10.81	394.00	1.33	0.78	0.22	0.58	15.00	6.08
17.80	116.00	0.58	0.78	0.22	0.64	15.50	6.63
19.10	138.00	0.64	0.78	0.22	0.63	15.70	6.65
15.70	261.00	0.97	0.78	0.22	0.39	15.30	5.11
15.90	352.00	1.22	0.77	0.23	0.56	14.40	6.21
12.90	295.00	1.07	0.75	0.25	0.46	13.90	5.00
14.90	299.00	1.08	0.75	0.25	0.57	14.90	6.66
13.80	270.00	1.00	0.75	0.25	0.54	14.80	5.77
14.80	264.00	0.99	0.75	0.25	0.59	15.30	6.25
18.50	242.00	0.92	0.77	0.23	0.68	15.40	6.81
14.40	95.00	0.53	0.77	0.23	0.61	15.10	6.29
11.80	39.00	0.38	0.76	0.24	0.58	14.80	5.99

f(T)	f(ed)	f(n/N)	Rnl	Rn	c	ETo	kc
16.70	0.10	0.60	1.00	5.07	1.10	7.84	0.00
16.70	0.12	0.69	1.38	5.24	1.03	6.56	0.20
16.70	0.13	0.64	1.39	5.26	1.03	7.01	0.34
16.30	0.12	0.42	0.82	4.28	1.03	6.91	0.65
16.30	0.13	0.69	1.46	4.75	1.04	8.44	1.01
15.90	0.13	0.51	1.05	3.95	1.04	6.66	1.09
15.90	0.15	0.60	1.43	4.23	1.03	7.18	1.09
15.90	0.14	0.60	1.34	4.44	1.04	7.04	1.03
15.90	0.15	0.64	1.53	4.73	1.03	7.40	1.03
16.30	0.15	0.69	1.69	5.13	1.03	8.11	0.00
16.30	0.13	0.64	1.36	4.93	1.03	5.71	0.00
16.30	0.12	0.60	1.17	4.82	1.04	4.92	0.00

Table III - 3 Cont'

f(A)(ha)	ETc	Days	ET -Mon	EFF-Rain	NIR(m^3) x 10^4
0.00	0.00	31.00	0.00	11.00	0.00
643	1.31	29.00	38.08	5.00	33.08
1286	2.38	31.00	73.88	34.00	39.88
18.30	4.49	30.00	134.75	81.00	53.75
2374	8.53	31.00	264.37	14.00	250.37
2137	7.26	30.00	217.72	5.00	212.71
1781	7.82	31.00	242.49	7.00	235.49
1187	7.26	31.00	224.94	2.00	222.94
119	7.62	30.00	228.58	19.00	209.58
0.00	0.00	31.00	0.00	26.00	0.00
0.00	0.00	30.00	0.00	58.00	0.00
0.00	0.00	31.00	0.00	45.00	0.00

APPENDIX IV: Monthly pumped volume of water (m3)
Table IV-I : Pump operation (Jan - Dec, 1988)
Pump Number/duty (m3/s)

Date	Pump Number 1	2	3	4	5	6	
	2.15	2.15	1.075	1.075	2.5	2.5	Total
4.1.88			23	23			49.5
5.1.88		7	23	10			50.5
6.1.88		10	23	23			71.0
7.1.88			22	18			43.0
8.1.88			23	23			49.5
9.1.88			22	22			47.3
10.1.88			18	22			43.0
11.1.88		5	23	23			60.2
12.1.88		23	23	23			98.9
13.1.88		23	23	20			95.7
14.1.88		23	23				74.2
15.1.88		23	23				74.2
16.1.88		23	23				74.2
17.1.88		23	23				74.2
19.1.88		23	23				74.2
20.1.88		23	23				74.2
21.1.88		23	23				74.2
22.1.88		10	10				32.3
23.1.88		10	10				32.3
24.1.88		10	10				32.3
25.1.88		11	11				35.5
26.1.88		11	11				35.5
27.1.88		10	10				32.3
28.1.88		10	10				32.3
29.1.88		10	10				32.3
30.1.88		10	10				32.3
31.1.88		10	10				32.3

1456.6

Table IV - I Cont'

1.2.88	10	10		32.3	
2.2.88	10	10		32.3	
3.2.88	10	10		32.3	
4.2.88	10	10		32.3	
5.2.88	15	15		48.4	
6.2.88	22	22	10	81.7	
7.2.88	15	15	15	64.5	
8.2.88	15	15	15	64.5	
9.2.88	15	15	15	64.5	
10.2.88	15	15	15	64.5	
11.2.88	15	15	15	64.5	
12.2.88	10	10	10	43.0	
13.2.88	15	15	15	64.5	
14.2.88	15	15	15	64.5	
15.2.88	15	15	15	64.5	
16.2.88		12	23	37.6	
17.2.88	16	16	23	76.3	
18.2.88	15	15	15	64.5	
19.2.88	15	6	15	54.8	
20.2.88	15	15	15	64.5	
21.2.88				0.0	
22.2.88				0.0	
23.2.88				0.0	
24.2.88	12		12	38.7	
25.2.88				0.0	
26.2.88				0.0	
27.2.88	14		14	45.2	
28.2.88	23	15	23	90.3	
29.2.88					1290.0

Table IV - I Cont'

1.3.88	20	20	20	86.0	
2.3.88	15	15	15	64.5	
3.3.88	15	15	15	64.5	
4.3.88	15	15	15	64.5	
5.3.88	20	20	20	86.0	
6.3.88	20	20	20	86.0	
7.3.88	18	18	18	77.4	
8.3.88	18	18	18	77.4	
9.3.88	15	15	15	64.5	
10.3.88	20	20	20	86.0	
11.3.88	20	20	20	86.0	
12.3.88	20		20	64.5	
13.3.88			23	24.7	
14.3.88				0.0	
15.3.88			23	24.7	
16.3.88		10	18	30.1	
17.3.88		23	6	31.2	
18.3.88		20	23	46.2	
19.3.88		23	23	49.5	
20.3.88		20	20	43.0	
21.3.88		23	23	49.5	
22.3.88		23	23	49.5	
23.3.88		23	23	49.5	
24.3.88		23	23	49.5	
25.3.88		23	23	49.5	
26.3.88		23	23	49.5	
27.3.88		23	23	49.5	
28.3.88		23	23	49.5	
29.3.88		23	23	49.5	
30.3.88		23	23	49.5	
31.3.88		20	20	43.0	
					1694.2

Table IV - I Cont'

1.4.88			20	15		37.6	
2.4.88			20	6		28.0	
3.4.88			20	20		43.0	
						0.0	
18.4.88				16	16	57.2	
19.4.88				16	16	57.2	
20.4.88				16	16	57.2	
21.4.88				16	16	57.2	
22.4.88				16	16	57.2	
23.4.88				16	16	57.2	
24.4.88				16	16	57.2	
							509.0
2.5.88	16			16		51.6	
3.5.88	16			16		51.6	
4.5.88	16			16		51.6	
5.5.88	16			16		51.6	
6.5.88	16			16		51.6	
7.5.88	16			16		51.6	
8.5.88	16			16		51.6	
9.5.88				18	18	64.4	
10.5.88				18	18	64.4	
11.5.88		13		22	22	106.6	
12.5.88		12		6	18	77.3	
13.5.88				12	12	42.9	
14.5.88				18	18	64.4	
15.5.88				18	18	64.4	
16.5.88					18	45.0	
17.5.88					18	45.0	
18.5.88					18	45.0	
19.5.88		18			18	83.7	
20.5.88		12			12	55.8	
21.5.88					18	45.0	
22.5.88					18	45.0	
23.5.88		12		23		50.5	
24.5.88		12		23		50.5	
25.5.88		10		23		46.2	
26.5.88						0.0	
27.5.88						0.0	
28.5.88						0.0	
29.5.88						0.0	
30.5.88					19	47.5	
31.5.88					23	57.5	
							1462.1

Table IV - I Cont'

1.6.88			0.0	
2.6.88			0.0	
3.6.88	3	18	51.5	
4.6.88		23	57.5	
5.6.88		23	57.5	
6.6.88		23	57.5	
7.6.88		23	57.5	
8.6.88		23	57.5	
9.6.88		16	40.0	
10.6.88		12	30.0	
11.6.88		16	40.0	
12.6.88		16	40.0	
13.6.88		16	40.0	
14.6.88		16	40.0	
15.6.88		16	40.0	
16.6.88		16	40.0	
17.6.88		14	35.0	
18.6.88		16	40.0	
19.6.88		16	40.0	
20.6.88		23	57.5	
21.6.88		17	42.5	
22.6.88		17	42.5	
23.6.88		17	42.5	
24.6.88		13	32.5	
25.6.88		17	42.5	
26.6.88		17	42.5	
27.6.88		17	42.5	
28.6.88		12	30.0	
29.6.88		4	10.0	
30.6.89		17	42.5	
				1191.5

Table IV - I Cont'

1.7.88	12	30.0	
2.7.88	17	42.5	
3.7.88	12	30.0	
4.7.88	17	42.5	
5.7.88	17	42.5	
6.7.88	12	30.0	
7.7.88	17	42.5	
8.7.88	12	30.0	
9.7.88	17	42.5	
10.7.88	12	30.0	
11.7.88	15	37.5	
12.7.88	17	42.5	
13.7.88	24	60.0	
14.7.88	24	60.0	
15.7.88	24	60.0	
16.7.88	22	55.0	
17.7.88	11	27.5	
18.7.88	22	55.0	
19.7.88	16	40.0	
20.7.88	22	55.0	
21.7.88	22	55.0	
22.7.88	22	55.0	
23.7.88	13	32.5	
24.7.88	15	37.5	
25.7.88	22	55.0	
26.7.88	15	37.5	
27.7.88	22	55.0	
28.7.88	13	32.5	
29.7.88	15	37.5	
30.7.88	23	57.5	
31.7.88	20	50.0	
			1360.0

Table IV - I Cont'

1.8.88	20	50.0	
2.8.88	16	40.0	
3.8.88	20	50.0	
4.8.88	16	40.0	
5.8.88	16	40.0	
6.8.88	20	50.0	
7.8.88	16	40.0	
8.8.88	16	40.0	
9.8.88	16	40.0	
10.8.88	20	50.0	
11.8.88	16	40.0	
12.8.88	16	40.0	
13.8.88	20	50.0	
14.8.88	16	40.0	
15.8.88	16	40.0	
16.8.88	14	35.0	
17.8.88	16	40.0	
18.8.88	14	35.0	
19.8.88	16	40.0	
20.8.88	16	40.0	
21.1.88	14	35.0	
22.8.88	20	50.0	
23.8.88	14	35.0	
24.8.88	18	45.0	
25.8.88	18	45.0	
26.8.88	14	35.0	
27.8.88	16	40.0	
28.8.88	16	40.0	
29.8.88	10	25.0	
30.8.88		0.0	
31.8.88	23	57.5	
			1247.5

Table IV - I Cont'

1.9.88	19			40.9	
2.9.88	14			30.1	
3.9.88	4			8.6	
4.9.88	20			43.0	
5.9.88			14	35.0	
6.9.88			16	40.0	
7.9.88			14	35.0	
8.9.88			20	50.0	
9.9.88			14	35.0	
10.9.88			14	35.0	
11.9.88			20	50.0	
12.9.88			14	35.0	
13.9.88			12	30.0	
14.9.88			10	25.0	
15.9.88			10	25.0	
16.9.88			10	25.0	
17.9.88			10	25.0	
18.9.88			10	25.0	
19.9.88		10		25.0	
20.9.88		12		30.0	
21.9.88		7		17.5	
22.9.88		6		15.0	
23.9.88		6		15.0	
24.9.88		6		15.0	
25.9.88		6		15.0	
26.9.88			10	25.0	
27.9.88			6	15.0	
28.9.88			10	25.0	
29.9.88			6	15.0	
30.9.88			10	25.0	
					830.1

Table IV - I Cont'

1.10.88		6	15.0	
2.10.88		10	25.0	
3.10.88	10		25.0	
4.10.88	6		15.0	
5.10.88	10		25.0	
6.10.88	6		15.0	
7.10.88	8		20.0	
8.10.88	6		15.0	
11.10.88	10		25.0	
12.10.88	5		12.5	
13.10.88	10		25.0	
14.10.88	6		15.0	
15.10.88	10		25.0	
16.10.88	6		15.0	
17.10.88	10		25.0	
18.10.88	6		15.0	
19.10.88	6		15.0	
20.10.88	10		25.0	
21.10.88	11		27.5	
22.10.88	10		25.0	
23.10.88	6		15.0	
24.10.88	4		10.0	
25.10.88	6		15.0	
26.10.88	6		15.0	
27.10.88	6		15.0	
28.10.88			0.0	
29.10.88	1	6	17.5	
30.10.88	1		2.5	
31.10.88	5		12.5	
				507.5

Table IV - I Cont'

1.11.88		5	12.5	
2.11.88			0.0	
3.11.88		1	2.5	
4.11.88		1	2.5	
5.11.88		4	10.0	
6.11.88		1	2.5	
7.11.88	8		17.2	
8.11.88	5		10.8	
9.11.88			0.0	
10.11.88			0.0	
11.11.88			0.0	
12.11.88			0.0	
13.11.88			0.0	
14.11.88		3	7.5	
15.11.88		5	12.5	
16.11.88		1	2.5	
17.11.88			0.0	
18.11.88			0.0	
19.11.88			0.0	
20.11.88			0.0	
21.11.88			0.0	
22.11.88			0.0	
23.11.88		6	15.0	
24.11.88			0.0	
25.11.88		6	15.0	
26.11.88			0.0	
27.11.88		6	15.0	
28.11.88		6	15.0	
29.11.88		6	15.0	
30.11.88		6	15.0	
				170.5

Table IV - I Cont'

1.12.88		6		15.0	
2.12.88		6		15.0	
3.12.88		6		15.0	
4.12.88		6		15.0	
5.12.88	6			12.9	
6.12.88	12			25.8	
7.12.88	12			25.8	
8.12.88	10			21.5	
9.12.88	6			12.9	
10.12.88	6			12.9	
11.12.88	6			12.9	
12.12.88		6		15.0	
13.12.88		6		15.0	
14.12.88		6		15.0	
15.12.88		8		20.0	
16.12.88		6		15.0	
17.12.88		8		20.0	
18.12.88		8		20.0	
19.12.88		4	4	20.0	
20.12.88		4	4	20.0	
21.12.88		4	4	20.0	
22.12.88		4	4	20.0	
23.12.88		3	3	15.0	
24.12.88				0.0	
25.12.88				0.0	
26.12.88			8	20.0	
27.12.88			8	20.0	
28.12.88			5	12.5	
29.12.88				0.0	
30.12.88				0.0	
31.12.88			4	10.0	
					462.2

Appendix - V

(A) Procedure on current meter.

The following precautions were taken:-

1. The conditions of the current meter bearings were checked at the beginning of each measurement by a spin test to ensure no damage had occurred.

2. The stop watch was checked regularly for accuracy.

3. At every point three observations were made and recording was done for consistent readings.

4. For low and irregular readings, the period of observation was lengthened to obtain a more accurate average count.

(B) Formular for computing discharge.

The midsection method of computing discharge was applied as suggested by USBR (1984). In this method the depth and mean velocity were measured for each of a number of verticals along the cross section.

The depth at each vertical is applied to a sectional width which extends half way to the preceeding vertical and half way to the following vertical to develop a cross-sectional area. The following formula was applied to determine the discharge:-

$$Q_i = \frac{V_I + V_2}{2} * \frac{[(L_2 - L_1) + (L_3 - L_2)]}{2} * d_i$$

where

L_1, L_2 and L_3 = distances in metre from the initial point, for any consequtive verticals; d_2 = depth of water at vertical L_2, V_1 and V_2 = velocities (m/s) at 0.2 and 0.8 of the water depth, respectively, at vertical L_2.

q = discharge in m^3/s through section of average depth d_2.

Canal discharge is obtained by

$$\sum_{i=1}^{i=n} q_i$$

Where;

n = number of sections;

q_i = section discharge.

Seepage rate computation formular:

This is computed using the equation below:

S = scepage (l/s/km)

Q_{in} = incoming flow (m^3/s)

Q_{out} = outgoing flow (m^3/s)

L = length of the reach (m).

Velocity determination

The counter readings are made (revolutions/time)

Let r = number of revolutions

t = time lapse (e.g 30 sec)

n = revolutions/second

n = r/t

if r = 180 revolutions

t = 30 seconds

n = 180/30 = 6.00 revolutions/second.

Velocity in m/s is then computed using current meter equations:

Example:

Equations of Neyrpic current meter are given as:

if n < 2.00 V = 0.0617n + 0.017 (m/s)

if n > 2.00

< 5.88 V = 0.0577n + 0.025 (m/s)

if n > 5.88 V = 0.0543n + 0.045 (m/s)

Appendix VI: Inflow-outflow results

Table VI-1 : Current meter flow measurements

Station 1+000 Date: 15-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.51	37.40	13.50	0.094	0.045	0.033
1.50	0.89	93.00	40.50	0.204	0.100	0.123
2.50	0.95	99.00	89.00	0.215	0.196	0.196
3.50	1.02	103.50	89.00	0.224	0.196	0.214
4.50	1.09	109.00	83.00	0.235	0.185	0.223
5.50	1.05	83.00	67.00	0.185	0.154	0.173
6.50	0.90	44.00	24.00	0.107	0.066	0.075
7.50	0.59	39.00	31.00	0.097	0.081	0.046
8.00	0.00	0.00	0.00			
				Total		1.083

Station 11+00 Date: 15-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.24	37.40	26.18	0.094	0.071	0.021
1.50	0.56	118.06	82.64	0.252	0.184	0.113
2.50	0.71	115.50	80.85	0.247	0.181	0.144
3.50	0.72	112.94	79.06	0.242	0.177	0.143
4.50	0.58	126.50	88.55	0.268	0.195	0.133
5.50	0.41	121.74	85.22	0.259	0.189	0.097
6.50	0.34	70.40	49.28	0.160	0.118	0.038
7.00	0.00	0.00	0.00	0.000	0.000	0.000
				Total		0.690

Station 21+000 Date: 15-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.24	37.00	10.00	0.093	0.038	0.016
1.50	0.65	56.33	31.25	0.133	0.081	0.070
2.50	0.77	67.00	52.00	0.154	0.124	0.107
3.50	0.95	71.67	48.00	0.163	0.116	0.132
4.50	0.74	55.33	26.40	0.131	0.071	0.075
5.50	0.41	29.25	13.20	0.077	0.044	0.025
6.50	0.00	0.00	0.00	0.017	0.017	0.000
				Total		0.424

Station 31+000 Date: 15-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.32	17.00	1.50	0.052	0.020	0.012
1.50	0.62	33.00	23.33	0.085	0.065	0.046
2.50	0.74	52.50	25.67	0.125	0.070	0.072
3.50	0.85	56.33	34.00	0.133	0.087	0.093
4.50	0.81	52.00	37.33	0.124	0.094	0.088
5.50	0.67	45.00	39.20	0.110	0.098	0.069
6.50	0.47	25.50	5.50	0.069	0.028	0.023
7.50	0.00	0.00	0.00	0.000	0.000	0.000
				Total		0.404

Station 0+930 Date: 23-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	Current Meter Reading 0.8D	Velocity 0.2D	Velocity 0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.69	121.88	65.65	0.259	0.151	0.122
1.50	1.00	187.61	121.20	0.385	0.258	0.310
2.50	1.17	187.61	155.54	0.385	0.324	0.400
3.50	1.18	182.13	150.49	0.375	0.314	0.406
4.50	1.18	181.30	160.34	0.373	0.333	0.435
5.50	1.38	185.84	157.36	0.381	0.328	0.472
6.50	1.38	157.50	120.44	0.328	0.257	0.303
7.00	0.00	0.00	0.00	0.000	0.000	0.000
				Total		2.447

Station 0 11.00 Date: 23-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	Current Meter Reading 0.8D	Velocity 0.2D	Velocity 0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.61	120.67	111.00	0.257	0.238	0.131
1.50	0.89	185.75	161.00	0.381	0.335	0.321
2.50	1.20	175.50	154.00	0.363	0.321	0.375
3.50	1.10	180.33	149.00	0.371	0.312	0.384
4.50	1.10	179.50	158.75	0.370	0.330	0.367
5.50	0.89	184.00	155.80	0.378	0.325	0.304
6.50	0.58	150.00	119.25	0.314	0.254	0.169
7.50	0.33	119.40	103.00	0.000	0.000	0.000
8.40	0.00	0.00	0.00			
				Total		2.051

Station 0 21.00 Date: 23-11-88

easuring oint	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.67	142.00	121.00	0.298	0.258	0.170
1.50	1.10	190.00	145.00	0.389	0.304	0.344
2.50	1.10	211.60	191.00	0.428	0.391	0.450
3.50	1.10	182.00	145.00	0.374	0.304	0.354
4.50	0.87	176.25	132.00	0.364	0.279	0.284
5.50	0.69	109.00	99.00	0.235	0.215	0.127
6.10	0.00	0.00	0.00	0.017	0.017	0.003
				Total		1.730

Station 031+000 Date: 23-11-88

suring nt	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.60	121.00	112.53	0.258	0.241	0.132
1.50	0.92	145.00	134.85	0.304	0.284	0.257
2.50	1.05	191.00	177.63	0.391	0.367	0.390
3.50	1.10	145.00	134.85	0.304	0.284	0.320
4.50	1.10	132.00	122.76	0.279	0.261	0.287
5.50	0.95	99.00	92.07	0.215	0.202	0.181
6.50	0.47	31.33	29.14	0.081	0.077	0.037
7.50	0.00	0.00	0.00	0.000	0.000	0.000
				Total		1.604

Station 0+930 Date: 25-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.78	26.00	20.00	0.070	0.058	0.044
1.50	1.15	79.75	76.33	0.178	0.172	0.194
2.50	1.35	150.20	137.00	0.314	0.288	0.393
3.50	1.37	153.00	127.00	0.319	0.269	0.399
4.50	1.34	143.00	123.00	0.300	0.262	0.378
5.50	1.33	121.00	111.00	0.258	0.238	0.333
6.50	1.37	78.00	63.00	0.175	0.146	0.193
7.50	0.73	44.00	34.00	0.107	0.087	0.069
7.90	0.00	0.00	0.00			
				Total		2.002

Station 0 11.00 Date: 25-11-88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.40	65.33	48.33	0.151	0.116	0.048
1.50	0.64	150.00	129.00	0.314	0.273	0.190
2.50	0.91	210.33	172.67	0.426	0.357	0.343
3.50	1.05	202.00	168.33	0.411	0.349	0.381
4.50	1.00	205.00	158.33	0.416	0.330	0.369
5.50	0.91	196.00	181.33	0.400	0.373	0.334
6.50	0.64	151.75	118.67	0.317	0.253	0.180
7.20	0.33	80.33	65.00	0.000	0.000	0.000
8.40	0.00	0.00	0.00			
				Total		1.845

Station 0+930 Date: 10.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	1.16	95.00	78.33	0.208	0.176	0.177
1.50	1.38	117.67	82.33	0.251	0.183	0.286
2.50	1.34	125.00	129.75	0.265	0.275	0.364
3.50	1.33	151.00	126.67	0.315	0.269	0.384
4.50	1.26	140.75	126.67	0.296	0.269	0.370
5.50	1.39	128.00	137.00	0.271	0.288	0.371
6.50	1.26	109.00	84.00	0.235	0.187	0.206
7.50	0.00	0.00	0.00	0.000	0.000	0.000
				Total		2.157

Station 0 11.00 Date: 10.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.46	76.00	75.33	0.171	0.170	0.082
1.50	0.99	172.00	108.00	0.356	0.233	0.262
2.50	1.11	198.00	151.00	0.403	0.315	0.392
3.50	1.14	212.33	162.00	0.429	0.337	0.435
4.50	1.14	188.00	127.67	0.385	0.271	0.370
5.50	1.08	182.00	144.00	0.374	0.302	0.334
6.50	0.65	134.33	97.67	0.283	0.213	0.147
7.50	0.00	0.00	0.00	0.000	0.000	0.000
				Total		2.022

Station 0 21.00 Date: 10.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.58	81.67	72.00	0.182	0.163	0.110
1.50	1.38	138.67	121.00	0.292	0.258	0.327
2.50	1.43	196.50	180.00	0.401	0.371	0.547
3.50	1.44	150.00	135.00	0.314	0.285	0.425
4.50	1.38	133.67	111.00	0.282	0.238	0.319
5.50	0.71	76.00	65.67	0.171	0.151	0.113
6.10	0.00	0.00	0.00	0.017	0.017	0.003
				Total		1.843

Station 0 31.00 Date: 10.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.85	117.45	81.00	0.251	0.181	0.154
1.50	1.16	169.65	117.00	0.351	0.250	0.335
2.50	1.30	164.21	113.25	0.341	0.243	0.367
3.50	1.27	148.44	102.38	0.311	0.222	0.326
4.50	1.05	107.66	74.25	0.232	0.168	0.209
5.50	0.79	63.08	43.50	0.146	0.106	0.083
6.50	0.00	0.00	0.00	0.017	0.017	0.003
				Total		1.477

Station 0+930 Date: 14.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	1.24	105.50	95.33	0.228	0.208	0.210
1.50	1.37	104.83	115.75	0.227	0.248	0.316
2.50	1.35	171.00	170.00	0.354	0.352	0.471
3.50	1.27	172.00	174.00	0.356	0.360	0.464
4.50	1.30	158.67	141.00	0.330	0.296	0.410
5.50	1.37	135.00	136.00	0.285	0.287	0.363
6.50	1.05	68.00	57.75	0.156	0.136	0.126
7.00	0.00	0.00	0.00	0.000	0.000	0.000
				Total		2.361

Station 0 11.00 Date: 14.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.52	83.00	71.00	0.185	0.162	0.092
1.50	1.10	148.75	123.00	0.311	0.262	0.283
2.50	1.24	192.25	146.75	0.393	0.307	0.424
3.50	1.27	184.50	133.50	0.379	0.282	0.417
4.50	1.27	187.80	155.25	0.385	0.324	0.444
5.50	1.20	175.50	152.67	0.363	0.319	0.374
6.50	0.72	129.00	104.00	0.273	0.225	0.164
7.50	0.00	0.00	0.00	0.000	0.000	0.000
				Total		2.199

Station 21.00 Date: 14.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.53	80.23	71.00	0.179	0.162	0.098
1.50	1.25	168.09	148.75	0.348	0.311	0.357
2.50	1.30	217.24	192.25	0.438	0.393	0.536
3.50	1.31	208.49	184.50	0.422	0.379	0.517
4.50	1.25	212.21	187.80	0.429	0.385	0.453
5.50	0.65	86.45	76.50	0.191	0.172	0.116
6.00	0.00	0.00	0.00	0.000	0.000	0.000
				Total		2.077

Station 31.00 Date: 14.12.88

Measuring Point	Depth	Current Meter Reading 0.2D	0.8D	Velocity 0.2D	0.8D	Discharge (m^3/s)
0.0	0.00	0.00	0.00	0.000	0.000	0.000
0.50	0.85	113.20	94.33	0.243	0.206	0.160
1.50	1.16	161.04	134.20	0.335	0.283	0.344
2.50	1.30	146.80	122.33	0.307	0.260	0.363
3.50	1.37	152.80	127.33	0.319	0.270	0.390
4.50	1.27	115.60	96.33	0.247	0.210	0.284
5.50	1.05	100.50	83.75	0.218	0.186	0.211
6.50	0.79	73.50	61.25	0.166	0.143	0.102
7.50	0.00	0.00	0.00	0.000	0.000	0.000
				Total		1.855

Appendix VII: Soil constants

Table VII - 1: Constants in moritz formula

Soil type	Value of C
Cemented gravel and hardpan with sandy loam	0.34
Clay and clay loam	0.41
Sandy loam	0.66
Volcanic ash	0.68
Sand or volcanic ash or clay	1.20
Sandy soil with rock	1.68
Sandy and gravelly soil	2.20

Table VII - 2 Soil permeability constants

	Low	Medium	High
a	0.70	1.90	3.40
m	0.30	0.40	0.50

Appendix VIII: Seepage meter computations

Table VIII - 1: Seepage meter reading

Measuring points km	Average seepage rate ml/hr
15.00	240.0
20.00	237.0
26.00	243.0
38.60	245.0
Average	241.3

Determination of area covered by the seepage meter.

Diameter of meter = 0.30 m

Area (A) covered by the meter is obtained as:

$A = \Pi D^2/4\ m^2$

$= 3.14 * 0.30^2/4\ m^2$

$= 0.0707\ m^2$

Determination of wetted area of supply canal per km:

Dimensions of the canal (before operation):

Bed width (B) = 4.0 m

Slope 2:1

Depth of flow = 1.40 m

Length side slope (x) = 3.13 m

Surface water width (w) = 9.60 m

Wetted perimeter = (3.13 + 4.0 + 3.13) m

= 10.26 m

Wetted area of the canal per km

= 10.26* 1000 m

= 10260 m^2

Computation of seepage (l/s/km).

Average seepage rate recorded = 241.3 ml/hr.

This is equivalent to 241.3*1 * 1/1000* 3600 = $6.70*10^{-5}$ l/s.

But $6.70*10^{-5}$ l/s is seepage through the area covered by the seepage cup (0.0707 m^2).

Therefore seepage rate per km,

= $6.70*10^{-5} * 10260/0.0707$

= 9.68 l/s/km.

Appendix IX

Table 6: Seepage rates in $l/s/m^2$

Canal reach km	wetted perimeter m	wetted area m^2	seepage $l/s/m^2$
0 - 10	8.95	89500	$8.8*\ 10^{-3}$
10 - 20	6.75	67500	$10.0*\ 10^{-3}$
20 - 30	7.85	78500	$5.8*\ 10^{-3}$

Appendix x

Table 7:. Effective porosity ranges for different materials (after Johnson, 1966).

Material	Effective porosity %	
	Range	Mean
Clay	0 - 5	2
Silt	3 - 19	8
Sandy clay	3 - 12	7
Fine sand	10 - 32	21
Medium sand	15 - 32	26
Coarse sand	20 - 35	27
Gravelly sand	20 - 35	25
Fine gravel	17 - 35	25
Medium gravel	13 - 26	23
Coarse gravel	12 - 26	22

Source: ILRI Publication 16 Vol.III, Drainage Principles and Applications.

Appendix XI

Table 8. Fuel consumption at Nanigi pump station for Feb - July, 1988.

Months	Pump 2			Pump 3			Pump 4		
	Litres/ month	Operating hrs/month	L/hr	L/month	Operating hrs/month	L/hr	L/month	Operating hrs/month	L/hr
Feb	7600	231	32.9	11600	294	39.5	14200	257	55.3
Mar	13400	241	55.6	13450	565	23.8	17078	533	32.0
Apr	-	-	-	-	-	-	32460	757	42.9
May	48020	747	64.3	-	-	-	-	-	-
June	45580	472	96.7	-	-	-	-	-	-
Jul	41820	539	77.6	-	-	-	-	-	-

NB.

1. Average fuel consumption = 52.1 L/hr (Table 8).
2. Price of diesel in 1988 at Bura was Kshs. 4.839 + 1.00 per litre on transport. Total cost = Kshs. 5.84 per litre.
3. Total number of hours of pumping from 6 months of cotton crop production = 4636 hours.
4. Total fuel consumption = 4636 * 52.1 litres/hour = 241,535.6 litres.
5. Cost of pumping in 6 months = 241,535.6 * 5.84 = Kshs. 1,410,567.90.

Appendix XII

Table 1:. Flow velocities for calibration.

Propellor No.	Distance from East-bank (m)	Depth of flow (cm)	Velocity Rev/30 sec 0.2D	Velocity Rev/30 0.8D
1	3.0	101 20.2, 80.8	213 (A)	173.7 (B)
C-2 small				
A.OTT				
current meter				
to be calibrated	4.0	101 20.2, 80.8	215.7 (C)	196 (D)
3	3.0	101 20.2 80.8	45.0 (A)	42.3 (B)
A.OTT				
current meter				
(standard)	4.0	101 20.2 80.8	29.8 (C)	42.3 (D)
2	3.0	101 20.2 80.8	41.3 (A)	32.0 (B)
C-2 large				
Neypric				
current meter				
(standard)	4.0	101 20.2 80.8	46.3 (C)	37.4 (D)