INVESTIGATIONS AND DESIGN OF AN APPROPRIATE DRAINAGE SYSTEM FOR THE UNIVERSITY OF NAIROBI'S VETERINARY FARM -KANYARIRI.

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Thesis submitted to the Department of Agricultural Engineering in partial fulfillment for the degree of Master of Science in Agricultural Engineering (Soil and Water Engineering),
UNIVERSITY OF NAIROBI.

March, 2000

DECLARATION

I hereby, declare that this thesis is my original work and has not been presented for a degree in any other University and that, all citations have been appropriately acknowledged.

Date 17/4/2000

Josiah Willie Mulwa

This thesis has been submitted for examination with my approval as University supervisor.

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DEDICATION.

I salute my Mother, Mutany'a, my Father, Mulwa and uncle Mitula Mbunza for the efforts they made to make my school life possible through the unfavourable agriculture practiced in Kitui District. This work is dedicated to them all.

ACKNOWLEDGEMENTS.

I am very grateful to SAREC (Swedish Agency for Research Cooperation with Developing Countries) for providing me with a two- year master of science scholarship which enabled me to carry out this study.

I am deeply grateful to my principal supervisor, Mr. R.K. Muni of the Department of Agricultural Engineering for his guidance throughout the performance of this work and for his advice and encouragement.

Appreciation is also made to Dr. T.C. Sharma for his assistance and advice especially in preparing the computer programme which was used to analyze rainfall data for this study. Special thanks are due to Ngigi and Sang for their contribution during the execution of the field work, and to the Management of the Veterinary farm for allowing me to carry out my research in their farm .

I express my sincere appreciation and gratitude to my wife, Annastacia Mbua for her encouragement, endurance and understanding during the time I was undertaking this study and more appreciation goes to my children Ndanu, Mulwa and Muthui for their patience and endurance during my absence. I also appreciate the efforts of Peter, my brother who spent much of his precious time going through this work searching for any editorial mistakes and therefore thank him for all the corrections he proposed.

I sincerely recognize the help and cooperation received from Jonah Kitheka of ***Provide International*** in providing his personal computer to me and especially so in times of peak demand when.continuity of work could not be maintained from my Department.

I would also like to acknowledge the help received from the Department of Soil Science. In particular, I would like to thank Tirop and Mainga for their untiring efforts in the handling of soils data.

Finally, but not the least, I would like to register my sincere gratitude to all the Staff and fellow students of the Agricultural Engineering Department for their Constructive criticism, support of whatever nature and all others who directly or indirectly contributed to the final success of this work.

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LIST OF ABBREVIATIONS AND SYMBOLS

ASAE	= American Society of Agricultural Engineers.
ASCE	= American Society of Civil Engineers.
FAO	= Food and Agricultural Organisation of the United Nations.
MFLP	= Matrix Flux Potential.
PVC	= Polyvinyl Chloride.
SAREC	= Swedish Agency for Research Cooperation with Developing Countries.
USBR	= United States Bureau of Reclamation.
USDA	= United States Department of Agriculture.
USDA-SCS	= United States Department of Agriculture-Soil Conservation Service.
USDI	= United States Department of the Interior.
A	= Cross-sectional area (m^2), Area of catchment (ha).
a	= Texture specific empirical constant for soil (cm/day), a constant which is a measure of magnitude of infiltration as used in Kostiakov's formula.
α	= Reaction factor (day^{-1}) as used in Glover - Dumm formula, albedo as used in Penman method, texture specific empirical constant for soil used in Rejitema method (cm^{-1}).
Cap, CR	= Capillary rise (mm).
C	= Runoff coefficient in relational formula, geometry factor in auger hole method.
D	= Diameter of a circle equal in area to a given non- circular catchment as used in Bransby- Williams formula (Km), Thickness of an aquifer, average flow depth (m).
d	= Distance from drain to barrier (m), rate of decline of watertable level (cm/day).

dt/dy	= Change in height of water table over distance y along water table(-)
δhm	= Fall in water level (m).
ΔGws	= Change in ground water storage(mm).
ΔSm, ΔSsm	= Change in moisture storage (mm).
Δ Ssw	= Change in water storage (mm).
ΔW	= Change in soil water storage (mm).
e	= Rate at which infiltrating water vertically enters the saturated zone and is negative for evapotranspiration or deep seepage (m/day).
E_o, Et_o	= Open water Evaporation (mm/day).
E_t	= Potential Evapotranspiration (mm).
F	= Average fall per 100 m length of the main water course as used in Bransby-Williams formula.
f,μ	= Drainable porosity (-).
g	= Ground water flow (mm/day).
GRW	= Ground water runoff (mm).
H	= Total heat converted to depth units as used in Penman method, difference in head (m), depth of hole below ground water table as used in Auger hole method (cm).
H_o, M_o	= Water table depth before instantaneous recharge (m).
H_t, M_t	= Water table after time t (m).
h	= Position of water table below soil surface (cm), soil- water pressure head (cm).
h(t)	= Available hydraulic head in a drain as used in Kraijenhoff formula (m).

I	= Interception (mm).
i	= Hydraulic gradient (-), rainfall intensity (mm/hr).
I_{cum}	= Cumulative infiltration (mm).
Inf	= Infiltration (mm).
Inst	= Instantaneous infiltration rate (mm/hr).
I_t	= Infiltration rate at time t(mm/hr).
j	= Ground water reservoir coefficient (days).
K	= Hydraulic conductivity of a medium (m/day), frequency factor corresponding to a given return period and for a given probability distribution.
K_c	= Crop coefficient which depends on the stage of growth of a crop.
K_o	= Texture specific hydraulic conductivity of a soil (cm/day).
$K(\Theta)$	= Hydraulic conductivity at a given moisture content (m/day).
$K(\psi)$	= Hydraulic conductivity at matric suction (ψ) (cm/day).
L	= Drain spacing (m), Length of soil core in constant head permeameter method (cm), Longest distance from outlet in a runoff channel (km).
M	= Actual area of catchment as used in Bransby-Williams formula (km^2).
Mb	= Millibars.
M_s	= Weight of oven- dried soil (gm).
μ	= Drainable pore space, mean of parent data in rainfall storms (mm).

n	= Day number when a ground water level is required, exponent reflecting the change of infiltration rate with time.
N_d	= Number of potential drops in a flownet.
N_f	= Number of channels in a flownet.
Θ	= Soil water content.
P, Pr	= Precipitation (mm).
p	= Rainfall rate (cm/day).
Perc	= Percolation from precipitation (mm).
ρ_b	= Bulk density. (g/cm^3).
Q	= Rate of flow passing a given cross-section (m^3/day)
q	= Peak discharge rate as used in rational formula (m^3/s), discharge rate (m^3/day), deep seepage rate (cm/day).
Q_{inf}, Q_{in}	= Inflow towards an area (mm/day).
Q_{out}, q_{out}	= Outflow from an area (mm/day).
q_m	= Discharge rate (m^3/day).
R	= Runoff from an area (mm), Recharge rate (m/day), design rainfall rate (mm/day).
r	= Adjustments value for surface runoff depending on slope of the area, radius of hole in Auger hole method (cm), pore space drained during a decline of water table (%).
RD	= Rooting depth (cm).
γ	= Barometric constant = 0.66 mb/°C .
S	= Drain spacing (m), specific yield in the zone of water table fluctuation (-).

s = Pore spacing available for storage of water between the soil surface and water table level, a chosen safety factor for siltation which depends on soil type and used for calculation of design drainage rate.

S_f = Surface water flow (mm).

SS_F = Subsurface water flow.

t = Time water table takes to fall from one position to another, drain out period (days).

T_c = Time of concentration (hrs).

t_r = Period of steady recharge (days).

V = Flow velocity, velocity describing ground water movement (m/day).

V_t = Volume of soil core as obtained from the field (cm^3).

X = required storm value, estimated (mm), distance in the horizontal direction (m).

ψ = Matric suction (cm).

ψ_{max} = A texture specific suction limit (cm).

σ = Standard deviation of the storm data (mm).

z = Height of the capillary rise from the bottom of a hole as used in Auger hole method (cm).

ABSTRACT.

The study was carried out in the University of Nairobi's Veterinary Farm at Kanyariri. The area lies in the upper catchment of Mathare river four Kilometres north of upper Kabete campus. The farm is approximately 50 ha of which 6 ha are seasonally waterlogged.

The area under study is a bottom land lying below a springline where the water table is within 50 cm for at least two months every wet season.

The present study was carried out to establish the causes of the poor drainage in the bottom land, determine relative contribution of each causative factor and to give remedial measures to improve the drainage situation in the area.

Drainage investigations which involved monitoring of ground water behaviour in the bottom land, water balance analysis to ascertain the sources of the excess water, soil and topographic studies to ascertain the causes of the drainage problem were carried out.

It was found that upslope seepage into the bottom land and direct rainfall were the main sources of the excess water. Low hydraulic conductivity, infiltration rates and presence of depressions within the bottom land were the major contributors to the poor surface drainage whereas the virtually impermeable clay (glei) layer, over most of the profile, resulting to low internal drainage was the cause of the poor subsurface drainage.

Soil loosening accompanied by mole drainage was recommended as a solution owing to the small drain spacings (7.5 m) required due to the low hydraulic conductivity value of 0.06 m/day.

Based on long term rainfall data as well as storm rainfall data from a neighbouring weather station, surface drainage system showing drain dimensions, grades and spacings were proposed. An interceptor drain was proposed to serve both as a cutoff and interceptor drain for subsurface flow. The designed system was thought to be adequate to control ground water rise and prevent waterlogging in the 50 cm root zone.

The drainage coefficient as determined from the rainfall analysis was 51.7 mm/day giving a drainage modulus of 6 l/s/ha for the surface drainage system.

1. INTRODUCTION.

As population of animals grow, the pressure on the available grazing lands will become more pronounced leading to the need to reclaim more lands which have otherwise been unproductive for pasture production. These lands may be reclaimed from waterlogged areas. Thus, much of the waterlogged agricultural lands of Kenya can economically and practically be used for grazing animals if proper water management practices are employed. Proper water management for waterlogged lands will ultimately require a component of drainage at one stage or another.

However, it is a recognized fact that drainage in Kenya and many other third world countries is a relatively new technology whose application requires clearly laid down procedures and guidelines to enable the overall user to appropriately use it for his betterment. In this respect, adequate information on drainage planning and performances are lacking and most of the work on drainage achieved to date as Van Schilfgaarde (1978) puts it are based on the rules of the thumb derived from experience rather than on sound analytical formulation of the problems at hand. Having very little experience in Kenya, this kind of situation has led to inappropriate application of drain spacing in some areas not suitable and subsequent failure of the whole drainage scheme as experienced in the Kisii valley bottom of Nyanza province of Kenya.

Adequate information would therefore enable government planners and administration to establish firm objectives for drainage projects, implementation of drainage plans, direction of drainage activities and to operate and manage drainage schemes properly.

Drainage investigations made for design of an appropriate system would take time if all the required investigations are to be carried out as some investigations may require periods of time ranging from months to years. Therefore, for preliminary designs, the most important investigations are the ones which yield information indicating the causes of the problem, the sources and the quantities of the damaging waters, the frequency and duration of the problem and hence revealing the likely solution to the problem.

This project was undertaken with a view to isolate some of these investigations with their procedures to determine the above required information for a bottomland in the University farm in Kanyariri, Kiambu district and then formulate design procedures which will be applied in the study area. Although these investigations will be for Kanyariri area, allowances for transferability to other areas with similar conditions will be possible with only slight changes .

1.1 Background information about the area.

The project is a continuation of various studies which have been carried out in the area. Studies by Ngigi (1991), produced a topographical map of the area at grid spacing of 20 m by 20 m for the poorly drained area and 40 m by 40 m for the adjacent catchment. From the above study, the ground surface configuration and the direction of flow of the surface runoff into the poorly drained area could be determined.

A further study on the same area by Gichuki (1992), saw the establishment of water table observation wells network over the entire waterlogged area at a grid spacing of 40 m by 40 m and a resultant network of 54 observation wells. The study also tried to quantify the various water balance quantities which was not possible owing to the short duration available to undergraduates for project work. Based on the latter study's recommendation, this project was borne with a few additions to the required investigations suggested.

1.2 The Problem.

In-situ rainfall and lateral seepage from adjacent high lying areas of the catchment has resulted in the development of seasonal shallow water tables in the valley bottoms of the University's Veterinary Farm at Kanyariri. This causes increasing losses in grazing productivity for dairy cattle and sheep as the land is left under utilised and vegetated with unpalatable vegetation such as water loving reeds and wire grass for most part of the year.

For any meaningful use of the bottomland a way of controlling the watertable during the wetter parts of the year through drainage must be sought. However, during dry periods, the impacts of drainage may produce harmful effects and thus re-use of the drained water for animal production should also be considered. That is, the drained water to be impounded in a reservoir downstream of the waterlogged area for animal watering. Sub-irrigation should also be a candidate in the overall water management strategy in the considered project.

1.3 Justification of the study.

One of the greatest problem in agricultural development of the study area was the inadequacy of the parameters for water control and drainage system design. In order to reach a better information and identification of the causes of the drainage problems, it was imperative to undertake some surveys to reveal the much needed information pertaining to water control measures.

It was for this reason that the present study was carried out in the present site. Its scope ranged from watertable monitoring to determination of soil physical parameters which are used in drainage design purposes .

1.4 Objectives.

The general objective of the study was to provide a theoretical design of the drainage system for the poorly drained area based on hydrological soil properties measured by conventional methods and assumed drainage criteria.

The specific objectives of the study are :-

(i) To study Ground water fluctuations with rainfall during the study period in the study area.

(ii) To determine experimentally the hydrological soil properties; depth to impervious layer, saturated hydraulic conductivity, drainable porosity and the infiltration rate of the top stratum which are required for drainage design.

(iii) To determine the extent, degree and the nature of the drainage problem.

(iv) To analyze the ground water system and assess the ground water balances quantitatively, to determine the causes of the drainage problem.

(v) To design a drainage system for the area.

2. LITERATURE REVIEW

2.1 Land Drainage

The term land drainage may be defined as the removal of excess water from the soil or from the land surface, by artificial means. Its objective being to make the land more suitable for use by man (Anon, 1979). Land drainage, therefore, may be categorised into two types:-

i) Surface drainage, which is the evacuation of excess water over the ground surface sometimes in part through the top soil to open drainage system with an adequate outlet. It is necessitated by a combination of certain climatic, hydrologic, and soil conditions and of topography and land use. Thus if the rate of rainfall exceeds infiltration rates of the soil in question, runoff will result and hence the need for surface drainage system (Raadsma and Schulze, 1980).

ii) Subsurface drainage or ground water drainage, is the removal of the excess water below the soil surface to promote favourable soil-water relations. The temperatures of poorly drained soils, for examples are 4° - 8°C lower than those of comparable well drained soils (Van Beers, 1979).

Whether surface or subsurface, drainage, results into new areas being put under cultivation or a complete change in agricultural pattern of an area as conditions will have changed for better. Poor drainage also affects soil structure, aeration, organic matter content and even temperature as mentioned above (Van der Goor, 1979).

2.1.1 Hydrogeology and Drainage

The drainage problems of an area are closely related to its geomorphological and geogenetical conditions. Thus the presence or absence of layers with good water transmitting properties, or barrier to ground water flow, or springs, as well as the relationship between ground water and surface water will directly or indirectly affect the water conditions in or near the rootzone. When this water is in excess, it leads to drainage problems (Ridder, 1979).

2.1.2 Causes of Waterlogging.

In most of the drainage problems, the major concern is to determine the soil moisture pressure (or Suction) and moisture content at relevant points in the soil due to the operation of Darcy's law in a medium in which such conditions are imposed (Childs, 1957). This may be done by employing tensiometers accompanied by piezometers.

The tensiometers will give the water tension(suction) whereas the piezometers will indicate whether the watertable is under artesian pressure or it is a free watertable.

In the present study, the watertable was observed using observation wells which showed response to applied recharge and the possibility of an artesian aquifer was discarded.

The major causes of waterlogging in an area may be from a single factor or a combination of many factors. The factors can be summarised as (Castle et al.,1984; Childs,1957):-

i) Watertable problem, where the watertable boundary is in the soil profile between unsaturated soil overlying saturated soil. This causes watertable fluctuations into the rootzone in times of heavy rainfall.

ii) Soils of low permeability, fine textured with poorly developed structure thus impending downward flow of water causing it to stand for prolonged hours or days on the rootzone.

iii) Springs or springlines caused by geological features such as faults, dips, slopes and generally occurring on sloping lands.

iv) A stratum of relatively high hydraulic conductivity resting on a substratum which is relatively non-conducting. When rain falls, the above stratum will accept water in a rate which is in excess of the negligible rate which can be accepted by the lower substratum and consequently a body of groundwater is built up in the rootzone.

v) Artesian water entering from below and bringing water into the rootzone.

vi) An upper stratum of high hydraulic conductivity rests on a sloping impermeable bed down which flows a body of ground water from a foreign catchment area, bounded by water table, so that at the outcrop of the substratum, the groundwater is intersected by the surface of the permeable upper stratum leading to a spring or springline.

vii) Other special causes of high watertable with undesirable effects is the so called **Glyben-Herberglens**, where the watertable bounds a rainfed body of fresh groundwater based on a sea-fed body of salty groundwater (Childs, 1957).

For a drainage system to be effective, each of these likely causes of waterlogging problem should be investigated so that the one causing the problem can be identified.

2.2 Investigation procedures.

In an attempt to establish the factors causing the drainage problem of an area, the most important activity that must be undertaken is the stratum survey. This is necessitated by the fact that waterlogging is a complicated process involving many factors which in turn interact and cause the problem. Therefore in attempting to establish the causative factors, which might have acted singly or in combination, it requires a clear understanding of the following (Van der Meer and Van de Graaf, 1980; Smedma and Rycroff, 1983; SCS, 1973):-

i) The topography to show mostly the slopes, areas' flatness or evenness or the presence of depressions.

ii) The soil characteristics especially in relation to the permeability characteristics of both the top and subsoil.

iii) The rainfall- runoff characteristics especially the rainfall intensity, duration, the frequency of occurrence and infiltration rates as they affect discharge during the same period.

iv) The groundwater table fluctuations in relation to precipitation and time.

The study investigated most of these aspects except the discharge aspect. The discharge monitoring was made impossible by the nature of the landscape especially towards the outlet, where one concentration point could not be located.

2.2.1 Determination of the extent, degree and nature of the drainage problem.

Delineation of areas requiring drainage raises considerable difficulties as some investigators consider drainage necessary to the entire wet spot whereas others may use arbitrary criterion to decide on which area to be considered as requiring drainage. However, many specialists agree on one point, that the starting point is the groundwater table level monitoring using a grid system of observation wells or piezometers (USDA-SCS, 1973; Boersma et al, 1972; Armstrong, 1985; Galvin,1985; Bognár and Toth, 1987).

Dickey (1977) suggested that subsurface agricultural drainage problem is assumed to exist when the groundwater table is less than 1.5 m below the ground surface during the crop growing season. Armstrong(1985) on the other hand suggests that, all that area with depth to watertable being less than 0.5 m below the ground surface requires drainage which is a further refinement of the Dickey's criterion. However, Bognár and Toth (1987) suggested the use of different probabilities of occurrence of watertable at a given depth, which is crop dependent to be the basis. In this criterion, the occurrence of groundwater table in the crop's rooting depth with the required probability is the deciding factor.

The criterion suggested by Bognár and Toth(1987) was adopted for the current study since the delineation of the area suffering from waterlogging can be solved in a more accurate way. The system designer is able to base the project on an appropriate hydrological study and carry out efficiency calculations by weighting the investment against damages.

For high valuable lands, probabilities of water table occurring at the crop rootzone of between 10 and 20 percent are used, whereas for less valuable lands, like grazing, probabilities of 33 or 50 percent can be used.

2.2.2 Determination of the causes of high groundwater table.

The presence of excess water that causes a drainage problem can ordinarily be traced to precipitation, irrigation applications, seepage from surface water bodies, hydrostatic pressure from an artesian aquifer or a combination of any of these sources. The source of the damaging waters must be known so that the proper protective measures can be taken (USDI,1978).

Determination of each of these possible causes is established through groundwater studies in the problematic area. Therefore by correlating groundwater levels with the various factors suspected to be the causes, the actual cause can be identified.

That is, if groundwater levels are found to respond positively to precipitation and not the other suspected causes, the rainfall is the likely cause (USDI, 1978; Dickey,1977; USDA-SCS,1973).

In the present study, where irrigation was not applicable as it is not practiced, the competitors were precipitation, lateral groundwater seepage from the adjacent high grounds and the artesian flow from underground.

A groundwater contour map which is a useful tool in detecting various causes of high ground water table problems was suggsted for use. Thus, if the problem was due to hydrostatic artesian pressure or lateral seepage, the seepage line will be reflected in the contour map as a groundwater mound (USDA-SCS,1973; Ridder and Aart,1980).

2.2.3 Determination of the location of the impervious layer.

The barrier or the impervious layer as defined by the United States Bureau of Reclamation, is a layer which has a hydraulic conductivity one-fifth or less of the weighted hydraulic conductivity of the strata above it (USDI,1978).

Using the above definition, the determination of the barrier layer would entail the determination of the hydraulic conductivity of the various soil strata found in the problematic areas soil profile. The observed hydraulic conductivities can later be compared in magnitude and the layer with properties suiting the above conditions designated as the impervious layer. A profile that does not have a layer meeting the above description can be reported as not having a barrier layer within the investigated depth.

Another approach of detecting and locating an underground barrier or impermeable material causing a high watertable is by use of ground-water contours. Ground water contours inspection, can reveal a region where the contours are closely spaced, a situation depicting steep water table gradient. It can also show that region with widely spaced contours which indicates a flat water table gradient.

The sharp break of the water table gradient from steep to flat or vice versa in a region is an indication of the presence of a less permeable material or a barrier to ground water flow in that region of the slope break (USDA-SCS,1973).

As watertable contours are actually elevations down the profile, the depth of the barrier layer can be determined from the contour reading where the break occurs.
The above described method requires the establishment of ground water monitoring points arranged in a grid in the study area. The method was employed in addition to the hydraulic conductivity

determination method since the watertable data was also to be employed in other applications of the study.

2.2.4 The water balance equation.

The law of conservation of mass states that in a specified period of time, all water entering a specified area must either be consumed therein, be exported there from or kept in joint storage within its boundaries. The water balance equation to represent the above definition can be written as (Bredero,1991):-

$$P - E_t = S_F + SS_F + \Delta WS \qquad 1$$

Where
- P = precipitation (mm)
- E_t = Potential evapotranspiration (mm)
- S_F = surface water flow (mm)
- SS_F = sub-surface water flow (mm)
- ΔWS = change in soil water storage (mm)

The various components of the water balance equation can be calculated or estimated using well established procedures. A more commonly used procedure for water balance analysis in humid areas of the tropics where the excess water mainly comes from excess rainfall over the amount that is evaporated, is the one described by Thornthwaite and Mather in 1957 and modified by Dunne and Leopold (1978) as explained by the latter authors.

The formula relating the various terms can be written as:-

$$Pr = I + Et + Of + \Delta SM + \Delta GWS + GWR \qquad 2$$

Where
- Pr =precipitation (mm)
- I =Interception (mm)
- Et =Evapotranspiration(mm)
- Of =Overland flow (mm)

ΔSM =change in soil moisture storage (mm)

ΔGWS =Change in ground water storage (mm)

GRW =groundwater runoff (mm)

Based on the above described procedure, and the knowledge of available water capacities of the rootzone, for different crops and soils and on accumulated potential water loss, each of the above factors can be evaluated.

2.2.4.1 ***Rainfall determination.***

Precipitation is one of the weather elements which is easy to measure. This is so because rain gauging is a well developed technology. The use of autographic rain gauges or non-recording rain gauges is normally employed for the purposes of measuring rainfall in an area.

For the purposes of the current study an autographic rain gauge was installed in the project site and monitored during the study period.

2.2.4.2 ***Evapotranspiration.***

Evapotranspiration of crops or Evaporation of bare surface can not be determined directly with the same ease as rainfall. Thus for the determination of evapotranspiration parameter in equation (2) calculations are done based on one or more of the existing procedures. The procedures include the one described by Jansen and Haise, Blanney and Criddle, Thornthwaite and the Modified Penman methods as presented by Doorenbos and Pruitt (1977).

In the case of the present study, Modified Penman method was selected for its accuracy is said to be approximately 10 percent over or below the correct value whereas the other methods have large deviations (Doorenbos and Pruitt, 1977).

The Penman equation may be represented as:-

$$E_O = \frac{\Delta}{\Delta + \gamma} H + \frac{\gamma}{\Delta} E_a \qquad (3)$$

Where E_o = Open water Evaporation or pan evaporation (mm/day)

γ = Barometric constant = 0.66 mb/°C

Δ = Slope of the vapour pressure curve

$H = Q_S - \alpha Q_S - Q_O$ (mm/day)

Q_s= Incoming radiation as measured by Solarimeter (mm/day)

Q_o = out going radiation (mm/day)

α = albedo

E_a = The wind term, which is normally determined from tables, given wind run and vapour pressures (mm/day)

The above quantities can be determined, given the meteorological data, which can obtained from Kabete University Farm Field station about 4 km away. The pan evaporation can then be converted to potential crop evapotranspiration by use of crop coefficient, thus;

$$E_t = K_c E_o \qquad 4$$

Where, E_t is the potential evapotranspiration from the crop (mm/day) and

K_c is the crop coefficient which depends on the growth stage of the crop.

For the determination of the reference Evapotranspiration of grass, which is going to be the major crop after drainage has been done, can be performed using a FAO Computer Programme based on the Penman _Monteith formula and termed "*CROPWAT*". The procedures for data entry and analysis are explained by Smith (1992).

2.2.4.3 Change in soil moisture storage.

For Poorly drained soils, which are assumed to be saturated, the change in soil moisture storage is assumed negligible. This assumption is fairly true especially during the wet period when drainage is necessary.

2.2.4.4 Sub-surface flow.

The sub surface water flow is reflected by the rise or fall of the watertable level (Bredero,1991). The rise or fall of the watertable as recorded from observation wells can be used to construct flownets, where net subsurface flow can be estimated (Das,1989).

The flownet may simply be defined as a combination of a number of flowlines (a line along which water particle travels from upstream to downstream of a permeable soil medium) and equipotential lines (a line along which the potential head is the same at all points) (Cedegren,1977; Das,1989).

The Seepage flow from a flownet is estimated from the equation (Das,1989):-

$$q = K\left(\frac{HN_f}{N_d}\right) \qquad 5$$

where, q is the total rate of seepage through all the channels per unit width (m^2/day).
K is the hydraulic conductivity of the medium (m/day)
H is the difference of head between upstream and downstream sides(m)
N_f is the number of channels in a flownet
N_d is the number of potential drops

The greatest difficulty in the above estimation lies on the construction of the flownet itself. However, the construction follows laid down procedures as given in Heath and Trainer(1968), Cedegren(1977) and Das(1989). Thus :-

(i) A flowline crosses an equipotential line at right angle.

(ii) Spaces bounded by flowline and an equipotential line should approximate squares.

(iii) Distortion of squares is either caused by differences in transmissivity or by recharge to or by discharge from the aquifer.

(iv) Flowlines run parallel to an impermeable boundary and equipotential lines intersect the such a boundary at right angles.

(v) Flowlines originate from a recharge boundary; equipotential lines run parallel to it at the point on the boundary closest to the discharging point.

(vi) No flow takes place across a flowline

(vii) The boundary of an impervious layer is a flowline.

Despite the above laid down procedures, there are many limitations in flownet construction since changes in permeability present difficulties in construction of flownets, moving boundaries cause even greater complications, but perhaps the greatest limitation is the impossibility of obtaining direct estimates of quantities of fluid flowing (Rushton and Redshaw,1979).

Another approach to the solutions to the seepage and drainage problems is the use of simplified cross sections that are hopefully and reasonably representative of the real-life section. These will enable determination of the real water transmitting properties of the formations and masses that influence flow. This, coupled with judicious application of seepage principles, will have the best probability of developing structures and water control systems that will perform as expected (Cedegren,1977).

The above simplification to seepage or groundwater flow may be achieved through a wide range of techniques, each of which is suited to a particular class of problems and may well lead to inaccurate results when applied in other situations (Rushton and Redshaw,1979).

The most commonly applied techniques are discussed below:-

2.2.4.4.1 Analytical techniques.

These involve the use of analytical expressions obtained by direct integration of the appropriate differential equations (Rushton and Redshaw,1979). However, there are many situations for which analytical solutions cannot be obtained. This is because the number of unknown functions on which the problem depends is limited (Cedegren,1977) or because seepage problems have a number of non-linear features which are not amenable to inclusion in analytical solutions such as variable permeability, moving boundaries or long term time-dependent effects (Rushton and Redshaw,1979).

2.2.4.4.2 Mathematical models.

Mathematical models utilise the solution of the equations describing flow through the aquifer with appropriate initial and boundary conditions being expressed mathematically (Rushton and Redshaw,1979). However, Cedegren (1977), pointed out that despite the rapid development of these models for a number of cases, the solutions resulting from them are cumbersome and more often than not approximate. The approximations are introduced by describing the continuous function of seepage and ground water potential in terms of values on some form of discrete grid and digital approach by dividing time into discrete steps. Although this is normally the case, when care is taken, errors due to these approximations can be made sufficiently small to be neglected (Rushton and Redshaw,1979).

2.2.4.4.3 Physical models.

This involves the modelling of the actual physical shape of the medium and then the boundary conditions are simulated as heads of water or drains. These may include viscous flow models and sand tanks(Cedegren,1977 and Rushton and Redshaw,1979).

The method however, suffers the disadvantage of being complicated in construction and operational procedures.

2.2.4.4.4 The Darcy Model.

The model utilises Darcy law of flow through porous medium. The assumption here is that all other things being equal, the effective underground water flow velocity is proportional to hydraulic gradient.

Thus (USDA-SCS,1973):-

$$V = Ki \qquad 6$$

where v= effective flow velocity (m/day)

K= hydraulic conductivity (m/day)

i= hydraulic gradient (-)

When the cross section through which the flow takes place is known, then the flow rate passing through a given cross section is given by the following equation:-

$$Q = KiA \qquad 7$$

With Q= Rate of flow passing a given cross sectional area of the saturated soil (m^3/day)

A= saturated cross sectional area (m^2) and other factors as defined in equation (7) above.

The saturation is assumed to be from the impermeable bed up to where the watertable reaches. The equation (8) above is used to estimate the quantity of flow in simple drainage problems, such as might be found in hillside interception over a sloping impermeable layer. For more complex problems other methods may be followed.

2.2.4.4.5 Non-Equilibrium method of finite differences for underground flow.

The approach utilises the hydrodynamic analysis of the observed fluctuations in the water level. The method is based on the non- equilibrium formula of the finite differences for underground flow developed by Kamensky in 1940 and reported in Altovsky and Konoplyntev(1959). It is claimed to be the best from the hydrogeological point of view. The method requires a minimum of three observation wells distributed along the line of ground water flow and sunk to the watertight bed.

When the data of water levels in the three observation wells for an average moment of the given time interval (denoted by s+1), as well as the level of groundwater in the middle well (denoted by n) at the initial time, s and ultimate moment, s+2 of the same time interval are available, then the value

a) if the impervious layer or watertight bed is horizontal,

$$\omega = \mu \frac{h_{n,s+2} - h_{n,s}}{\Delta t} - \frac{K}{l_{n-1,n} + l_{n,n+1}} \left[\frac{h^2_{n-1,s+1} - h^2_{n,s+1}}{l_{n-1,n}} - \frac{h^2_{n,s+1} - h^2_{n+1,s+1}}{l_{n,n+1}} \right]$$

$$- \frac{h^2_{n,s+1} - h^2_{n+1,s+1}}{l_{n,n+1}} \Big] \qquad (8)$$

Where $h_{n,s+2}$ = the saturated thickness of the aquifer flow measured down to the watertight bed in the middle well (n) at the ultimate moment of time interval (s+2), (m)

$h_{n,s}$ = the saturated thickness of the aquifer flow in the same well at initial moment (s) of the time interval Δt, (m).

K = Hydraulic conductivity (m/day).

$l_{n-1,n}$ = The distance from the upper well (the first along the flow (n-1) to the middle well (n),(m).

$l_{n,n+1}$ = distance from the middle well (n) to the lower well (n+1) (the last along the flow), (m).

$h_{n-1,s+1}$ = The saturated thickness of the aquifer flow in the upper well (n-1) measured from the watertight bed to the watertable at an average moment (s+1) of the time interval, (m).

$h_{n,s+1}$ = The same for the middle well(n), (m).

$h_{n+1,s+1}$ = The same for the lower well(n+1), (m).

t = The time interval in days between two chosen moments of observation (s and s+2).

μ = drainable porosity (-)

ω = The infiltration rate m/day.

b) if the water tight bed is sloping ,then the infiltration rate can be estimated from :-

$$\omega = \mu \frac{H_{n,s+2} - H_{n,s}}{\Delta t} - \frac{2K}{l_{n-1,n} + l_{n,n+1}} [\frac{h_{n-1,s-1} + h_{n,s+1}}{2} X \frac{H_{n-1,s+1} - H_{n,s+1}}{l_{n-1,n}} - h_{n,s+1} + h_{n+1,s+1}$$

$$- h_{n,s+1} + h_{n+1,s+1} X \frac{H_{n,s+1} - H_{n+1,s+1}}{l_{n,n+1}}] \quad (9)$$

Where, $H_{n-1,s+1}$, $H_{n,s+1}$ and $H_{n+1,s+1}$ = Level of groundwater table at the average moment (s+1) in upper,middle and lower wells for a given time interval respectively.

$H_{n,s+2}$ = Level of ground water table in the middle well at the ultimate time (s+2) and other designations remain the same.

Depending on the nature of the impermeable layer, the value of infiltration from rainfall can be established.

Positive values of ω obtained from calculations points to the predominance of seepage over evaporation during the period under consideration, while negative values will testify to the contrary.

After determination of the rainfall factor in the drainage problem, the remaining problem is to establish the contribution of the underflow from the adjacent areas and from the area to the neighbouring lower grounds.

The inflow of free ground waters towards the selected element of flow is determined on the basis of the following formulae:-

a) With plane flow and horizontal impervious bed :

$$q_{in} = K \frac{h^2_{n-1,s-1} - h^2_{n,s-1}}{2 l_{n-1,n}} \quad (10)$$

The outflow of free groundwater from the given element is computed from:

$$q_{out} = K \frac{h^2_{n,s-1} - h^2_{n-1,s-1}}{2 l_{n,n-1}} \tag{11}$$

$$q_{in} = K \frac{(h_{n-1,s-1} + h_{n,s-1})}{2} X \frac{(H_{n-1,s-1} - H_{n,s-1})}{l_{n-1,n}} \tag{12}$$

The outflow under these circumstances is given by ;

$$q_{out} = K \frac{(h_{n,s-1} + h_{n-1,s-1})}{2} X \frac{(H_{n,s-1} - H_{n-1,s-1})}{l_{n,n-1}} \tag{13}$$

2.2.5 Groundwater Balance Assessment.

Assessment of groundwater balance of an area allows the determination of the cause(s) of the drainage problem quantitatively. The method used involves the determination of all inflow and outflow components into and out of the area respectively and by using the groundwater balance equation the components contributing to the problem in the area are identified (Kessler and Ridder, 1980). Thus;

$$(P_r-R)-E_t-(Perc-Cap)=\Delta S_{sm} \tag{14}$$

Where P_r is the precipitation in the area (mm).

R is the runoff from the area (mm).

E_t is the potential evapotranspiration (mm).

Perc · is the percolation of water from precipitation through the unsaturated zone towards the watertable (mm).

Cap is the capillary rise from the shallow groundwater table to the overlying unsaturated zones (mm).

ΔS_{sm} is the change in moisture storage (mm).

Determination of the above components will lead to identification of the causes of the drainage problem in the area.

*2.2.5.1 **Runoff Determination.***

Runoff into an area can be measured with flow measuring meters such as flumes, weirs, and flowmeters. This is however possible when the flow cross-section is well defined (Ridder,1979). In catchment areas however, flow cross-sections are rarely defined and hence such runoff measurement is not possible .

When the above cases prevail, runoff is estimated from empirical formulae or other established procedures. From one of these procedures, the water balance of the surface system can be represented as (Ridder and Aart,1980):-

$$(Q_{inf} - Q_{out}) - I_{nf} - Et_o = \Delta S_{sw} \tag{15}$$

where, Q_{inf} is inflow from adjacent areas as runoff (mm)

Q_{out} is outflow through channels as runoff (mm)

I_{nf} is Infiltration (mm)

Et_o is Open water evaporation (mm)

ΔS_{sw} is the change in soil water storage (mm)

The inflow from adjacent areas can be determined by measurements, using enclosed plots as in runoff plots or using open plots such as Gerlach troughs (Lal, 1990; De Ploey and Gabriels, 1980).

The Gerlach troughs are located on the sloping sides of the adjacent area where slopes are mostly parallel to runoff flowlines. This method however, is not in any way better than the empirical methods of runoff prediction.

The outflow from channels if they exist can be measured using hydraulic structures while the amount of water that has infiltrated after a certain time can be estimated from the infiltration parameters obtained by conducting experiments in the study area as described by Amoozegar and Warrick (1986) and Oosterbaan and Stalkman (1987).

Some of the most common procedures for predicting runoff are discussed below:-

(i) **The Rational method** (Ward,1967; Hudson,1971; Chow et al.,1988):

$$q = (C\ i\ A)/360 \quad (16)$$

where, q is peak discharge rate (m^3/s)
i is rainfall intensity (mm/hour) for a duration equal to the time of concentration for the catchment.
A is area of catchment (ha)
C is the runoff coefficient

The above described formula is applicable to small catchments of sizes up to 800 ha. The catchment under study is approximately 100 ha and hence the method can be applied without problems (Chow et. al., 1988)

(ii) **Unit hydrograph method**

This method as described by Ward (1967), Wilson (1974), Chow et al.(1988), Shaw (1983) considers surface runoff in a form of normal hydrograph with a steep rising limb and a shallow falling limb and hence obviating from the static behaviour inherent in other models.

The model has the ability to predict the peak rates of runoff and at the same time predict the total volume of runoff resulting from a given storm. It is actually a useful tool for the determination of the hydrograph of surface runoff that results from any given amount of rainfall excess (Wisler and Brater,1959).

Although assumption must be made about the relationship between rainfall intensity and infiltration capacity, and therefore about volume of precipitation excess available for surface runoff during the course of the storm, the method is still considered powerful for runoff prediction (Wisler and Brater,1959; Ward, 1967).

(iii) **Distribution graph method.**

This method as described by Ward (1967), Wisler and Brater (1959), Todd (1980) and Khushalani et.al(1984) is basically a modification of the unit hydrograph as it uses the same time scale and ordinates as the Unit hydrograph if catchment is same. However, their main difference is that whereas in a Unit hydrograph the rainfall amount is plotted against successive equal units of time, in a distribution graph, the runoff resulting from a unit storm is plotted as a percentage of the total volume of runoff during a number of successive equal time units. Thus the basic concept involved in a unit hydrograph is actually that, all unit storms produce virtually identical distribution graphs for a given catchment area (Ward, 1967). This follows that once a distribution graph is derived for a drainage basin, it serves as a means of converting any expected volume of runoff into a hydrograph of runoff discharge.

2.3 Ground Water Dynamics

Water table depth is a dynamic soil feature that fluctuates greatly from year to year and throughout the year. This applies when the depth is measured in the same soil series as well as between soils of different soil series (Zobeck and Ritchie,1984).

To study the ground water dynamics, a survey of the groundwater levels should be conducted to offer an insight in to how the levels are changing in relation to time and the other factors such as rainfall (Ridder and Aart,1980). The water levels can be observed from existing wells, open bore holes, piezometers, surface waters such as lakes, streams and canals, and observation wells.

Observation wells are dug using an auger and can be cased with perforated plastic pipes or left uncased if the soils are stable. Water table observation wells intercept water present in the soil regardless of its location and thus the water table levels measured, indicate the highest level of saturated soil (Zobeck and Ritchie,1984).

2.3.1 Groundwater fluctuations.

The rise and fall of groundwater level will mostly depend on recharge and discharge (Bowen, 1986;Ward, 1967).

The relationship between storage and water level are complicated by the fact that the latter responds to other factors other than storage changes (Ward,1967). Meinzer cited by Ward (1967) noted that *"the water level in a well is sensitive to every force that acts upon the body of water with which the well communicates "*. Thus the observed watertable levels, as the water fluctuates can provide many useful information pertaining to drainage design.

When the ground water table fluctuations are taken for a considerable length of time such as one year including both wet and dry season, the information serves as a useful guide in formulating a water table height prediction model for the area (Coleman and Fenton,1982)

Proper land use and development especially in seasonally waterlogged areas will depend in part upon the ability of the land user to predict watertable levels under a wide range of conditions (Nelson et al.,1973). This requires that,watertable level data be collected and processed for use in deriving such a prediction model. If the water table data is to be of use, then the method proposed by Nelson et al.(1973) can be adopted. Other methods of predicting the water table level is by use of equations such as the one given by Boersma et al.(1972) :-

$$h_n = h_{n-1} + 100(q/r) - 100(p/s) \quad (17)$$

where h =Position of water table below the soil surface (cm)

n =day number when the level is required

r= pore space drained during a decline of water table (%)

q =deep seepage rate (cm/day)

p = rainfall rate (cm/day)

s =pore space available for storage of water between the soil surface and water table level.

The above equation is only applicable when surface runoff and lateral ground water flow are not taking place. When rainfall and evapotranspiration are equal and lateral flow not taking place, the deep seepage rate can be represented by :

$$q = d \times (r/100) \tag{18}$$

where q and r are as defined above and d is the rate of decline of the water table level (cm/day)

In a place where lateral seepage is a contributor to the drainage problem as is the case in the University farm under study, the above method is not applicable unless modifications are done to allow for that component.

2.3.2 Groundwater movement and direction.

Groundwater movement is a variable and as observed by researchers, its velocity can range from thousandth of a centimetre per day to as much as 5400 metres per day even in the same formation (Ward, 1967). Ground-water velocity and discharge are controlled primarily by the height (slope) of the water table and by permeability. The water table height depends partly upon permeability and partly upon incoming and outgoing percolation.

Thus, fluctuations in water table level and consequent changes in discharge may be observed after a storm (Weyman,1975). The movement of ground water can be said to be mostly governed by established hydraulic principles.

Most of the flow through aquifers, which are natural porous media, can be expressed by what is known as Darcy's law; the application of which enables the rate and direction of flow to be assessed (Bowen,1986; Todd,1959).

The movement of groundwater can be followed direction wise by the addition of a dye such as Sodium fluorescein to the flow and tracing its movement in space and time (Todd,1959; Bowen, 1986; USDA-SCS,1973). The use of the dyes may however, not be always possible given that most of them are very expensive. This calls for a cheaper way of determining both the speed and direction of the ground water flow.

Since ground water movement may be very slow in some formations, it is sometimes difficult to determine the direction of flow and yet such knowledge is of considerable importance especially, where for example seepage from adjacent high porous grounds is to be intercepted (Ward,1967). As it is this direction which determines proper placement of interceptor drains and the speed which determines the discharge reaching the interceptor and hence useful in dimensioning of these channels. For this purpose, several methods are available, but the commonly used one is the cartographic method described by Ward (1967); USDA-SCS(1973); Ridder and Aart(1980) and Marshall et al.(1986), where the area under consideration is grided with a network of observation wells and the water level monitored in the wells.

Groundwater contours are then drawn connecting points with same water levels. The direction perpendicular to these contours is the direction of groundwater movement. With the value of hydraulic conductivity of the medium determined using other established procedures and the hydraulic head being defined as the quotient of the difference between any two adjacent contours and the distance between them, the velocity of the groundwater movement can be established from the relation (Bowen,1986;Weyman,1975):-

$$V = - K\, dz/dy \qquad (19)$$

where K = hydraulic conductivity of the medium (m/day)

dz/dy = change in height of water table over distance y along the watertable (-)

V= velocity describing the groundwater movement (m/day)

2.3.3 Capillary rise.

The upward flux or capillary rise, from a water table is an important phenomenon in water table management schemes, as it plays a partly role in watertable and ground water storage fluctuations (Memon et al.,1986). The determination of this upward flux has presented considerable problems to researchers . Hillel (1982) presented a mathematical formula describing the capillary rise which is a modification of the Darcy's equation and written as :-

$$V = K(\theta)\frac{dh}{dz} - K(\theta) \qquad (20)$$

Which can be re-arranged to express the height of the capillary rise as ;

$$Z = \int_{h=0}^{h=Z} \frac{K(\theta)}{V + K(\theta)} dh \qquad (21)$$

where V= the vertical velocity (m/day)

$K(\Theta)$ = hydraulic conductivity at a given moisture content (m/day)

Θ = Soil water content

h=0 is the surface of watertable as base (datum)

Z = height of the capillary rise from the datum (m)

The height at which the flux will rise is only determinable when the relationship between K and Θ is well established. This has rendered the above method not readily applicable.

Another approach would be the solution of Richard's one-dimensional unsaturated flow equation. Skaggs et al.,(1978) devised numerical and analytical procedures for obtaining the solution of Richard's equation, but his procedures are tedious and time consuming and hence reducing the applicability of his method to practical problems.

A further development in this area was reported by Ragab and Amer (1986) and entails the use of a model that utilizes the relationship between hydraulic conductivity and the water content, depth of the watertable below the crop rooting zone and the matric suction at the bottom of the root zone to calculate the capillary rise. However, the modified formula of Rijtema (1969) which employs the use of some numerical constants and obtained from prepared tables is an easily applicable method for planning though approximate. The resulting capillary rise is then estimated as:

$$CR = \frac{K_O(e^{-\alpha\psi} - e^{-\alpha(Z_t-RD)})}{e^{-\alpha(Z_t-RD)} - 1}$$

The above equation is however applicable when ψ is less than ψ_{max}. For high suction range however, the relationship between CR, ψ and Zt-RD is best calculated using numerical integration and using a slightly different equation presented by Rijtema (1969) as:-

$$CR = K(\psi)(\frac{\psi}{Z_t - RD} - 1) \qquad (23)$$

where , CR =capillary rise (cm)

K_o = The texture specific saturated hydraulic conductivity (cm/day)

$K(\psi)$ = hydraulic conductivity at matric suction , ψ (cm/day)

$K(\psi) = a\psi^{-1.4}$

a = A texture specific empirical constant ($cm^{2.4}$/day)

α= A texture specific empirical constant (cm^{-1})

Z_t = Ground water depth at the beginning of time interval, t (cm)

RD = Rooting Depth (cm)

ψ_{max} = A texture specific suction limit (cm)

ψ = Mean suction in the increment (Z_t -RD) (cm)

The above parameters are obtained from table 2.1 shown below.

Table 2.1. Values of suction limit, ψmax, saturated hydraulic conductivity, K_o, constants a and α for various soil texture classes

Soil texture class	ψ_{max}(cm)	K_o (cm/day)	a ($cm^{2.4}$/day)	α (cm^{-1})
Coarse sand	70	1120.0	0.080	0.224
Fine sand	175	50.0	10.9	0.0500
Loamy sand	200	26.5	16.4	0.0398
Fine sandy loam	290	12.0	26.5	0.0248
Silt loam	300	6.5	47.3	0.0200
Loam	300	5.0	14.4	0.0231
Loess loam	130	14.5	22.6	0.0490
Sandy clay loam	200	23.5	33.6	0.0353
Silty clay loam	170	1.5	36	0.0237
Clay loam	300	0.98	1.69	0.0248
Light clay	300	3.5	55.6	0.0174
Silty clay	50	1.3	28.2	0.0480
Heavy clay	80	0.22	4.46	0.0380
Peat	50	5.3	6.82	0.1045

(After Rijtema, 1969)

Memon et al.(1986) proposed a slightly simplified method which utilizes the concept of the matric flux Potential. The method requires that unsaturated hydraulic conductivity function, K(h) of the soil be known before hand. The method stems from the steady-state, one-dimensional unsaturated flow, which expresses the governing equation as:

$$\frac{d}{dz}\left[K(h)\frac{dh}{dz} - K(h)\right] = 0 \qquad (24)$$

Where h =soil-water pressure head (cm)

Z = depth to watertable and measured downwards from the soil surface (cm)

$K(h)$ = Unsaturated hydraulic conductivity function which can be determined using many approaches, though many of them employ empirical relationships.

There are various methods proposed by researchers for determining the function, many of which employ empirical relationships .

The approach of using Matric Flux Potential (MFLP) as described by Shaykewich and Stroosnijder in 1977 and reported by Memon et al.(1986) defines MFLP as:-

$$MFLP(h) = \int_{h=0}^{h_{max}} K(h)\, dh \tag{25}$$

where h_{max} = maximum allowable pressure head (cm) at the centre of the effective root zone

$h=0$ = pressure head at the watertable (cm)

Integration of the Darcy-Buckingham equation and some mathematical manipulation, the MFLP model can be written in finite difference form as (Memon et al.,1986):

$$Z_{i+1} = \frac{MFLP_{i+1} - MFLP_i}{q + \frac{[K(h_{i+1}) + K(h_i)]}{2}} \tag{26}$$

Where q = upward flux from the water table.

Z_{i+1} = watertable depth at which the corresponding upward flux occurs (cm).

$K(h_i)$ = Hydraulic conductivity as a function of pressure head (cm/day).

h_i, h_{i+1} =pressure heads (cm) corresponding to $MFLP_i$, $MFLP_{i+1}$ respectively.

The above equation is easier to solve using a computer than both the numerical and analytical methods. But, still suffers the same disadvantage as the numerical methods in that K(h) at one point must be known. The method of Rijtema (1969) proves superior to the others, especially where most of the soil parameter relations are not known except the soil texture which is easy to establish.

2.4 Determination of soil hydrological parameters.

The most important soil physical constants that must be determined for any drainage design purposes are the hydraulic conductivity of the soil profile at, and below the proposed drain depth, the thickness of the water transmitting layer and the porosity or the drainable porosity (Van der Meer and Van de Graaff,1980).

2.4.1 Measurement of hydraulic conductivity

Hydraulic conductivity of a soil is a measure of the soil's ability to transmit water through it. This property of the soil depends on the soil conditions. Thus for homogeneous saturated soils, hydraulic conductivity can be assumed constant, for a given position in the field, at a given time (Amoozegar and Warrick,1986). Hydraulic conductivity is actually a key parameter for aspects of water and solute movement through the soil and hence for drainage purposes it should be determined as accurately as it is practically possible.

There are many methods in use at present for measuring the saturated hydraulic conductivity of the soil. Some of which are discussed below with a few being chosen for the current study due to their merits over the others. The methods may thus be classified as hydraulic methods or correlation methods (Oosterbaan and Stalkman,1987).

Some of the correlation methods are described by Mualem and Dagan (1978); Mualem (1976); Brunce (1972) and Kunze et al.(1968) in which matching factors are employed to known hydraulic conductivities. The most common hydraulic methods involve the use of soil samples in the laboratory and employing either the constant head or the falling head approaches for the hydraulic conductivity determination or the in-situ approaches where holes are augured in the field and the rate of rise or drawdown measured as in the auger hole ,inverse auger hole and Piezometer methods (Oosterbaan and Stalkman,1987).

In the present study the method chosen were the Laboratory method of constant head, Inverse auger hole, and the auger hole methods as described by Oosterbaan and Stalkman(1987), Dieleman and Trafford (1984) depending on specific site requirements.

2.4.1.1 The laboratory methods.

The most common methods used for hydraulic conductivity tests among the laboratory methods are constant head and falling head permeameters. However, Laboratory methods may only be useful for making comparisons on relative basis between different soil hydraulic conductivities and also identifying soil anisotropy, slowly permeable soil layers, problem soil conditions and not for design purposes (FAO,1980).The major limitations to these laboratory methods are the small sample sizes involved and the partial destruction of the soil structure during sampling and thus rendering the values of hydraulic conductivities so obtained less representative of the actual situation (Ahuja et al.,1988 and FAO,1980).

2.4.1.2 Field methods

For proper modelling of water flow in the soil for drainage purposes, the saturated hydraulic conductivity of the in-situ soil is of paramount importance as it nears the reality. The most widely used methods in this category are the auger hole method and the piezometer method. However,other methods are still used depending on the accuracy of results required and the site conditions (Amoozegar and Warrick,1986). Some of these methods are discussed below.

(i) **The auger hole method**

This method introduced by Diserens (Boersma,1965) is the procedure most widely used to measure saturated hydraulic conductivity of a soil profile up to 5 m in depth. It has however undergone modifications and improvements. Most prominent improvements to the method have been done by various researchers with the latest version being the one presented by Kessler and Oosterbaan (1980). Despite the above modifications, some limitations with the method still exist. Some of the limitations unresolved to date are that, the flow to the auger hole is three-dimensional whereas the equations used describe two-dimensional flow and sometimes the flow properties could be different in each direction due to soil's anisotropy inducing errors into the measured value of hydraulic conductivity. The method used in the present study is the one described by Kessler and Oosterbaan (1980), which involves bailing the water from a previous dug hole and repeatedly measuring the rate of rise of the water until consecutive readings become consistent. The hydraulic conductivity is then calculated from the formula

$$K = c\frac{\Delta h}{\Delta t} \tag{27}$$

Where K = hydraulic conductivity of soil profile in m/day

C = geometry factor = **f(h,H, r, S)**.

$\Delta h/\Delta t$ = rate of water rise in the auger hole in (cm/s.)

This method is commonly preferred due to its simplicity and wide application (Kessler and Oosterbaan, 1980).

The above formula is however, applicable when standard sized augers are used in making the auger hole, as the factor C in the formula is a function of the hole radius among other things as presented above. A more generalized formula is the one given by ASAE (1990) which when written in metric units is of the form:-

$$K = \frac{4000\,r^2}{(H + 20r)(2 - \frac{Y}{H})Y}\frac{\Delta y}{\Delta t}$$

Where Z > 1/2 H and

K = hydraulic conductivity (m/day)

Y = average distance between ground water table and level water in the hole (cm)

H =depth of hole below groundwater table (cm)

Δt =selected time interval (s)

Δy = average change in water level in selected time (cm) r = radius of the hole (cm)

Z = depth to impermeable layer below the bottom of the hole (cm)

When the auger hole reaches the impermeable layer, Z= 0 and the equation is slightly modified to :

$$K = \frac{3600 r^2}{(H + 10r)(2 - \frac{Y}{H})Y} \frac{\Delta y}{\Delta t} \tag{29}$$

For cases where Z < 1/2 H but greater than zero, K is calculated from both formulae and the average obtained.

However, where watertable is above the surface or is under artesian conditions , the method cannot be used as reliable results are not likely to be obtained (Amoozegar and Warrick,1986).

(ii) **The piezometer method**

This Method involves driving an 8 cm diameter pipe into the auger hole, leaving a small cavity of known length at the bottom of the hole. The rate of water rise in the hole is measured as in the other methods and the hydraulic conductivity calculated as:

$$K = \frac{\pi r^2}{[c(t_n - t_l]} \ln \frac{(h_l)}{(h_n)} \tag{30}$$

Where, K = hydraulic conductivity in (cm/s)

r = inside radius of the pipe (cm)

h_l and h_n = depth of water in the pipe below equilibrium groundwater level at time t_l and t_n respectively (cm)

C = is a geometry factor = **f(H, r, L, S)** .

t_n-t_l = time interval of measurements (Kessler and Oosterbaan, 1980; Amoozegar and Warrick,1986). This method is mostly useful when dealing with stratified soils or where the water table is above the ground surface or is in artesian conditions (Amoozegar and Warrick,1986).

(iii) **The inverse auger hole method.**

This method is used in places where watertable is below 2.0 m or are dry for most of the time. Here, the rate of fall of the water added is measured and the corresponding hydraulic conductivity calculated from the formula below: Thus;

$$K = 1.5\, r \tan\alpha \tag{31}$$

Where K = hydraulic conductivity (cm/s)

r = radius of hole (cm) and

$$\tan\alpha = \frac{[\ln\ h(t_1)+\frac{r}{2}]-[\ln\ h(t_n)+\frac{r}{2}]}{(t_n-t_1)} \tag{32}$$

$h(t_1)$ = initial water level in the hole (cm)

$h(t_n)$ = the water level in the hole at time tn (cm)

t_1 = the initial time when water level is read (sec)

t_n = the final time (sec) (Kessler and Oosterbaan,1980).

As in the area under investigation, the water table reaches the said depth during dry season or during abnormal years like the year of present investigation and the method was also used for the investigations.

2.4.1.3 Hydraulic conductivity classification.

Hydraulic conductivity of the soil can be classified according to many methods all using the rate as the measure to designate an hydraulic conductivity class. The most notable method in this classification is the one proposed by O'Neal (1952) and is presented in the following table below. The most may be considered sufficient for most practical purposes.

Table 2.2 Classification of saturated hydraulic conductivity of soils.

Class	**Intake Designation**	**Hydraulic conductivity**	
		cm/hour	m/day
0	Very slow	< 0.25	< 0.06
I	Slow	0.25-0.5	0.06-0.12
II	Moderately slow	0.5-2.0	0.12-0.48
III	Moderate	2.0-6.25	0.48-1.50
IV	Moderately Rapid	6.25-12.5	1.50-3.00
V	Rapid	12.5-25.0	3.00-6.00
VI	Very Rapid	>25.0	>6.00

(after O'Neal, 1952).

2.4.2 Determination of Drainable porosity.

Drainable porosity is a soil parameter defined as the volume of water per unit area that is released when the water table falls by a unit distance. It is assumed constant for most drainage designs, although studies done by Childs and Taylor in 1960 showed that it varies with watertable depth (Skaggs et al.,1978). There are, however, many methods suggested for the measurement of drainable porosity by researchers. Some of which include the following (Skaggs et al.,1978)

a) Measurements of water table depth and drain outflow and the parameters in Brook's and Corey's relationship.

b) Continuous water table depth measurements and drain out flow.

c) Determining hydraulic conductivity-drainable porosity ratio from watertable drawdown measurements.

d) Use of water table -drainage discharge rates relationship (Dieleman, 1980).

e) Determination from empirical formula using the average value of hydraulic conductivity (Smedema and Rycroff, 1983).

Drainable porosity may also be expressed implicitly as a ratio of hydraulic conductivity as obtained from ground watertable drawdown measurements (Skaggs,1976), and thus be defined by ;

$$f\frac{\partial h}{\partial t} = K\frac{\partial}{\partial x}[h\frac{\partial h}{\partial x}] + e \qquad (33)$$

Where : h= Elevation of the water table above the impermeable layer(m) t= time (S)

x= Distance in the horizontal position(m)

e= rate at which infiltrating water vertically enters the saturated zone and is negative for evapotranspiration or deep seepage.

K= Hydraulic conductivity of the soil profile (m/day)

f= Drainable porosity (-)

However, solution to equation (33) above can only be used for cases where the distance between the drain and impermeable layer, is small. Where the distance is large, errors will be induced by convergence near the drain tubes and the consequent failure of Dupuit-Forcheimmeir assumptions which are the bases for solution to the above equation (Skaggs,1976).

Johnston et al., (1965) however, proposed the explicit expression for drainable porosity as;

$$f = \frac{Qt}{a(m_0 - m_t)} \qquad (34)$$

where, Q= the average drain outflow rate (m^3/s)

a= The area being drained (m^2).

t= the time for the water table to drop from m_o to m_t (Seconds)

f= The average porosity for the soil layer between m_o and m_t

m_o, m_t are the water table heights above the drain depth at times t_o and t_t respectively (m)

The above equation could not be applied in current case as it is only applicable in areas where existing drains are available.

Porosity may also be determined from observations of drain discharge after a rainfall event, which involves observing discharge rates and water table depths and using the formula developed by Kraijenhoff in 1971 as cited by Dieleman (1980).

The formula defines the ground water reservoir-coefficient, j, as;

$$j = \frac{1}{\alpha} = f\frac{L^2}{(\pi KD)} \tag{35}$$

With the ground water reservoir-coefficient being defined as above, the discharge from the area and the rate of fall of the watertable are respectively described by equations (36) and (37) shown below thus; (Dieleman, 1980) :

$$q(t) = \frac{8R}{\pi^2}[\sum_{n=1,3,5,...}^{\infty}\frac{1}{n^2}(1-e^{\frac{-n^2t}{j}}) - \sum_{n=1,3,5,...}^{\infty}\frac{1}{n^2}(1-e^{\frac{-n^2(t-t_r)}{j}})] \tag{36}$$

$$h(t) = \frac{4Rj}{\pi f}[\sum_{n=1,3,5,...}^{\infty}\frac{1}{n^3}(1-e^{\frac{-n^2t}{j}} - \sum_{n=1,3,5,...}^{\infty}\frac{1}{n^3}(1-e^{\frac{-n^2(t-t_r)}{j}}] \tag{37}$$

Where q(t) is discharge rate (m/day)

h(t) is available hydraulic head (m)

R is recharge rate (m/day)

t_r is period of steady recharge (days)

t is time (starting from the beginning of recharge.) (days)

f is the drainable porosity to be determined (-)

j is ground water reservoir coefficient (days)

K is the hydraulic conductivity (m/day)

D is distance to the impervious base (m)

L is drain spacing (m)

In the above formulae, it is further assumed that for time duration which is t = 0.4*j, that is , 40% of the reservoir coefficient after cessation of the recharge, the second and further terms of the two series represented above by equations (36) and (37) are small enough to be neglected, Hence equations (37) and (38) respectively reduce to :

$$q(t) = \frac{8R}{\pi^2}[(1-e^{\frac{-t}{j}})-(1-e^{\frac{-(t-t_r)}{j}})] \tag{38}$$

$$h(t) = \frac{4\,R\,j}{\pi f}\left[\left(1 - e^{\frac{-t}{j}}\right) - \left(1 - e^{\frac{-(t-t_r)}{j}}\right)\right] \quad \ldots\ldots\ldots\ldots\ldots \tag{39}$$

When the values of the time, t are substituted in equations (38) and (39) as t_1 and t_2 with t_1 and t_2 being respectively greater than t_r+0.4j and further manipulated mathematically, the following expressions result:

$$\frac{(t_1 - t_2)}{j} = \ln \frac{q(t_1)}{q(t_2)} \tag{40}$$

$$\frac{(t_1 - t_2)}{j} = \ln \frac{h(t_1)}{h(t_2)} \tag{41}$$

Equations (40) and (41), though simpler than their corresponding counterparts, they are not very useful in solving the problems encountered. The difficulty is however, circumvented by combining equations (38) and (39) and manipulating them further to yield equation (42) below, thus the relationship between discharge and hydraulic head can be plotted from which the value of the gradient is a measure of the product of f and 1/j.

$$q(t) \;=\; \frac{2f}{\pi j}\, h(t) \tag{42}$$

With the value of the 1/j being defined as in equation (44) with tan a being the slope of the graph then the value of the drainable porosity, f can be established.

The method however, requires the construction of drains in the area under investigations, so that hydraulic heads can be measured between the two drain channels. This requirement may not be met in many cases as in the present study and thus other methods have to be used although they yield inferior results.

$$\frac{1}{j} \;=\; \tan\alpha \tag{43}$$

The drainable pore space (μ) may also be found from the relation (Oosterbaan and Stakman,1987):-

$$\mu = \frac{-q_m}{(\delta h_m / \delta t)} \tag{44}$$

Where μ= drainable pore space (-)

q_m = discharge rate (m/day)

δh_m = fall in water table (m)

δt = change in time since the falling water table has acquired the shape of 2nd degree parabola.

Determination of the above parameters is not any easy task, especially where the discharge rate is underground. In such cases other methods are employed such as those employing the determination of hydraulic conductivity which is a relative easy factor to determine and then correlate it to drainable pore space.

As may be observed from all the above methods considered, determination of outflow from drains is required which is later correlated to groundwater drawdown. The determination of outflow from drains for independent determination of drainable porosity is only applicable in areas with already existing drains. In an area like the one where the present study was being conducted, such conditions did not exist, and therefore such methods could not be used. The method used for the final determination of the drainable porosity for the area was the one described by Smedema and Rycroff(1983), which expresses the drainable porosity as a function of the average saturated hydraulic conductivity as obtained from the field, Thus;

$$\mu=\sqrt{K} \tag{45}$$

Where, μ is the drainable pore space

K is the saturated hydraulic conductivity of the aquifer (m/day).

$\sqrt{}$ is positive square root

Another method also used is the one involving graphical determination of effective drainable porosity from hydraulic conductivity as described in FAO (1980) and ASAE (1990). The graphical representation of the set up is shown in figure 2.1 below.

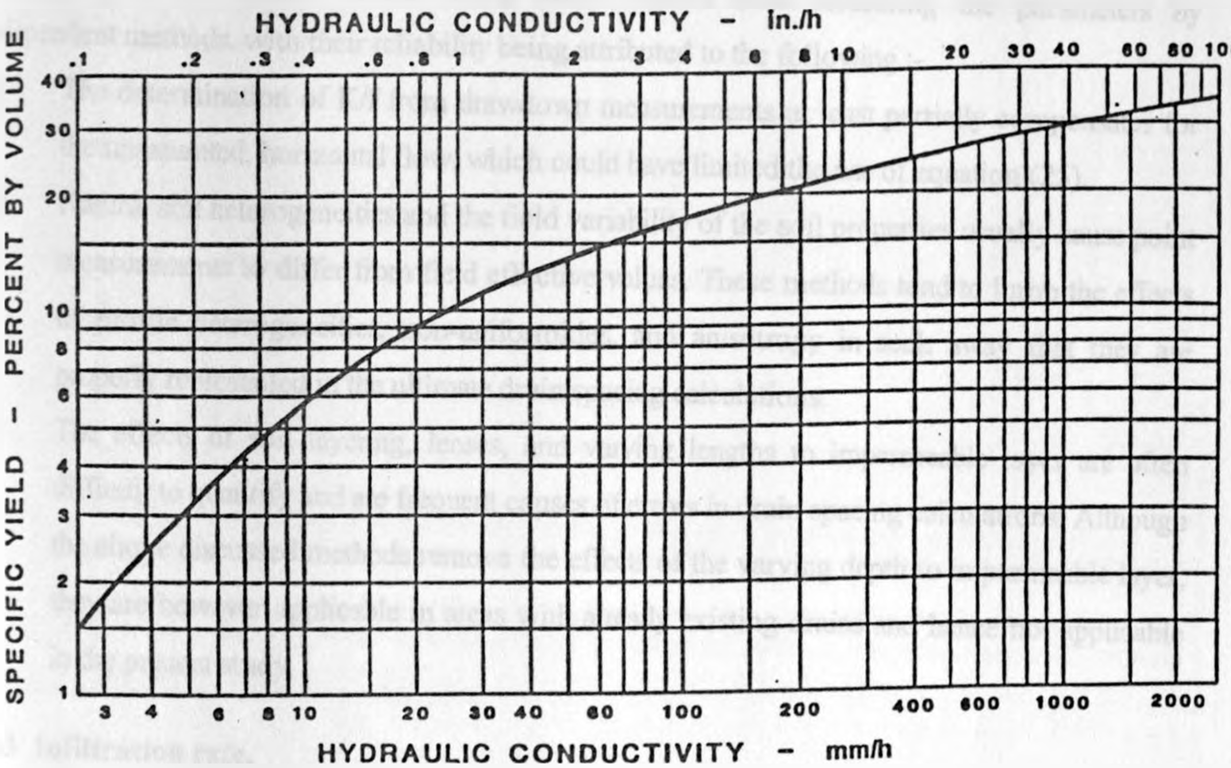


Figure 2.1 Curve showing general relationship between specific yield and hydraulic conductivity(after ASAE,1990).

Although the above methods were chosen for the determination of drainable porosity, they have one limitation in that, they depend on empirical relations which were developed from other places and may therefore not be reflecting the actual situation. However, they may still be applied in areas where other methods are not possible.

The most accurate methods however, are those employing soil-water characteristics (Dieleman, 1980) and the hydraulic conductivity-drainable porosity ratio from watertable drawdown measurements (Skaggs,1976). But, they also have their limitations in that, one or more drainage channels are required in the area under study if they are to be used and it is for that reason that the methods were not chosen despite being more reliable than measuring the parameters by independent methods, with their reliability being attributed to the following :-

(i) The determination of K/f from drawdown measurements at least partially compensates for the unsaturated, horizontal flow, which could have limited the use of equation (25).

(ii) Natural soil heterogeneities and the field variability of the soil properties usually cause point measurements to differ from field effective values. These methods tend to lump the effects of profile heterogeneities, non-uniformities, and anisotropy in such away that they are properly represented in the ultimate drain spacing calculations.

iii) The effects of soil layering, lenses, and varying lengths to impermeable layer are often difficult to quantify and are frequent causes of errors in drain spacing calculations. Although the above discussed methods remove the effects of the varying depth to impermeable layer, they are however applicable in areas with already existing drains and hence not applicable in the present study.

2.4.3 Infiltration rate.

In any drainage project it is essential to have an insight into the rate at which rain water enters into the soil, as water that does not infiltrate creates a nuisance on the land surface. Hence infiltration tests are required to establish this rate (Van der Meer and Van De Graaff,1980).

There are three common methods of estimating infiltration characteristics of the soil. The methods in common use are the Cylinder infiltrometer, measurement of subsidence of free water in a large basin, and estimation of accumulated infiltration from the water front advance data (Michael,1983).

The cylinder infiltrometer method is widely used owing to its simplicity. The cumulative infiltration can then be given by the Kostiakov equation written as (Van der Meer and Van De Graaff,1980) :-

$I_{cum} = a\,t^{n}$ (46)

where, I_{cum} = cumulative infiltration over time t (mm)

a = a constant and is a measure of magnitude of infiltration and for homogeneous soil is independent of time.

t = time that has elapsed since the ponding started (hours)

n = an exponent reflecting the change of infiltration rate with time.

The instantaneous infiltration rate can be derived from the above equation using calculus, as it is the rate of change of infiltration with time, that is :-

$$I_{nst} = \frac{d}{dt} I_{cum} = ant^{n-1} \quad (47)$$

Where I_{nst} is the instantaneous infiltration rate (mm/hr) and other parameters as described above.

The quantity mostly used in hydrologic studies is the basic infiltration rate assumed to be reached when the following conditions are fulfilled:

$I_t - I_{t-1} < 0.1 I_t$ (48)

where I_t = infiltration rate at time t (where t is in hours).

This gives the time that must elapse before infiltration becomes approximately constant to be (Van der Meer and Van De Graaff,1980):

$t = 10(1-n)$ (49).

where, t =time in hours

n= exponent in the cumulative infiltration formula.

The values of a and n can be determined from the plot of cumulative infiltration against the elapsed time on a double logarithmic graph, where n is the slope and a is the intercept. The same values may also be determined from linear regression techniques.

2.5 *Drainage systems Design.*

2.5.1 Drainage system Design approaches.

It is a well known fact that drainage problems are extremely complicated, hence in the design of a drainage scheme, the first thing to do is to form a simpler idealized picture of the flow system in such a way that; the most essential features are preserved and a mathematical or experimental approach becomes possible (Van Schilfgaarde, 1957). For artificial drainage, the drainage engineer therefore must attempt to make conditions opportune to accelerate seepage by increasing the energy gradient and thus enhancing faster removal of the water from the problematic area (Peterson, 1957). However, most of the drainage projects implemented to date in third world countries are more often than not been based on guidelines derived from experience rather than on sound analytical formulations(Van Schilfgaarde,1978).

It is also surprising to find that it is in the same countries where these experiences have not been gained which has resulted to mere use of rules of the thumb and consequently outright failures when a drainage design is chosen outside the safe range of options (Oosterbaan and Wind,1978).

The failures in most drainage projects are therefore not solely due to insufficiency of precise data, but due to the unawareness of the effects that changes in the water regimes may have and also due to lack of understanding of the broad inter-relationship between the various technical components as well as between the non-technical factors involved in drainage design (Dieleman,1978).

For example, the capacity of an open ditch or pipe drain as designed should be adequate to remove surface and subsurface water at a rate which will not cause serious damage to crops. This capacity will therefore be depended on the following interrelated factors:-

a) Precipitation
b) Size of the contributing area
c) Topography
d) Soil characteristics
e) Vegetation
f) Degree of protection warranted
g) Frequency and height of tidal and flood waters from rivers, lakes, creeks , and other outlets if they occur.
h) And in irrigated areas, the leaching requirements (Schwab et al, 1957 and Reeve and Luthin, 1957).

Effective drainage would therefore be required to accomplish two main objectives. Thus; remove the factors contributing to the problem and establish a satisfactory outlet-system that will prevent future flooding or waterlogging in the area under consideration (Edminster, 1957). The required performance of the resulting groundwater drainage system will then be defined by the combination of the minimum groundwater depth to be maintained in the critical periods and the amount of the excess water to be drained during those critical periods (Boumans,1987). The discharge rate also termed the drainage coefficient is therefore the rate to discharge the drainage surplus in the critical period. The depth to watertable for such situation is governed by the drainage criterion applied.

The design and the operation of an efficient agricultural water management system, which includes drainage is becoming more and more critical as competition for water resources increases. Although drainage is necessary to permit farming of some of the nation's most productive soils otherwise considered wastelands and left idle, and also provides trafficable conditions for seedbed preparations and planting in wet seasons, ensures suitable environment for plant's growth during the growing season and even permits harvest in the fall, it may however be undesirable when excessively done. The undesirable effects of over drainage results from excess reduction of soil water available to growing plants, leaching of fertilizer nutrients and hence carrying them to streams where they become pollutants (Skaggs, 1974).

Therefore any design of a drainage system must take such consequences into consideration. It is from such deficiencies as the ones discussed above that Van Schilfgaarde (1978),called for a better definition of drainage criteria and expansion of the data base to incorporate both crops and trafficability. This requires drainage engineers to be able to identify those most important factors for design and those which are not, and concentrate on the most important ones for successful drainage design.

2.5.2 Drainage design criteria.

In any drainage system, it is desirable that the designed drainage rate (drainage coefficient) be no larger than necessary to remove excess water adequately and soon enough for the needs of the crops or the field machinery. This means that the desired drainage rate logically should depend on climatic regime of the area, the drainage porosity and available water holding capacity of the particular soils and the watertable depth requirements of a particular crops or field machine operations (Chieng et al.,1978).

One of the most important aspect concerning drainage systems is the ability for the designer to determine the appropriate dimensions of the structures concerned, as it is those which account for the quality of the installation as well as the economic acceptability of the investment (Dierickx and Knops,1978). Drainage designs are mostly based on the Hooghoudt's equation due to its relative simplicity and versatility, that is it can be altered to fit both layered and unlayered soils. In this case, two mostly and widely used criteria are:-

i) Intensity criteria, (q/h) where q is the discharge rate and h is hydraulic head of groundwater mid-way between drains.

The above criteria enables the depth and intensity of drains to be chosen in such away that the drainage aims are fulfilled - Thus:-

a) avoiding too wet conditions during rainy periods
b) obtaining workable conditions shortly after rainy periods
c) controlling salinity in arid climates.

This drainage intensity is the most influential factor governing the groundwater table depths (Wind and Buitendijk, 1978).

ii) The unsteady state flow, which is expressed as the fall of the watertable required within a certain period after the watertable has risen near the surface. From the use of rainfall data, soil conditions and the use of basic drainage equations, these criteria can be established (Kessler, 1979). The establishment of the criteria will require a relationship to be established between the factors playing a role in the drainage problem. However, in an attempt to relate the above factors, the following questions would have to be answered:-

a) How do we characterise a soil profile consisting of a large number of different layers changing in position and magnitude from one place to another?
b) How do we measure the soil physical "constant"?
c) How do we formulate the agronomic requirements in respect to "excess water"?

To provide the answers to the above questions, which we must do in order to solve our drainage problems, lead to estimation of the various quantities which results in the prevailing inaccuracies in our designs (Anon, 1979). The kind of situation that results and is faced in everyday life as one tries to tackle drainage problems leads to the conclusion made by Clyde Horston in 1961 cited by (Anon, 1979) that "***Although excellent progress has been made in recent years in developing drainage criteria and investigational tools, it still takes good judgement, local experience and***

trial and error -along with thorough understanding of the basic principles to design a successful drainage system".

As of now, it is not uncommon to find drainage systems being designed using drainage coefficients selected many years ago, or based on experience from other areas which aggravates the situation further.

In humid areas, drains should remove excess water and provide a well aerated root zone, hence the design criterion here is formulated in terms of water table depth (Durnford et al.,1987).

When grazing lands are considered the various water table depths suggested by researchers to avoid poaching in grasslands are 80 cm below the surface (Kuntze, 1967) and 50 cm below surface reported by Massey and others in 1974 (Trafford, 1975).

According to research done by Browswijk (1988), waterlogged clay soils for grazing cattle requires a bearing capacity of the top soil of at least 0.6 Mpa. This value corresponds to a depth of not less than 35 cm of ground water below the ground surface which supports the above research work with only refined findings.

The methods used for estimation of water table positions after a given recharge are many, but the most popular ones used in situations of unsteady-state conditions are :-

(i) **De Zeeuw and Hellinga formula** as reported by Smedema and Rycroft (1983), thus;

$$H_t = H_{t-1}e^{-\alpha\Delta t} + \frac{R_{\Delta t}}{0.8\mu\alpha}(1 - e^{-\alpha\Delta t}) \tag{50}$$

where, H_t = water table depth at time t (m)

H_{t-1} = water table depth the previous day (m)

$R_{\Delta t}$ = Recharge (rainfall) during t-(t-1) (m)

μ= drainable pore space (-)

α= reaction factor = KD/L^2 (day^{-1})

K= hydraulic conductivity (m/day)

D = depth to impervious layer (m)

L = drain spacing (m)

The criteria will be taken as t = 4 days and H = 0.5 m as suggested for grass and animal destruction (Smedema and Rycroft, 1983).

(ii) Glover-Dumm equation, which is the most commonly used equation to evaluate drain spacing and is given as (Gupta and Gupta,1987):

$$S^2 = \frac{\pi^2 KDt}{\mu \ln(\frac{4}{\pi}\frac{H_o}{H_t})} \qquad (51)$$

where, S= drain spacing (m)

π^2 and $4/\pi$ = numerical constants

K= hydraulic conductivity (m/day)

D= saturated thickness (m)

t= Time in days when the water should be lowered to the required depth.

μ = drainable porosity.

H_o, H_t = hydraulic head above drain level after instantaneous recharge and at any time respectively (m).

As this equation does not take into consideration the radial resistance of flow towards drains not reaching the impermeable layer, the thickness of the aquifer, D, is replaced by the equivalent depth, d, as in Hooghoudt's equation. This equivalent depth is either determined from Nomographs or complicated formulae which are also depended on the drain spacing to be established . This requires many iterations before the right solution is obtained and hence a disadvantage to the approach.

(iii) Boussinesq equation .

Drainage design for unsteady- state conditions, which are normally the case with drainage problems resulting from precipitation is possible with the application of Glover-Dumm equation. Especially in cases where the drains rest in the impermeable layer, the Boussinesq equation is widely used (Gupta and Gupta.,1987). The equation is given as :

$$L^2 = \frac{4.46\, H_t\, H_o / t}{\mu (H_o - H_t)} \qquad (52)$$

Where, L = drain spacing (m)

H_o= hydraulic head above drain level after instantaneous recharge (m).

H_t= hydraulic head above drain level mid-way between drains at any time (m).

t= time in days when optimum level should be reached.

μ= drainable pore space (-).

(iv) Transient Flow Method (USBR).

This method takes into account the transient nature of ground water recharge and discharge. The method gives drain spacings which produce dynamic equilibrium of the true fluctuating watertable while fitting the requirements of keeping the watertable below a specified depth. It employs the relationships between the dimensionless parameters of y/y_o versus KDt/SL^2 and Z/H vs KHt/SL^2 where the parameters are defined as (ASAE,1990):

y_o and H = mid point water table height at the beginning of any drain-out period (m)

y and Z = mid point water table height at the end of any drain out period (m)

L = drain spacing (m)

d = distance from drain to barrier (m)

$D = d+y_o/2$ = average flow depth (m)

K = hydraulic conductivity in flow zone (m/day)

S = specific yield in the zone of watertable fluctuation (-)

t = drain out time (days)

For the purposes of this study, the last two methods were used.

2.5.3 Rainfall analysis for drainage design.

Rainfall analysis for drainage design is restricted to that part of the hydrologic year during which excess rainfall may cause damage. The period is depicted by rainfall exceeding evapotranspiration (Kessler and Raad,1979).

There are various methods of analysing daily rainfall data depending on the nature of the work to be undertaken. Thus; For irrigation and water supply purposes, rainfall data of a normal, wet, and dry years are normally used, where an estimate of the respective rainfall data can be obtained by computing and plotting probabilities from the rainfall records of many years at the required station. The rainfall at 20, 50, and 80 percent probabilities of exceedance are considered the wet, the normal and the dry year respectively (Smith,1992).

However, in drainage, the critical factor is the duration the crops in question can withstand inundation and the watertable depth that once maintained for a certain duration can cause damage. Therefore, the approach used in drainage is different from the one of water supply and irrigation. Thus; for drainage design purposes, the rainfalls for the critical durations can be analyzed using daily rainfall records where available and their frequency of occurrence determined.

Three methods of computing rainfall amounts are in use, successive totals, moving totals and maximum moving totals (Kessler and Raad, 1979). Mostly, successive maximum moving totals are used because the method takes into consideration all possible totals and thus do not result in an underestimation of any high rainfall total (Beltran,1978; Kessler and Raad,1979). The method of rainfall analysis put forward by Kessler and Raad (1979) is commonly adopted and the number of successive days used depends on the crop to be grown in the area, as sensitivity to waterlogging is crop depended. Rainfall values measured in the area and previous records of rainfall can be used in the determination of the following:

i) design drainage rate

ii) design rainfall

A design rainfall as obtained from the method put forward by Kessler and Raad (1979), is used to determine the design drainage rate using the method postulated by Farr and Henderson (1986). Thus;

$$q= [((R \times r) + g)/S] \tag{53}$$

Where, q=design drainage rate (mm/day)

R=design rainfall rate (mm/day)

r=adjacent value for surface runoff as a factor depending on slope of the area slopes.

S=the chosen safety factor for siltation which depends on soil type

g=groundwater flow (mm/day)

2.5.4 Storm Analysis.

Different characteristics of rainfall are important to specialists in various fields, and therefore the number of ways of analysing rainfall data are virtually unlimited. The method chosen depends upon the nature of the available data and the purpose of the investigations (Dunne and Leopold,1978).

The data available may range from daily rainfall values to storm data with their corresponding intensities and durations. Rainfall storm durations are assumed to be randomly distributed and thus can be estimated stochastically with the method described by Chow et al.(1988), presented as :

$$X = \mu + K\sigma \tag{54}$$

where X = the required storm (mm)

μ = the mean from the parent data (mm)

σ =The standard deviation (mm)

K = a frequency factor corresponding to the return period and the distribution.

In many occasions, the data available is for short durations and may not be usable for design purposes unless it is extended to cover a longer duration. As it is also uneconomical to design structures to cope with extreme events in low priority, low investment situation, A designer takes a calculated risk and designs a structure that will accommodate the largest rain storm that can be expected in some generally agreed upon time interval. In such cases an approximating frequency distribution must be used (Dunne and Leopold,1978).

The data to be fed into the approximating distribution may not necessarily need to come from the catchment in question, but may come from an area with the same climatic regime and hence the resulting frequency curve will approximate the true curve for the study catchment (Linsley et al.,1988).

The Gumbel Extreme - Value frequency distribution is not the only one that can be used for estimating large events, but is the most popular and has received widest applications in various parts of the world and better still it is found to yield results of acceptable accuracy (Dunne and Leopold,1978).

3. METHODOLOGY

3.1 The Project Site.

The project was undertaken in the University of Nairobi's Veterinary Farm in Kanyariri, which is located in Kiambu district of the Central Province of Kenya. The nearest shopping centre is Kanyariri market, about four kilometres North west of Upper Kabete Campus. The total area of the farm is more than 100 ha with the catchment contributing the damaging water being approximately 70 ha. The study area is a bottomland which extends to about 620 m in length and 100 m in width and occupying an estimated area of 6 ha of which the drainage problems are on approximately 5 ha of the land area for most of the wet seasons of the year.

The experimental site lies at longitude 36° 42' East and Latitude 1°14' South. It lies at an average altitude of 1880 m above the mean sea level with average slopes being 1.5% and slopes towards the bottomland at about 13.5% (Ngigi, 1991).

The soils in the affected area are predominantly clay, with red clay soils on the higher parts and black clay in the depression.

During wet seasons, seepage spots are observed at the transition between the higher parts and the depression which continue into the black clays where the waterlogging occurs.

Due to waterlogging problems, most of the area is left vegetated with reeds and some spots of wire grass (*Pennisetum Schimperi*). The wire grass may be used for grazing or not depending on the stage of growth, but the reeds are unpalatable to livestock.

The area receives an average annual rainfall of about 980 mm which is bimodal in nature, and from past rainfall records, long rains begin in mid March and end in May whereas the short rains begin in October continue through November and end in December (Jaetzold and Schimidt,1986). The wettest months are April, May in the long rains and October and November in the short rains with much of the rainfall being concentrated in the long rain months.

In the past, there seemed to have been an attempt to control the drainage problem, by use of open ditches which also seemed not to have functioned as expected since the problem is still present. The problem could have persisted due to improper maintenance of the ditches or due to improper installation of the control measures used owing to improper design. The study area is shown in figure 3.1.

3.2 Methods of study.

3.2.1 Inventory of soil properties.

Three Profile pits were dug in the experimental site; one on each different soil group as suggested by the presence of different plant vegetation. The distribution of the observation sites for both hydraulic conductivity and other soil properties is shown in figure 3.2 The pits were described according to FAO guidelines as described in FAO/UNESCO (1977) and sampled for the determination of various soil properties as described below :

3.2.2 Saturated hydraulic conductivity of soil cores.

Undisturbed soil samples were obtained in metal core rings fitted into a core sampler and driven into the soil using a plastic mallet. The average size of the rings used to take the samples was 5.0 cm in height and 5.0 cm in diameter. The samples were taken from each of the three horizons found in the three profile pits.

Six core samples were taken from each horizon, three vertically and three horizontally. Care was taken to avoid disturbance of the soil as it was being transported from the field to the laboratory.

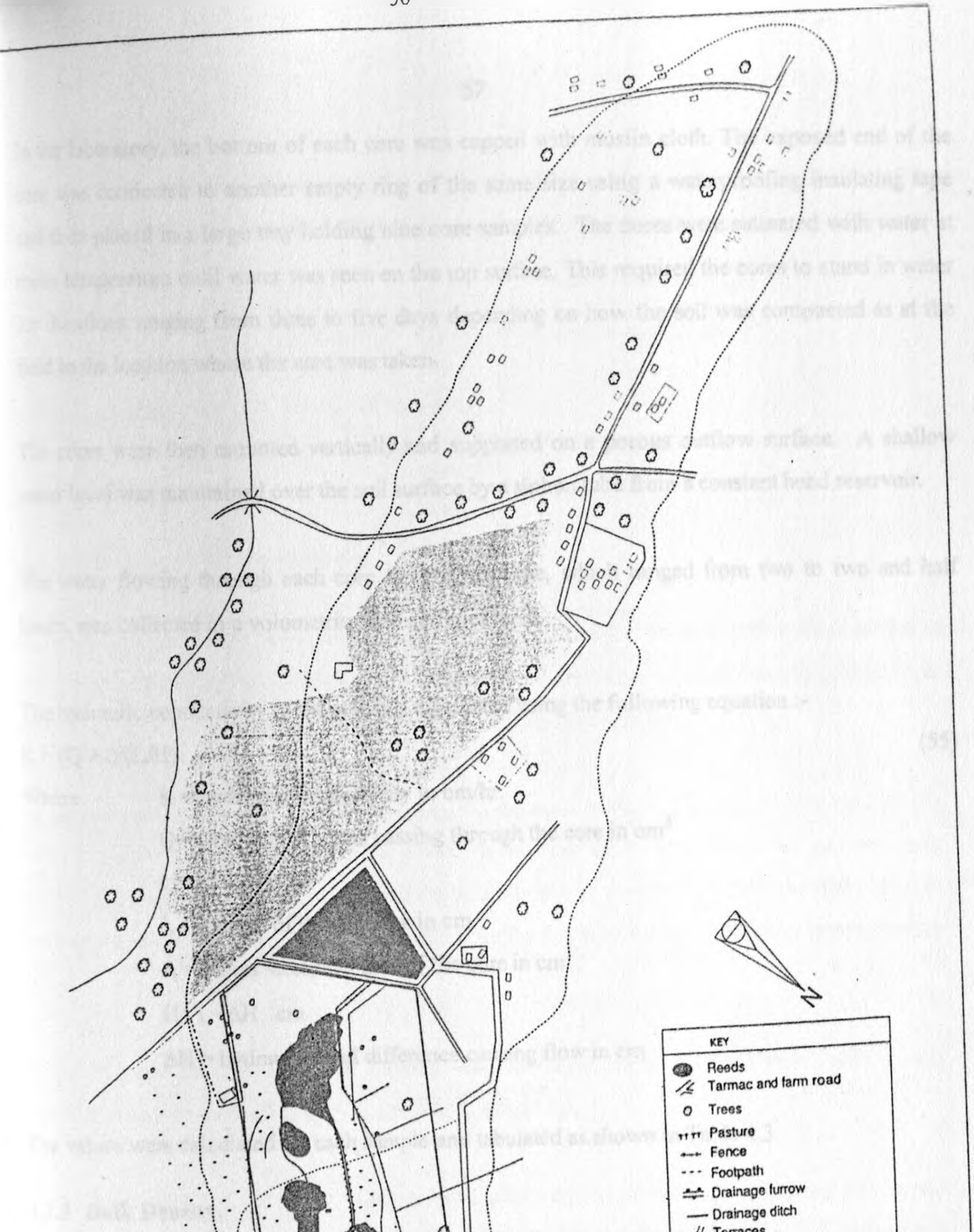


Figure 3.1: The location of the project area

In the laboratory, the bottom of each core was capped with muslin cloth. The exposed end of the core was connected to another empty ring of the same size using a waterproofing insulating tape and then placed in a large tray holding nine core samples. The cores were saturated with water at room temperature until water was seen on the top surface. This required the cores to stand in water for durations ranging from three to five days depending on how the soil was compacted as at the field in the location where the core was taken.

The cores were then mounted vertically and supported on a porous outflow surface. A shallow water level was maintained over the soil surface by a siphon tube from a constant head reservoir.

The water flowing through each core for a given time, which ranged from two to two and half hours, was collected in a volumetric flask and measured.

The hydraulic conductivity of the soil was calculated using the following equation :-

$$K = (Q/At)/(L/H) \tag{55}$$

Where K = hydraulic conductivity in cm/hr.

Q = Volume of water passing through the core in cm^3

t =time in hours

L = Length of the soil core in cm

A = Cross -sectional area of the core in cm^2

$H = L + \Delta H$ cm

ΔH = hydraulic head difference causing flow in cm

The values were calculated for each sample and tabulated as shown in Table 4.3

3.2.3 Bulk Density.

The same samples used for hydraulic conductivity determination were also used for the determination of bulk density.

The mass of each core was determined after drying to 105·°C and the volume taken as that of the sample as obtained from the field. The bulk density of each sample was calculated from the following equation :-

$\rho_b = M_s/V_t$ (56)

where ρ_b = bulk density, g/cm^3

M_s =weight of oven dried soil, gm

V_t = volume of soil core as obtained from the field. (cm^3).

3.2.4 Infiltration rate measurement.

The infiltration rates were conducted during April when rainfall was supposed to have started to reflect the practical field conditions. The measurements were done adjacent to each profile pit so that readings could be compared to the ones obtained from core samples.

The measurements were done using double ring infiltrometer method. The method involved driving two concentric metal cylinders into the ground to a depth of 10 cm. The inner and the outer cylinders had a diameter of 30 cm and 100 cm respectively. A column of water was maintained in the sleeve between the two cylinders to act as a buffer and hence keep infiltration approximately vertical.

The fall in water level in the inner cylinder was monitored using a rubber float with a stem graduated in centimetres and kept in position by a metal clip.

The fall was measured after every five minutes until three to four hours depending on the site conditions. The same observations were repeated the following day and average soil parameters were obtained.

Using the relationship for cumulative infiltration shown in equation (46), the parameters were determined. The basic infiltration, which is the relatively constant rate of infiltration that develops after long time depending on the type of soil, was determined from a plot of the infiltration rate against time.

3.2.5 Saturated hydraulic conductivity (The auger hole method)

Three sites in the problematic area where the watertable was near the surface were chosen.

(i) The area was cleaned from trash, loose soil and plant materials.

(ii) An auger hole 5.08 cm in diameter was bored to at least 40 cm below water table.

(iii) Water was bailed out of the hole using an improvised bailer and groundwater was allowed to fill the hole, this was allowed to continue until equilibrium reached.

(iv) After equilibrium was attained, that is when ground water was not raising any more, water was removed from the hole and the rate of rise measured by measuring the changes in water level during a given period, one minute.

The readings were however, completed before the hole was half full and the hydraulic conductivity was determined from equation (28).

3.2.6 Hydraulic conductivity (Inverse Auger hole method)

Ten sites were chosen with at least three from each of the areas infested by different vegetation. Holes were bored to depths varying from 50 cm to 126 cm, depending on the site conditions. That is shallower where the murrum layer was near the surface and deeper where the soil was easily workable.

The holes were filled with water for three to four times before the experiment was started. After saturation was assumed to have been attained, the hole was filled with water and the rate of fall of the water level was measured using a rubber float with a graduated stem to determine the fall with time.

In the study area the experiment was repeated at least twice in every site and sometimes thrice as the soils were predominantly clay in most of the study area. The data was plotted against time on a

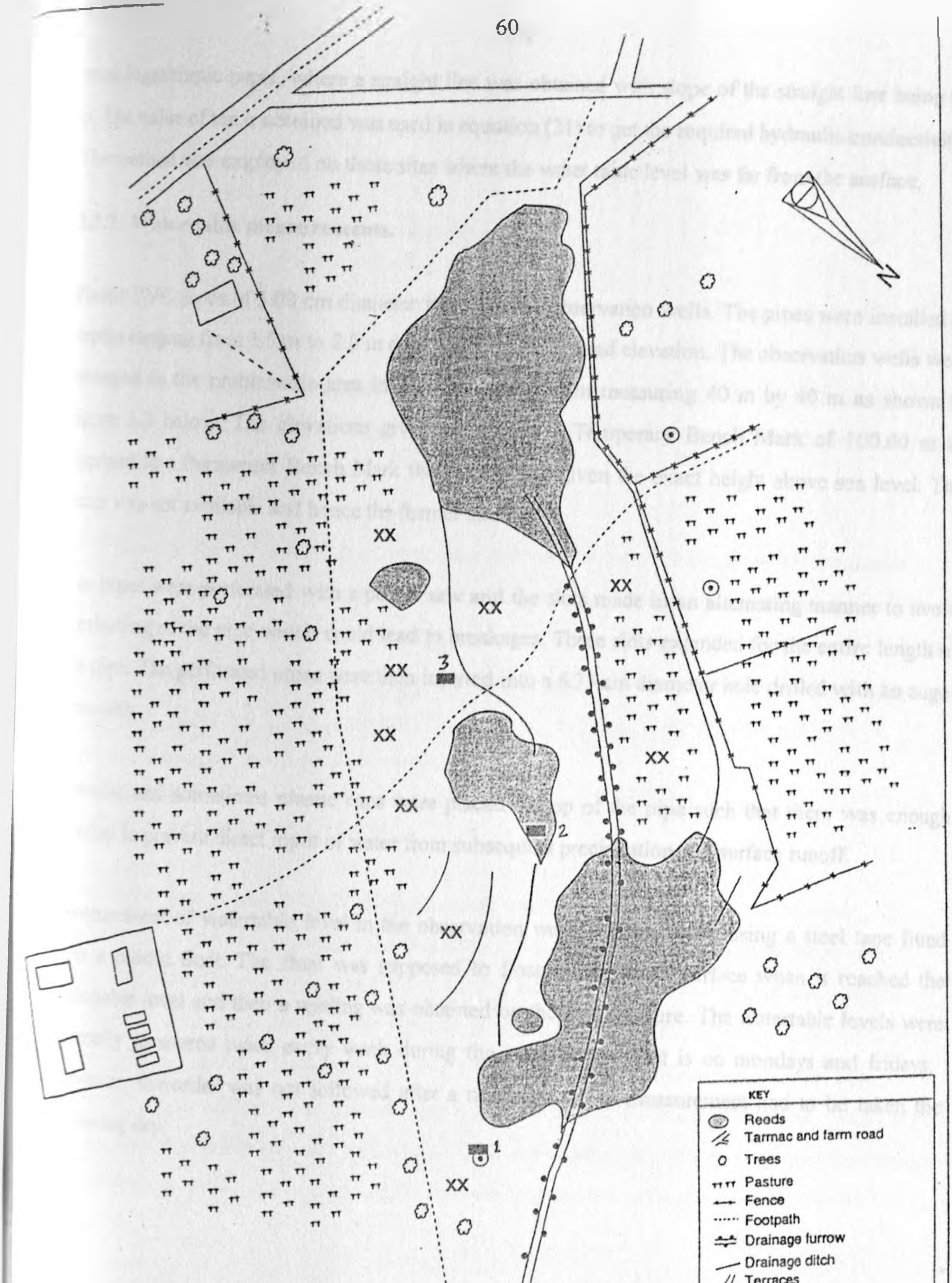


Figure 3.2: Distribution of observation sites for the poorly drained area

semi-logarithmic paper, where a straight line was obtained with slope of the straight line being tan α. The value of tan α obtained was used in equation (31) to get the required hydraulic conductivity. The method was employed on those sites where the water table level was far from the surface.

3.2.7 Water table measurements.

Plastic PVC pipes of 5.08 cm diameter were used as observation wells. The pipes were installed to depths ranging from 1.5 m to 2.0 m depending on the ground elevation. The observation wells were arranged in the problematic area in a square grid system measuring 40 m by 40 m as shown in figure 3.3 below. The elevations given were from a Temporary Bench Mark of 100.00 m as opposed to a Permanent Bench Mark that could have given the exact height above sea level. The latter was not available and hence the former used.

The pipes were perforated with a power saw and the slots made in an alternating manner to avoid weakening of the pipe which could lead to breakages. These slots extended for the entire length of the pipe. The perforated pipes were then inserted into a 6.35 cm diameter hole drilled with an auger manually.

Metallic and sometimes plastic caps were placed on top of the pipe such that there was enough overlap to prevent direct input of water from subsequent precipitation and surface runoff.

Measurement of watertable level in the observation wells were made by using a steel tape fitted with a plastic float. The float was supposed to float on the water surface when it reached the watertable level and then a reading was obtained on the tape measure. The watertable levels were normally measured twice every week during the study period, that is on mondays and fridays. However, the order was not followed after a rainfall event as measurement had to be taken the following day.

Watertable level for two selected days, that is one when the watertable was highest and the other when the water level was almost drying, but present in most wells were used for plotting of ground water contour map, which was used for the drainage design.

3.2.8 Overland flow from adjacent high ground.

Runoff from four selected landscape positions was to be measured with Gerlach troughs, which are small run off troughs measuring 50 cm by 50 cm. The troughs were fixed to the A-horizon of the slope by means of a metal flange and protected from splash erosion and direct precipitation by a movable metallic lid. The flange was placed in such a way that it corresponded to the slope surface of the ground.

Runoff flow and the accompanied sediment were supposed to drain via plastic hoses to a plastic bucket dug into the downward slope. The runoff from the adjacent area was to be determined from the knowledge of slope and the contributing area, which was to run up to the divide. The divide was taken to be were the slope shape changed to indicate a ridge in the contour map. The layout of the arrangement is shown in figure 3.4 below. The levels are based on Temporary Bench Mark of 100.00 m as explained in the previous section.

3.2.9 Runoff estimation procedures.

Runoff from adjacent areas towards the bottomland was estimated by use of two empirical procedures. This was necessitated by the lack of runoff data during the study period as gerlach troughs did not catch any runoff owing to the rainfall showers which fell during the study period. The methods used were the Rational formula (Hudson,1971) and the water balance method of Thornthwaite and Mather (Dunne and Leopold,1978). In the Rational formula, equation (16), the time of concentration was calculated using Kirpich and Bransby-Williams formula, which were compared and the value from the later was adopted due to its superiority as suggested by Hudson (1971). Bransby-Williams method gives the formula for time of concentration as (Hudson,1971):

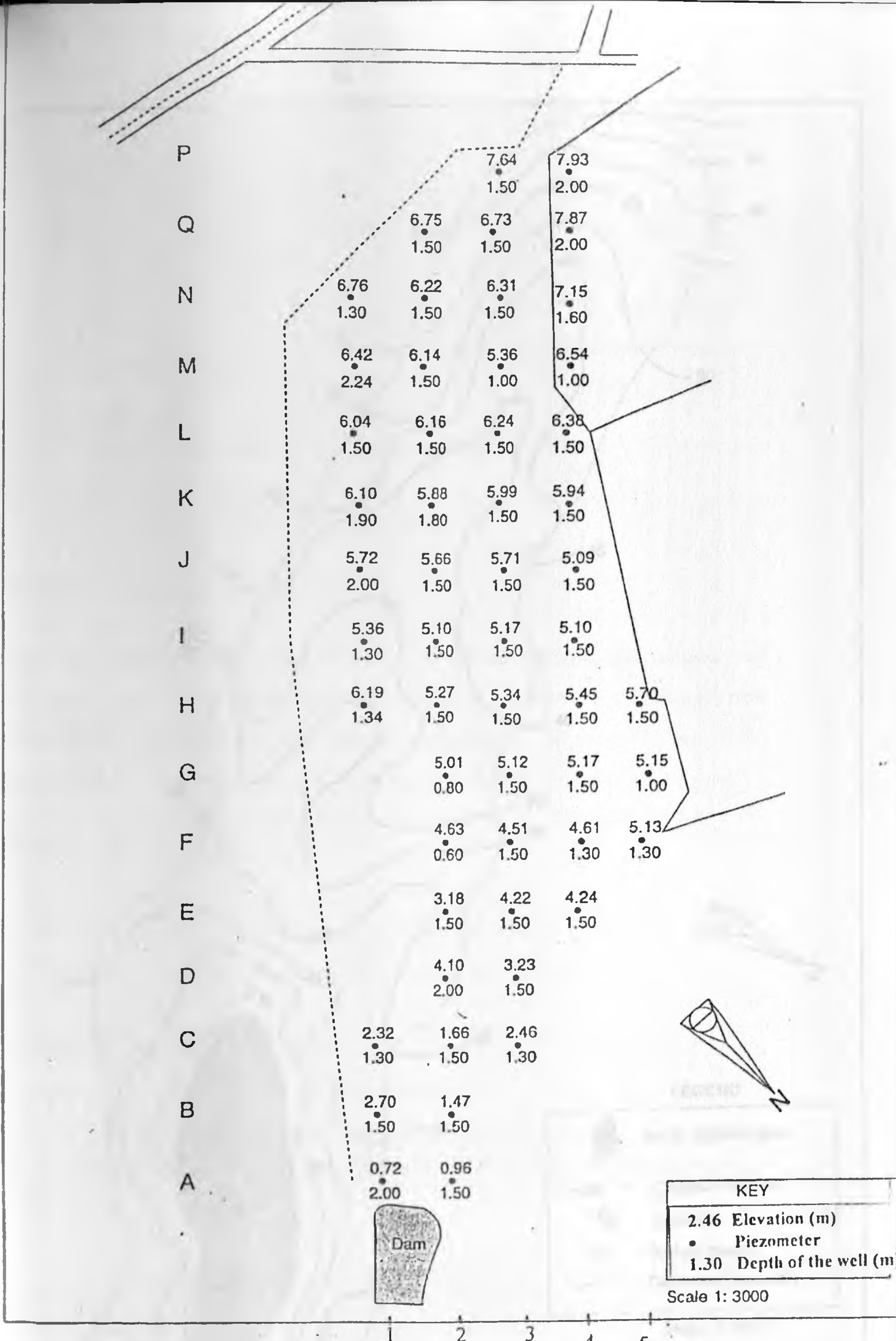


Figure 3.3 Detailed layout of the observation wells in the poorly drained area.

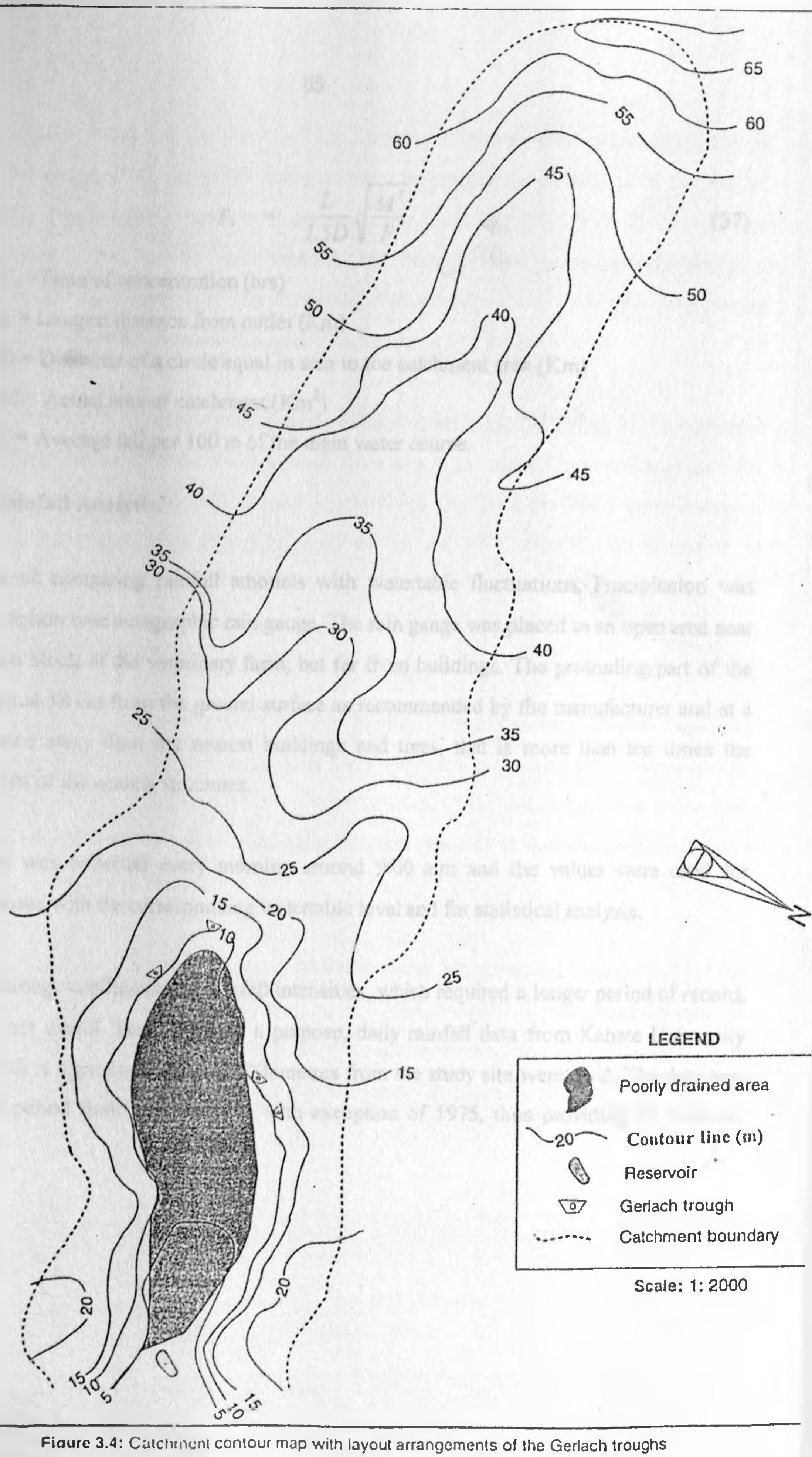


Figure 3.4: Catchment contour map with layout arrangements of the Gerlach troughs

$$T_c = \frac{L}{1.5D}\sqrt[5]{\frac{M^2}{F}} \qquad (57)$$

where T_c =Time of concentration (hrs)

L = Longest distance from outlet (Km)

D = Diameter of a circle equal in area to the catchment area (Km)

M = Actual area of catchment (Km^2)

F = Average fall per 100 m of the main water course.

3.2.10 Daily Rainfall Analysis.

For the purpose of comparing rainfall amounts with watertable fluctuations, Precipitation was measured using siphon type autographic rain gauge. The rain gauge was placed in an open area near the administration block of the veterinary farm, but far from buildings. The protruding part of the raingauge was set at 58 cm from the ground surface as recommended by the manufacturer and at a reasonable distance away from the nearest buildings and trees, that is more than ten times the approximate height of the nearest structures.

The rainfall data was collected every morning around 9.00 a.m and the values were used for comparison purposes with the corresponding watertable level and for statistical analysis.

However, for Drainage coefficient and rainfall intensities, which required a longer period of record, the values were not useful. Thus, for such a purpose, daily rainfall data from Kabete University field station, which is approximately four Kilometres from the study site were used. The data was available for the period from 1971 to 1993 with exception of 1975, thus providing 22 years of continuous data.

Moving daily rainfall totals for durations of one day up to eleven days were computed as recommended by Kessler and Raad (1979) using a computer programme developed by the author. The full Computer Programme is shown in Appendix 5. The maximum value for each duration was chosen. Maximum values for the same duration, but from different years were ranked in a descending order. The same procedure was repeated for all durations and every time determining the probability of exceedance, non-exceedance and the return period for each value.

These results were plotted in a normal graph paper of Depth-return period relations from which data for a return period could be obtained for all rainfall durations. The latter information was used for the development of Depth-Duration-Frequency relationships for the area. The Depth-Duration-Frequency curves accompanied by storage capacity of the soil were used for determination of drainage coefficient for the area.

3.2.11 Storm rainfall analysis.

The determination of rainfall intensity was done in a slightly different manner. In that from daily autographic rainfall charts for the area, storm data were extracted and analyzed for maximum 15 minutes, 30 minutes and other multiples of 15 up to 180 minutes. This was necessitated by the need to obtain lateral run-off into the area from adjacent high grounds which depends on rainfall intensity.

The values were subjected to Gumbel frequency distribution and the values for 0.25, 0.5,1,1.5,2 and 3 hour maximum storms with return periods of 2, 5, 10, 15, 20, 50 and 100 years were determined. The values so obtained were used to establish the drainage coefficient and for calculation of the runoff into the area which depends on time of concentration of the storm in question.

4. RESULTS AND DISCUSSIONS.

4.1 Climate.

Average rainfall and standard Meteorological data as obtained from a neighbouring weather station situated about four Kilometres away are shown in Tables A4.1 and A4.2, in Appendix 4. The tables show an average data for the period 1971 to 1993.

Figure 4.1 below shows the rainfall and evapotranspiration diagram for the area and was prepared using data for the same duration. From figure 4.1, the wettest months, that is months when rainfall exceeded evapotranspiration, or when the two parameters were nearly equal, were found to be April and May in the long rains season and November in the short rains season.

The rainfall during the study period is shown in Table A4.3 of Appendix 4 and confirms the message conveyed by the rainfall and evapotranspiration diagram shown in figure 4.1. From the figure, it can also be inferred that in six out of 12 months, excess water is available in the area under study, and hence drainage is required during that time. Also clearly seen from the rainfall and evapotranspiration diagram is that, the other half of the year Evapotranspiration is greater than rainfall and thus depending on the level of the ground water table either irrigation or drainage would be required in the same place.

As design rainfall is the most critical rainfall event that drainage system should cope with, and observing that drainage coefficients in the long rains period are higher than in the short rains period as depicted by figure 4.1, it was decided that any design that could take care of waterlogging during the long rains period would as well take care of the same during the short rains period. It is for this reason that short rains period were discarded from the calculations. This decision was arrived at after a comparison between the average rainfall and average evapotranspiration in the study area was done. From it, it became evident that April and May were the months when rainfall exceeds evapotranspiration by more than 40 percent. Thus during such months, if the soil does not allow for

...tural drainage as the one in the bottomlands of the Veterinary farm in Kanyariri, artificial ...inage is evidently necessary.

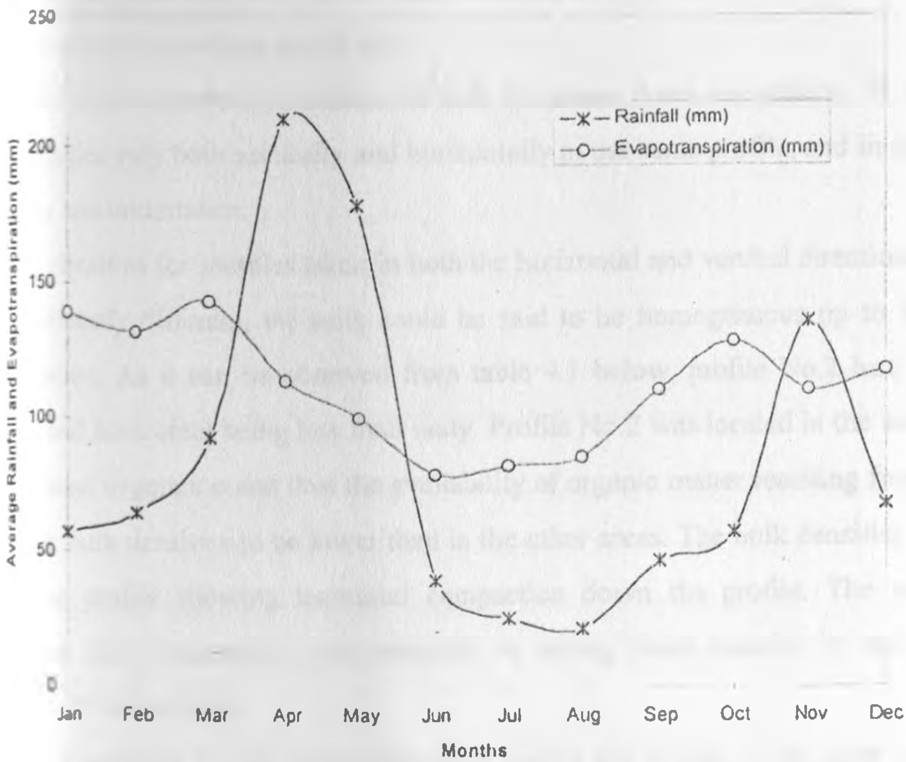


Figure 4.1 **Average variation of Rainfall and Evapotranspiration over the year**

4.2 *Soil Characterisation at the experimental site.*

4.2.1 Soil Properties

The soil texture as analyzed according to the method of 'feel' was found to be predominantly clay. This was true in over 90 percent of the bottomland. However, where waterlogging continued for long periods, organic soils, mainly peat were prevalent.

The bulk density of the soils was found to range from 1.17 to 1.52 g/m^3 in most of the profiles investigated and increases with the profile depth, with exception of profile pit number two which was predominantly in an area infested by water loving reeds. In that area the bulk density ranged

0.73 to 0.98 g/m^3 for the top horizon, a fact that could be attributed to the top soil being ominantly organic peat. The presence of such soils in an area requiring drainage is an ation that care should be taken when draining such portions as subsidence may result when utionally measures are not taken.

4.1 below shows the variation of bulk densities down the profile. It also shows how the densities vary both vertically and horizontally in the same profile, and in the entire area where udy was undertaken.

ulk densities for samples taken in both the horizontal and vertical directions were not found to ignificantly different, the soils could be said to be homogeneous up to 1.2 m, the depth of stigation. As it can be observed from table 4.1 below, profile No.2 had bulk densities both cal and horizontal being less than unity. Profile No.2 was located in the area with reeds as the ominant vegetation and thus the availability of organic matter resulting from decomposed peat e the bulk densities to be lower than in the other areas. The bulk densities generally increased n the profile showing increased compaction down the profile. The soil properties were mined both horizontally and vertically by taking three samples in each of the respective ction for all horizons.

soil description for the three observation profile pits chosen in the study area are presented in e A1.1 to A1.3 of Appendix I and all depicted clayey soils (Vertisols) with poor drainage.

areas were considered representative as they represented the three major vegetation classes rring in the project site;

mely, overgrown wire grass (pit no.1); growing wire grass (pit no.2); and reeds (pit no.3). The ation of these pits is shown in figure 3.2 described in previous sections.

e profile data as used in the determination of hydraulic conductivities also helped to have an ght as to how the hydraulic conductivity varied with depth from the surface both horizontally d vertically. It also indicated to which depth, the strata may be classified impermeable.

Table 4.1 Average bulk densities of the soils in the study site (ρ_b g/cm^3)

Direction/	Vertical				Horizontal			
Replicate Number	1	2	3	Av.	1	2	3	Av.
	Pit 1							
Depth (cm): 0-15	1.31	1.33	1.42	1.35	1.20	1.28	1.17	1.22
15-78	1.32	1.27	1.25	1.28	1.35	1.33	1.23	1.30
78-120$^+$	1.35	1.42	1.34	1.37	1.56	1.48	1.53	1.52
	Pit 2							
Depth (cm): 0-16	0.73	0.81	0.8	0.78	0.89	0.97	0.98	0.95
16-26	1.30	1.28	1.23	1.27	1.30	1.31	1.16	1.26
26-120$^+$	1.38	1.30	1.43	1.37	1.47	1.36	1.57	1.47
	Pit 3							
Depth (cm): 0-10	0.97	0.98	0.91	0.95	1.10	1.28	1.29	1.22
10-22	1.13	1.22	1.23	1.19	1.04	1.18	1.13	1.12
22-134$^+$	1.14	1.20	1.28	1.21	1.17	1.19	1.15	1.17

The soil as depicted by low hydraulic conductivity values indicates a soil that cannot be drained adequately by sub-surface drainage alone. (The lowest value of hydraulic conductivity for successful subsurface drainage should be more than 0.096 m/day).

4.2.2 Soil hydrological properties useful in Drainage Design.

4.2.2.1 Saturated Hydraulic conductivity.

The hydraulic conductivities as determined from the inverse auger hole method and the auger hole method are summarised in Table 4.2 below. The inverse auger hole was employed in many sites as watertable was only high in some concentrated points. Thus the determination of hydraulic conductivity in all such areas would not have yielded a representative value. The auger hole method was conducted in three selected sites where the watertable was high and were representative for the whole area as they were done in areas with different soils as depicted by differences in vegetation. The average saturated hydraulic conductivity from the study area can be classified as moderately slow to very slow as can be seen from table 4.2, which implies difficulties with drainage of the area

...er consideration. However, exceptional hydraulic conductivity of more than 4 m/day were found ...areas inhabited by overgrown wire grass. After close examination of the areas with such ...ceptional hydraulic conductivity, it was found that there was a layer of weak murrum at 50 cm ...low the surface and hence the reason for the high hydraulic conductivity. The overall saturated ...draulic conductivity obtained as a geometric mean was then 0.06 m/day.

...e values starred (**) were not used for computing the average value of hydraulic conductivity as ...e site was different from the others with murrum layer at 50 cm below the surface and hence ...ster water transmission.

...e raw data from which the various hydraulic conductivity values presented in table 4.2 were ...tained are shown in Tables A2.1 to Table A2.18 in Appendix 2.

...2.2.2 Depth to Impervious Layer.

...e depth to the impervious layer could not be located precisely as the different profile pits gave ...fferent results as shown in table 4.3 below. Thus for profile pit 1, and using the definition of ...SDI, the vertical hydraulic conductivity values indicated no barrier to water flow down to a depth ... 1.2 m (the investigated depth); whereas the horizontal hydraulic conductivity indicated an ...npervious strata after 15 cm. In pit 2 and 3, the results indicated an impervious strata 10-16 cm ...sing both vertical and horizontal Hydraulic conductivity values.

...om Table 4.3 below, the hydraulic conductivity of the soils in the area were generally observed to ...ecrease with depth in both vertical and horizontal directions. There were, however, some cases ...here the trend was not clearly observable. Such cases could be attributed to soil compaction by ...razing animals to a lesser extent, which preferred some areas more than others and mostly to the ...oils general anisotropy as reflected in the k_r ratio also shown in the same table.

Table 4.2 Hydraulic conductivity values from the inverse auger hole and auger hole methods (m/day).

Location	Trial 1	Trial 2	Trial 3	Average value	Method used	Remarks
1	0.140	0.230	-	0.190	AH	m.slow
2	0.065	0.056	0.069	0.063	IAH	slow
3	0.003	0.005		0.004	IAH	v.slow
4	0.006	0.005		0.006	IAH	v.slow
5	0.143	0.071		0.107	IAH	slow
6	0.053	0.052		0.053	IAH	v.slow
8	0.080	0.077		0.079	IAH	slow
9	0.094	0.050		0.072	IAH	slow
10	4.110**	4.690 **		4.400 **	IAH	rapid
11	0.715	0.822	0.828	0.788	AH/IAH	moderate
12	0.060	0.027		0.044	IAH	v.slow

The symbols AH, IAH, m. Slow and v. Slow were used to mean auger hole method, inverse auger hole method, moderately slow and very slow respectively.

ble 4.3 Saturated hydraulic conductivity (K-value) for the study area from constant head thod (m/day).

ofile pit No./depth(cm)	Direction of measurements		Ratio $K_r = K_H/K_V$
	Vertical (K_V)	Horizontal(K_H)	
Pit No.1			
-15	0.015	4.669	311.30
5-78	0.012	0.033	2.75
78-120$^+$	0.017	0.046	2.70
Pit No.2			
-16	8.084	6.221	0.77
16-26	0.010	0.070	7.00
26-120$^+$	0.033	0.008	0.24
Pit No.3			
-10	1.050	0.481	0.46
10-22	0.040	0.027	0.68
22-134$^+$	0.011	0.015	1.36

ing the watertable contour maps, shown in figures 4.2 and 4.3 below which indicate the water ble levels on 5/2/93 and 29/3/93 when it was highest and lowest respectively, a barrier in the top cm is not noticed as suggested by the table above. The barrier is however, noticed at 1.0 m low the soil surface (marked by close contours then wide contours) around A1--B1. Since the tertable contour method is more realistic than the saturated hydraulic conductivity ratio method, depth of the impervious layer was taken to be 1.0 m at the lower end of the poorly drained area d deeper at the upper part of the bottomland as it was not detected at the 1.0 m depth in the upper ts of the bottomland.

ce it could not be established precisely in the entire area, an average value of 1.2 m was chosen.

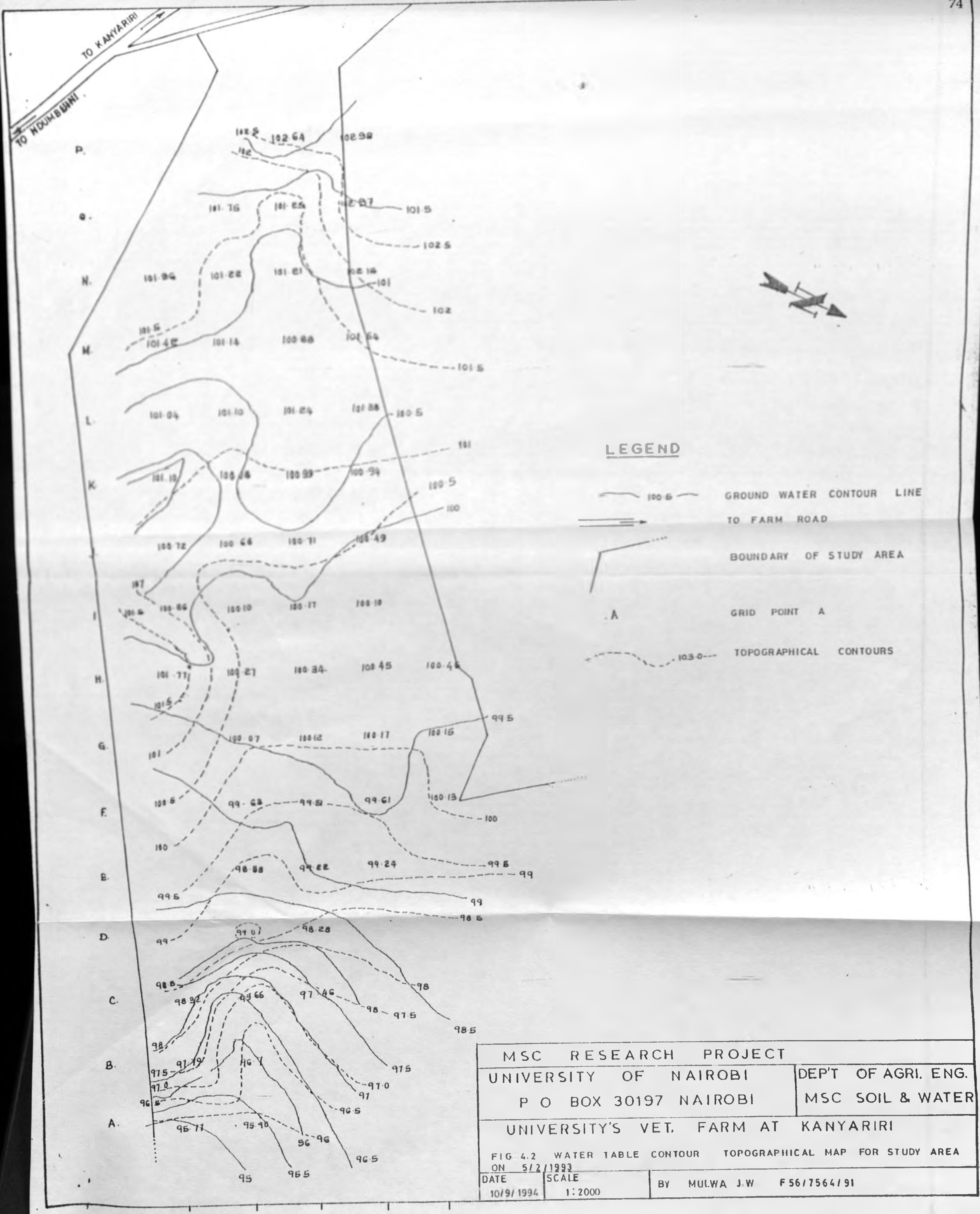
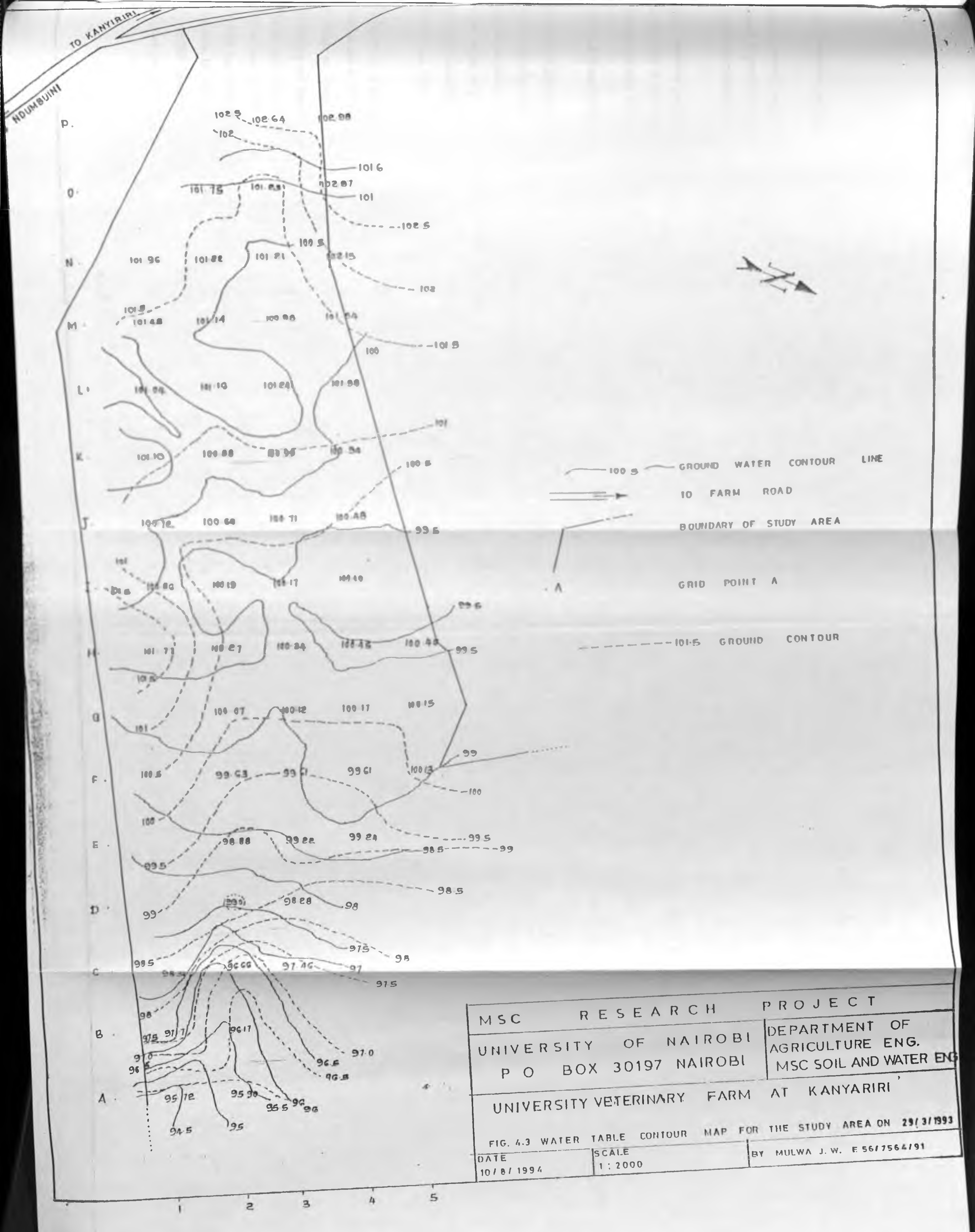


FIG 4.2 WATER TABLE CONTOUR TOPOGRAPHICAL MAP FOR STUDY AREA ON 5/2/1993



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DEPARTMENT OF AGRICULTURE ENG.
MSC SOIL AND WATER ENG

UNIVERSITY VETERINARY FARM AT KANYARIRI

FIG. 4.3 WATER TABLE CONTOUR MAP FOR THE STUDY AREA ON 29/3/1993

DATE 10/8/1994

SCALE 1 : 2000

BY MULWA J. W. F 56/7564/91

4.2.3 Drainable porosity.

Drainable porosity was determined from two empirical methods ; calculation from equation (46) discussed in previous chapters and from graphical determination using the relationships between the required parameter and hydraulic conductivity as depicted in figure 2.1 of chapter 2.

Thus the drainable porosity ranged from 0.6% to 21% with an average of 3% from the calculation and 1.8% to 20 % from the chart method.
An average value of 3% was taken as the average drainable porosity for the area.

4.2.4 Soil Water Intake rate.

From the soil infiltration test data, Shown in Appendix III, Cumulative infiltration depth, I_{cum} (cm) and the infiltration time t (Min.) were plotted on normal graph and the equation of best fit determined as shown in figure 4.4 below. The equation of the best fit was obtained as $I_{cum} = at^n$, with the values of a and n being approximately 0.24 and 0.78 respectively. The equation was further differentiated with respect to time and the equation for the instantaneous infiltration rate was obtained as $I_{rate} = an\ t^{n-1}$ (mm/hr).

From figure 4.4, it can be seen that the infiltration reaches equilibrium after 2.27 hours. This short duration is due to the clayey nature of the soils. The final or basic infiltration rate was obtained as 0.292 mm/hour. These values were also obtained by use of data presented in table 4.4 below.
By comparing the basic infiltration rate of the soils from adjacent catchment as presented in table 4.4 below, and rainfall intensities, surface runoff resulting from excess of rainfall rate over infiltration rate of the soil to the poorly drained area was considered to be negligible. This observation was further justified by the fact that a rainfall value which caused the watertable to rise dramatically, had surface runoff being recorded as zero in the Gerlach troughs. Thus showing most of the rainfall falling in the adjacent catchment infiltrated and possibly appeared as lateral seepage to the ground watertable and surface runoff.

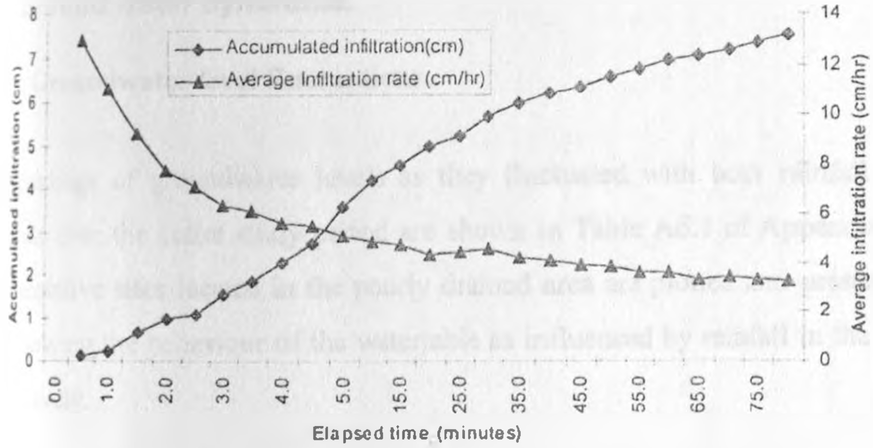

Figure 4.4 **Cummulative and Instantaneous infiltration as a function of time.**

Table 4.4 **Infiltration constants as obtained from regression analysis of the field data**

Location	a	n	r	R^2
Pit 1 (trial 1)	0.238	0.774	0.991	0.983
Pit 1 (trial 2)	0.853	0.662	0.997	0.995
Pit 2	1.037	0.842	0.997	0.994
pit 3	0.303	0.774	0.998	0.995
From adjacent catchment				
Test 1	0.674	1.001	0.992	0.984
Test 2	0.263	0.759	0.956	0.913

where a and n are the Kostiakov's constants in the equation $I_{cum} = at^n$ and I_{cum} =cumulative infiltration (cm) ; t is time taken for water to infiltrate (sec); r is coefficient of linear correlation and R^2 is the coefficient of determination.

4.3 Ground water dynamics.

4.3.1 Groundwater level fluctuations.

The readings of groundwater levels as they fluctuated with both rainfall and time over 4-day intervals over the entire study period are shown in Table A6.1 of Appendix 6. The data from 18 representative sites located in the poorly drained area are plotted and presented in figures 4.5 and 4.6, showing the behaviour of the watertable as influenced by rainfall in the peat soil and clay soil respectively.

As it is observed from figures 4.5 and 4.6 below, Rainfall had a marked influence on the dynamics of ground water flow below the poorly drained area. Rainfall contributed both to the groundwater recharge and to the surface reservoir and thus affected the drainage of the area significantly.
The ground water level fluctuated from 6 cm above the ground surface in February to below 1.5 m in June in most of the observation wells which indicated that the area does not only require drainage, but also requires sub-irrigation during the periods when the watertable level is lower than 1.5 m below the ground surface.

The two figures below, indicate that rainfall had remarkable influence over the watertable level fluctuation and hence a greater contributor to the drainage problem as all wells responded to rainfall by either rising water table after rainfall event or gradual decrease when rainfall ceases. The control of excess rainfall would be a major solution to the drainage problem of the area under investigation.

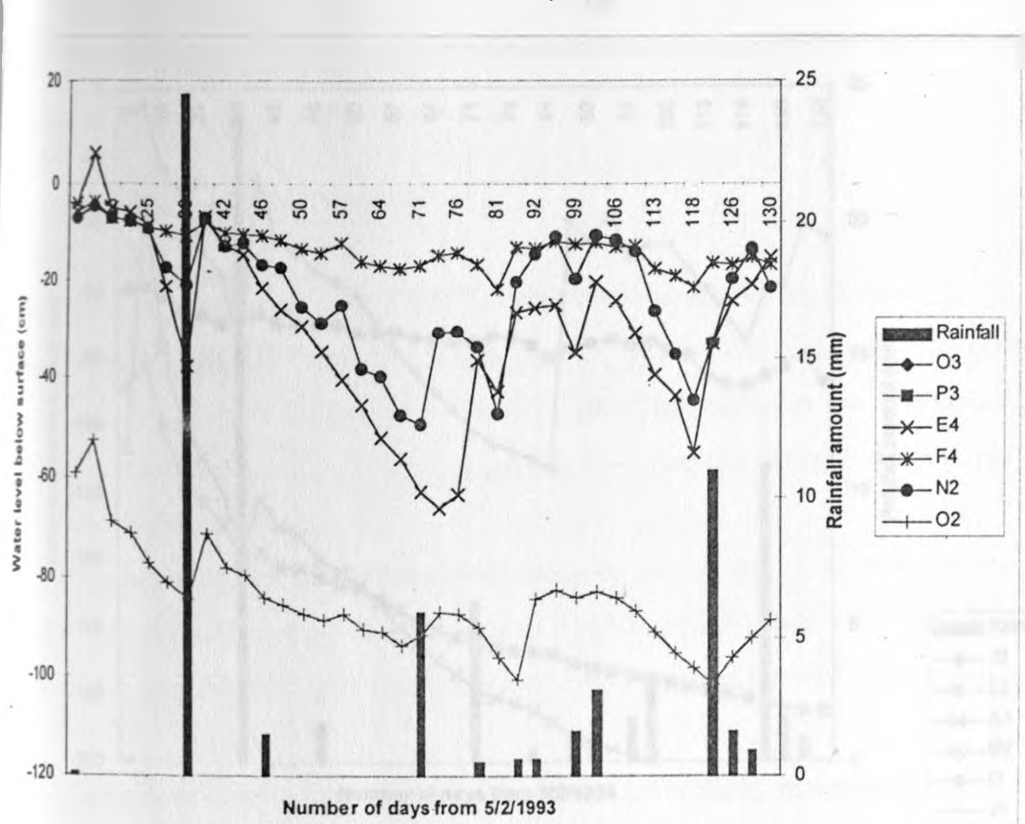

Figure 4.5 Watertable fluctuation behaviour as influenced by rainfall(Reeds dominated site).

4.3.2 Watertable level data analysis.

Correlation between rainfall and ground watertable rise and falls was observed to be existing. This correlation was confirmed by physical observation as the water levels were found to respond not significantly to any rainfall amount that fell to some extent especially the observation wells in the upper part of the study area or those which were in areas inhabited by reeds. This led to the suggestion that, some areas were recharge points, that is where water was entering the area and others discharge points, that is where the water was leaving the area under consideration.

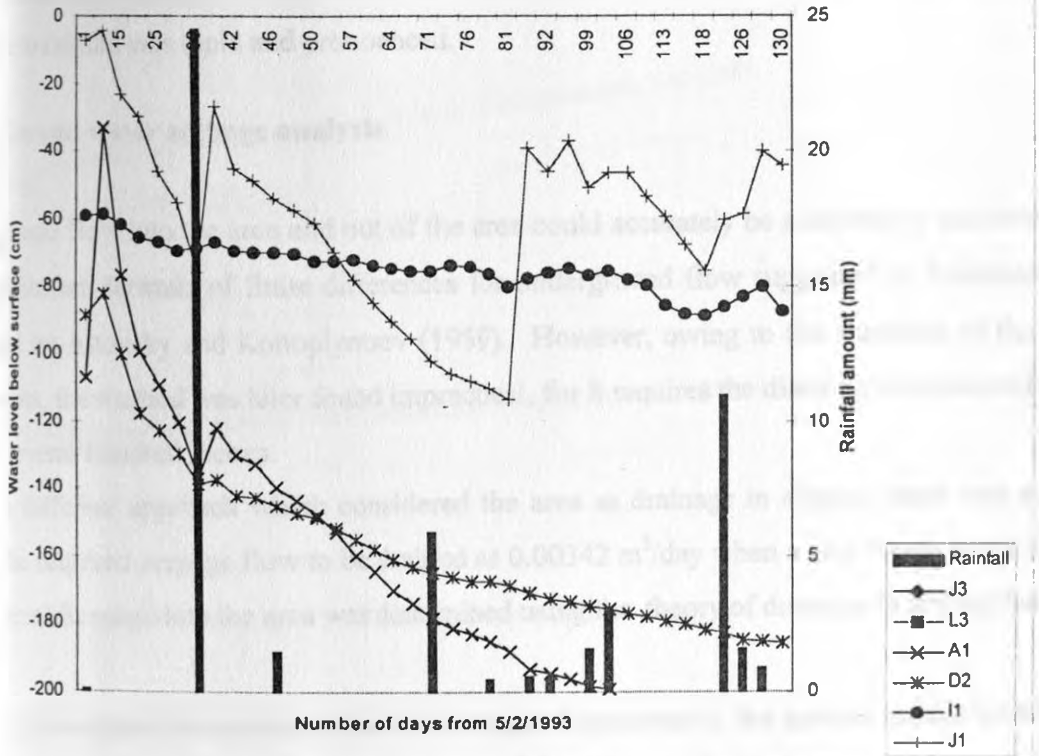

Figure 4.6 Watertable fluctuation behaviour as influenced by rainfall (Wire grass dominated site).

However, the lag in response to rainfall events was not uniform in all observation wells. This was attributed to differences in water transmission properties of the soils in the study area resulting from the soil's heterogeneity as evident from the wide range of hydraulic conductivity values obtained and the various values of the same property as investigated down the profile.

Thus, those wells which showed little or no sharp response to rainfall were assumed to have fissures or cracks where water would find a way out whenever recharge occurred. The presence of fissures and cracks was also exemplified by the value of Kostiakov's, "n" in the infiltration equations of about 0.8 as found in the previous section.

The water levels in virtually all the observation wells never remained constant, except for wells N2, O3, F4, and E4 which showed fairly slow response to evaporation and lateral discharge. The areas

...re considered seepage sites as unlike the evaporation where their response was small, their ...ponse to rainfall was rapid and pronounced.

...3 Ground water seepage analysis.

...derground flow into the area and out of the area could accurately be analyzed by the method of ...n-equilibrium formula of finite differences for underground flow suggested by Kamensky and ...scribed by Altovsky and Konoplyntsev (1959). However, owing to the shortness of the farms ...mensions, the method was later found impractical, for it requires the distance between wells to be ...least several hundred metres.

...ence a different approach which considered the area as drainage in sloping lands was adopted ...ving the required seepage flow to be drained as 0.00342 m^3/day when a unit length is considered. The Lateral Seepage into the area was determined using the theory of drainage in sloping lands.

The impervious layer is assumed to have an average slope same as the ground surface towards the ...ottom land. The average hydraulic conductivity is 0.06 mm/day. The interceptor is to maintain the ...atertable at 0.5 m below the soil surface, to avoid adverse effects of the soil as a result of "poaching" by animal hooves. The overall situation is schematically presented in figure 4.7 below; ...hus, the inflow into the area from upstream, q is given as q_1 and outflow as q_2 respectively. Hence;

$$q_1 = K_1 H \tan\alpha_1 \quad and \quad q_2 = K_2 H \tan\alpha_2$$

where	K_1 =	Hydraulic Conductivity of adjacent grounds = 0.05 m/day
	H_1 =	Thickness of the draining layer (obtained from bore hole information in the catchment) = 6 m
	$\tan\alpha_1$=	General slope of the ground surface towards the bottom land = 0.015 or (1.5%)

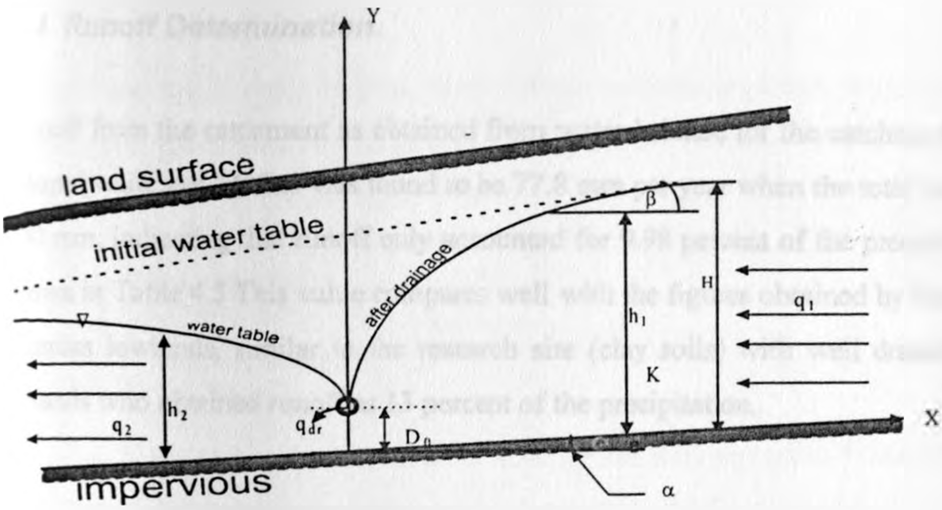

Figure 4.7 Lateral seepage from the sloping higher grounds

K_2 =	Hydraulic conductivity of the soils in the poorly drained area as obtained from experiment = 0.06 m/day
H_2 =	Depth of the area to be drained (deepest wells in the poorly drained area) = 1.5 m
$Tan\alpha_2$ =	General slope of the ground surface in the poorly drained area = 0.012 or (1.2%).

Thus;

$q_1 = 0.05 * 6 * 0.015$ m^2/day and $q_2 = 0.06 * 1.5 * 0.012$ m^2/day. This gives $4.5 * 10^{-3}$ m^2/day and $1.08 * 10^{-3}$ m^2/day respectively.

Hence discharge to be intercepted by interceptor drain is the difference of the two quantities and is given by, $q_1 - q_2 = q_{drain}$

which amounts to $4.5 * 10^{-3} - 1.08 * 10\text{-}3$ m^2/day = $3.42 * 10^{-3}$ m^2/day and equal to 0.00342 m^2/day.

Therefore the Seepage rate is 0.00342 m^2/day or 0.00432m^3/day when a unit length is considered and Run-off is 78 mm per year.

4.4 Runoff Determination.

Runoff from the catchment as obtained from water balance for the catchment using the method of Thornthwaite and Mather was found to be 77.8 mm per year when the total yearly precipitation was 780 mm, indicating that runoff only accounted for 9.98 percent of the precipitation into the area as shown in Table 4.5 This value compares well with the figures obtained by Home(1991) working in Mumias lowlands, similar to the research site (clay soils) with well drained soils in the higher grounds who obtained runoff as 13 percent of the precipitation.

Using the rational formula the peak runoff rate to the bottomland was found to be 0.25 m^3/day, such a figure coming into an area of 6 ha would just translate into only 1.5 mm and hence direct runoff is not the major cause of the problem.

4.5 Rainfall analysis.

4.5.1 Daily Rainfall Analysis.

Table 4.6. below shows the maximum rainfall figures for 1-day up to 11-days maximum successive rainfall totals for the month of May for 23 years (1971-1993). The table also includes the expected mean, (Yn) and the expected standard deviation (Sn) of the reduced variate for a sample size of 23. Similar data for the months of April and June are shown in Tables A4.3 and A4.4 Respectively in Appendix 4

The data was assumed to fit Extreme Value Type I Gumbel distribution as rainfall totals are found to obey the same in literature. Expected means (Yn) and standard deviations (Sn) of the reduced extremes for the sample size of 23 were also determined and included in each of the respective tables.

The estimation was based on the expected mean Yn = 0.5276 and expected standard deviation Sn = 1.076 of the reduced extremes of a sample size 23.

Table 4.5 Monthly water balance for University of Nairobi Veterinary farm at Kanyariri, for a clay soil with available water capacity of 200 mm per metre and rooting depth of 0.7 m.

Param.	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Prec., P	60.0	63.0	92	205	177	42	27	23	39	56	133.0	68	780.0
PET	136.4	128.8	142.6	114.0	99.2	81.0	80.6	86.8	108.0	127.1	111.0	114.7	1334
P-PET	-76.4	-65.8	-50.6	91	77.8	-39.0	-53.6	-63.8	-69.0	-71.1	22.0	-46.7	-345.2
Acc Pot WL	-123.1	-188.9	-239.5	0	0	-39.0	-92.6	-156.4	-225.4	-296.5	0	-46.7	
SM	129.0	80	70	200	200	166	143	95	65	45.0	200.0	185.0	
ΔSM	-56.0	-49	-10	130	0	-34	-23	-15	-30	-20	155.0	-15.0	
AET	116.0	112	102	114	99.2	76	50	38	69	76	111.0	83.0	1079.2
DEF	20.4	16.8	40.6	0	0	5	30.6	48.8	39.0	51.1	0	31.7	284.0
Surplus	0	0	0	0	77.8	0	0	0	0	0	0	0	77.8
Tot.av. RO	0	0	0	0	77.8	15.6	3.1	0.6	0.1	0	0	0	
RO	0	0	0	0	62.2	12.5	2.5	0.5	0.1	0	0	0	77.8
Detention	0	0	0	0	15.6	3.1	0.6	0.1	0	0	0	0	

Table 4.6 Maximum successive rainfall totals for durations 1 to 11 days for Kabete field station for the Month of May during the period 1971-1993.

	Duration of successive rainfall										
Year	1-day	2-day	3-day	4-day	5-day	6-day	7-day	8-day	9-day	10-day	11-day
1971	55.7	75.9	89.0	97.2	116.5	134.3	147.4	152.4	174.9	188.8	191.3
1972	26.6	49.6	58.4	58.6	60.7	69.0	75.9	98.5	99.3	107.2	108.1
1973	18.6	19.0	34.0	34.2	34.2	36.2	36.2	38.5	38.7	38.7	40.7
1974	43.5	69.8	88.9	92.8	98.6	108.1	112.0	112.0	112.0	117.5	119.5
1976	37.7	44.0	79.0	82.1	82.1	82.1	89.5	98.8	104.7	104.7	105.9
1977	77.2	101.7	113.1	116.1	120.9	124.9	141.7	176.6	202.6	211.6	211.6
1978	17.0	17.0	18.3	19.2	19.2	23.3	30.1	31.7	31.7	35.8	37.5
1979	38.5	50.8	65.0	80.1	80.1	92.0	100.2	101.7	102.6	103.5	103.6
1980	79.6	120.6	152.8	169.7	190.6	220.0	223.9	228.0	229.4	230.1	242.9
1981	58.7	72.2	90.7	97.6	97.7	101.0	117.1	163.0	169.9	170.6	175.6
1982	61.4	68.1	70.2	75.8	75.9	77.6	101.0	107.7	111.5	115.0	121.7
1983	11.4	11.8	12.1	16.7	17.1	18.0	21.2	25.4	27.3	27.4	30.7
1984	4.1	4.1	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2
1985	15.8	16.2	31.5	32.4	33.8	34.6	35.3	37.4	46.8	46.9	46.9
1986	87.7	154.8	160.2	161.1	167.9	173.3	177.6	179.4	179.8	179.8	180.9
1987	73.5	74.4	74.4	76.6	76.6	76.6	78.0	95.4	106.2	121.5	121.5
1988	53.6	95.4	122.8	137.4	137.4	142.4	152.5	178.4	178.4	183.4	190.4
1989	158.2	169.7	169.7	176.4	205.6	254.5	273.6	309.2	311.2	355.5	371.5
1990	90.0	97.0	122.3	142.0	149.8	174.3	182.1	204.5	206.9	206.9	206.9
1991	45.7	85.9	109.2	115.5	124.7	129.1	145.9	165.4	185.3	188.5	196.8
1992	133.4	161.3	170.2	171.0	171.0	171.0	171.0	171.0	171.0	172.1	172.1
1993	17.3	17.9	22.0	22.0	22.5	23.1	24.3	31.9	31.9	31.9	32.7
Est(α)	0.027	0.022	0.021	0.020	0.018	0.016	0.015	0.014	0.014	0.013	0.012
Est(μ)	35.6	47.6	59.2	63.3	65.8	69.9	76.2	85.3	89.8	92.2	94.0
Mean	54.8	71.7	84.5	89.9	94.9	103.2	110.9	123.2	128.5	133.7	137.0
Std	39.3	49.2	51.7	54.4	59.5	67.9	70.8	77.3	79.1	84.8	87.7

The Gumbel distribution which uses the above information can be transformed into its linear form by being presented as follows

$$y = \alpha (p - \mu) \tag{57}$$

Where estimates for α and μ are given respectively by:

$$Est(\alpha) = Sn/Sp \text{ and } Est(\mu) = Pn - Yn/Est(a) \tag{58}$$

Sp = Standard deviation of the sample.

Pn= Mean of the sample.

Yn= expected mean of the reduced extremes as a function of sample size= 0.527 for a sample of size =23 and

Sn =expected standard deviation of the reduced extremes as a function of sample size = 1.076 for a sample size = 23.

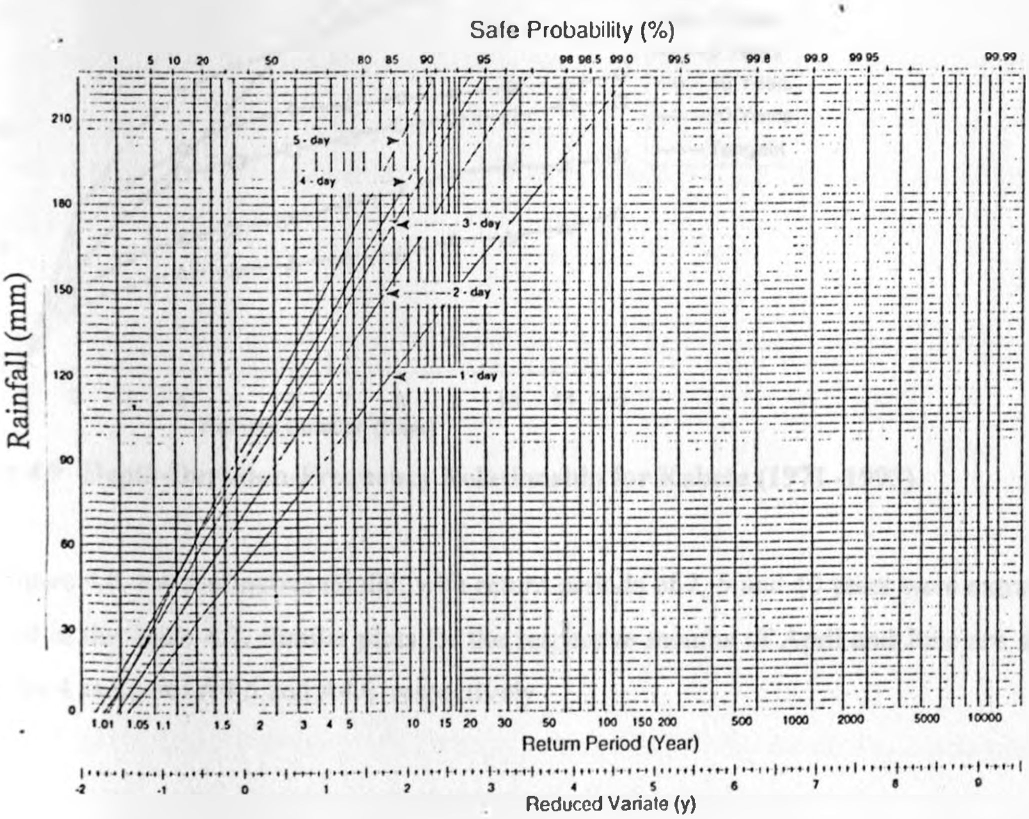

Figure 4.8 Continuous rainfall for duration 1-5 days for May 1971 to 1993.

A plot of the linear equation for the month of May which had the highest coefficient on a Gumbel paper is shown in figure 4.8 above from which rainfall of durations 1-5 days with return periods of 2, 3, 5, 10 and 20 years were extracted and plotted in figure 4.9. It is from there that the design rainfall of 5 successive days with return period of 5 years was obtained as 54.9 mm/day.

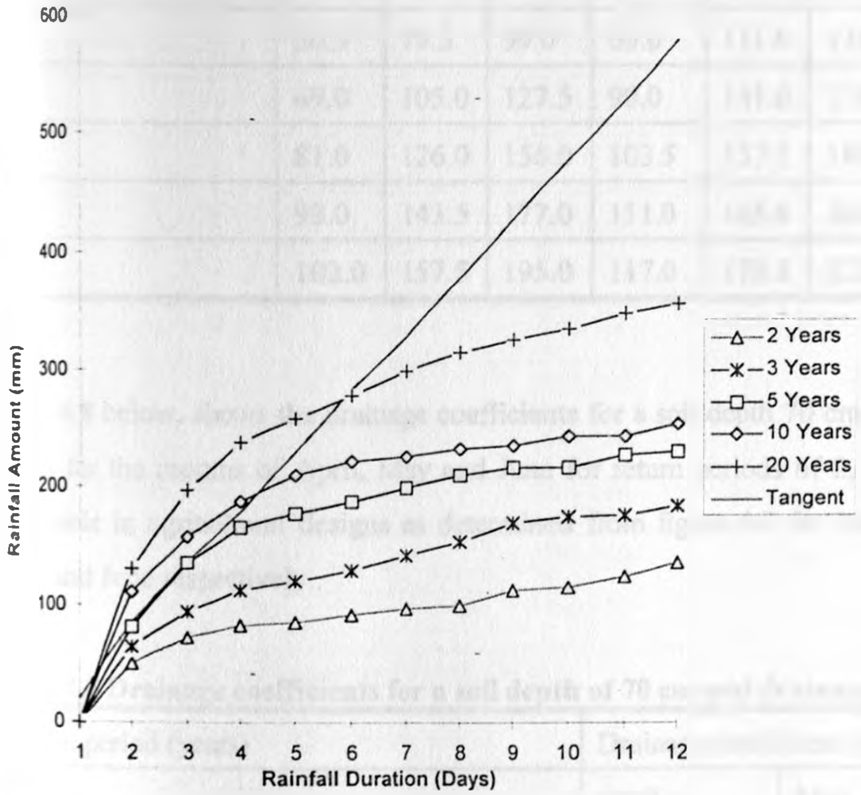

Figure 4.9 Depth-Duration-Frequency Relationship for Kabete (1971 -1993)

From figure 4.8, the continuous rainfall with return periods of 2, 5 and 10 years were extracted and tabulated in the Table 4.7. Similar plots for the respective months of April and June are shown in Appendix 4 as figures A4.5 and A4.6 respectively

Table 4.7 The maximum continuous rainfall amounts for April, May and June.

Duration of maximum continuous Rainfall (days).	Return Period (years).								
	APRIL			MAY			JUNE		
	2	5	10	2	5	10	2	5	10
1	50.3	79.5	99.0	69.0	111.0	138.0	24.0	46.0	61.0
2	69.0	105.0	127.5	90.0	141.0	174.0	29.0	52.0	69.0
3	81.0	126.0	156.0	103.5	157.5	192.5	31.0	56.0	73.0
4	93.0	143.5	177.0	111.0	165.0	204.0	31.0	56.0	73.5
5	102.0	157.5	195.0	117.0	178.5	222.0	32.0	58.0	76.0

Table 4.8 below, shows the drainage coefficients for a soil depth 70 cm and drainage porosity (Θ) 2.4 % for the months of April, May and June for return periods of 2, 5 and 10 years which are applicable in agricultural designs as determined from figure 4.9 for May and similar figures for April and June respectively.

Table 4.8 Drainage coefficients for a soil depth of 70 cm and drainage porosity 2.4 %.

Return period (years)	Drainage coefficient (mm/day).		
	April	May	June
2	25.8	27.1	-
5	47.2	54.9	8.6
10	73.6	90.9	17.6

Since May had the highest drainage coefficients as seen from Table 4.8 above, it was chosen for design purposes. A period of 5 years was chosen for the high valued pasture.

The Table 4.9 below, shows the expected means (Yn) and the standard deviation (Sn) of the reduced extremes as a function of sample size (n). The table was used to linearly interpolate the values to obtain the expected mean and the expected standard deviation of sample of size 23 using the Langrage method.

Table 4.9 Expected mean(Yn) and Standard deviation (Sn) of a reduced extremes as a function of sample size (n).

sample Size (n)	Standard deviation (Sn)	Mean (Yn)
10	0.950	0.495
15	1.021	0.513
20	1.063	0.524
25	1.092	0.531
30	1.112	0.536
40	1.141	0.544
50	1.161	0.548
70	1.185	0.555
100	1.206	0.560
∞	1.282	0.577

(after Kessler and Raad,1980)

4.5.2 Rainfall Storm data analysis.

Table A7.1 shows the statistical parameters of the storm data, mean and standard deviation for rainfall storms obtained from Kabete University Field weather station.

Considering rainfall storms as extreme events, Extreme Value type 1 Gumbel distribution was used in conjunction with the above discussed parameters to obtain Table 4.10 below, which shows the storm intensities for durations of 15 minutes to 180 minutes as functions of their return periods. These storm data was further transformed into storm intensities and expressed in mm/hr for comparison purposes as shown in table 4.11 below.

Table 4.10 Storm intensities as a function of return period and duration.

T (Years)	Storm Duration (Minutes)												
	15	30	45	60	75	90	105	120	135	150	180	240	300
2	2.10	3.63	4.95	6.13	7.21	8.20	9.12	9.99	10.81	11.58	13.00	15.45	17.49
5	4.8	8.22	11.14	13.73	16.07	18.22	20.20	22.08	23.76	25.37	28.31	33.25	37.20
10	6.6	11.26	15.25	18.79	21.98	24.90	27.60	31.10	32.43	34.62	38.59	45.25	50.54
15	7.6	12.97	17.54	21.59	25.24	28.57	31.64	34.49	37.14	39.62	44.11	51.60	57.51
20	8.3	14.17	19.16	23.58	27.56	31.19	34.54	37.64	40.53	43.22	48.11	56.25	62.66
25	8.8	15.09	20.41	25.11	29.35	33.21	36.77	40.07	43.14	46.00	51.20	59.85	66.65
50	10.5	17.88	24.11	29.59	34.51	38.99	43.08	46.87	50.37	53.62	59.49	69.09	76.45
100	12.1	20.68	27.86	34.16	39.81	44.93	49.62	53.94	57.94	61.64	68.30	79.12	87.33

Table 4.11 Storm intensities X_T mm/hr.

T (Years)	Storm Duration (Minutes)												
	15	30	45	60	75	90	105	120	135	150	180	240	300
2	8.40	7.26	6.60	6.13	5.76	5.47	5.21	5.00	4.80	4.63	4.33	3.86	3.35
5	19.15	16.44	14.85	13.73	12.86	12.14	11.54	11.02	10.56	10.15	9.44	8.31	7.44
10	26.26	22.53	20.34	18.79	17.58	16.60	15.77	15.05	14.41	13.85	12.86	11.31	10.11
15	30.28	25.93	23.39	21.59	20.19	19.05	18.08	17.25	16.51	15.85	14.70	12.90	11.50
20	33.09	28.33	25.55	23.58	22.04	20.79	19.74	18.82	18.01	17.29	16.04	14.06	12.53
25	35.25	30.18	27.21	25.11	23.47	22.14	21.01	20.03	19.17	18.40	17.07	14.96	13.33
50	41.92	35.75	32.15	29.59	27.61	25.99	24.62	23.43	22.39	21.45	19.83	17.27	15.29
100	48.54	41.35	37.14	34.16	31.85	29.95	28.36	26.97	25.75	24.66	22.77	19.78	17.47

For each return period, the logarithmically transformed data of storm duration and intensity were regressed and curves whose equation are represented in table 4.12 below were obtained. The given equations were plotted in figure 4.10.

Table 4.12 Indices for the intensity - duration frequency relationship from linear regression.

Return Period (yrs).	Equation	R^2
2	$i = 5.78\ t^{-0.269}$	0.731
5	$i = 9.15\ t^{-0.297}$	0.723
10	$i = 12.38\ t^{-0.306}$	0.716
20	$i = 15.32\ t^{-0.312}$	0.714

Figure 4.10 below shows the relationships between the intensity- duration and frequency of a storm and the chart was used to obtain the design intensities for the determination of peak runoff rates from the catchment.

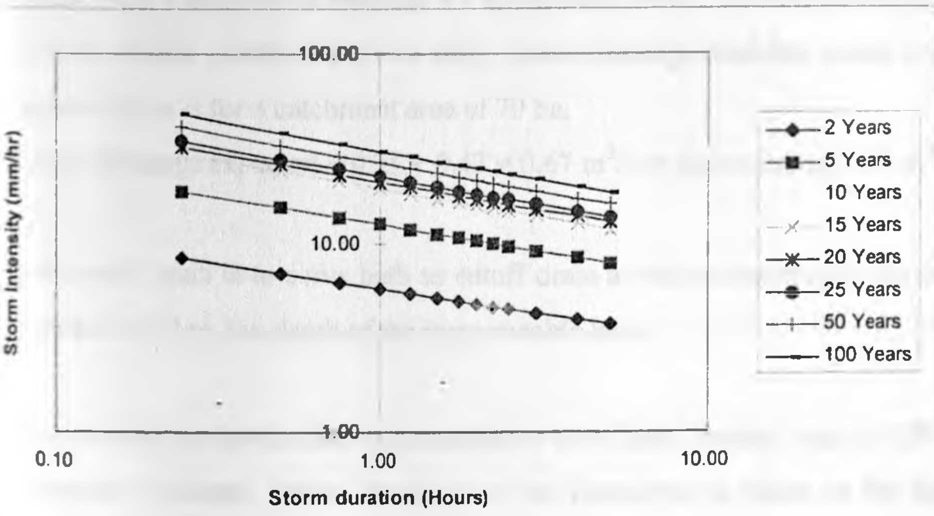

Figure 4.10 Intensity-Duration _Frequency chart for the study area.

5. DRAINAGE SYSTEM DESIGN PROPOSALS.

5.1 *Proposed surface Drainage system.*

A cutoff drain is proposed at the uppermost part of the bottomland to cater for both overland flow and seepage from adjacent sloping lands.

From rainfall intensities calculated earlier, and a return period of five years for less sensitive crops such as grass, the surface runoff towards the bottomland is 0.25 m^3/s. The contribution from rainfall analysis for a soil of depth 70 cm and porosity 3 percent, maximum rainfall storage would be 700 X 3/100 = 21 mm

With the obtained storage and Depth- Duration -Frequency curves, given before, a drainage coefficient of 54.9 mm/day is obtained for the critical month. The evapotranspiration during May, which is the critical month is 3.2 mm /day hence drainage modulus would be 54.9 - 3.2 = 51.7 mm/day or 0.42 m^3/s for a catchment area of 70 ha.

Total Peak discharge expected = 0.25 + 0.42 = 0.67 m^3/s or approximately 0.7 m^3/s.

Since the cutoff drain is to serve both as cutoff drain as well as interceptor for subsurface flow, Its depth would be 1.2 m, the depth of the impermeable layer.

The ground slope perpendicular to groundwater flow from contour map is 0.99 percent which is approximately 1 percent. Hence the slope of the interceptor is taken as the slope of the natural ground slope in the direction of the interceptor and thus one percent or 0.01 and this will also be the slope of all laterals which run parallel to this interceptor.

5.1.1 The design.

The design of the cross- section of the channel to convey the damaging flows is presented below, and the resulting channel being presented in figure 5.1 below.

The quantity of peak discharge rate, q = 0.7 m^3/s as obtained from previous section. The channel that can deliver this quantity safely and at a permissible velocity will have dimensions as given from the application of Mannings' formular as follows:

$$q = KAR^{2/3}S^{1/2} \tag{59}$$

Where q is the discharge capacity (m^3)

K is the Mannings' roughness coefficient of the proposed channel (-)

A is area of the cross-section of the channel (m^2)

R is the hydraulic mean radius of the channel cross-section (A/P) (m)

P is the wetted perimeter of the channel (m)

With assumed Side slopes = 1:1.5 depending on the soil type (Clay), K=10 and S=0.01 (approximately the slope of the natural ground surface towards the direction of the cut-off drain) and b/d ratio of 1, and application of the above formula, the dimensions are as follows:-

Depth of flow, d = 0.5 m, base, b = 0.5 m, top width,T = 2.0 m

The wetted perimeter, P = 2.303 m

and channel cross-sectional area , A = 1.15 m^2

Hydraulic radius, R =A/P = 0.499

using Manning's equation, with K = 10 and S = 0.01,

a velocity of 0.63 m/s is obtained which is safe for stiff clay soils as given in table A8.2 (Appendix 8).

A cross-section of the Interceptor drain is shown in figure 5.1 below.

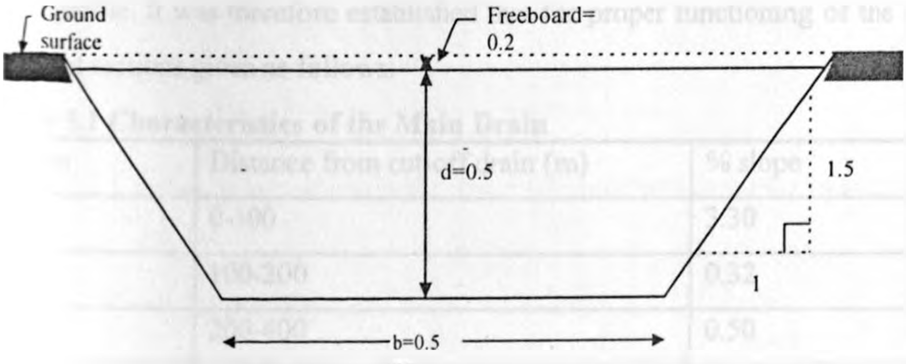

Figure 5.1 Cross-section of the Interceptor drain (measurements in metres and not to scale)

5.1.2 Design of the Laterals.

The capacity of the lateral must equal the ground water intercepted from the area served by that lateral, which acts as any interceptor drain for the small area it is preventing from waterlogging. The area being protected by the lateral from waterlogging is given by; $L * S + S^2/2$, with L being the length of the lateral and S being the lateral spacing (m).

Thus in our case, L =100 m, S = 7.5 m and hence area served by the drain is 778.1 m^2

With an assumed drainage coefficient for humid areas of 10 mm/day (USDA/SCS,1973), the discharge per lateral is 0.09 l/s.

With 166 laterals, the total capacity from lateral flow is 14.95 l/s or 0.015 m^3/s.

5.1.3 Main drain design.

The main drain which have to carry all the drainage water from the interceptor and all lateral flows from lateral drains and deliver it to the outlet at the reservoir must have a capacity of (0.7 m^3/s (Interceptor drain) + 0.015 m^3/s (lateral drains)) 0.715 m^3/s to perform its work. The average length would be 620 m, the lateral length of the poorly drained area.

The average slope down the bottomland, that is, parallel to ground water flow will vary depending on the ground surface slope which was found to vary from one place to the other along the entire drain profile. It was therefore established that for proper functioning of the drain, it will have five different sections given as follows:

Table 5.1 Characteristics of the Main Drain

Section	Distance from cut-off drain (m)	% slope
I	0-100	3.30
II	100-200	0.32
III	200-400	0.50
IV	400-500	1.06
V	500-620	1.66

For efficient deliverly of water to the outlet, the drain was designed to have two drop structures of 0.2 m each and placed at 300 m and 450 m from the cut-off drain respectively. The slope of the main drain would be taken as average of the natural ground slope and hence s = 0.012 .

Capacity of main drain = 0.715 m^3/s or 0.72 m^3/s

Slope, s =0.012

With assumed Side slopes = 1:1.5 depending on the soil type (Clay), K=10 and S=0.012 (approximately the slope of the natural ground surface towards the outlet) and b/d ratio of 1, and application of the Manning's formula as outlined in the previous section, the dimensions and other characteristics of the designed channel are as follows:-

Depth of flow, d = 0.45 m, base, b = 0.6 m, top width, T = 1.95 m, The wetted perimeter, P = 2.22 m, channel cross-sectional area , A = 1.08 m^2 Hydraulic radius, R =A/P = 0.486 and velocity of flow of 0.68 m/s which is safe for stiff clay soils. These dimensions allows for a maximum discharge of 0.73 m^3/s slightly more than the designed flow.

The cross-section of the channel is shown in figure 5.2 below.

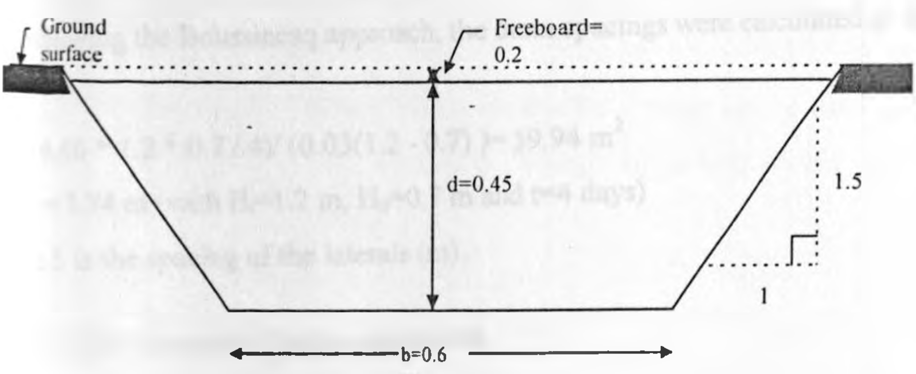

Figure 5.2 Cross-section of the main drain (dimensions in metres not to scale)

5.2 *The proposed subsurface drainage system.*

The soils are stiff clay with low hydraulic conductivity and hence shallow subsurface drainage is recommended.

Soil depth = 70 cm

Drainable porosity = 3%

Hydraulic conductivity = 0.06 m/day

Assumptions:

Interceptor has trapped all seepage from upslope and all overland flow towards the poorly drained area.

The rootzone should not remain saturated for more than 4 days; watertable should fall by 0.5 m during the same period (as suggested by Smedema and Rycroft,1983).

The drain base will coincide with the impervious layer.

5.2.1 Boussinesq approach.

By employing the Boussinesq approach, the drain spacings were calculated as follows:

$S^2 = (4.46 * 1.2 * 0.7 / 4)/ (0.03(1.2 - 0.7)) = 59.94\ m^2$

or $S = 7.74$ m (with H_t=1.2 m, H_0=0.7 m and t=4 days)

Where S is the spacing of the laterals (m).

5.2.2 USDI Transient Design approach.

Approaching the same problem with the USDI method, the drain spacing were calculated as:

Drain depth below soil surface = 1.2 m

H= initial water table above drain =1.2 m (worst case when water is on the soil surface)

Final water table above drain, Z =0.7 (0.5 m for rootzone of grass).

Drain out period = 4 days

$Z/H = 0.7/1.2 = 0.583$

From figure 5.3 below, which gives the value of KHt/SL^2 of 0.18

With the values of K ,H,t and S being known, the value of L was obtained from :

$L^2 = (0.06 \times 1.2 \times 4)/(0.18 \times 0.03) = 53.3\ m^2$

$L = 7.3$ m

Since the values of the drain spacing from these formulae are comparable an average value was obtained. Thus average spacing = (7.74 +7.3)/2 = 7.5 m

The drain spacing of 7.5 m so obtained is too small to be economically feasible, based on FAO (1971) recommendations, thus drains of spacing less than 6 m or placed in soils with hydraulic conductivity less than 0.096 m/day will not be economically feasible. In such cases, mole drains at the same spacing of 7.5 m should be tried and also at double spacing with soil disruption since the formula used only gives an indication as to the magnitude and not the absolute values.

If the given spacing gives unfavourable results, the spacing can be tried in combination with pipe drainage. In all cases, soil disruption through subsoiling should be tried to increase soil permeability.

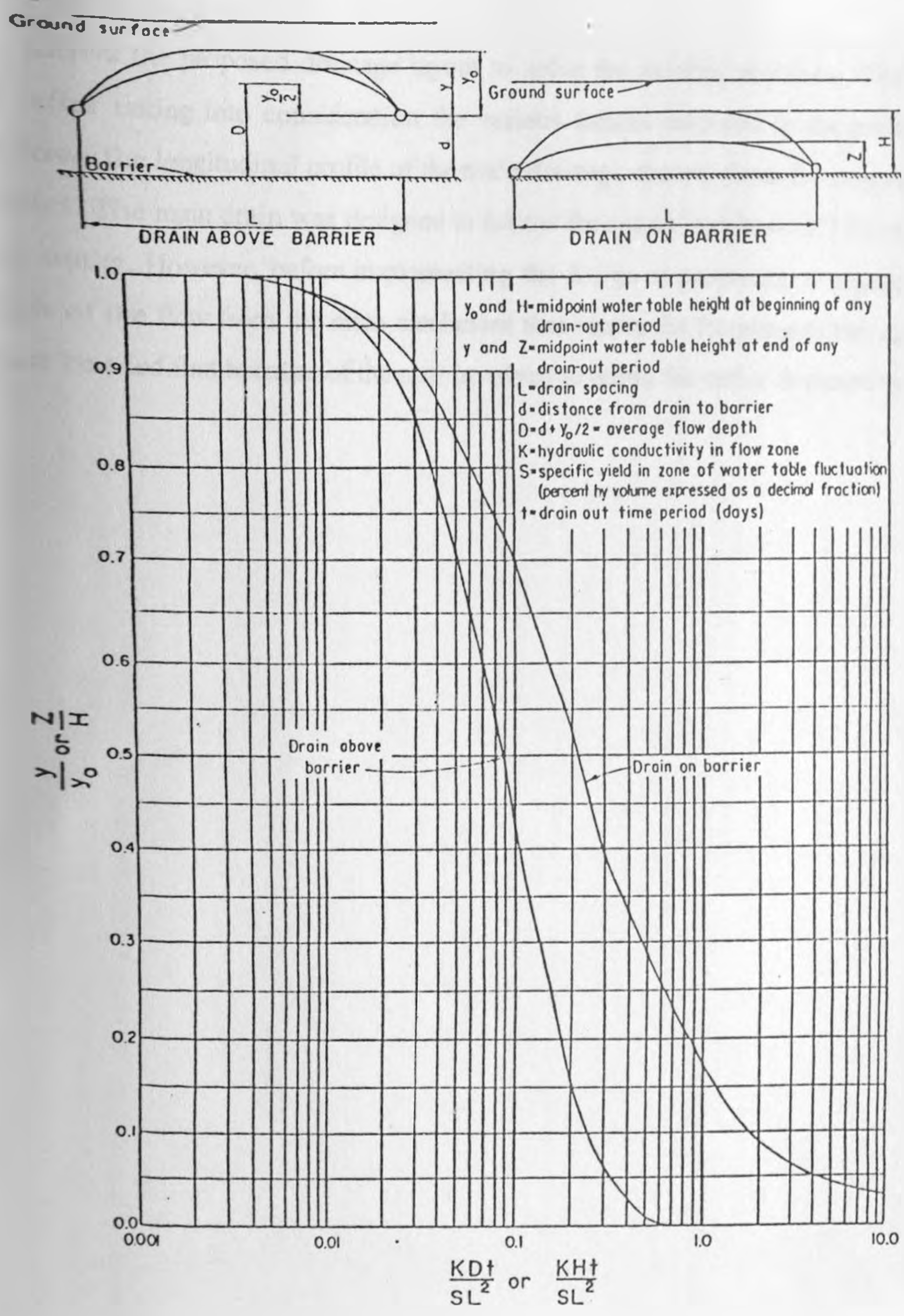

Figure 5.3 Curves showing relationship of parameters for drain spacing using transient flow theory.

5.3 *Proposed Drainage layout.*

Figure 5.4 below shows the proposed drainage layout to solve the existing problems. The design has been reached after taking into consideration the various factors involved in the problematic area. Figure 5.5 shows the longitudinal profile of the main drainage channel from the proposed cut-off drain to the outlet. The main drain was designed to follow the natural gradient (1.2 %) and thus drain at the shown outlet. However, before implementing the design as proposed, an experimental drain to relief much of the flow from the main catchment that causes the flooding of the upstream of the lowland must be tried first because of the cost involved in doing the entire drainage works as recommended.

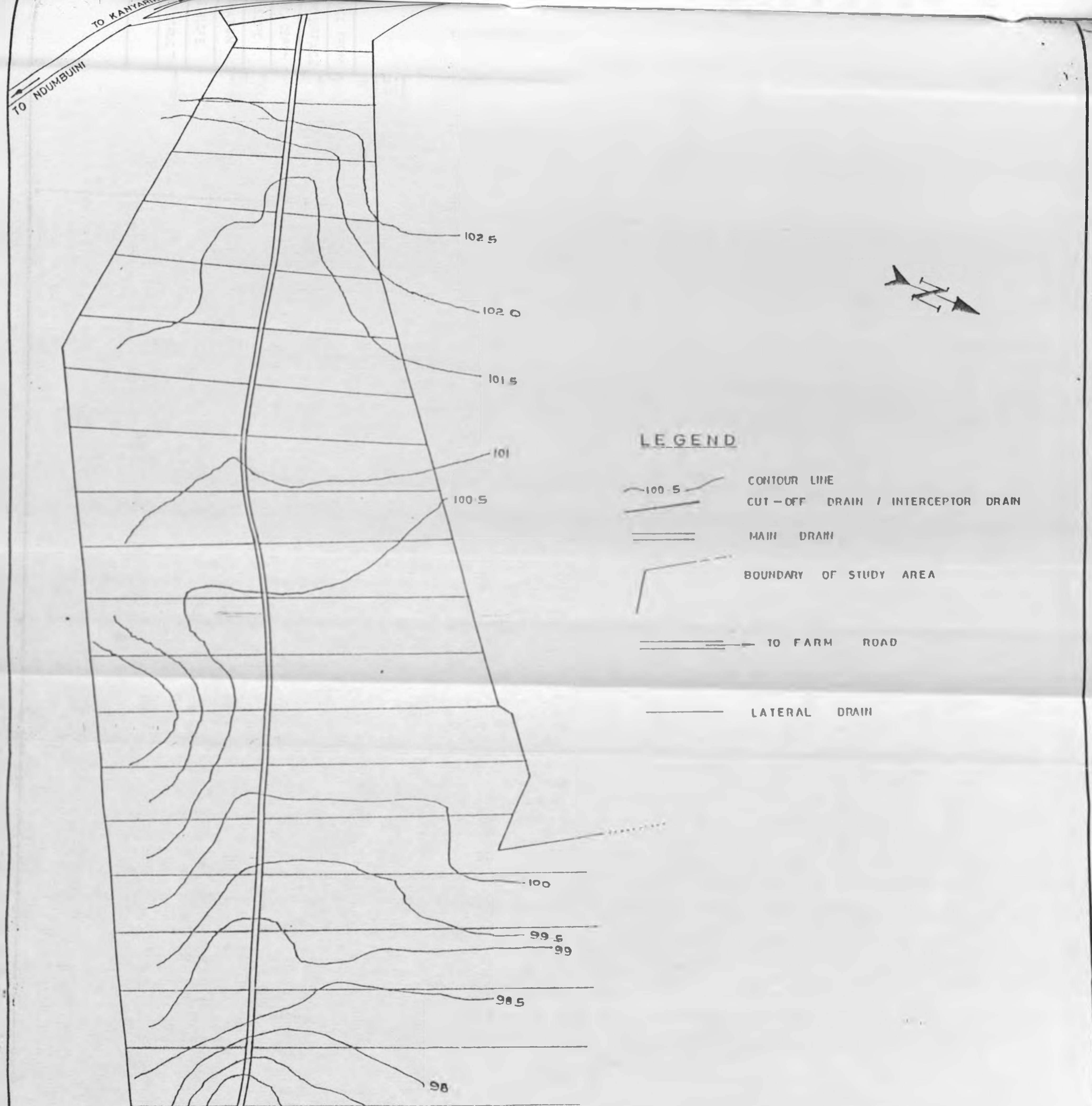

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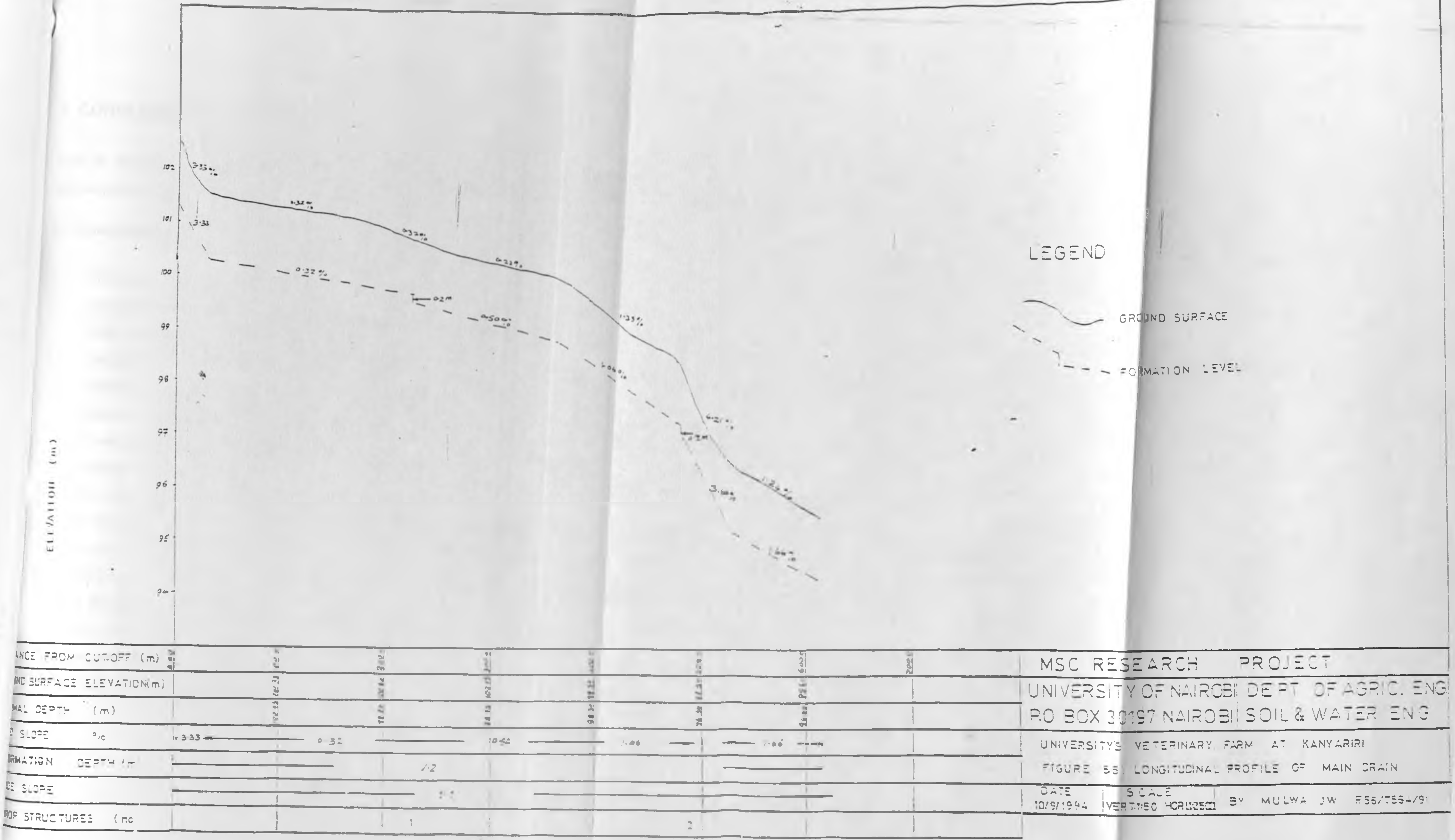
FIGURE 5.4 PROPOSED DRAINAGE LAYOUT MAP

DATE	SCALE	
10/9/1994	1:2000	BY MULWA J W F56/7564/91


ELEVATION (m)
LEGEND
GROUND SURFACE
FORMATION LEVEL
MSC RESEARCH PROJECT
UNIVERSITY OF NAIROBI DEPT OF AGRIC. ENG
P.O BOX 30197 NAIROBI SOIL & WATER ENG
UNIVERSITY'S VETERINARY FARM AT KANYARIRI
FIGURE 5.5: LONGITUDINAL PROFILE OF MAIN DRAIN
DATE 10/9/1994
SCALE VERT 1:50 HOR 1:2500
BY MULWA JW F56/7564/91

6. CONCLUSIONS AND RECOMMENDATIONS

From the results of the study, the following conclusions can be drawn and the following recommendations made. Also suggestions to the areas requiring further research can be made.

6.1 Conclusions.

1. The major contributor to the drainage problem in the study area was found to be precipitation, both direct and seepage from upslope resulting from poor soil natural drainage and low horizontal hydraulic conductivities.
2. The soils in .the bottomland are predominantly heavy clays with very low saturated hydraulic conductivity and considerably varying over a short distance. The average hydraulic conductivity and drainable porosity were 0.06 m/day and 2.4% respectively. However, owing to the wide variation of the drainable porosities so obtained, a value for the drainable porosity of 3 % was adopted for design purposes.
3. The relative contributions of the various factors to the drainage problem were direct precipitation, Runoff which accounts for about 10% of the rainfall, and Lateral underground seepage accounting for 2.15 m^3 per year if a unit length is considered.
4. The critical drainage period is from April to June when rainfall exceeds evapotranspiration by 90 mm in April, 75 mm in May and evapotranspiration exceeds rainfall by 45 mm in June on the average. Using daily rainfall figures as a basis and for the months in questions, a drainage modulus of 51.7 mm/day was obtained. This value was used in designing the system for the area under study.

5. The ground water levels in the observation wells were found to follow closely the rainfall pattern during the study period, although in some wells it was much delayed. Some wells also showed little fluctuations as the dry weather progressed. These were found to be recharge points or areas of low water transmission.
6. The continuous depletion of the ground water in all the observation wells with the progressing dry weather showed the absence of a perched watertable and hence the presence of a continuous area of high ground watertable which can then be drained with the designed system.

6.2 Recommendations.

1) The bottomland was found to have heavy clay which restricts both horizontal and vertical flow of water. To improve the soil permeability, soil disruption is recommended through deep subsoiling.

2) An interceptor drain placed at the very top of the problematic area and dug down to a depth of 1.2 m can intercept all the upslope seepage and in cases of overland flow, act as a cut- off drain for the same.

3) Mole drains, which are less expensive drainage measures should be tried at spacings of 7.5 m and 15 m and then evaluate the effect on the watertable control.
4) Further research should be done to ascertain the following:-
   a) The effectiveness of the recommended drainage system as far as animal trampling and grass growth is concerned.
   b) Extension of the watertable fluctuation data so as to come up with a meaningful ground watertable prediction model for the area and similar areas, so that given a certain amount of rainfall on the catchment, the likely rise of the ground water table is predictable without necessarily going to measure in the observation wells. This should however include such players as Evapotranspiration, sunshine hours,

Temperature both minimum and maximum, Previous rainfall, Relative humidity, and Atmospheric pressure changes among other things as they likely to influence ground water table fluctuations.

c) Investigation into the possibility of a total water management schedule encompassing both drainage and sub-irrigation. This stemmed from the fact that a prolonged dry spell followed during the drier months of July to September which virtually caused all the grass to dry. This kind of a scenario looked odd as barely three months away, during the rainy season, water was in excess as to cause damage to the same grass. Hence the possibility of using the temporary excess water for sub-irrigation during times of deficit was conceived. The same channels designed for drainage could be used for irrigation, but with module arrangement to allow raising of water level in times of sub-irrigation.

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APPENDICES

APPENDIX 1. SOIL OF THE STUDY AREA FROM 3 PROFILE PITS (ACCORDING TO FAO/UNESCO GUIDELINES, 1977).

Topography : Flat, with wire grass as predominant vegetation
Drainage: Moderate to poorly drained
Soil classification: Vertisol.
Location : 20 m from the dam

Table A1.1 Soils of profile pit 1.

Horizon	Depth (cm)	Type	Grade of structure	Size	Colour
1	0 - 15	Sub-angular blocky	Weak to Moderate	Medium to coarse	10 YR 4/3 dark to dark brown with many grass roots 2 cm layer of murrum 7.5 YR 8/1 light grey predominant
2	17 - 78	Sub angular	Weak	Coarse	7.5 YR 3/2 dark brown and mottled with iron
3	80+	Sub-angular blocky	Moderate	Fine to medium	7.5 YR 4/4 brown to dark brown mixture of soil, weathering material and smeary manganese

Topography : Flat, with reeds as predominant vegetation
Drainage: Moderate to poorly drained
Soil classification: Predomintaly Vertisol, with patches of Histosol
Location : 60 m from the dam

Table A1. 2. Soil of profile pit 2

Horizon	Depth (cm)	Type	Grade of structure	Size	Colour
1	0-16	Sub angular blocky	Moderate	Medium to coarse	10YR 2/2 very dark brown (moist), many roots and characteristics of long duration of inundation
2	16-26	Sub angular blocky	Weak	Coarse	10 YR 5/2 (moist) greyish brown with whitish layer resulting from clay being washed from the layer.
3	26-58+	Sub angular blocky	Moderate	Coarse	10 YR 2/2 very dark brown with coarse, prominent yellow mottles

Topography : Flat, with a mixture of wire grass and reeds
Drainage: Moderate to poorly drained
Soil classification: Verisol.
Location: 80 m from the dam.

Table A1. 3. Soil of profile pit 3.

Horizon	Depth (cm)	Type	Grade of structures	Size	Colour
1	0-10	Sub angular blocks	Weak - moderate	Medium to coarse	5 YR 2.5/2 dark reddish brown with many grassroots
2	10-22	Sub angular blocks	Moderate to strong	Medium to coarse	5 YR 2.5/2 dark reddish brown
3	22-58	Sub angular blocky	Weak	Coarse	10 YR 2/2 very dark brown, with presence of slickensides
4	58-134+	Sub angular blocky	Weak to moderate	Coarse	5 YR 2.5/2 dark reddish brown

APPENDIX 2: SATURATED HYDRAULIC CONDUCTIVITY DATA.

Table A2. 1. Hydraulic conductivity data for location 2 (trial 1).

i	Ti	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)
1	0	18.0	108.5	109.77
2	40	18.2	108.3	109.57
3	80	19.0	107.5	108.77
4	150	19.2	107.3	108.57
5	250	20.0	106.5	107.77
6	350	20.8	105.7	106.97
7	550	21.8	104.7	105.97
8	750	22.0	104.5	105.77
9	975	22.5	104.0	105.27
10	1200	22.8	103.7	104.97
11	1500	23.1	103.4	104.67
12	1950	23.5	103.0	104.27
13	2100	23.6	102.9	104.17

Table A2. 2. Hydraulic conductivity Data For location 2 (Trail 2)

i	Ti	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)
1	0	6.0	120.5	121.77
2	40	9.0	117.5	118.77
3	80	10.5	116	117.27
4	150	13.2	113.3	114.57
5	250	15.2	111.3	112.57
6	300	16	110.5	111.77
7	360	16.6	109.9	111.17
8	420	17.2	109.3	110.57
9	480	17.6	108.9	110.17
10	540	18.1	108.4	109.67
11	600	18.4	108.1	109.37
12	660	18.7	107.8	109.07
13	720	18.9	107.6	108.87
14	780	19.3	107.2	108.47
15	840	19.6	106.9	108.17

Table A2. 3. Hydraulic conductivity data for location 2 (trail3)

i	Ti	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)
1	0	6	120.5	121.77
2	40	7.8	118.7	119.97
3	80	8.5	118	119.27
4	150	10.8	115.7	116.97
5	250	12.8	113.7	114.97
6	300	13.6	112.9	114.17
7	360	14.3	112.2	113.47
8	420	14.9	111.6	112.87
9	480	15.2 -	111.3	112.57
10	540	15.7	110.8	112.07
11	600	16.4	110.1	111.37
12	660	16.7	109.8	111.07
13	720	17	109.5	110.77
14	780	17.3	109.2	110.47
15	840	17.6	108.9	110.17

Table A2. 4. Hydraulic conductivity data for location 9 (trial 1).

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	39.8	1.3	60.2	61.47
2	40	40.2	1.7	59.8	61.07
3	80	40.4	1.9	59.6	60.87
4	150	40.9	2.4	59.1	60.37
5	250	41.1	2.6	58.9	60.17
6	350	41.4	2.9	58.6	59.87
7	550	41.6	3.1	58.4	59.67
8	750	41.9	3.4	58.1	59.37
9	975	42.0	3.5	58.0	59.27

Table A2. 5. Hydraulic conductivity data for location 9 (trial 2).

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	39.0	0.5	61.0	62.27
2	150	39.5	1.0	60.5	61.77
3	300	39.8	1.3	60.2	61.47
4	450	39.9	1.4	60.1	61.37
5	630	40.0	1.5	60.0	61.27
6	760	40.2	1.7	59.8	61.07
7	990	40.3	1.8	59.7	60.97

Table A2. 6. Hydraulic coductivity Data for location 10 (Trail 1)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	42	3.5	52	53.27
2	10	43.2	4.7	50.8	52.07
3	20	44.3	5.8	49.7	50.97
4	30	45.2	6.7	48.8	50.07
5	40	46.1	7.6	47.9	49.17
6	50	47.4	8.9	46.6	47.87
7	60	48.3	9.8	45.7	46.97
8	70	49	10.5	45	46.27
9	80	49.7	11.2	44.3	45.57
10	90	50.3	11.8	43.7	44.97
11	100	51	12.5	43	44.27
12	110	51.8	13.3	42.2	43.47
13	120	52.4	13.9	41.6	42.87

Table A2. 7. Hydraulic conductivity Data For location 10 (Trail 2)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	42	3.5	52	53.27
2	10	43.5	5	50.5	51.77
3	30	45.6	7.1	48.4	49.67
4	45	47.5	9	46.5	47.77
5	60	48.8	10.3	45.2	46.47
6	80	49.8	11.3	44.2	45.47

Table A2. 8. Hydraulic Conductivity data For Location 11

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	41	2.5	79.6	80.87
2	40	45	6.5	75.6	76.87
3	80	47	8.5	73.6	74.87
4	90	47.8	9.3	72.8	74.07
5	120	48.9	10.4	71.7	72.97
6	135	49.5	11	71.1	72.37
7	150	50	11.5	70.6	71.87
8	180	50.9	12.4	69.7	70.97
9	210	51.6	13.1	69	70.27
10	240	51.8	13.3	68.8	70.07
11	270	52	13.5	68.6	69.87

Table A2. 9. Hydraulic Conductivity Data For Location 12 (Trail 1)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	39.1	0.6	69.6	70.87
2	40	39.3	0.8	69.4	70.67
3	80	39.4	0.9	69.3	70.57
4	150	39.6	1.1	69.1	70.37
5	250	39.8	1.3	68.9	70.17
6	300	40.1	1.6	68.6	69.87
7	550	40.4	1.9	68.3	69.57
8	750	40.8	2.3	67.9	69.17
9	975	40.9	2.4	67.8	69.07
10	1200	41.1	2.6	67.6	68.87

Table A2. 10. Hydraulic conductivity data for location 12 (Trails 2)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	41.1	2.6	67.6	68.87
2	150	41.3	2.8	67.4	68.67
3	300	41.4	2.9	67.3	68.57
4	450	41.5	3	67.2	68.47
5	600	41.6	3.1	67.1	68.37
6	750	41.7	3.2	67	68.27
7	900	41.8	3.3	66.9	68.17
8	1050	41.9	3.4	66.8	68.07
9	1200	42	3.5	66.7	67.97
10	1350	42.1	3.6	66.6	67.87

Table A2. 11. Hydraulic Conductivity Data Location 3 (Trail 1)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	39	0.9	80.6	81.87
2	140	39.15	1.05	80.45	81.72
3	300	39.2	1.1	80.4	81.67
4	500	39.45	1.35	80.15	81.42
5	650	39.65	1.55	79.95	81.22
6	900	39.85	1.75	79.75	81.02
7	1090	39.95	1.85	79.65	80.92
8	1300	40.05	1.95	79.55	80.82
9	1520	40.1	2	79.5	80.77
10	1800	40.15	2.05	79.45	80.72
11	2100	40.2	2.1	79.4	80.67

Table A2. 12. Hydraulic conductivity data for location 6 (trial1)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	42.10	4.00	82.00	83.27
2	40	43.10	5.00	81.00	82.27
3	80	43.60	5.50	80.50	81.77
4	150	44.20	6.10	79.90	81.17
5	250	44.55	6.45	79.55	80.82
6	350	44.75	6.65	79.35	80.62
7	550	45.00	6.90	79.10	80.37
8	750	45.15	7.05	78.95	80.22
9	975	45.25	7.15	78.85	80.12
10	1200	45.35	7.25	78.75	80.02
11	1500	45.50	7.40	78.60	79.87

Table A2. 13. Hydraulic conductivity data for location 8 (trial 1).

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	40.30	2.20	81.00	82.27
2	40	41.00	2.90	80.30	81.57
3	80	41.70	3.60	79.60	80.87
4	150	42.40	4.30	78.90	80.17
5	250	43.30	5.20	78.00	79.27
6	350	44.10	6.00	77.20	78.47
7	550	45.40	7.30	75.90	77.17
8	750	46.40	8.30	74.90	76.17
9	975	47.35	9.25	73.95	75.22
10	1200	48.15	10.05	73.15	74.42
11	1500	49.05	10.95	72.25	73.52

Table A2. 14. Hydraulic conductivity data for location 8 (trial 2).

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	38.70	0.60	82.60	83.87
2	140	39.70	1.60	81.60	82.87
3	300	40.60	2.50	80.70	81.97
4	500	41.50	3.40	79.80	81.07
5	650	42.15	4.05	79.15	80.42
6	900	44.00	5.90	77.30	78.57
7	1090	45.15	7.05	76.15	77.42
8	1300	46.10	8.00	75.20	76.47
9	1520	46.90	8.80	74.40	75.67
10	1800	48.25	10.15	73.05	74.32
11	2100	49.25	11.15	72.05	73.32

Table A2. 15.Hydraulic conductivity data for location 5 (trial 1)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	39 10	1.00	89.80	91.07
2	40	42.20	4.10	86.70	87.97
3	80	43 85	5.75	85.05	86.32
4	150	45.20	7.10	83.70	84.97
5	250	45.40	7.30	83.50	84.77
6	300	46.95	8 85	81.95	83.22
7	350	47.45	9.35	81.45	82.72
8	550	49 60	11.50	79.30	80.57
9	750	51.60	13.50	77.30	78.57
10	900	53 10	15.00	75.80	77.07

Table A2. 16. Hydraulic conductivity data for location 5 (trial 2).

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	39 50	1.40	89.40	90.67
2	140	42 25	4.15	86.65	87.92
3	250	43 40	5.30	85.50	86.77
4	300	43 85	5.75	85.05	86.32
5	360	44.30	6.20	84.60	85.87
6	420	44 65	6.55	84 25	85.52
7	480	45.10	7.00	83.80	85.07
8	600	45.65	7.55	83.25	84.52
9	720	46 15	8.05	82.75	84.02
10	900	46 75	8.65	82.15	83.42
11	960	46 95	8.85	81.95	83.22
12	1200	47 65	9.55	81 25	82.52

Table A2. 17. Hydraulic conductivity data for location 3 (trial 2).

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	39 60	1.50	80.00	81.27
2	40	39.70	1.60	79.90	81.17
3	80	39 85	1.75	79.75	81.02
4	150	40 00	1.90	79.60	80.87
5	250	40.05	1.95	79.55	80.82
6	350	40 10	2 00	79.50	80.77
7	550	40.15	2.05	79.45	80.72
8	750	40 20	2.10	79.40	80.67
9	975	40 25	2.15	79 35	80.62
10	1200	40 28	2.18	79.32	80.59
11	1500	40 30	2.22	79.28	80.55

Table A2. 18. Hydraulic Conductivity Data For Location 4 (Trail 1)

i	Ti	h'(ti)	h"(ti)	h(ti)	h(ti)+r/2
Index	sec	(Cm)	(Cm)	(Cm)	(Cm)
1	0	40.5	2.40	68.10	69.37
2	140	40.55	2.45	68.05	69 32
3	300	40.65	2.55	67.95	69.22
4	500	40 80	2.70	67.80	69.07
5	650	40 85	2.75	67.75	69.02
6	900	40.95	2.85	67.65	68.92
7	1090	41.00	2.90	67.60	68.87
8	1300	41.05	2.95	67.55	68.82
9	1520	41.1 0	3.00	67.50	68.77

APPENDIX 3: SOIL INFILTRATION DATA .

Table A3. 1. Infiltration data for location 2 (trial 2).

Elapsed time (min)	Reading (cm)	Infiltrated depth (cm)	Average infiltration rate (cm/hr)	Accumulated infiltration (cm)
0.0	40.6	0.00		
0.5	40.8	0.20	24.0	0.20
1.0	40.9	0.10	12.0	0.30
1.5	41.0	0.10	12.0	0.40
2.0	41.1	0.10	12.0	0.50
3.0	41.3	0.20	12.0	0.70
4.0	41.5	0.20	12.0	0.90
5.0	41.7	0.20	12.0	1.10
10.0	42.7	1.00	12.0	2.10
15.0	43.7	1.00	12.0	3.10
20.0	44.2	0.50	6.0	3.60
25.0	44.8	0.60	7.2	4.20
30.0	45.4	0.60	7.2	4.80
35.0	46.1	0.70	8.4	5.50
40.0	46.8	0.70	8.4	6.20
45.0	41.5	0.90	10.8	7.10
50.0	42.4	0.90	10.8	8.00
55.0	43.3	0.20	2.4	8.20
60.0	43.5	0.30	3.6	8.50
65.0	43.8	0.50	6.0	9.00
70.0	44.3	0.50	6.0	9.50
75.0	44.8	0.60	7.2	10.10
80.0	45.4	0.70	8.4	10.80
85.0	46.1	0.40	4.8	11.15
90.0	46.5	0.20	2.4	11.35
95.0	46.7	0.30	3.6	11.65
100.0	47.0	0.50	6.0	12.10
110.0	47.4	0.80	4.8	12.85
115.0	43.5	0.80	9.6	13.60
120.0	44.2	0.80	9.6	14.35

Table A3. 2. Infiltration Data for location 12 (Trail 2)

Elapsed time (min)	Infiltrated depth (cm)	Average infiltration rate (cm/hr)	Accumulated infiltration (cm)
0.00			
0.50	0.2	24.0	0.20
1.00	0.10	15.4	0.30
1.50	0.75	11.5	1.05
2.00	0.55	7.7	1.60
2.50	0.65	6.9	2.25
3.00	0.30	5.4	2.55
4.00	0.50	4.8	3.05
5.00	0.75	4.7	3.80
6.00	0.50	4.7	4.30
7.00	0.65	4.7	4.95
8.00	0.85	4.2	5.80
9.00	0.65	4.3	6.45
10.00	0.30	4.4	6.75
15.00	0.65	5.1	7.40
20.00	0.45	4.7	7.85
25.00	0.30	4.3	8.15
30.00	0.35	4.2	8.50
35.00	0.15	4.4	8.65
40.00	0.25	4.4	8.90
45.00	0.30	4.2	9.20
50.00	0.40	4.2	9.60
55.00	0.25	4.2	9.85
60.00	0.30	4.1	10.15
65.00	0.20	3.2	10.35
70.00	0.20	3.7	10.55
75.00	0.10	3.8	10.65
80.00	0.30	3.3	10.95
85.00	0.15	3.5	11.10
86.00	0.20	3.2	11.30
87.00	0.15	3.8	11.45
88.00	0.25	3.6	11.70
89.00	0.20	3.6	11.90
90.00	0.10	3.7	12.00
91.00	0.25	4.2	12.25
95.00	0.10	3.6	12.35
100.00	0.20	3.4	12.55

Table A3. 3. Infiltration data from Site 3

Elapsed time (min)	Infiltrated depth (cm)	Average infiltration rate (cm/hr)	Accumulated infiltration (cm)
0.0			0.0
0.5	0.40	48.0	0.4
1.0	0.40	48.0	0.8
1.5	0.30	36.0	1.1
2.0	0.30	36.0	1.4
2.5	0.20	24.0	1.6
3.0	0.20	24.0	1.8
3.5	0.20	24.0	2.0
4.0	0.20	24.0	2.2
4.5	0.20	24.0	2.4
5.0	0.20	24.0	2.6
6.0	0.40	24.0	3.0
7.0	0.30	18.0	3.3
8.0	0.40	24.0	3.7
9.0	0.30	18.0	4.0
10.0	0.20	12.0	4.2
12.0	0.40	12.0	4.6
14.0	0.50	15.0	5.1
15.0	0.15	9.0	5.3
20.0	0.95	11.4	6.2
25.0	1.10	13.2	7.3
30.0	1.00	12.0	8.3
35.0	1.00	12.0	9.3
40.0	0.80	9.6	10.1
45.0	0.60	7.2	10.7
50.0	0.60	7.2	11.3
55.0	0.50	6.0	11.8
60.0	0.40	4.8	12.2
65.0	0.70	8.4	12.9
70.0	0.60	7.2	13.5
75.0	0.70	8.4	14.2
80.0	0.50	6.0	14.7
85.0	0.60	7.2	15.3
90.0	0.50	6.0	15.8
95.0	0.50	6.0	16.3

Table A3. 4. Infiltration data from adjacent site 1

Elapsed time (min)	Infiltrated depth (cm)	Average infiltration rate (cm/hr)	Accumulated infiltration (cm)
0.0	0.00		0.0
0.5	0.10	12.0	0.1
1.0	0.10	12.0	0.2
1.5	0.10	12.0	0.3
2.0	0.10	12.0	0.4
2.5	0.10	12.0	0.5
3.0	0.05	6.0	0.6
3.5	0.10	12.0	0.7
4.0	0.05	6.0	0.7
4.5	0.10	12.0	0.8
5.0	0.10	12.0	0.9
6.0	0.20	12.0	1.1
7.0	0.10	6.0	1.2
9.0	0.30	9.0	1.5
11.0	0.30	9.0	1.8
13.0	0.20	6.0	2.0
15.0	0.20	6.0	2.2
17.0	0.20	6.0	2.4
19.0	0.20	6.0	2.6
21.0	0.20	6.0	2.8
23.0	0.10	3.0	2.9
25.0	0.20	6.0	3.1
30.0	0.30	3.6	3.4
35.0	0.30	3.6	3.7
40.0	0.30	3.6	4.0
45.0	0.30	3.6	4.3
50.0	0.20	2.4	4.5
55.0	0.30	3.6	4.8
60.0	0.20	2.4	5.0
65.0	0.20	2.4	5.2
70.0	0.20	2.4	5.4
75.0	0.20	2.4	5.6

Table A3. 5. Infiltration Data From Adjacent Site 2

Elapsed time (min)	Infiltrated depth (cm)	Average infiltration rate (cm/hr)	Accumulated infiltration (cm)
0.0			0.00
0.5	0.10	12.90	0.10
1.0	0.10	11.04	0.20
1.5	0.10	10.08	0.30
2.0	0.10	9.48	0.40
2.5	0.10	9.00	0.50
3.0	0.05	8.64	0.55
3.5	0.10	8.34	0.65
4.0	0.05	8.10	0.70
4.5	0.10	7.86	0.80
5.0	0.10	7.68	0.90
6.0	0.20	7.38	1.10
7.0	0.10	7.14	1.20
9.0	0.30	6.72	1.50
11.0	0.30	6.48	1.80
13.0	0.20	6.24	2.00
15.0	0.20	6.00	2.20
17.0	0.20	5.88	2.40
19.0	0.20	5.70	2.60
21.0	0.20	5.58	2.80
23.0	0.10	5.46	2.90
25.0	0.20	5.34	3.10
30.0	0.30	5.16	3.40
35.0	0.30	4.98	3.70
40.0	0.30	4.86	4.00
45.0	0.30	4.68	4.30
50.0	0.20	4.62	4.50
55.0	0.30	4.50	4.80
60.0	0.20	4.44	5.00
65.0	0.20	4.32	5.20
70.0	0.20	4.26	5.40
75.0	0.20	4.20	5.60

APPENDIX 4: RAINFALL AND OTHER METEOROLOGICAL DATA.

Table A4. 1 (a) Rainfall for Kabete University Field station(1992-1993)

Date	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
1	0	0	0	2.6	0	28.8		0	0
2	0	0	0	50.6	140.9	0	14.0	0	0
3	0	0	0	1.9	8.6	0	12.9	0	0
4	0	0	0	0	1.5	2.5	0	0	0.5
5	0	38.8	0	3.5	0	2.0	0	0	0
6	0.5	0	0	5.1	0	0	0	0	0
7	0	0	0	39.4	0	0	0	0	0
8	0	0	0	37.5	0	0	0	0	0
9	0	27.5	0	15.7	0	0	0	0.9	0
10	0	0	0	42.1	1.0	0	0	0	1.3
11	0	0	0	0	0	1.2	0	0	13.0
12	0	0	0	0	0	1.6	0	0	0
13	0.8	0	0	0	9.5	3.7	0	0	0
14	0	0	0	0	4.0	4.6	0	0	0
15	5.0	0	0	0	20.9	0	0	1.6	0
16	0	0	0	0	2.5	0	0	0.8	0
17	0	0	1.5	0	0	0	0.3	0	0
18	0	0	0	15.5	0	0	0	0	0
19	0	0	0	0	0	0	0	0	0
20	0	0	0	21.5	1.3	2.3	0	0	0
21	0	0	0.6	11.6	0	0.7	0	0	0
22	0	0	2.3	0	0	0	0	0	0
23	0	0	0	0	0	0	0	0	0
24	0	0	0	0.7	0	0	0	0	0
25	0	0	0	0	0	0	0	0	0
26	0	0	0	51.2	0	0	0	0.6	0
27	0	0	0	16.4	0	0	0	0	0
28	0	0	0	45.9	0	0	0	0	0
29	0	0	0	9.6	8.7	0	0	0	0
30	0		0.3	40.6	1.3	0	0	0	0
31	0		0		0		0	0	
Total	6.3	66.3	4.7	411.4	226.8	18.6	30.0	3.9	14.8

Table A4.1 (a) Rainfall for Kabete Kabete University Field station (1992-1993)

Date	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun
1	0	0.5	1.5	12.3	0.7	0	1.5	7.6	0
2	1.2	1.9	0	0.4	0	0	0	1.2	0
3	2.1	26.3	0	0	0	0	0	0.6	0
4	7.2	8.2	0	0	0	0	0	0.5	30.7
5	0	6.5	0	3.5	0.1	0	0	0	0.8
6	0	5.6	1.4	20.6	1.3	0	5.0	4.1	0
7	0	0	8.0	28.0	0	0	0	0.6	0
8	0	0.4	3.7	0	15.7	12.9	0	17.3	0.7
9	0	2.5	4.2	1.6	0.3	0	0	0	7.1
10	0	0	14.3	5.7	16.7	0	0	0	11.0
11	0	1.1	1.4	0	4.1	0	3.4	0.8	1.6
12	5.9	2.4	0.5	2.2	0	24.5	0	0	0.9
13	0	0.5	3.8	7.0	0	0	2.1	4.5	0
14	0	5.2	0	0.6	0	0	2.3	1.6	0.5
15	0.6	0	0	18.1	0	0	3.7	0	0
16	0	26.3	34.3	0	0	0	5.9	0	0
17	8.8	0	3.3	8.5	0	0	1.0	3.1	0
18	7.6	0	0.7	19.5	0	0	1.8	0	0
19	0	0	0	4.4	0	0	0	0	0
20	0	0	0	40.0	0	0	0	0	0
21	0	0	0	3.1	0	0	0	0	0
22	0	0	0	0	0	1.5	0	0	0
23	0	0.3	0.4	0	0	0.4	0.5	0	0
24	0	4.0	0	0	12.1	0	0.8	0	0
25	0	0	0	10.6	0	0	0	0	0
26	0	0	0	5.9	0	0	0	0	0
27	21.4	1.1	0	0	0	0.4	0	0	0
28	7.2	0	0	0.8	0	0	0	0	0
29	5.9	0	2.5	5.8		0	0	0	0
30	5.5	6.0	3.8	0.9		13.1	20.5	0	0
31	0		7.4	2.1		1.6		0	0
Total	73.4	98.8	91.2	201.6	51.0	54.4	48.5	44.4	53.3

Table A4. 2. Reference Evapotranspiration ETo according to Penman -Monteith

Country: Kenya Altitude: 1942 m				Meteo.Station:Kabete University Field station (23 yr) Coordinates:1.5°S, 36.44°E.				
MONTH	Max. Temp.	Min. Temp	Humidity %	Windrun km/day	Sunshine hours	Sol.Radia. (MJ/m/day)	Eto-Pen-Mon mm/ day	ETo mm/month
January	24.9	13.0	63	144	8.8	22.4	4.4	136.4
February	25.9	13.4	62	141	8.4	22.5	4.6	128.8
March	26.0	13.9	62	138	8.3	22.6	4.6	142.6
April	24.8	14.6	73	110	6.8	19.5	3.8	114.0
May	23.4	13.9	75	83	5.6	16.7	3.2	99.2
June	22.1	12.0	75	72	4.3	14.3	2.7	81.0
July	21.4	11.4	75	76	3.7	13.6	2.6	80.6
August	21.6	11.2	72	84	3.6	14.2	2.8	86.8
September	24.2	11.8	66	94	5.9	18.4	3.6	108.0
October	25.0	13.2	63	119	7.2	20.6	4.1	127.1
November	23.4	13.8	72	132	6.4	18.8	3.7	111.0
December	23.7	13.4	70	144	6.4	18.5	3.7	114.7
YEAR	23.9	13.0	69	111	6.3	18.5	3.7	1334

Table A4. 3. Maximum successive rainfall totals for durations 1 to 11 days for Kabete field station for the Month of April during the period 1971-1993.

Year	Duration of successive rainfall										
	1-day	2-day	3-day	4-day	5-day	6-day	7-day	8-day	9-day	10-day	11-day
1971	73.6	77.0	77.0	81.7	94.8	103.7	131.0	151.0	155.3	160.8	173.9
1972	9.6	18.1	18.1	18.1	18.1	18.1	18.1	18.1	18.1	18.1	18.1
1973	102.0	108.3	108.3	122.3	129.3	153.3	161.3	183.3	195.3	195.6	199.1
1974	25.0	40.5	57.2	73.0	86.6	91.1	109.2	126.9	140.9	151.1	157.9
1975	57.0	78.2	85.2	109.5	116.5	123.7	134.2	155.0	165.5	180.5	181.0
1976	33.0	55.5	57.7	65.4	69.6	87.8	90.0	90.0	90.0	95.6	97.5
1977	84.9	149.3	173.3	178.7	182.1	187.5	191.2	231.6	241.5	246.9	250.1
1978	70.5	75.6	76.1	101.9	101.9	133.7	147.7	177.7	183.1	183.6	183.6
1979	29.8	57.6	63.6	66.0	71.7	96.5	102.5	103.7	129.4	135.4	136.6
1980	55.9	79.0	95.6	111.0	113.7	121.7	137.1	140.4	143.1	144.7	144.9
1981	128.3	133.1	175.0	190.9	221.1	279.8	284.6	319.7	326.1	327.2	327.4
1982	66.1	69.5	124.4	135.6	171.0	182.2	183.8	185.0	185.3	186.5	194.3
1984	20.9	23.7	25.2	26.2	26.9	27.0	29.7	29.8	34.5	37.3	38.8
1985	35.1	54.6	77.6	97.1	104.9	111.0	112.1	116.9	136.4	137.5	137.5
1986	41.0	79.7	94.2	95.6	120.0	120.0	120.7	136.4	136.8	136.8	137.3
1987	37.5	69.5	101.9	115.7	121.0	121.5	132.3	133.5	157.1	157.5	157.5
1988	63.8	115.9	123.1	173.8	204.3	204.3	225.8	260.9	260.9	282.4	293.6
1989	48.2	58.3	72.7	79.8	91.9	103.9	104.3	107.4	109.8	109.8	110.6
1990	47.7	68.1	70.5	97.6	118.0	118.5	121.5	122.0	130.5	133.2	133.4
1991	52.4	75.2	80.1	80.1	82.2	82.2	84.3	92.3	97.2	119.0	123.9
1992	60.5	77.9	118.5	130.2	157.4	157.4	159.1	159.1	190.7	193.5	193.5
1993	20.5	20.5	20.5	20.5	20.5	20.5	20.5	21.3	21.8	21.8	22.4
Est(a)	0.038	0.033	0.026	0.023	0.020	0.018	0.017	0.015	0.015	0.014	0.014
Est(m)	39.0	55.9	66.0	75.8	83.7	90.5	96.6	103.6	111.6	115.7	117.7
Mean	52.8	72.1	86.2	98.7	110.2	120.2	127.3	139.2	147.7	152.5	155.1
Std	28.2	33.1	41.3	46.7	54.1	60.6	62.6	72.6	73.8	75.2	76.3

Table A4. 4. Maximum successive rainfall totals for durations 1 to 11 days for Kabete field station for the Month of June during the period 1971-1993.

	Duration of successive rainfall										
Year	1-day	2-day	3-day	4-day	5-day	6-day	7-day	8-day	9-day	10-day	11-day
1971	6.6	8.1	8.1	8.1	8.1	8.1	11.6	15.4	15.4	15.4	15.4
1972	97.8	105.9	113.1	113.1	113.4	113.4	113.4	113.4	113.4	113.4	113.4
1973	20.8	29.8	30.8	32.8	32.8	32.8	32.8	32.8	32.8	32.8	32.8
1974	20.0	25.6	26.7	29.7	38.1	40.0	41.8	41.8	41.8	41.8	41.8
1975	3.5	3.5	3.5	3.5	4.2	4.2	4.6	5.7	5.7	5.9	7.3
1976	18.0	21.7	23.3	27.0	27.2	27.2	29.8	31.2	31.4	31.4	32.4
1977	29.3	38.9	38.9	38.9	38.9	38.9	38.9	38.9	38.9	40.6	40.6
1978	3.9	5.8	6.6	7.0	7.0	7.0	7.0	7.0	7.0	7.0	9.0
1979	14.7	16.6	16.6	17.6	17.6	21.6	21.6	26.1	28.6	28.6	28.6
1980	14.6	14.6	14.8	14.8	15.0	15.0	20.0	21.0	21.0	21.2	21.2
1981	3.8	4.1	4.8	4.8	5.0	5.2	5.6	5.6	5.6	6.0	6.0
1982	6.2	6.6	6.6	6.7	11.4	11.9	12.3	12.9	13.3	13.6	13.7
1983	35.0	36.7	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.3
1984	3.0	4.0	4.0	4.0	4.0	4.0	5.5	5.7	5.7	5.7	5.7
1985	11.4	12.5	13.5	15.9	16.1	16.1	16.1	16.1	16.1	16.1	16.1
1986	7.6	8.9	8.9	8.9	9.5	10.0	10.0	10.0	10.6	12.2	12.2
1987	37.1	44.0	54.4	54.6	54.8	57.0	58.1	61.0	61.2	61.4	63.4
1989	24.6	25.4	25.9	25.9	26.1	26.1	26.1	26.1	26.1	26.1	26.1
1990	3.1	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.7
1991	6.4	8.2	8.4	8.4	10.8	11.0	11.0	11.0	11.0	11.0	11.0
1992	5.1	8.1	10.4	11.6	11.6	11.8	11.8	11.8	13.0	14.2	17.0
1993	30.7	31.5	31.5	31.5	32.2	39.3	50.3	51.9	52.8	52.8	53.3
Est(α)	0.051	0.047	0.044	0.044	0.044	0.043	0.043	0.043	0.043	0.043	0.043
Est(μ)	8.1	9.9	10.3	11.0	11.8	12.5	13.6	14.3	14.6	14.9	15.3
Mean	18.3	21.1	22.3	23.0	23.8	24.6	25.9	26.6	26.9	27.2	27.6
Std	20.9	22.8	24.6	24.6	24.6	24.8	25.1	25.2	25.2	25.1	25.1

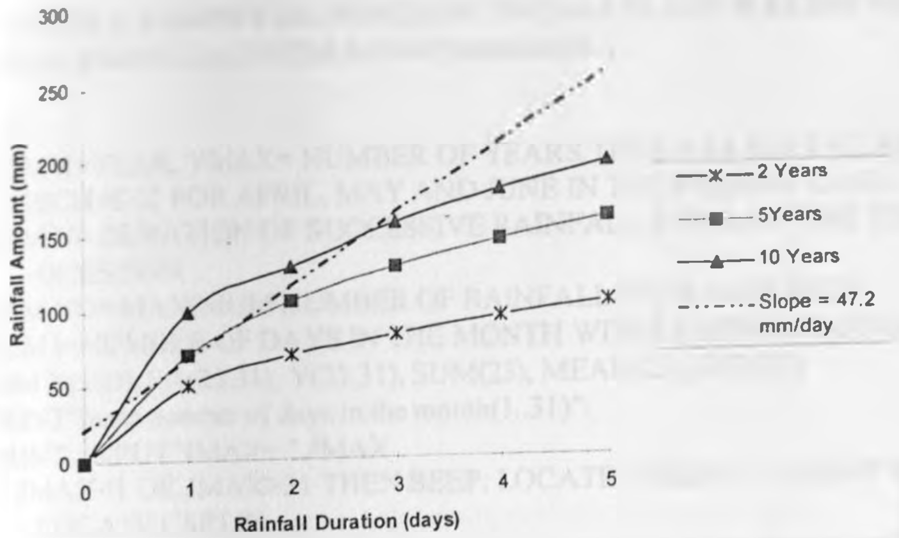

Figure A4.5. Depth-Duration- Frequency Relationship for Kabete for the month of April

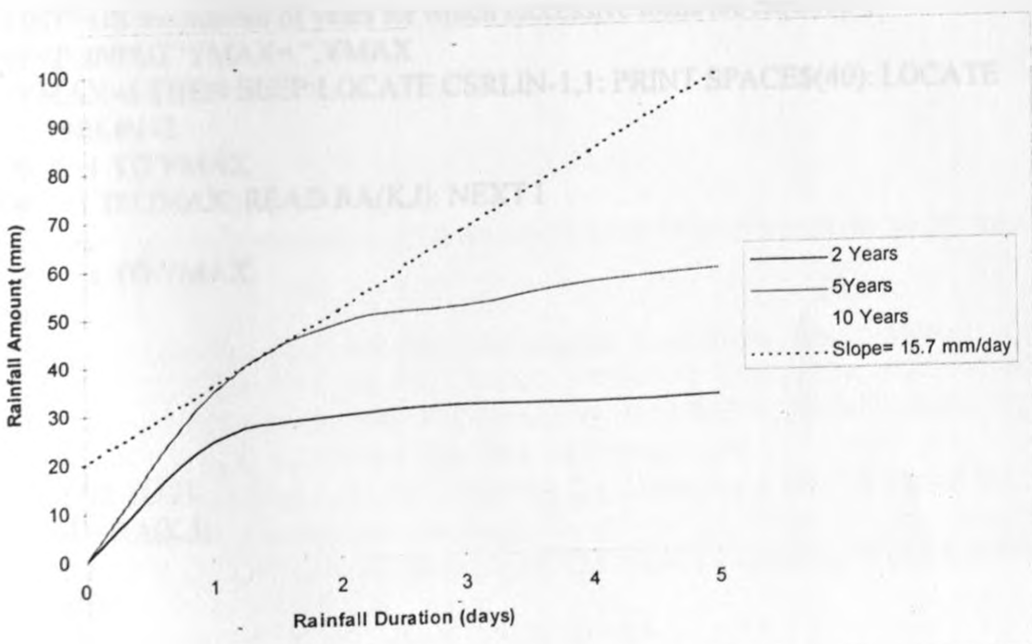

Figure F4.6. Depth-Duration- Frequency Relationship for Kabete for the month of June.

APPENDIX 5: COMPUTER PROGRAM TO CALCULATE MAXIMUM SUCCESSIVE MOVING RAINFALL TOTALS (MS QuickBASIC) .

```
5 CLS
10 REM K=YEAR, YMAX= NUMBER OF YEARS THE DATA IS AVAILABLE
15 ' WHICH IS 22 FOR APRIL, MAY AND JUNE IN THE PRESENT CASE
20 REM N= DURATION OF SUCCESSIVE RAINFALL TOTAL OF THE YEAR IN
      QUESTION
25 REM NO= MAXIMUM NUMBER OF RAINFALL DAYS REQUIRED
30 REM I=NUMBER OF DAYS IN THE MONTH WITH 31 BEING MAX=JMAX
31 DIM X(100), RA(23,31), Y(23,31), SUM(23), MEAN(23), STD(23)
33 PRINT"Input number of days in the month(1..31)":
34 PRINT: INPUT"JMAX= ",JMAX
35 IF JMAX<1 OR JMAX>31 THEN BEEP: LOCATE CSRLIN-1,1: PRINT SPACE$(40):
      LOCATE CSRLIN......
36 PRINT"Input the maximum duration for which successive totals are required";
37 PRINT: INPUT"NO= ",NO
38 IF NO <1 OR NO >31 THEN BEEP:LOCATE CSRLIN-1,1: PRINT SPACE$(40):
      LOCATE CSRLIN .......
39 PRINT"Input the number of years with continuous rainfall data";
40 PRINT" OR the number of years for which successive totals are required";
41 PRINT: INPUT"YMAX= ",YMAX
42 IF YMAX<1 THEN BEEP:LOCATE CSRLIN-1,1: PRINT SPACE$(40): LOCATE
      CSRLIN -2
44 FOR K=1 TO YMAX
45 FOR I=1 TO JMAX: READ RA(K,I): NEXT I
50 NEXT K
55 FOR K=1 TO YMAX
60 N=1
70 J1=N
80 J=1
90 I1=1
100 SU=0
110 FOR I=I1 TO J1
120 SU=SU+RA(K,I)
130 NEXT I
140 X(J)=SU
150 I1=I1+1
160 J1=J1+1
170 IF J1>JMAX THEN 200
175 J=J+1
180 GOTO 100
```

```
200 REM SORTING THE RAINFALL TOTALS BY COMPARISON AND THEN PICKING
THE MAXIMUM
210 FOR I=1 TO JMAX-N
215 FOR J=I+1 TO JMAX- (N-1)
220 IF X(J)>X(I) THEN 250
225 TEM=X(I)
230 X(I)=X(J)
240 X(J)=TEM
250 NEXT J
260 Y(K,N)=X(I)
261 REM SUM=SUM+Y(K,N):SUMM=SUMM+(Y(K,N))^2
265 IF N=NO THEN 280
270 N=N+1
275 GOTO 70
280 NEXT K
290 PRINT TAB(1);"YEAR";TAB(6);"1-DAY"; TAB(12);"2-DAY";
300 PRINT TAB(18);"3-DAY"; TAB(24);"4-DAY";TAB(30);"5-DAY";
310 PRINT TAB(36);"6-DAY"; TAB(42);"7-DAY"; TAB(48);"8-DAY";
312 PRINT TAB(54);"9-DAY"; TAB(60);"10-DAY"; TAB(67);"11-DAY"
315 FOR K=1 TO YMAX
316 PRINT TAB(1);K;
320 FOR N=1 TO NO
325 PRINT TAB(6*N);Y(K,N);
330 NEXT N
335 NEXT K
336'DATA FOR THE MONTH OF MAY (1971-1993)
337'-------------------------------------
340 DATA sets are given in this section on a daily basis for each month for the 22 years
    considered.
350 DATA
7980 REM THIS SECTION OF THE PROGRAM WAS NOT SUCCESSIFUL AND HENCE
7985 REM SHOULD NOT BE EXECUTED WITHOUT MODIFICATION. HOWEVER,THE
7986 REM UPPER PART OF THE PROGRAM WAS ALRIGHT AND GENERATED THE 7990
REM CORRECT INFORMATION FOR USE IN THE STUDY.
8000 REM SUBROUTINE FOR RETRIEVING DATA FROM A SEQUENTIAL DATA
8005 REM FILE, WHOSE NAME SHOULD.....
8500 REM THE DATA WILL BE USED TO CALCULATE MAXIMUM SUCCESSIVE
8505 REM RAINFALL TOTALS
9000 REM FOR DURATIONS 1-DAY TO NO -DAYS
9100 PRINT:PRINT
9500 REM PRINT" FILE RETIEVAL SECTION"
9600 N=1
9700 SUM=0 : SUMM=0
```

```
9800 FOR K=1 TO YMAX
9900 SUM=SUM+Y(K,N): SUMM=SUMM+(Y(K,N)^2)
10000 NEXT K
10100 SUM(N)=SUM: SUMM(N)=SUMM
10200 MEAN(N)= SUM(N)/YMAX
10250 STD=SQRT((SUMM(N)-YMAX*(MEAN(N)^2))/(YMAX-1))
10300 N=N+1
10400 IF N=NO+1 THEN 1060
10500 GOTO 970
10600 REM FOR K=1 TO YMAX
10650 REM FOR N=1 TO NO
10700 REM PRINT MEAN(N);
10800 REM NEXT N
10900 FOR N=1 TO NO
10950
11000 PRINT SUM(N);
11100 PRINT STD(N);
11200 NEXT N
11400 REM SORTING OUT THE MAXIMUM TOTALS FOR A PARTICULAR DURATION
11500 REM AND ARRAGING THEM IN ORDER OF MAGNITUDE STARTING WITH THE
11600 REM HIGHEST AND ENDING WITH THE LOWEST
11700 REM FOR PROBABILITY DETERMINATION
11800 REM M=1 TO NO
11900 FOR N=1 TO NO
12000 IF Y(K,N)<Y(K,N+1) THEN 1240
12100 H=Y(K,N)
12200 Y(K,N)=Y(K,N+1)
12300 Y(K,N+1)=H
12400 NEXT N
12500 NEXT M
12600 REM PRINT TAB(5);"RAINFALL AMOUNT"; TAB925);"RANK"
12700 FOR N=1 TO NO
12800 REM PRINT TAB(10);Y(K,N);TAB(30);N
12900 NEXT N
40000 END
```

APPENDIX 6: WATERTABLE LEVEL DATA COLLECTED DURING THE STUDY PERIOD.

Table A6. 1. Water table information

Date	Observation well designation								
	A1	A2	B1	B2	C2	C3	D2	D3	E2
5/2/93	106.9	68.2	85.2	44.2	31.2	15.6	88.5	9.6	20.5
12/2/93	32.9	57.6	78.1	29.8	17.0	10.1	82.0	4.9	6.6
19/2/93	76.4	62.7	86.2	46.9	38.0	22.0	100.0	15.0	24.5
26/2/93	99.0	58.0	93.1	54.3	55.3	29.6	117.5	30.0	38.3
1/3/93	108.9	61.0	95.4	55.4	62.7	38.8	122.8	36.7	40.0
5/3/93	120.0	63.5	95.7	58.7	68.2	43.4	129.8	41.4	37.9
12/3/93	137.0	71.2	99.1	66.7	73.7	50.8	138.5	52.5	40.0
15/3/93	121.9	58.5	80.7	47.9	62.3	39.0	137.5	14.7	31.2
18/3/93	130.0	62.6	84.5	53.0	66.9	40.2	142.1	32.8	37.5
19/3/93	132.5	64.0	85.4	54.4	67.5	44.0	142.6	34.3	37.9
22/3/93	139.5	66.6	88.3	58.4	70.6	46.5	145.1	39.0	39.6
24/3/93	143.5	70.0	90.4	61.4	73.2	49.0	147.2	43.5	40.3
26/3/93	147.7	72.2	91.8	64.2	74.6	52.8	148.5	46.7	41.8
29/3/93	153.0	76.2	94.1	69.9	76.1	58.0	151.4	52.9	43.9
2/4/93	159.7	80.9	97.4	76.9	78.5	62.7	155.1	60.6	46.2
5/4/93	164.3	85.0	99.4	81.5	81.5	66.9	158.0	66.1	48.0
9/4/93	169.9	94.5	104.4	85.1	84.9	d	160.8	72.2	50.3
12/4/93	173.9	d	107.5	d	87.5	d	163.0	76.3	51.7
16/4/93	178.1	d	113.3	d	d	d	164.8	81.0	53.5
19/4/93	181.2	d	d	d	d	d	166.2	83.5	53.8
21/4/93	183.0	d	d	d	d	d	167.2	86.0	54.3
23/4/93	185.0	d	d	d	d	d	167.5	88.0	55.6
26/4/93	188.0	d	d	d	d	d	168.6	91.2	57.2
3/5/93	193.2	99.5	d	d	d	d	171.0	72.1	51.0
7/5/93	194.4	72.6	d	d	d	d	172.4	76.3	53.7
10/5/93	196.1	78.0	d	d	d	d	173.2	78.2	46.3
14/5/93	198.2	82.2	d	d	d	d	174.5	83.4	48.8
17/5/93	199.0	83.7	d	d	d	d	175.7	85.0	46.8
21/5/93	d	90.7	d	d	d	d	177.0	88.8	48.5
24/5/93	d	98.5	d	d	d	d	177.8	92.0	47.9
28/5/93	d	109.1	d	d	d	d	178.9	97.2	50.6
31/5/93	d	d	d	d	d	d	180.0	100.0	52.8
2/6/93	d	d	d	d	d	d	181.7	d	53.0
9/6/93	d	d	d	d	d	d	182.6	d	56.5
10/6/93	d	d	d	d	d	d	184.5	d	51.9
11/6/93	d	d	d	d	d	d	184.8	d	51.5
14/6/93	d	d	d	d	d	d	185.6	d	56.1

d:denotes dry well and hence no reading taken

(Cont'd)

Observation well designation

Date	E3	E4	F3	F4	F5	G3	G4	G5	H2
5/2/93	24.0	4.0	42.1	5.8	68.2	45.6	59.4	68.0	37.5
12/2/93	2.3	-6.0	24.5	3.7	72.1	13.4	43.0	62.5	22.2
19/2/93	30.0	5.0	43.8	7.0	72.5	43.1	63.5	71.0	38.5
26/2/93	47.8	5.8	50.4	7.5	83.4	60.4	65.5	72.0	33.5
1/3/93	54.9	8.6	53.8	8.7	85.7	67.6	68.3	d	40.8
5/3/93	62.9	21.2	57.2	9.7	88.8	77.0	72.1	d	43.4
12/3/93	74.6	37.5	63.0	10.6	94.5	93.7	79.0	71.2	52.5
15/3/93	37.3	6.6	48.0	7.5	86.3	66.9	61.8	d	38.3
18/3/93	47.5	12.4	52.6	10.0	90.5	76.0	65.4	d	44.4
19/3/93	50.1	14.6	54.5	10.2	92.4	79.4	67.0	d	44.7
22/3/93	58.3	21.3	58.8	10.5	95.7	88.9	70.2	d	49.3
24/3/93	64.0	25.8	62.2	11.6	98.0	96.1	72.4	d	52.6
26/3/93	68.8	29.2	65.6	13.4	99.9	103.1	75.9	d	56.5
29/3/93	74.7	34.5	71.8	14.2	100.8	112.9	82.5	d	61.3
2/4/93	80.1	40.2	77.7	12.1	101.0	125.2	91.6	d	65.7
5/4/93	83.8	45.3	d	16.1	101.3	132.5	97.2	d	69.6
9/4/93	85.6	51.8	d	16.9	101.6	141.5	103.8	d	73.8
12/4/93	d	56.2	d	17.6	d	147.9	107.6	d	75.1
16/4/93	d	62.6	d	17.0	d	154.8	112.0	d	76.1
19/4/93	d	66.0	d	14.9	d	157.9	114.4	d	76.5
21/4/93	d	63.2	d	14.2	d	158.2	116.0	d	d
23/4/93	d	36.3	d	16.5	d	162.8	117.6	d	d
26/4/93	d	42.8	d	21.8	d	166.8	119.5	d	d
3/5/93	d	26.6	d	13.0	d	177.0	d	83.4	61.4
7/5/93	d	25.7	d	13.4	d	179.7	d	d	66.5
10/5/93	d	25.1	d	11.9	d	d	d	d	60.1
14/5/93	d	34.9	d	12.3	d	d	d	d	67.9
17/5/93	d	20.3	d	12.0	d	d	d	d	67.8
21/5/93	d	24.0	d	13.0	d	d	d	d	74.0
24/5/93	d	30.7	d	12.7	d	d	d	d	78.4
28/5/93	d	39.5	d	17.4	d	d	d	d	83.6
31/5/93	d	43.6	d	19.0	d	d	d	d	87.2
2/6/93	d	54.8	d	21.3	d	d	d	d	90.4
9/6/93	d	33.0	d	16.3	d	d	d	d	78.0
10/6/93	d	24.2	d	16.6	d	d	d	d	76.5
11/6/93	d	20.8	d	15.2	d	d	d	d	62.0
14/6/93	d	15.1	d	16.6	d	d	d	d	67.4

(Cont'd)

Date	Observation well designation									
	H3	H4	H5	I1	I2	I3	I4	J1	J2	J3
5/2/93	38.6	52.5	65.7	59.0	50.0	23.1	68.5	7.2	42.6	34.0
12/2/93	9.5	32.0	59.5	58.4	31.5	8.2	87.7	4.5	25.0	17.5
19/2/93	43.5	49.2	69.3	61.5	47.0	27.0	87.0	23.3	42.0	35.7
26/2/93	57.2	50.5	67.4	65.2	47.2	36.2	95.9	29.7	39.5	33.9
1/3/93	64.1	53.3	72.7	66.4	55.3	40.8	99.0	45.8	45.7	46.3
5/3/93	72.4	57.4	78.8	69.0	66.0	45.8	103.0	54.9	56.8	60.0
12/3/93	81.9	67.0	90.3	68.4	87.1	54.4	99.5	75.6	68.6	73.5
15/3/93	59.0	55.5	58.6	66.6	55.0	46.4	96.5	26.5	47.1	53.8
18/3/93	66.3	60.2	65.2	69.1	65.5	49.6	100.2	45.0	60.1	63.4
19/3/93	68.4	62.3	67.8	69.4	69.1	51.4	101.5	48.3	62.5	66.5
22/3/93	75.4	68.9	74.6	69.6	80.2	55.8	104.3	53.5	71.3	74.9
24/3/93	78.6	73.8	79.5	70.1	86.1	59.2	107.0	57.0	77.3	78.3
26/3/93	82.0	78.4	84.1	71.9	93.8	63.9	109.0	61.0	82.4	82.4
29/3/93	87.1	85.4	91.6	72.0	104.9	71.4	112.2	69.7	89.6	89.0
2/4/93	92.1	96.5	102.7	71.8	114.4	80.0	116.2	77.5	96.1	90.0
5/4/93	94.4	108.0	114.2	73.3	119.5	86.0	119.3	84.7	102.7	96.6
9/4/93	d	118.8	125.2	74.4	123.8	99.0	123.0	90.1	108.3	89.0
12/4/93	d	128.3	132.0	75.2	124.8	97.8	126.0	95.3	112.4	96.1
16/4/93	d	135.2	d	74.9	d	103.0	129.4	101.7	117.0	101.0
19/4/93	d	137.4	d	73.5	d	104.6	131.8	105.5	119.4	102.7
21/4/93	d	138.6	d	74.0	d	106.0	133.0	107.9	121.0	106.0
23/4/93	d	140.4	d	75.9	d	106.6	134.9	110.0	124.0	109.3
26/4/93	d	143.8	d	79.9	d	108.7	138.3	113.3	128.7	115.0
3/5/93	d	147.5	d	77.2	115.3	111.9	144.4	39.0	101.0	97.3
7/5/93	d	d	d	75.5	122.3	113.0	146.5	46.0	100.4	100.0
10/5/93	d	d	d	74.3	119.5	115.5	148.2	37.0	108.9	90.5
14/5/93	d	d	d	76.6	125.5	118.0	d	50.5	116.0	98.3
17/5/93	d	d	d	75.0	d	d	d	46.3	119.4	99.1
21/5/93	d	d	d	77.5	d	d	d	46.5	124.4	105.8
24/5/93	d	d	d	79.3	d	d	d	53.3	129.4	111.3
28/5/93	d	d	d	85.5	d	d	d	59.4	134.0	120.0
31/5/93	d	d	d	88.0	d	d	d	67.5	135.9	125.5
2/6/93	d	d	d	88.6	d	d	d	75.0	d	131.0
9/6/93	d	d	d	86.0	d	d	d	60.3	d	130.2
10/6/93	d	d	d	83.0	d	d	d	58.5	d	128.6
11/6/93	d	d	d	79.9	d	d	d	40.0	d	116.1
14/6/93	d	d	d	87.2	d	d	d	44.3	d	121.3

(Cont'd)

Observation well designation

Date	J4	K1	K2	K3	K4	L1	L2	L3	L4	M1
5/2/93	55.8	-6.5	41.8	2.0	51.0	80.1	27.6	11.3	76.4	41.0
12/2/93	42.9	-6.3	11.5	-4.9	28.5	97.5	6.4	8.4	87.6	24.5
19/2/93	60.6	-0.5	40.0	22.9	49.4	110.4	25.5	11.8	87.8	49.2
26/2/93	72.7	0.8	52.4	7.0	66.6	123.2	31.0	14.0	104.6	51.1
1/3/93	76.8	4.6	57.1	20.0	71.3	127.9	38.0	17.1	111.2	55.9
5/3/93	81.4	7.4	62.9	36.3	76.8	d	59.5	18.2	118.1	59.5
12/3/93	90.3	11.8	72.3	55.9	86.2	d	d	22.5	130.1	66.2
15/3/93	70.7	6.6	46.7	19.9	59.5	74.8	38.6	16.8	134.9	53.3
18/3/93	77.1	9.5	53.2	33.3	68.8	91.6	50.5	18.3	139.2	58.2
19/3/93	79.0	11.0	55.1	37.2	71.5	95.2	53.0	19.7	140.8	59.0
22/3/93	84.2	13.3	61.6	47.1	78.2	104.6	62.8	21.5	143.6	63.0
24/3/93	87.4	14.9	66.0	54.0	82.0	111.0	65.5	22.7	146.4	65.1
26/3/93	91.1	16.0	70.1	60.3	86.0	116.8	d	23.9	148.5	69.3
29/3/93	92.0	20.4	75.8	70.0	92.0	124.4	d	26.4	151.4	70.0
2/4/93	106.8	24.1	82.2	78.5	98.5	d	d	27.8	154.8	d
5/4/93	107.0	38.7	83.6	82.5	103.8	d	d	32.0	157.8	d
9/4/93	117.0	46.2	d	d	108.6	d	d	33.4	d	d
12/4/93	121.0	51.1	d	d	113.1	d	d	35.0	d	d
16/4/93	128.0	54.2	d	d	118.9	d	d	38.5	d	d
19/4/93	136.5	55.7	d	d	122.1	d	d	37.2	d	d
21/4/93	140.5	57.2	d	d	124.6	d	d	38.8	d	d
23/4/93	144.0	59.1	d	d	126.1	d	d	42.3	d	d
26/4/93	150.0	62.1	d	d	131.2	d	d	52.7	d	d
3/5/93	d	38.0	d	92.9	135.0	d	101.0	35.4	d	78.2
7/5/93	d	40.0	d	95.3	137.8	d	101.6	36.5	d	77.0
10/5/93	d	37.5	d	97.3	140.0	d	102.5	36.6	d	71.6
14/5/93	d	48.0	d	99.2	142.6	d	105.7	43.0	d	80.0
17/5/93	d	49.1	d	100.1	144.1	d	107.0	42.0	d	82.5
21/5/93	d	53.0	d	101.0	147.0	d	110.0	48.5	d	88.5
24/5/93	d	55.8	d	102.0	148.6	d	113.0	54.1	d	93.4
28/5/93	d	61.1	d	102.9	d	d	116.7	68.5	d	99.2
31/5/93	d	65.1	d	104.0	d	d	119.9	73.0	d	d
2/6/93	d	69.2	d	105.2	d	d	125.2	77.4	d	101.7
9/6/93	d	72.0	d	109.5	d	d	123.0	64.3	d	
10/6/93	d	72.4	d	109.1	d	d	122.7	52.3	d	99.5
11/6/93	d	55.0	d	109.4	d	d	114.4	50.3	d	91.1
14/6/93	d	57.5	d	110.0	d	d	116.1	53.2	d	94.6

(Cont'd)

Observation well designation

Date	M2	M3	M4	N2	N3	O2	O3	P3	P4
5/2/93	17.6	8.5	83.6	7.0	37.1	59.2	17.9	39.3	118.6
12/2/93	8.0	-1.6	89.0	4.5	19.4	52.5	12.7	35.5	119.6
19/2/93	23.5	6.5	93.2	7.0	37.6	68.8	19.1	55.5	126.2
26/2/93	30.5	8.5	97.5	7.6	47.2	71.1	20.5	59.5	d
1/3/93	38.5	20.8	100.0	9.1	53.5	77.1	24.4	64.2	d
5/3/93	47.0	48.2	101.0	17.2	64.1	81.0	27.4	68.4	d
12/3/93	60.6	79.8	104.8	20.9		84.4	33.4	71.5	d
15/3/93	22.5	7.2	94.9	7.1	35.1	71.1	21.5	58.6	d
18/3/93	37.0	18.5	97.0	12.8	47.6	78.0	31.5	64.2	d
19/3/93	39.2	36.8	98.2	12.2	52.2	79.5	31.2	65.5	d
22/3/93	44.8	72.0	100.9	16.5	66.5	84.1	31.6	68.0	d
24/3/93	49.5	84.1	102.0	17.2	75.4	85.6	33.2	69.8	d
26/3/93	53.2	93.0	104.5	25.3	81.6	87.4	42.1	70.5	d
29/3/93	59.1	99.5	104.8	28.5	82.9	88.5	45.0	72.0	d
2/4/93	67.8	d	d	24.9	d	87.2	38.1	73.0	d
5/4/93	76.1	d	d	38.3	d	90.4	50.7	74.4	d
9/4/93	84.4	d	d	39.6	d	90.8	48.5	75.3	d
12/4/93	90.6	d	d	47.6	d	93.5	47.0	76.5	d
16/4/93	d	d	d	49.3	d	91.4	43.5	77.8	d
19/4/93	d	d	d	30.6	d	86.9	34.6	78.4	d
21/4/93	d	d	d	30.3	d	87.4	36.2	79.4	d
23/4/93	d	d	d	33.8	d	90.0	41.5	79.8	d
26/4/93	d	114.8	d	47.2	d	96.0	52.5	89.9	d
3/5/93	83.0	77.0	d	20.1	d	100.4	28.1	77.3	d
7/5/93	84.7	113.0	d	14.6	d	84.3	27.5	78.0	d
10/5/93	81.7	82.5	d	10.8	d	82.5	26.0	77.0	d
14/5/93	88.8	111.5	d	19.5	d	84.0	31.8	79.0	d
17/5/93	94.0	114.1	d	10.7	d	82.9	25.5	79.0	d
21/5/93	99.5	109.0	d	11.5	d	84.0	29.7	77.3	d
24/5/93	103.9	d	d	13.8	d	86.8	33.2	77.5	d
28/5/93	108.6	d	d	26.1	d	91.0	45.3	78.5	d
31/5/93	d	d	d	35.2	d	95.0	49.1	79.0	d
2/6/93	d	d	d	44.5	d	98.0	53.2	80.5	d
9/6/93	d	d	d	32.7	d	101.5	43.0	82.3	d
10/6/93	d	d	d	19.5	d	96.0	37.0	82.3	d
11/6/93	d	d	d	13.2	d	92.0	30.9	82.0	d
14/6/93	d	d	d	21.4	d	88.5	31.0	79.8	d

APPENDIX 7: SOME STATISTICAL PARAMETERS OF THE STORM DATA.

Table A7. 1. Rainfall storm durations and their corresponding parameters.

Storm duration (Minutes)	Mean amount (mm)	Number of storms analyzed	standard deviation (mm)	Average intensity (mm/hr)
15	2.6	104	2.04	10.30
30	2.7	65	2.33	5.34
45	2.6	46	2.48	3.48
60	4.3	35	3.84	4.29
75	4.8	25	4.34	3.84
90	8.4	19	7.51	5.59
105	15.0	9	13.47	8.55
120	8.5	31	8.26	4.22
135	8.8	6	6.69	3.90
150	14.4	5	14.25	5.78
180	11.4	20	10.61	3.80
240	16.9	7	12.13	4.23
300	11.1	7	6.55	2.22

APPENDIX 8. SOME SELECTED DESIGN STANDARDS

Table A8. 1 . Agricultural Land drainage standards

Land Potential	Crops	Design Flood Freq.not more than	
		Wet Period	Whole year
Very High	All Agricultural & Horticultural crops	No flood allowed	-say 1 in 100 years
High	Root crops, Cereals & grass	1 in 25 years	1 in 10 years
Medium	cereal, grass	1 in 10 years	1 in 5 years
Low	Grass	1 in 5 years	1 in 2 years
Very low	Grazing Land	1 in 3 years	1 in 1. year

(after Shaw, 1983)

Table A8. 2. Maximum Permissible velocities for various soil textures

Materials (bare channel)	Maximum permissible velocity (m/s)
Sand and Silt	0.45
Loam, Sand loam and Silt loam	0.60
Clay loam	0.65
Clay	0.70
Gravely Soil	1.00

(Adapted from Murty, 1985)