RE-DESIGN OF CHECK BASIN IRRIGATION SYSTEM FOR EDOZHIGI PILOT IRRIGATION PROJECT, GBAKO LOCAL GOVERNMENT, NIGER STATE, NIGERIA

BY

IBRAHIM NAKORDI MOHAMMED
M.ENG/SEET/2004/1149

A THESIS SUBMITTED TO THE POST-GRADUATE SCHOOL, FEDERAL UNIVERSITY OF TECHNOLOGY MINNA, NIGER STATE, NIGERIA. IN PARTIAL FULFILLMENT FOR THE REQUIREMENT OF THE AWARD OF MASTERS DEGREE IN AGRICULTURAL AND BIORESOURCES ENGINEERING.

(SOIL AND WATER ENGINEERING OPTION)

NOVEMBER 2009

DECLARATION

I, Ibrahim Nakordi Mohammed (M.ENG/SEET/2004/1149). Solemnly declare that this thesis ed "Re-Design of Check Basin Irrigation system for Edozhigi Pilot Irrigation Project, Gbako Local vernment, Niger State, Nigeria, is a study carried out by me in order to meet with the regulations verning the award of Masters Degree in Agricultural and Bioresources Engineering (Soil and Water gineering Option). Federal University of Technology, Minna.

29 / 3/ 09

....

BRAHIM NAKORDI MOHAMMED DATE

.ENG/SEET/2004/1149

CERTIFICATION

This thesis titled "Re-Design of Check Basin Irrigation System for Edozhigi Pilot Irrigation Project, Gbako Local Government, Niger State," by Ibrahim Nakordi Mohammed (M.ENG/SEET/2004/1149), in the Department of Agricultural and Bioresources Engineering, Federal University of Technology, Minna, which was approved for its contribution to knowledge and literary presentation.

.... 27/2/09 ....

Prof. O.J Mudiare — **_Date_**

Supervisor

.... 11/11/09 ....

Engr. Dr. Ayuba Balami — **_Date_**

Head of Department

.... 12/11/09 12/11/09. ....

Prof. O Usifo — **_Date_**

Ag. ***Dean, School of Engineering***

.... 9/2/2010 ....

Prof. S. Lamai — **_Date_**

Dean, Post Graduate School

DEDICATION

This project is dedicated to my parents, Late Alh. Mohammed Nakordi, Mallama Zainab M. Nakordi, Fatima M. Nakordi and Saratu M. Nakordi, for their support right from my childhood to date.

ACKNOWLEDGEMENT

My unlimited thanks goes to Almighty for giving me the strength and perseverance especially, throughout this trying period of my academic pursuit, more importantly, in writing this report.

I acknowledge with gratitude the kind, willing, understanding and able assistance offered me by my supervisor, Prof. O.J Mudiare, for his untiring efforts in going through this manuscript and making necessary corrections, constructive criticisms and supplying relevant information which would have been left out, that made this report a living reality. My sincere gratitude also goes to my Head of Department, Dr. Eng. Ayuba Balami, Departmental P.G Co-ordinator, Associate Prof. B.A Alabadan and other course lecturers; Prof. E.A.S. Ajisegiri, prof M.G. Yisa, Associate Prof. N.A Egharevba, Dr. D. Adgidzi, Dr. (Mrs) Z. D. Osunde, Dr. O. Chukwu, Mr. Peter Idah and the entire members of staff of Agricultural and Bioresources Engineering Department for their invaluable knowledge imparted on me which contributed in no small measure toward achieving this feat. May Almighty Allah reward you all.

I am highly indebted to my Director, Engr. N. Bima (Director Engineering Services, Niger State Ministry of Agricultural), Alh. Mohammed Ndako (Deputy Director), Abdulkareem Mairiga, and Mohammed Isah Kobo for their professional advice and contributions towards this research work.

My special appreciation goes to my wives, Amina Ibrahim, Saratu Ibrahim and all my brothers and sisters for their prayers and assistance towards achieving this goal.

Furthermore, I wish to express my acknowledgement to Dr. Isah N. Kolo and Dr. Aliyu Umar both of N.C.R.I. Badeggi for their contributions, all my friends and other well-wishers whose names are not mention here, but offered me assistance in different ways. My immense appreciation and acknowledgement alos goes to my project colleagues for their great concern about my success.

This work will be incomplete if I forget to appreciate the laudable efforts of my parents, Late Alh. Mohammed Nakordi, Hajiya Zainab M. Nakordi, Hajiya Fatima M. Nakordi, Hajiya Saratu M. Nakordi and Hajiya Aisha Yagaie, in giving me adequate moral and every kind of support right form my childhood to date. God bless you all (Amen).

ABSTRACT

From the analysis of the field data used for this research, the soil in the site was found to be andy loam, with Bulk density of $1.62g/cm^3$, Infiltration rate of 3.7cm/hr, Permanent wilting point of .94%, Field capacity of 7.49%, discharge through the main canal to be $0.6m^3/sec$, Crop water equirement and Consumptive use (ET) was also determined to be 353.844cm and 294.87mm/month. After redesigning, the values of infiltration rate and Permanent wilting point falls within the standard ange of 1.3-7.6cm/hr and 5-15%. It was also concluded that the Bulk density (BD) is influenced by the soil structure, texture and degree of compaction of the soil. The average monthly Irrigation frequency was found to be three (3) days and the Irrigation application time of 0.44hrs was redesigned. The economic analysis of the project gave a cost benefit ratio of 1:8 indicating that the project is viable.

TABLE OF CONTENT

LIST OF FIGURES

LIST OF TABLE

LIST OF PLATE

PLATE

PAGE

APPENDICES

CHAPTER ONE

1. INTRODUCTION

Irrigation can be used to supply water to an area where crops are grown, so as to reduce the length and frequency of the periods where lack of soil moisture is the limiting factor to plant growth (Ruthenberg, 1980). It can be total irrigation, when all crop water requirement is supplied through irrigation or supplemental when irrigation is applied in addition to natural precipitation or soil profile contribution to enhance crop yield.

The importance of irrigation development for food production is no longer an issue in Nigeria. The issue is how to sustain irrigated agriculture for the permanent benefit of the population. In the historical and current development of irrigation in Nigeria, it is noted that the setting up of the River Basin Development Authorities for irrigation was proposed to put about 2 million hectares of land under irrigation between 1980 – 1985 and beyond. To this end, substantial capital has already been invested in development of irrigation facilities.

Surface irrigation can be broadly classified as check basin irrigation, Border irrigation and Furrow irrigation. A surface irrigation event is composed of advance phase, wetting phase, depletion phase and recession phase. When water is applied to the field, it advances across the surface until the water extends over the entire area. Then the irrigation water either runs off the field or begins to pond on the surface. The interval between the end of the advance and the time the inflow is cut off is called the wetting or ponding phase. The volume of water on the surface begins to decline after the water is no longer being applied. It either flows from the surface (runoff) or infiltrates into the soil. For the purpose of describing the hydraulics of the surface flows, the drainage period is segregated into the depletion phase (vertical recession) and recession phase (horizontal recession). Depletion is the interval between cut off and the appearance of the first bare soil under the water. Recession begins at that point and continues until the surface is drained (Walker, 2007).

1.1 Statement of Problem

Edozhigi irrigation project has been in existence since 1966. It was first constructed during the colonial-era and reconstructed in 1980/81. Presently, most of the water control and conveyance structures have collapsed due to lack of maintenance. This is an ambitious scheme designed for 1,600ha of rice production. As an alternative, a pilot scheme was commenced near the command area since there is good and reliable source of water to supply the entire area and the topography is also favourable, but there has been no design plan.

Therefore, for effective performance of the scheme to guarantee large scale rice production, there is the need to have a proper re-design plan for the project.

1.2 Objectives

The general objective of this project is to re-design an effective surface irrigation method for large-scale rice production. The specific objectives include:

(i) To re - design a check basin irrigation system for the project area.

.(ii) To estimate the cost/benefit ratio in order to justify the feasibility of implementation of the scheme.

1.3 Justification

The focus of this project is to re - design an effective surface irrigation method that can be applied both now and in the future. Check basin system of surface irrigation was selected because it is highly economical, feasible, generally easy and cheap to install where conditions are favourable.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Check Basin Irrigation,

In check basin irrigation, the field to be irrigated is divided into units surrounded by small levees or dikes. Gated outlets, siphon tubes, spiles, and hydrants conduct water from delivery channels or pipelines into each basin (James, 1993). Basin may be either level or graded. In level basins, water is introduced into the basin as rapidly as possible and then held until it infiltrates or is drained away. High application efficiencies are possible primarily because runoff losses are minimized (James, 1993).

Graded basins are construction with two levees parallel and two perpendicular to field contours. Water enters graded basins along the upper contour and flows to the lower contour until the irrigation is completed. Water is then removed with surface drains located along the low contour levee. Graded basins are sometimes arranged in several rows or layers placed one above the other so that the drained water from upper basins is used to irrigate lower basins. For paddy rice, water is usually circulated through basins throughout most of the irrigation season. Graded check basin irrigation is sometimes called contour levee irrigation (James, 1993).

The field to be irrigated by the basin method is divided into level rectangular areas bounded by dikes or ridges. Water is turned in at one or more points until the desired gross volumes has been applied to the area. The flow rate must be large enough to cover the entire basin in approximately 60 to 75 percent of the time required for the soil to absorb the desired amount of water. Water is pounded until infiltrated (Jensen, 1993).

Check basin irrigation is the most common form of surface irrigation, particularly in regions with layouts of small fields. There are few crops and soils not amenable to check basin irrigation, but it is generally favoured by moderate to slow intake soils, deep-rooted and closely spaced crops. Crops which are sensitive to flooding and soils which form a hard crust following an irrigation can be basin irrigated by adding furrowing or using raised bed planting. Reclamation of salt-affected soils is easily accomplished with check basin irrigation and provision for drainage of surface runoff is unnecessary. Of course, it is always possible to encounter a heavy rainfall or mistake the cut –off time by having too much water in the basin. Consequently, some means of emergency surface drainage is good design practice. Basins can be served with less command area and field water courses than can border and furrow systems because their level nature allows water applications from anywhere along the basin perimeter (walker, 2007).

2.2 Infiltrations

Infiltration, usually defined as the entry of water into soil profile, is a process of great practical importance to irrigation design. It is the infiltration capacity of the soil that determines the rate at which water can be applied to the surface without runoff. Failure to adequately consider the infiltration process may result in non-uniform distribution of water in the field as well as excessive water loss due to deep percolation and runoff. Many of the soil-related factors that control infiltration also govern soil water movement and distribution during and after the infiltration process. Hence, an understanding of infiltration and the factors affecting it is important to the design and operation of efficient irrigation systems.

Walker (2006) stated that infiltration is the most important process in irrigation. It essentially controls the amount of water entering the soil reservoir, as well as the advance and recession of the overland flow. Irrigation of initially dry soil exhibits an infiltration rate with a high initial value which decreases with time until it becomes fairly steady, which is termed the basic infiltration rate. Infiltration is a complex process that depends upon physical and hydraulic properties of the soil, moisture content, previous wetting history, structural changes in the layers and air entrapment. In surface irrigation, infiltration rates. changes dramatically throughout the irrigation season. The water movements alter the surface structure and geometry, which in turn affects infiltration rates. The term 'intake' is often used interchangeably with infiltration, particularly where the geometry of the field influences the irrigation process.

2.2.1 **Infiltration function**

Both the procedures for interpreting filed data and those covering surface irrigation design require that infiltration be described mathematically. There are a number of mathematical equations to choose from, probably none is versatile as the Kostiakov – Lewis relationships (walker, 2006). The simplest approximation of cumulative infiltration is written as:

$$Z = Kt^{a} \qquad (2.1)$$

Where:- Z = cumulative infiltration in units of volume per unit area.

t = intake opportunity time in minute

K and a = empherical constants

Equation 2.1 is simple, easy to define, and widely used. Its major disadvantage is its inadequacy in describing infiltration over long time periods. The infiltration rate based on Eq. 2.1 is:

$$I = a\,Kt^{a-1} \qquad (2.2)$$

Since a is always less than unity, I approaches zero at infinite time. This is a condition not typically encountered in the field, since some soils do, however, have extremely small infiltration rates after a period of time Eqs. 2.1 and 2.2 can be used effectively (Walker, 2006).

2.3 Basin Sizes

Basin sizes are normally determined by the infiltration characteristics of the soil, the stream size and the type of soil. Relatively small basins are required on soils with high infiltration capacities, such as sands, even when large stream sizes are available. Basin on fine textured soils can be small or large depending on the stream size (James, 1993).

Basins vary in size from one (1) square meter (1m x 1m) for intensive crops such as vegetables to as much as 1 hectare for the production of rice and other crops. Many different crops including cotton, grains, maize, orchards, and pastures are suited to this system of irrigation (James, 1993).

Soils with high infiltration rates, such as sands require limited basin size even when large flows of water are available. Basins on clay soils can be large or small, depending on the water flow rate. Sandy loam soils with high infiltration rates permit only small size basins (Michael, 2006). The objective in selecting the basin size is to be able to flood the entire area in a reasonable length of time so that the desired depth of water can be applied with a high degree of uniformity over the entire basin (Jensen, 1983). The basin size of 4m x 4m is to be used for this project, base on the soil type (sandy loam) and infiltration rate as suggested by Michael, (2006).

2.4 Irrigation efficiency

The overall efficiency of a farm irrigation system is defined as the percent of water supplied to the farm that is beneficially used for irrigation on the farm. Overall system efficiency, also known as the irrigation efficiency, is defined mathematically by (James, 1993) as:

$$E_i = 100\left(\frac{I + L}{S}\right) \quad (2.3)$$

or

$$E_i = 100\left(\frac{S - DP - RO - O}{S}\right) \quad (2.4)$$

Where:- E_i = Irrigation efficiency (percent)

I = Irrigation requirement

L = Leaching requirement

S = Amount of water supplied to the farm

DP = Total deep percolation on farm

RO = Total run - off from farm

O = Operational losses due to planned and accidental Spillage from open channels and pipelines.

When evaluating the performance of a farm irrigation system, it is often useful to examine the efficiency of each system component. This allows component s that are not performing well to be identified. The system component include: reservoir, and the conveyance system.

2.4.1 Reservoir Storage Efficiency

The efficiency with which water is stored in a reservoir is reduced by evaporation and seepage losses. It is defined mathematically as:

$$E_r = 100 \left[\frac{1 - V_s + V_e}{V_i} \right] = 100 \left[\frac{V + \Delta S}{V_i} \right] \quad (2.5)$$

Where:- E_r = Reservoir storage efficiency in percent

V_e = Evaporation volume from the reservoir, m^3

V_s = Seepage volume from the reservoir, m^3

V_i = Inflow to the reservoir during a time interval, m^3

V_o = Outflow volume from the reservoir during a time interval, m^3

ΔS = Change in reservoir storage during the time interval, that is, amount of water needed to maintain the water surface in the reservoir at the level that existed at the beginning of the time interval. (S is negative when water must be added to the reservoir, and positive when water must be removed).

The ΔS term is often neglected when long time periods are considered. This term should not, however, be neglected for short time periods, (James, 1993).

2.4.2 Conveyance Efficiency

Water conveyance efficiency (E_c) is the ratio, in percent, of the amount of water delivered by a canal to the amount of water delivered to the conveyance system.

E_c is computed using (James, 1993):

$$E_c = 100 \left[\frac{V_{c2}}{V_{ci}} \right] \quad (2.6)$$

Where: E_c = Conveyance efficiency in percent

V_{co}= Volume of water delivered by conveyance system to the field (that is outflow) m^3

V_{ci} = Volume of water delivery to the conveyance (that is inflow),m^3

2.4.3 Water use efficiency (E_w)

This concept has two (2) classes: -

i) Crop water use efficiency: -

This is the ratio of crop yield 'Y' to the amount of water depleted by the crop in the process of evapo-transpiration (E.T)

$$E_u = \frac{Y}{E.T} \qquad (2.7)$$

Where: - E_u = crop water use efficiency.

ii) Field water use efficiency: -

This is the ratio of crop yield 'Y' to the total amount of water used in the field (W.R)

$$E_f = \frac{Y}{E.R} \qquad (2.8)$$

Where: - E_f = Field water use efficiency.

2.4.4 Water application efficiency

Water application efficiency for an irrigated area (E_a) is the ratio, expressed in percent, of the volume of water beneficially used by the crop to the volume of water delivered to the field. Water application efficiency can be computed for each field of the farm or for the entire farm. Water application efficiency E_a is computed using (James, 1993):

$$E_a = 100\left[\frac{V_{bu}}{V_a}\right] = 100\left[\frac{I + L}{V_a}\right] \qquad (2.9)$$

Where:- E_a = Application efficiency in percent

V_{bu} = Volume of water beneficially used by crop(s) in an area, m^3

V_a = Volume of water applied in an area, m^3

I = Irrigation requirement for the area

L = Leaching requirement for the area

The overall system efficiency is the product of the above efficiencies as (James, 1993):

$$E_i = \left[\frac{E_r}{100}\right]\left[\frac{E_c}{100}\right]\left[\frac{E_a}{100}\right]\left[\frac{E_w}{100}\right] \qquad (2.10)$$

Where:- E_i = Irrigation efficiency in percent

E_r = Reservoir storage efficiency in percent

E_c = Conveyance efficiency in percent

E_a = Application efficiency in percent.

E_w = Water use efficiency in percent

The objective of these efficiency concepts is to show where improvements can be made which will result in more efficient irrigation. Such concepts include: adequate planning of the irrigation system, proper design of the irrigation method, adequate land preparation and efficient operation of the system.

2.5 Crop Water Requirement

Crop water requirement is defined as the depth of water needed to meet the water loss through evapo-transpiration (ET) of a disease-free crop, growing in large fields under non-restricting soil conditions including soil water and fertility and achieving full production potential under the given growing environment

(Doorenbos and Pruitt, 1977). In the design of an irrigation system, evapotranspiration (ET) or consumptive use is the principal factor to be considered in determining crop water requirement. Losses in storage, conveyance, in applying water, inability to apply water uniformly and the need for soil leaching are additional factors.

The most commonly used empirical formulae in estimating evapotranspiration (ET) are the Blaney- Criddle (1950), Penman (1948), Thornwaite (1948), Christiansen (1968) and Bleney –Morin (1942). Recently, an evapotranspiration model which parallels that proposed earlier by Blaney – Morin was developed for application in Nigeria by Duru (1984). The model, designated as the Blaney-Morin- Nigeria evapotranspiration model, predicts potential evapotranspiration with accuracy and consistency that are better than the Penman's model, under Nigeria conditions (Duru, 1984).

Evapotranspiration is a very complex phenomenon, as evidenced by the wide variety of formulae used to estimate it. These formulae ranges from simple equations, expressing ET as a function of temperature alone, to models requiring more extensive data. The formula developed by Blaney – Cradle (1950) is an example of the former group of formulae, hereafter termed temperature-based models. The Penman's (1948) formula is an example of the latter. There is substantial evidence, however, that temperature - based ET models, though simple, are not sufficiently sensitive in areas where the temperature is relatively constant while other meteorological factors in area that also promote evaporation vary (Michael,1978, Hashemi and Habibian, 1979). Duru and Yusuf (1980) have shown this to be true under Nigerian conditions.

The need to be able to compute ET rapidly and accurately remains undisputed. In Nigeria, and perhaps in other developing countries, there is added need to compute ET from these meteorological parameters which can be easily measured. In other words, a model that is easy to apply and requires a minimum of the commonly available meteorological parameters is to be preferred over a more complex and sophisticated one with comparable accuracy of prediction. A modified form of the Blaney Morin ET model satisfies these requirements (Duru, 1984).

The modified Blaney – Morin - Nigeria (BMN) ET model was compared with observed open water evaporation and with the penman's ET model, considered by many to be the most rational one. Other commonly used ET models were not included in the comparison because Duru and Yusuf (1980) had earlier compared these models under Zaria, Nigeria, conditions and found the Penman model to give superior numerical prediction of ET. The comparative analysis done by Duru showed that the weakness of the Blaney-Criddle model, for Nigeria was identified as its sole dependence on temperature as a variable while Blaney – Morin model includes relative humidity, a parameter that varies over a wide range in Nigeria, both in time and space.

From design and safety standpoints, a model that over-predicts to a lesser degree should be preferred to one that under predicts. It is readily evident from Duru's (1984) finding, that the Blaney –Morin – Nigeria is a better predictor under Nigeria conditions than the Penman model. Blaney –Morin – Nigeria model is given as:

$$ET = \frac{r_f\,(0.45T + 8)\,(520 - R^{1.31})}{100} \qquad (2.11)$$

Where:- r_f = Ratio of maximum possible radiation to the annual maximum.

T = Temperature, ^0C

R = Relative humidity (%)

$$ET_c = ET_o \times K_c \quad (2.12)$$

Where: - ET_C = Consumptive use for a specific crop mm/ month

ET_0 = potential ET or reference crop ET

K_C = Crop co-efficient

2.6 Method of the soil conservation services (USDA, 1974)

Design equations are based on equating the volume of water applied to a unit width of basin strip during the time period of water advance from the head to the end of the strip, and the volume of intake plus the water in temporary surface storage during the same period. The designer must know the cumulative intake characteristics of the soil, must select a Manning's roughness co-efficient (n) appropriate for the crops to be irrigated, and must select the net application depth to be used as a basis for design.

2.6.1 Cumulative intake (mm)

The basis of the soil conservation service (USDA, 1974) design is to classify soils into intake families. The equation of the intake rates for these families is as follows:-

$$F = aT^b + c \quad (2.13)$$

Where: - F = is the cumulative intake in (mm)

T = is the time water is in contact with the soil (min)

a, b, and c are constants unique to each intake family. Values of the constants are given in Table G and the intake families are plotted in fig. G.

2.6.2 Intake opportunity time (T)

Intake opportunity time required for intake of the selected net application depth can be estimated by the solution of the cumulative intake equation in the form:

$$T = [(F - c)/a]^{1/b} \qquad (2.14)$$

in which the terms are as defined earlier.

2.6.3 Advance Time (T_t)

The time required for the inflow rate per unit width to advance to the far end of the strip is called the advance time, T_t (min). The required advance time T_t for any desired water application efficiency is determined by multiplying the net opportunity time, T, by the efficiency advance ratio, R (Jensen, 1983).

$$T_t = T_n \ x \ R \qquad (2.15)$$

Where:- T, = is the net application times

$R = T_t / T_n$ = efficiency advance ratio.

2.6.4 Basin Length and Inflow Rate

The following mass balance equation can be used to estimate length of the basin strip as a function of unit inflow rate (Q_u) and advance time (T_t) (Jensen, 1983):

$$L = \frac{6 \times 10^4 Q_u T_t}{\frac{aT_t^b}{1+b} + 7.0 + 1798n^{3/8} Q_u^{9/16} T_t^{3/16}} \qquad (2.16)$$

Where L = basin length (m)

Q_u = the unit inflow rate (m^2 / s)

T_t = the required advance time for the desired efficiency (min)

a, & b, = constants in the cumulative intake equation

n= Manning's roughness co-efficient

2.6.5 Inflow Time (T_a)

The inflow time, that is, the time required to apply the gross application onto the basin strip, can be computed using the equation given by Jensen (1983):

$$T_a = \frac{F_n L}{600 Q_u E_i} \quad (2.17)$$

In which T_a = is the inflow time (min) and other terms as previously defined.

2.6.6 Maximum depth of flow (d)

The maximum depth of flow can be estimated from the following equation given by Jensen (1983):

$$d = 2250 n^{3/8} Q_u^{9/16} T_a^{3/16} \quad (2.18)$$

Where: - d = is the maximum depth of flow at the inlet end of the basin strip (mm), and other terms as defined previously.

If advance time (T_t) is greater than inflow time T_a, T_t is used in Eq. (2.18) in place of T_a

2.7 Walker's Design Model (2007)

Check basin irrigation system is somewhat simpler than either furrow or border

Irrigation to design. Tail water is prevented from exiting the field and the slopes are usually very small or zero. Recession and depletion are accomplished at nearly the same time and nearly uniform over the entire basin. However, because slopes are small or zero, the driving force on the flow is solely the hydraulic slope of the water surface, and the uniformity of the field surface topography is critically important.

Water movement over the basin is assumed to occur in a single direction like that in furrows and borders. Three further assumptions are usually made specifically for check basin irrigation design (Walker, 2007), they are;

1. The friction slope during the advance phase of the flow can be approximated by:

$$S_f = d / X \quad (2.19)$$

Where:- d = the depth of the flow at the basin inlet, m

X = the distance from inlet to the advancing front, m

S_{fs} = the friction slope, and

$$Q_o = \frac{60d^{2.167}}{nX^{0.5}} \quad (2.20)$$

Where:- Q_o = is the discharge per unit width in m^3 /min/m, and other terms as previously defined.

2. Immediately upon cessation of flow, the water surface assumes a horizontal orientation and infiltrates vertically, that is, the infiltrated depth at the inlet to the basin is equal to the infiltration during advance, plus the average depth of water on the soil surface at the time the water completes the advance phase, plus the average depth added to the basin following completion of advance. At the downstream end of the basin the application is assumed to equal the average depth on the surface at the time advance is completed plus the average depth added from this time until the time of cutoff.

3. The depth of water to be applied at the downstream end of the basin is equal to the required

Infiltrated volume per unit length and per unit width (Z_{req}).

Under the above assumptions, the time of cutoff t_{co} for check basin irrigation system is evaluated with X equal to L (Walker 2007)

$$t_{co} = \frac{Z_{req} L - 0.77 d L}{Q_o} + T_L \qquad (2.21)$$

in which all the parameters are as previously defined. The time of cutoff must be greater than or

equal to the advance time.

As a guide to basin design, the following steps are outlined:-

i. Field slope will not be necessary because basin's are dead level.

ii. The required intake opportunity time (T_n) is to be determined.

iii. The maximum unit flow should be calculated along with the associated depth near

the basin inlet. The maximum depth can be approximated by using Eq. 2.17, and then perhaps

increased by 10 – 20% to allow some room for post-advance basin filling. If the computed value of d is greater than the height of the basin perimeter dykes, then Q_u needs to be reduced accordingly. The maximum unit inflow Q_u is difficult to assess. During the initial part of the advance phase, flow velocities will be greater than letter in advance because of the roughness nature of the soil surface, that is, it retards water movement. As a general guideline, it is suggested that Q_u be based on the flow velocity in the basin when the advance phase is one ninth completed (Jensen, 1983).

iv. Select several field layouts that would appear to yield a well organized field system and for each determine the length and width of the basins.

(v) Compute the advance time, T_t, for each field layout, cutoff time, t_{co}, from Eq. (2.21) (if $t_{co} < T_t$, set $t_{co} = T_t$) and application efficiency using Eq. 2.14.

The layout that achieves the highest efficiency while maintaining a convenient configuration for the irrigator/farmer should be selected (Walker,2007).

Water should be applied at a rate that will advance over the basin in a fraction of the infiltration time to achieve high efficiency. The volume of water applied must be equal to the average gross application. The intake opportunity time at all point in the basin must be greater than or equal to the time required for the net irrigation to enter the soil. The longest intake opportunity time at any point on the basin area must be sufficiently short to avoid scalding and excessive deep percolation. The depth of water flow must be contained by the basin ridges (Jensen 1983).

2.8 Applicability

Most crops can be irrigated with check basin irrigation. It is widely used for close-growing crops such as alfalfa and other legumes, grasses and rice. It is used for row crops that can withstand some inundation, such as sugar beets, corn, grain sorghum, and cotton and for other row crops. Also, it is well suited for irrigation of tree crops such as grapes, and barriers (Jensen, 1983). This irrigation method is the best suited to soils of moderate to low intake rate of 50mm/h or less. It is an excellent way of applying water to soils that have a moderately high to high in intake rate, but basin areas may need to be very small.

Check basin irrigation is best suited to smooth, gently, uniform land slopes. Undulating or steep slopes can be prepared for check basin irrigation, provided the soils are deep enough to permit the needed land leveling (Jensen 1983).

2.9 Advantages

High application efficiency can be obtained easily with little labour. Check basin irrigation can be used efficiently by inexperienced workers, and can easily be automated. When basins are leveled with laser-controlled scrapers, basins can be as large as 16ha (Erie and Dedrick, 1979). Many different kinds of crops can be grown in sequence without major changes in design, layout, or operating procedures. There is no irrigation run –off and the deep percolation loss is small, if no excess water is applied, and maximum use can be made of rainfall. Leaching is easy and can be done without changing the layout or operation method (Jensen, 1983).

2.10 Disadvantages

The principal disadvantage of check basin irrigation is that levees interfere with the movement of farm equipment. The presence of levees and ditches can also reduce the area available for crop production (James, 1993).

Accurate initial land leveling is essential and level surface must be maintained. Adequate basin ridge height may be difficult to maintain on sandy soils and fine textured soils that crust or crack when dry. Prolonged ponding and crop scalding can occur if the system is poorly managed. In some areas special provision must be made for surface drainage.

CHAPTER THREE

MATERIALS AND METHODS

3.1 General Description of the Project Site

The Project is Edozhigi pilot Irrigation scheme, located between latitude 8^0 $4\ ^1$N and longitude $6^0 3^1$E and about 12km south-east of Bida- Kutigi – Mokwa road and taking a laterite road by the signboard opposite Etsu Nupe's farm before Wuya. The pilot scheme covers a total area of 100ha of rice cultivation.

The area is located in the Guinea savanna zone in the middle belt of Nigeria where the vegetation type is a proportionate combination of trees and grasses. It consists of trees such as locust bean and shear nut tree in a scattered form. Grasses abound and the vegetation appears to be park land. The climatic type is the tropical inter-land where rainfall of between 6 to 8months is enjoyed and a dry season of between 4 to 5 months usually between November and March. The temperature is high throughout the year and ranges between 27^O C and 32^o C. The rainfall pattern influences to a great extent the agricultural practice in the study area. Most of the farming which is rainfed is carried on during the rainy season months of April to October while the irrigated agriculture takes over during the dry seasons when water needs are at its critical point. The terrain is generally a low lying area with almost all of it being a plain, that is, either level or undulating. This makes it substantially fevourable and suitable for siting of irrigation schemes.

3.1.1 Source of Water for the Project

The water source for the proposed project is River Ejiko which is 1 km from the field guided by a canal and located at a height in such a way that the basins can be irrigated by gravity.

3.2 Soil Texture

Soil samples used for this experiment were taken in such away as to represent the entire field U.S.D.A method of soil classification was used to determine the soil textural class.

3.3 Parameters Determination

The bulk density, infiltration rates, field capacity and permanent wilting points were determined from field as follows:.

3.3.1 **Bulk Density**

This was determined by the undisturbed core method as described by vomocil (1954). In this method, twelve (12) soil samples obtained from the field after weighing were transferred to an oven at 105°C and left there for 24hrs. They were weighed again to determine their oven dry weight (ODW). Bulk density is the oven dry mass divided by the volume of the sample. The expression is given by Vomocil (1954):

$$B.D = \frac{O.D.W}{V} \qquad (3.1)$$

Where:- $O.D.W$ = Oven dry weight, gm

$B.D$ = Bulk density gm/cm^3

V = Volume of core cylinder cm^3

3.2.? **Infiltration Rates**

This was determined by a double ring infiltrometer using the Richard (1954) method. Double ring infiltrometer with the following dimensions was used for the experiment: diameter of the inner ring and outer ring is 35cm, and 45cm, respectively, the height of each of them was 45cm. The inner cylinder was first driven vertically downwards into the soil to depth of 15cm by the use of a wooden hammer and then the outer cylinder to the same depth. Enough water to fill each ring to a datum point was added and the water level in the inner ring measured at intervals, as the water infiltrated into the soil. The water level dropped and the time of measuring was recorded.

3.2.3 Field Capacity

In the lab, water was added to the twelve (12) samples collected from the field until saturation. The samples were allowed to drain freely and left for forty eight (48) hours after which they were transferred to the oven at 105 °C for twenty four (24) hours. The moisture content after drainage for 48hrs is the field capacity. The expression is given by Briggs and Shantz (1912):

$$F.C.P = \frac{W.F.C - O.D.W}{O.D.W} \times 100\% \qquad (3.2)$$

Where $F.C.P$ = Filed capacity percentage

$W.F.C$ = Weight at field capacity (fresh weight) gm

$O.D.W$ = Oven dry weight gm

3.2.4 Permanent Wilting Point

This was determined by the modified laboratory method of Briggs and Schantz (1912). The twelve (12) samples collected from the field were placed in containers which is open at the bottom. Water was added until saturation and free drainage was allowed to take place. Maize seed were selected and two (2) seeds were planted on each sample and regularly watered until germination was attained in five days (5). The surface was sealed with candle wax to prevent evaporation. The plant was left to wilt. The plants become permanently wilted at exactly seventeen (17) days. The soil moisture content was determined to be the permanent wilting percentage. The expression is given by Briggs and Shantz (1912):

$$P.W.P = \frac{W.F.C - O.D.W}{O.D.W} \times 100\% \qquad (3.3)$$

Where:- $P.W.P$ = Permanent wilting point

$W.P.W$ = Weight at permanent wilting point (fresh) gm

3.4 Topographic Survey

Land leveling survey of the project area was carried out using a leveling instrument, tripod, staff, ranging poles, chain, measuring tape and cutlass. The readings obtained were used to plot the topographic map.

3.5 Crop Water Requirement

Climatological data for 30 years (1975 – 2004) were obtained from the National Cereals Research Institute (NCRI), Badeggi Meteorological station, for the determination of crop water requirement.

3.6 Determination of Ponding Depth

Ponding was embarked upon by constructing 6 basins of 4m x 4m. Rice was planted in the 6 basins with varied ponding water depths of 6cm, 12cm, 18cm, 24cm, 30cm and 36cm, in order to ascertain the depth that will give the best yield. Pegs were fixed in each plot marked with paint to the desired depth. Water level was maintained in each plot throughout the planting period from 1st November 2006 to 11 April 2007. The depth of water that gave the highest yield (30cm) was taken as the required ponding depth.

3.7 **Basin Design Considerations**

Efficient irrigation by the basin method depends on the knowledge of the hydraulics of flow in the basin. The hydraulics of flow in basin may be considered to comprise of (Michael,1978):-

(i) Initial spreading of the entrance stream to cover the full width of the basin and simultaneous advance of the irrigation stream.

(ii) Advance of the water front after the initial spreading.

(iii) Rise of the water level after the advancing stream reaches the down stream

end, and

(iv) Subsidence of water after the irrigation stream is stopped.

If the check basin irrigation system is properly designed, it is possible to apply the right amount of water nearly uniformly throughout the basin. The problem of efficient irrigation by basin consists essentially of having the right size of basin to suit the available stream size for a particular set of soil and crop conditions.

Other variables to be considered in check basin irrigation design include (Jensen, 1983)

(i) Opportunity time (T_n) required for intake of the selected net application depth.

(ii) Basin length (L)

(iii) Inflow time required (T_a)

(iv) Maximum depth of flow (d).

3.7.1 Design Assumptions

For the design, the following assumptions are made:-

(i) Good quality and sufficient quantity of water for the project

(ii) Irrigation efficiency = 80%

(iii) Roughness co-efficient (n) for excavated earth canals straight and well maintained =0.023 (Chow, 1960)

(iv) Canals slope(s) = 0.001

(v) Side slope = 1.5:1 for shallow channels using sandy loan material.

3.7.2 Design Limitation

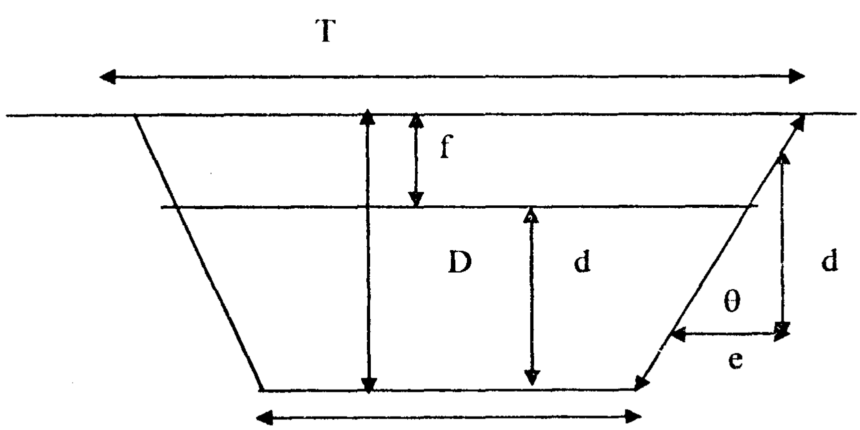
In theory, maximum depth of flow and maximum deep percolation both occur where water is introduced into a basin, usually considered as a "strip" of unit width for computational purposes. For any given set of site conditions, the depth of flow varies directly and the amount of deep percolation varies inversely with the inflow rate per unit width of basin strip. Thus, if a limit is set on flow depth, deep percolation may be reduced only by shortening the length of the basin strip. If limits are established for both depth of flow and deep percolation, then the design limit for length is determined (Jensen,1983). Flow at the head end of basin strips must not exceed some practical depths related to the construction and maintenance of basin ridges (Jensen, 1983). The average deep percolation should be, minimized. On some sites excess deep percolation causes acute drainage problems. In order to avoid this condition, the design efficiency usually should not be less than about 80% (Jensen, 1983). This efficiency can be obtained if the time required to cover the basin is not more than 60 percent of the time required for the net application to infiltrate the soil. A design efficiency of less than 70 percent should be considered only for soils having excellent internal drainage (Jensen, 1983) on sites where irrigation water supplies are limited or costly, where subsurface drainage problems are acute, or where crops can be damaged by prolonged surface flooding, design efficiencies in excess of 90% are often practical. These efficiencies are easily obtained when laser-controlled scrapers are used for land leveling (Erie and Dedrick, 1979).

Basin strips usually are designed to be level; however, they may be constructed with a slight grade in the direction of water flow. A slight grade in low areas or reverse grades, which result in a slower rate of advance, reduce efficiency, excessive deep percolation or prolonged flooding that may damage crops. The total fall in the length of the basin strips should not be greater than one-half the net depth of application used as a basis for design. No adjustment is made in the design to compensate for such slight grades.

Basin ridges, or levees, should be constructed so that the top width is at least as great as the ridge height. The settled height should be at least equal to the greater of (a) the design gross depth of application, or (b) the design maximum depth of flow plus a free board of 25 percent of the maximum depth of flow (Jensen, 1983).

3.7.3 Design Equations

(i) Channel Cross- Section:



b = Fig. 3.1

$$Q = A.V \tag{3.4}$$

Where: Q = Discharge in m^3 /sec

A = Cross sectional areas in m^2

V = Velocity of flow in m/sec.

$$A = d(b + zd) \tag{3.5}$$

Where:- A = Cross sectional area of flow in m^2

b = Bottom width of the channel in m

d = Flow depth in m

z = Side slope

$$P = b + 2d\sqrt{Z^2 + 1} \tag{3.6}$$

P = Wetted perimeter in m

b,d,z as defined above

$$R = \frac{A}{p} \tag{3.7}$$

Where:- R = Hydraulic radius in m

A and P as defined above

$$V = 1/nR^{2/3}S^{1/2} \tag{3.8}$$

Where:- V = Velocity of flow in m/s

n = The mannings roughness co-efficient

R= Hydraulic radius in m

S= Canal bed slope

$$D = 1,25d \tag{3.9}$$

D = Total depth of the channel from top to bottom, (m)

d = As previously defined

$$T = b + 2DZ \tag{3.10}$$

Where:- T = Top width of the channel, (m)

$$f = D - d \qquad (3.11)$$

Where:- f = Free board (m)

(ii) Design Discharge

$$Q = \frac{q.A}{1000} \qquad (3.12)$$

Where:- Q= Basin discharge, (m^3/sec)

q= Drainage co-efficient

A = Area in ha

(iii) Advance Time

$$T_t = T_n \text{ x R} \qquad (3.13)$$

Where:- T_t = Advance time minute

T_n = Net application time in minute

$R = T_t / T_n$ = efficiency advance ratio from table 3.1

(iv) **Infiltration Opportunity Time**

$$T = [(F\text{-c})/a]^{1/b} \qquad (3.14)$$

Where: - T = Opportunity time required for intake of the selected net application

depth in min

F = The desired net application depth in m

(v) Basin Length

$$L = \frac{6 \times 10^4 Q_u T_t}{aT_t^b + 7.0 + 17987^{3/8} Q_u^{9/16} T_t^{3/16}} \qquad (3.15)$$

$\overline{1+b}$

Where:- L = Basin length in m

Q_u = Unit inflow rate m^3/sec

T_t = Required advance time for the desired efficiency in min.

n = Manning's co-efficient

(vi) Inflow Time

$$T_a = \frac{F_n L}{600 Q_u E} \qquad (3.16)$$

Where:- T_a = The inflow time for the unit flow rate in min.

F_n = Net application depth mm

L = Basin length in m

E = Efficiency in %

(vii) Maximum depth of flow

$$d = 220n^{3/8}\,Q_u^{9/16}\,Ta^{3/16} \qquad (3.17)$$

Where:- d = the flow depth at the inlet end of the basin strip in m.

n,Q_u,T_a = As defined above

3.6.4 Formulae used in the computation of soil parameters

A. Determination of soil texture

This was determined using Bouyoucos Hydrometer method of (1951). To this end, 50gm of air-dry soil was weighed into a beaker and 50ml of calgon (sodium hexametaphosphate) was added, stirred and left overnight. The solution was stirred for 10 – 15min and then transferred to a 1000ml glass cylinder. Distilled water was added to the 1000ml mark. The cylinder was inverted to mix the solution with the added water using hand to close the mouth of the cylinder. The hydrometer was inserted in the cylinder and made up until it was suspended.

The temperature T_1 and hydrometer reading H_1 were taken after 40 sec. The hydrometer was allowed to remain in suspension for three (3) hours after which its reading T_2 and H_2 were taken by the use of thermometer.

(i) Percentage sand:-

$$= 100 - [H_1 + 0.36]T_1 - 20^0 C\ ] - 2.0\ ]2 \qquad (3.18)$$

(ii) Percentage clay:-

$$= H_2 + 0.36[T_2 - 20^0 C] - 2.0] \qquad (3.19)$$

(iii) Percentage silt:-

$$= 100 - [\text{Percentage sand + clay}] \qquad (3.20)$$

Where: - H_1 and H_2 are first second hydrometer readings at 40secs and 3hrs, respectively.

T_1 and T_2 are first and second temperature readings at 40secs and

3hrs,

respectively.

Soil textural triangle (U.S.D.A, 1951) was used to know the soil textural class.

B. $$A.W = D_{rz}\ (F.C - P.W.P)\ /100\ (\text{cm}) \qquad (3.21)$$

Where: - $A.W$ = Available Water

D_{rz} = Depth of the root zone, (cm)

C. $$F_n = AW\ x\ M.D \qquad (3.22)$$

Where: - F_n = Net water application, (cm)

$M.D$ = Moisture depletion usually taken as 50% Larry (1988)

D $$F_g = \frac{N.W.A}{E_l} \quad (3.23)$$

Where: - F_g = cross application in cm

E_l = Irrigation efficiency, (%)

E $$T_n = \frac{F_g}{I} \quad (3.24)$$

Where: - T_n = Irrigation application time (hr)

I = Infiltration rate, cm/(hr)

F. $$V = \prod r^2 h \quad (3.25)$$

Where: - V = Volume of core cylinder, (m^3)

r = Radius of circular base cm

h = Height of cylinder cm

G. $$I.F = \frac{F_n}{ET_0} \quad (3.26)$$

Where: - $I.F$ = Irrigation frequency (days)

ET_0 = consumptive use mm/day

H. $$E.T = r_f(0.45T + 8)(520 - R^{1.31})/100 \quad (3.27)$$

All parameters as previously defined.

3.8 ESTIMATION OF QUANTITIES AND COSTS

3.8.1 Economic Feasibility Assessment

Evaluation of the economic feasibility of an irrigation system requires estimating all of the costs and returns expected from the development. These economic studies should include comparisons of the costs and returns for the viable alternative farm irrigation and cropping systems, and be a part of the feasibility report for an economic feasibility report for a farm or project development. The results of an economic feasibility study will provide the farmer with the necessary information as to the attractiveness of proceeding with an irrigation development and for selecting the farm irrigation and cropping system (Jensen, 1983).

To estimate the Benefit-cost ratio, it is essential to convert all investment costs into annual costs. The capital or investment costs have to be recovered during the project life with a certain minimum attractive rate of return.

The capital-recovery factor (CRF) is used to convert a capital investment (CI) into an equivalent annual costs:

$$CRF = \frac{i(1+i)^n}{[(1+i)^n - 1]} \quad (3.28)$$

Where i is the interest rate per annum, n is the estimated lifespan (years) of the project.

Equivalent annual recovery cost (EARC) = CRF x CI (3.29)

Total annual costs (TAC) = EARC + Annual recovery (operational maintenance cost) (3.30)

$$\text{Benefit – cost ratio (B/C)} = \frac{\text{Discounted annual Benefit}}{\text{Discounted annual cost}} \quad (3.31)$$

B/C > 1, project is economical feasible

B/C < 1, project not viable

However, considering the social services/political issues of irrigated agriculture, Benefit-cost ratio equal to or greater than 1 (B/C = 1) may be adequate for a start.

Note: -

For irrigated project (usually), B/C ≥ 1.5 (Anova, 2006).

CHAPTER FOUR

4 RESULTS

4.1 Soil Physical Properties

Table 4.1: Soil textural Classification

Sample Number	Soil depth (cm)	% Sand	% Silt	% Clay	Textural Class
P_1	0 -15	70	28	2	Sandy Loam
	15 -30	69	24	7	”
	30 -45	68	20	12	”
	45 -60	67	16	17	”
P_2	0 - 15	91	8	1	Sand
	15 – 30	74	22	4	Sandy Loam
	30 – 45	58	35	7	”
	45 – 60	42	48	10	Loam
P_3	0 – 15	78	19	3	Sandy Loam
	15 – 30	75	21	4	”
	30 – 45	72	23	5	”
	45 – 60	69	25	6	”

Source: U.S.D.A Classification

4.2 Bulk Density (B.D), Field Capacity (FC) and Permanent Wilting point (PWP) are shown in table 4.2

Table 4.2: Bulk Density (B.D), Field Capacity (FC) and permanent Wilting Point (PWP)

Sample Number	Soil Depth (cm)	Bulk Density (g/cm3)	Field Capacity (Weight basis) (%)	Permanent Wilting Point (Weight basis) (%)
P_1	0 – 15	1.51	13.79	0.66
	15 – 30	1.48	9.84	4.43
	30 – 45	1.44	5.89	7.19
	45 – 60	1.40	1.94	9.30
P_2	0 – 15	1.43	11.93	3.11
	15 – 30	1.76	8.32	2.61
	30 – 45	2.09	4.71	2.31
	45 – 60	2.42	1.1	2.10
P_3	0 – 15	1.72	11.90	0.26
	15 – 30	1.56	5.45	0.14
	30 – 45	1.40	-	0.36
	45 – 60	1.24	-	0.79

Bulk density, field capacity and permanent wilting point= 1.62g/cm^3,7.49% and 1.94 % respectively.

4.3 Infiltration Rate (I)

The result of the infiltration rate experiment described in section 3.2.2 is given in Table

Table 4.3: Infiltration characteristic I

Time (Min) rate	Infiltration (cm)	Cumulative Infiltration	Infiltration (cm/hr)
0	0.0	-	-
10	2.0	2.0	12
20	1.25	3.25	7.5
30	1.0	4.25	6
40	0.75	5.0	4.5
50	1.0	6.0	6
60	1.0	7.0	6
70	0.75	7.75	4.5
80	0.75	8.5	4.5
90	0.70	9.2	4.2
100	0.70	9.9	4.2
110	0.68	10.58	4.1
120	0.65	11.23	3.9
130	0.63	11.86	3.8
140	0.62	12.48	3.7
150	0.62	13.1	3.7
160	0.61	13.71	3.7
170	0.61	14.32	3.7
180	0.60	14.92	3.6

The value of basic infiltration rate = 3.7cm/hr

4.4 Depth of Ponding experiment

Table 4.4: Depth of Ponding.

Plot Number (Kg/ha)	Water Depth (cm)	Yield
1	6	6.4
2	12	16
3	18	21
4	24	23
5	30	24
6	36	20

Required depth of ponding = 30cm which gave a yield of 24Kg/ha

4.5. Construction Cost

S/NO	Description	Quantity	Unit	Unit Rate ₦ K	Amount ₦ K
1.	Survey:-				
	Level survey of the entire area to produce the contour map used for the design work				
	a. Survey work for 5days	2	Men	1,500.00	15,000.00
	b. Chaining for 5 days	4	Men	7,00.00	14,000.00
	c. Draughtsman for plotting	1	Man	4,000.00	4,000.00
	d. Draughtsman for tracing	1	Man	1,000.00	1,000.00

2. Construction:-

Canal excavation by volume and embankment compaction.

a. Main canal

- Area = $0.3963m^2$ and 1,300m long.	515.19	M^3	170.00	87,582.30
b Field Channel: -				
- Area = $0.0753m^2$ and 400m long /one channel	30.12	M^3	170.00	5,120.40
We have 6 equal number of field channels	180.7	M^3	170.00	30.772.40
c. Field drain:-				
- Area = $0.1564m^2$ and 1,300m long /one	203.32	M^3	170.00	34,564.40
- for the two (2) field drains	406.64	M^3	170.00	69,128.80
d. Distribution box:-				
- Construction, labour and material	1	No	Lumpsum	15,000.00
- for the total number of 3 we have	3	No	15,000.00	45,000.00
e. Stilling Basin:-				
- Construction, labour and material	1	No	Lumpsum	50,000.00
3. Procurements:-				
- Irrigation equipments				
a A.c pipe of 5cm diameter for the field drains	2	No	3,500.00	7,000.00
b A.c pipe of 13.5mm diameter for the Field channels	10	No	8,000.00	80,000.00
c. H.R.3 Irrigation pump	2	No	500,000.00	1,000,000.00
			TOTAL =	**N1,403,433.50k**

4.6 Operation Cost

S/NO Description	Quantity	Unit	Unit Rate ₦ K	Amount ₦ K
1. Crop production cost:-				
a. Land preparation	1	ha	15,000.00	15,000.00
b. Seed for planting	1	ha	3,000.00	3,000.00
c. Planting	1	ha	2,000.00	2,000.00
d. Weeding	1	ha	5,000.00	5,000.00
e. Harvesting	1	ha	3,000.00	3,000.00
f. Threshing	1	ha	3,000.00	3,000.00
g. winnowing	1	ha	2,000.00	2,000.00
h. Bagging	1	ha	4,000.00	4,000.00
				37,000.00
- For the total land area	100	ha	37,000.00	3,700,000.00
2. Fertilizer:-				
a. 4 bags of N.P.K is required transportation and handling charges inclusive	1	ha	3,000.00	12,000.00
b 2 bags of urea required, including transportation & handling charges	1	ha	4,000.00	8,000.00
				20,000.00
- For the total land area	100	ha	20,000.00	2,000,000.00
3. Water supply to the field				
a. 2 irrigators are required	1	day	700.0	1,400.00
b. one (1) pump operator	1	day	1,000.00	1,000.00
Water supply throughout the irrigation period	162	days	2,400.00	388,800.00
4. fuel to be used for the operation				
a. Tank capacity	32	litres	70.00	2,240.00
b. full tank throughout the irrigation period	41 times		2,240.00	91,840.00
5. MAINTENANCE:				
a. Seasonal maintenance of the main canal				

field channels, drains and all the irrigation

structures for 4 weeks 10 Men 700.00 210,000.00

b. pump maintenance including cost

of servicing and labour per cropping season 5 Months Lumpsum 34,000.00

TOTAL = 6,424,640.00

4.7 Output

Description	Quantity	Unit	Unit Rate ₦ K	Amount ₦ K
1. Yield of rices in tones per hectare under normal circumstances is estimated to be 2 tonnes (40bags)	1	ha		
a. Total yield expected for the total land area of 100 ha = 4,000 bags				
b. Cost of one bag of unmitted rice	100	kg	3,000.00	3,000.00
c. Cost per hectare (40 bags)	4,000	kg	3,000.00	120,000.00
d. total land of 100ha (4,000bags)	4,000,000	kg	3,000.00	12,000,000.00
		TOTAL =		**12,000,000.00**

Equipment cost = N1,000,000.00 - Lifespan 5years

Construction cost of = N403,433.50- Lifespan 30years

Operation and Maintenance = N6,424,640.00

. Equipment cost.

$$\text{Capital-Recovery Factor (CRF)} = \frac{i(1+i)^n}{[(1+i)^n - 1]}$$

$$CRF = \frac{0.18(1+0.18)^5}{[(1+0.19)^5 - 1]} = \frac{0.4118}{1.2878} = 0.3198$$

Capital investment (CI) = N1,000,000.00

Equivalent annual recovery cost (EARC) = CRF x CI

EARC = 0.3198 x N1,000,000.00 = N319,800.00

EARC = N319,800.00

ii,Construction Cost

Capital recovery factor (CRF) = 0.1813 from equation ---- I = 0.18, n = 30years

Capital investment (CI) = N403,433.50

Equivalent annual recovery cost (EARC) = CRF x CI

Therefore, EARC = 0.1813 x N403,433.50 = N73,142.49

EARC = N73,142.49

Total EARC (i + ii) = N319,880.00 + N73,142.49 = N392,942.49

ii. Total annual cost TAC)

TAC = EARC + Annual recurring (operation & Maintenance cost)

= N392,942.49 + N6,424,640.00

= N6,817,582.49

iii. Benefit – Cost ratio (B/C) = $\frac{\text{Discounted annual Benefit}}{\text{Discounted annual cost}}$

= $\frac{\text{N12,000,000.00}}{\text{N6,817,582.49}}$ = 1.76

Since benefit – cost ratio is operation item one (1) (B/C ratio = 1.76), the project is economically viable.

Table 4.5 shows construction cost, 4.6 operation cost, 4.7 the output cost. From table 4.5, the total cost of construction was N1,403,433.50, the operation cost from table 4.6 was N6,424,640.00 and the output cost from table 4.7 was N12,000,000.00, this lead to a cost benefit ratio of 1.76.

4.8 SUMMARY OF DESIGNED VALUES

Summary of the designed values for the irrigation canals, field drains and basins as obtained from the preceding calculations are as follows:

Table 4.8.1: Channel Design

	Q (m^3/sec)	D(m)	d(m)	b(m)	f(m)	T(m)
Main Canal	0.6	0.49	0.39	0.2361	0.1	1.71
Field Channel 1 - 6	0.1	0.21	0.17	0.1029	0.04	0.73

Table 4.6.2: Field drain design:

	Q (m^3/sec)	D(m)	d(m)	b(m)	f(m)	T(m)
Field drain 1 - 2	0.25	0.31	0.245	0.1483	0.07	1.08

Table 4.8.3: Basin design for the 6 plots

	Q (m^3/sec)	T_n(min)	T_t (min)	T_a (min)
L(m d(m)				
Field channel 1 – 6 (Plot 1 - 6)	0.006	13	8	4.9
57 41.5				

See appendix C for calculations

CHAPTER FIVE

5.0 DISCUSSION OF RESULTS

From table 4.1 , the textural class of soil in the project site is sandy loam, this values were in agreement with the USDA textural classification. Appendix K. From table 4.2 above, the soil bulk density (BD) in influenced by the texture, structure and degree of compaction of the soil. The average value of Bulk density was 1.62g/cm^3. The values of field capacity of 7.49% fall within the standard range of – 15 % suggested by Michael (1978).

The value of permanent wilting point obtained from the experiment was 1.94%.Though, permanent wilting point is not a soil constant or a unique soil property. There is no single soil water content at which plant cease to withdraw water even though wilted, plant will absorb water, but not at rates sufficient to regain tugor (Hansen, 1983).

Table 4.4 shows value of plots, water depths (cm) and yield (kg/ha). From the table the estimated ponding depth is 30cm with a yield of 24kg/ha, this is the depth that produce the highest yield.

(i) The designed depth of water to apply was calculated to be 31.053cm, Inflow rate/time (T_a) is 4.9min, Available water (AW) was obtained to be 3.33cm, Net water application (F_n) is 1.7cm, Gross water application (F_g) is 2.13cm and Irrigation application time (T_n) is 0.58hrs. Appendix B_2 and E shows detail calculations of these parameters. These parameters were used in my design.

5.1 CONCLUSIONS AND RECOMMENDATIONS

5.1.1 Conclusions

Based on the findings in this study, the following conclusions are made:

1. Values of Field capacity, Permanent wilting point, Bulk density, Infiltration rate of 7.49%, 1.94%, 1.62gm cm^{-3} and 3.7cm/hr respectively, falls within the standard limit.
2. With the average monthly irrigation frequency obtained of three (3) days, the monthly irrigation frequency can serve as a guide.
3. With the discounted annual cost of the project at N6,817,582.49 and discounted annual benefit of N13,000,000.00, the cost benefit ratio is 1:76, that is, the project is viable.

5.1.2 5.1.2 Recommendations

1. Since the irrigation application time was calculated to be 0.58hrs, to achieve the desired irrigation efficiency, therefore, the irrigator must not deviate from said time by more than 20minutes.
2. The monthly irrigation frequency is recommended as a guide to the irrigator.
3. With value of the cost benefit ratio of 1:76, the project viability is recommended, hence a quick return on investment.

REFERENCES

Aisenbrey, A.J, Jr., R.B, Hayes, H.J, Warrem, D.L, Winsett, and Young, R.B. (1979): Design of Small Canal Structures, U.S. Dept. of the Interior Bureau of Reclamation, Denver, Colo, P.435.

Benami, A. and Ofen, A. (1993): Irrigation Engineering. Plates and Printing: Ram Offset Ltd., Tel – Aviv, Israel. P. 1-16.

Booher, L.J. (1974): Surface Irrigation, FAO (Food and Agriculture Organisation of the United Nations). Agr Develop Paper NO 95, P.80-92.

Bouyoucos, G.J (1951): A recalibration of the Hydrometern Method for Making Mechanical Analysis of Soils Agron J. 43:434-438.

Briggs, L.J and Shantz, H.L (1912): The wilting Co-efficient of Different plants and its Determination, U.S.D.A Agr Bur. Plant. Ind. Bul. P.230.

David, W.R. (1983): Land Drainage Planning and Design of Agricultural Drainage Systems. P. 167.

Dedrik, A,R, Erie, L.J. and Clemens, A.J. (1982): Level-basin irrigation. In Advances Irrigation, Vol. 1. D. Hillel (Ed.), Academic Press, New York, P. 105-145.

Doorenbos, J. and Pruitt, W.O. (1977): Guideline for Predicting Crop Water Requirement. Irrigation and Drainage Paper No. 24 of the Food and Agricultural organisation of the United Nations P.71.

Doorenbos, J. and Kassam, A.H. (1979):Yield Response to Water. Irrigation and Drainage Paper No. 33 of the Food and Agricultural Organisation of the United Nations. P. 125-130.

Duru, J.O. and Yusuf, S.A. (1970): Potential of Temperature-Based Methods in Estimating Evapotranspiration in Arid-Zone. Agricultural Development Project. Agric Meteorol 20: P. 237-247.

Duru, J.O. (1981): A simple Evapotranspiration Model for Northern Nigeria, Paper Presented at 5th Annual Conference Nigerian Society of Agric Engineers, University of Ife, Ile-Ife, Aug. 26-28.

Duru, J.O. (1984): Blaney-Morin-Nigeria Evapotranspiration Model. Journal of Hydrology, 70 Elsevier Science Publisher B.V. Amsterdam. Printed in the Netherlands. P. 71-83.

Erie, L.J. and Dedrick, A.R. (1979): Level Basin Irrigation – A Method for Conserving Water and Labour. Farmer Bul, 2261. U.S Government Printing Office, Washington, D.C. P. 24.

Etcheverry (1931): Land Drainage and Flood Protection, Copyright, McGraw Hill Book Co.

Glen, O, Schwab, Richard, K., Frevert, Edminster, T.W. and Barnes, K.K. (1981): Soil and Water Conservation Engineering. John Wiley and Sons, Inc. Canada. P.288,416-428.

Glen, O., Schwab and Richard, K.Frevert (1985). Elementary Soil and Water Engineering 3rd Edition John Wiley and Sons, Inc Canada P.76,79.

Haise, H.P, Donnan, W.W. Pelan, J.T. Lawhon, L.F, and Shockley, D.G (1956): The use of Cylinder Infiltrometers to Determine the intake Characteristic of irrigated soils Publ. Art 41-7, Agricultural Research Service and Soil conservation Service, USDA, Washington DC.

Hart, W.E, Collins, H.J, Woodward, g., and humpherys, A.J (1980): Design and Operation of Gravity on Surface System, Chapter 13, In: Design and

Operation of Farm Irrigation System. ASAE Monograph Number 3,St. Joseph, Michigan. P.829.

Internal Commission on Irrigation and Drainage (I.C.I.D), (1979): Fourth Afro-Asian Regional Conference Proceeding Vol. 1 Topic: Experience in River Basin Development in Nigeria.

Jenson, M.E, (1983):Design and Operation of Farm Irrigation systems, America Society of Agricultural Engineers, 2950 Niles Road, St. Joseph, Michiyan 49085 p. 501 – 511

Jensen, M.E (ed) (1973): Consumptive Use of Water and Irrigation Water Requirements. American Society of Civil Engineers, New York. P. 215.

Kraatz, D.B. and Mahaja V.I.K. (1975): Small Hydraulic Structure, Vol. 1 nad 2 FAO Irrigation and Drainage Paper 26/1and 26/2, Rome, P.293 and 407.

Lambart .K. Smedema and David W. Rycroft (1983); Land drainage, planning and design of agricultural drainage systems.

Larry G. James (1993): Principles of Farm Irrigation System Design. P. 301 – 308, 360 -362.

Martin, S.B and Ehaber (1965): FAO Report, Irrigation Development in Nigeria.

Martin Upton (1979): Agricultural Production Economics and Resource – Use, Oxford University Press.

Mazumder, S.K (1983): Irrigation Engineering First Edition. Tata McGraw Book Co., P. 160, 170-175.

Merriam, J.L. and Keller, J (1978): farm Irrigation System and Evaluation: A Guide For Management. Department of Agricultural and Irrigation Engineering, Utah State University, Logan, Utah.

Michael, A.M (1978): Irrigation Theory and Practice: Vikas Publishing House, 5 Ansari Road, New Delhi P. 482, 511 – 522, 546-549.

Michael, A.M. and Ojha, T.P. (2006): Principles of Agric Engineering, Vol II Survey, Irrigation, Drainage, Soil and Water Conservation. M/S Jain Brothers (New Delhi).

Richards, L.A (1954):Diagnosis and Improvement of Sahrie and Alkaline Soils. U.S.D.A. Hand Book 60.

Ruthenberg H. (1980): Farming System in the Tropics – Third Edition Clereden Press, Oxford London, P. 178-185.

Schwab, G.O, Frevet, R.K, Edminster, T.W and Barnes, K.K. (1981): Soil and Water Conservation Engineering. Wiley, New York, P. 525.

UDSA (1967): Planning farm Irrigation Farm Irrigations System Chap 3, Sect. 15 (Irrigation) Soil Conservation. Serv. Natl. Eng. Hondb.

Vaughn, E.H, Orson W. Isrealsen and Glen, E.S (1979): Irrigation Principles and Practice, John Wiley and Sons, Inc. U.S.D.A.P. 1-4, 52, 231, 277-284, 288-293.

Ven Te Chow (1955): A Note on the Manning Fromula, Transactions, American Geophysical Union, Vol. 36, No. 4, P. 688.

Ven Te Chow (1960): Open Channel Hydraulic. McGraw-Hill International Book Company P. 19-32,157-176.

Vomocil, J.A (1954): Insitu Measurement of soil Bulk Density. Agr Engr. 35. P 651-654.

Walker, W.R and Skogerboe, G.V. (1987): Surface Irrigation: Theory and Practice. Prentice-Hall, Englewood Cliffs, New Jersey, P. 386.

Walker, W.R (2007): Guideline for Designing and Evaluating Repository. P. 1-2, 12, 13, 16, 23, 33, 41- 43.

Worthington, E.B (1979): Arid Land in Developing Countries, environmental Problems and Effects, Pergamon Press, Oxford New York, Sydney, Toronto, Paris Frankfurt.

APPENDIX A

TABLE A_1: MEANS MONTHLY RAINFALL (MM) FROM 1975 – 2004

YEARS/M	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	MEANS
1975	0.0	23.1	1.8	7.1	122.2	161.2	185.9	214.9	343.5	100.8	0.0	0.0	96.71
1976	0.0	6.3	0.0	36.1	122.2	259.7	238.9	97.2	112.5	145.8	9.9	0.0	85.64
1977	0.0	0.0	0.4	18.7	93.2	175.2	87.9	25.0	196.7	94.1	0.0	0.0	57.60
1978	0.0	0.0	11.5	130.4	168.8	165.6	263.4	281.3	236.0	96.5	0.0	0.0	112.79
1979	0.0	0.0	12.51	19.0	143.2	174.8	174.8	116.3	113.1	73.4	26.0	0.0	79.53
1980	0.0	0.0	0.0	6.3	146.4	119.1	335.3	551.5	213.0	87.3	0.0	0.0	121.58
1981	0.0	0.0	0.0	6.3	133.8	166.6	281.9	147.1	196.9	34.5	0.0	0.0	80.59
1982	0.0	2.3	14.7	33.1	61.0	199.3	175.8	222.5	186.5	97.8	0.0	0.0	82.75
1983	0.0	0.0	0.0	0.3	171.1	174.0	170.3	144.0	175.1	0.0	0.0	0.0	69.57
1984	0.0	0.0	4.0	78.6	140.2	198.0	151.9	149.6	241.8	72.4	0.0	0.0	86.38
1985	0.0	0.0	175.7	6.9	99.8	322.1	195.7	234.5	305.8	25.0	0.0	0.0	113.79
1986	0.0	0.0	19.9	59.7	22.9	245.0	140.7	145.8	182.3	88.9	0.0	0.0	75.43
1987	0.0	2.7	28.0	28.0	123.9	103.2	247.7	405.1	156.3	91.3	0.0	0.0	98.85
1988	18.6	3.8	23.2	160.4	92.8	104.4	103.1	111.9	286.2	56.6	0.0	0.0	88.42
1989	0.0	0.0	4.0	104.5	102.4	129.9	287.2	288.9	136.7	74.3	0.0	0.0	93.99
1990	0.0	4.3	0.0	81.8	282.3	117.9	266.0	180.6	160.0	110.1	2.7	0.0	100.48
1991	0.0	0.5	68.3	80.5	205.9	331.5	232.0	244.7	149.6	75.7	0.0	0.0	115.73
1992	0.0	0.0	0.0	141.7	136.6	133.9	128.6	148.4	216.0	31.5	0.0	0.0	78.06
1993	0.0	0.0	61.6	8.9	154.7	241.8	206.9	308.4	240.4	152.8	0.0	0.0	114.63
1994	0.0	0.0	0.0	38.0	171.9	151.4	75.8	425.7	194.0	102.1	0.0	0.0	96.58

1995	0.0	0.0	22.9	43.8	93.3	128.[illegible]	236.7	307.5	152.2	105.6	12.3	0.0	91.83
1996	0.0	18.9	0.0	12.6	199.9	190.7	201.8	326.1	170.5	41.3	0.0	0.0	96.82
1997	0.0	0.0	64.9	53.9	129.3	279.2	219.0	227.2	147.5	135.4	7.2	0.0	105.30
1998	0.0	0.0	0.0	67.1	213.2	75.5	239.7	145.5	153.7	103.0	0.0	0.0	83.14
1999	0.0	2.8	9.8	112.1	135.4	196.8	261.1	194.5	143.7	98.0	0.0	0.0	97.27
2000	0.0	0.0	9.5	13.4	118.5	280.[illegible]	191.9	284.3	262.8	42.0	0.0	0.0	100.24
2001	0.0	0.0	0.0	62.4	115.9	119.3	245.9	345.5	301.6	66.0	0.0	0.0	104.72
2002	0.0	0.0	0.5	44.9	78.0	135.9	199.2	199.9	252.0	105.7	21.7	0.0	86.48
2003	0.0	0.0	0.0	20.2	210.2	169.1	138.4	151.7	162.8	72.9	36.0	0.0	88.47
2004	0.0	0.0	0.0	57.7	177.0	167.6	113.8	355.9	148.8	135.6	0.0	0.0	98.45
MEANS	0.6	2.16	17.77	54.48	138.83	180.44	207.72	232.722	198.27	83.88	3.83	0.0	

SOURCE: N C R I BADEGGI

TABLE A_2 : MEAN MONTHLY TEMPERATURE IN ^{0}C FROM 1975 – 2004

YEARS/M	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	MEANS
1975	23.7	27.2	29.9	29.8	28.4	29.8	26.3	26.0	26.2	27.3	27.3	25.0	27.24
1976	25.2	28.9	30.1	30.5	28.4	27.0	26.3	26.0	27.2	26.9	28.5	24.7	27.48
1977	26.3	27.6	29.3	31.3	29.5	27.3	26.9	26.3	26.7	27.6	25.6	24.6	27.42
1978	24.9	29.0	30.1	30.1	28.4	27.9	26.2	26.8	26.6	27.2	26.5	25.9	27.47
1979	26.0	28.0	30.0	30.5	28.5	27.5	27.0	27.0	27.0	28.0	28.0	24.5	27.67
1980	26.5	28.5	30.5	31.5	28.5	28.0	26.5	26.5	27.0	27.0	28.0	25.5	27.83
1981	24.0	27.0	30.0	31.5	29.0	27.5	26.5	27.0	27.0	28.0	26.5	25.0	27.42
1982	25.0	27.5	29.0	30.5	28.5	27.0	26.5	27.0	26.5	27.0	28.5	27.5	27.54
1983	26.3	29.1	29.0	31.5	30.5	28.0	27.0	26.5	26.5	28.0	27.0	25.0	27.87
1984	23.5	27.5	31.0	30.5	28.5	27.5	26.5	26.5	26.5	28.0	27.0	25.0	27.33
1985	27.0	27.0	30.5	30.0	29.0	26.5	26.5	27.0	26.5	28.0	27.5	24.5	27.50
1986	24.0	29.0	30.5	30.5	29.0	27.5	26.5	26.5	27.0	28.0	26.5	24.5	27.42
1987	24.5	29.0	30.0	30.5	30.0	28.5	27.0	27.0	27.0	28.0	26.5	24.5	27.71
1988	25.5	28.0	30.5	30.5	29.0	27.5	27.5	28.0	26.5	28.0	27.0	25.5	27.79
1989	23.0	26.5	30.0	31.0	28.5	27.5	27.0	27.0	27.0	27.0	27.5	25.0	27.50

1990	27.0	27.0	29.0	30.0	28.0	27.5	26.5	27.0	27.0	28.0	28.0	27.5	27.71
1991	30.0	30.5	29.5	27.5	27.5	27.0	26.5	25.5	26.0	26.5	23.5	23.5	27.17
1992	23.5	25.5	29.5	29.0	28.5	27.5	26.5	27.0	26.0	27.0	26.0	24.0	26.67
1993	24.0	27.0	29.0	31.0	30.0	27.0	26.5	27.0	27.0	28.0	27.5	26.5	27.54
1994	24.0	27.5	32.0	31.0	29.0	27.5	27.5	27.5	27.0	28.0	26.0	24.5	27.63
1995	24.5	26.5	31.5	32.0	29.5	28.5	27.5	27.0	27.5	28.0	27.0	26.0	27.96
1996	25.0	29.0	31.0	32.0	28.5	27.0	27.0	26.0	26.0	27.0	25.5	26.0	27.50
1997	29.0	28.5	30.5	30.0	28.5	27.0	27.5	27.5	27.5	28.0	280	26.0	28.21
1998	26.0	30.0	31.0	33.0	29.5	28.5	28.0	27.0	27.0	28.5	28.0	26.0	28.58
1999	26.0	28.5	31.0	31.0	29.0	27.5	27.0	26.5	27.0	28.0	28.0	25.5	27.92
2000	26.5	26.5	30.0	31.5	30.0	27.5	26.5	26.5	27.0	28.0	27.5	24.5	27.67
2001	24.0	26.5	30.5	31.0	29.5	28.0	27.0	26.5	26.5	28.0	27.0	25.5	27.50
2002	25.0	27.5	31.5	30.5	31.0	28.0	27.0	26.0	26.5	27.0	26.5	25.5	27.67
2003	26.0	29.0	31.0	31.5	30.0	27.5	27.0	26.5	26.0	28.0	27.5	24.5	27.88
2004	25.5	28.0	31.0	31.5	28.5	27.5	27.5	26.5	27.0	28.0	27.5	25.5	27.83
MEANS	25.5	27.89	0.31	30.82	29.02	27.65	26.89	26.72	26.74	27.68	27.16	25.19	

SOURCE: N C R I BADEGGI

ABLE A_3: MEAN MONTHLY PERCENTAGE RELATIVE HUMIDITY FROM

1975 - 2004

YEARS/M	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	MEANS
1975	55.0	65.0	60.0	70.0	77.0	79.0	87.0	72.3	85.0	82.0	78.0	59.0	73.28
1976	60.0	68.0	65.0	68.0	79.0	82.0	83.0	85.0	83.0	84.0	74.0	76.0	75.58
1977	66.0	53.0	61.0	70.0	77.0	81.0	84.0	84.0	86.0	80.0	67.0	66.0	72.92
1978	61.0	55.0	70.0	76.0	81.0	22.0	82.0	86.0	84.0	812.0	71.0	65.0	75.58
1979	66.0	60.0	66.0	71.0	83.0	85.0	87.0	86.0	86.0	84.0	80.0	63.0	76.42
1980	68.0	56.0	63.0	69.0	81.0	82.0	84.0	86.0	86.0	84.0	78.0	63.0	75.00
1981	64.0	56.0	63.0	72.0	82.0	83.0	85.0	85.0	86.0	82.0	69.0	67.0	74.50
1982	61.0	59.0	63.0	73.0	77.0	83.0-	85.0	58..0	86.0	82.0	68.0	65.0	73.92
1983	58.0	54.0	35.0	89.0	72.0	83.0	83.0	85.0	85.0	79.0	70.0	63.0	71.33
1984	43.0	46.0	61.0	62.0	79.0	83.0	83.0	83.0	82.0	80.0	75.0	55.0	69.33
1985	54.0	27.0	61.0	75.0	76.0	85.0	85.0	86.0	84.0	80.0	71.0	56.0	70.00
1986	62.0	68.0	73.0	75.0	76.0	82.0	86.0	85.0	84.0	82.0	75.0	79.0	75.58
1987	51.0	61.0	62.0	60.0	67.0	80.0	84.0	87.0	85.0	78.0	72.0	64.0	71.75
1988	61.0	52.0	64.0	73.0	78.0	82.0	84.0	86.0	84.0	83.0	71.0	69.0	73.92
1989	47.0	24.0	64.0	82.0	81.0	83.0	86.0	89.0	85.0	84.0	73.0	51.0	69.92

1990	72.0	52.0	47.0	72.0	84.0	86.0	89.0	87.0	87.0	86.0	86.0	81.0	77.42
1991	69.0	72.0	72.0	81.0	85.0	82.0	91.0	92.0	88.0	88.0	81.0	66.0	80.58
1992	65.0	53.0	66.0	82.0	84.0	86.0	88.0	87.0	87.0	82.0	68.0	69.0	76.42
1993	51.0	56.0	62.0	67.0	75.0	83.0	87.0	87.0	87.0	83.0	80.0	67.0	73.42
1994	66.0	48.0	68.0	69.0	80.0	81.0	84.0	85.0	85.0	81.0	69.0	53.0	72.42
1995	50.0	40.0	69.0	69.0	[illegible]	81.0	84.0	89.0	85.0	81.0	67.0	69.0	71.58
1996	57.0	65.0	65.0	65.0	78.0	83.0	86.0	88.0	87.0	7.0	65.0	68.0	74.50
1997	57.0	30.0	54.0	70.0	77.0	82.0	84.0	82.0	82.0	82.0	72.0	57.0	69.08
1998	52.0	43.0	33.0	68.0	82.0	82.0	85.0	84.0	85.0	82.0	74.0	59.0	69.08
1999	54.0	57.0	68.0	76.0	[illegible]	[illegible]	87.0	86.0	86.0	85.0	72.0	63.0	74.33
2000	65.0	36.0	45.0	66.0	73.0	86.0	87.0	87.0	86.0	82.0	72.0	61.0	70.50
2001	57.0	37.0	57.0	65.0	77.0	80.0	85.0	84.0	86.0	81.0	68.0	61.0	69.83
2002	41.0	42.0	60.0	69.0	67.0	81.0	86.0	88.0	86.0	84.0	76.0	67.0	69.75
2003	61.0	58.0	50.0	69.0	73.0	84.0	86.0	89.0	87.0	82.0	75.0	57.0	72.58
2004	56.0	40.0	41.0	67.0	80.0	84.0	89.0	88.0	86.0	84.0	73.0	67.0	71.75
MEANS	59.0	50.97	59.6	70.8	77.83	82.7	85.63	86.1	85.37	82.17	73.0	63.5	

SOURCE: NCRI BADEGGI

TABLE A_4: MONTHLY SOLAR RADIATION (mm) FROM 1975 – 2004

YEARS/M	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	MEANS
1975	17.7	14.6	15.8	16.8	15.3	14.3	10.9	9.2	11.9	15.6	13.6	14.3	14.17
1976	12.6	15.8	16.9	15.3	14.9	13.7	11.9	10.6	12.6	11.7	16.4	13.5	13.83
1977	11.3	12.5	14.0	15.1	14.8	12.8	11.9	10.2	12.2	15.6	15.7	12.7	13.23
1978	13.2	16.4	16.7	14.8	15.0	13.7	9.8	11.6	12.7	14.3	16.2	13.9	14.03
1979	13.5	16.7	15.5	16.1	13.8	12.9	13.1	11.5	14.1	15.3	15.2	12.4	14.18
1980	13.8	15.2	15.2	16.5	14.1	13.0	10.8	10.8	14.0	13.8	15.7	12.5	13.86
1981	12.0	15.2	15.2	16.5	14.1	13.0	10.8	10.9	13.3	15.6	14.0	12.6	13.76
1982	11.6	13.6	14.7	14.9	15.2	14.2	13.7	14.8	15.2	16.9	16.6	15.2	14.72
1983	12.4	15.0	18.2	17.4	16.7	13.7	13.9	12.5	15.3	17.2	16.4	14.2	15.24
1984	13.2	16.4	18.1	16.4	15.6	16.3	15.2	15.0	14.3	16.2	15.7	13.3	15.48
1985	13.8	14.9	15.4	15.2	14.7	12.5	12.5	13.5	14.2	16.2	17.0	13.4	14.44
1986	13.6	16.8	15.4	15.9	14.8	13.9	11.6	13.0	14.2	16.5	15.6	13.9	14.60
1987	14.1	16.9	16.3	17.4	16.[illegible]	14.1	13.6	13.3	14.8	17.0	16.7	14.1	15.40
1988	12.2	15.6	17.3	16.6	15.0	15.9	11.7	10.1	13.8	17.2	17.3	13.5	14.68
1989	13.0	16.0	18.0	16.1	14.6	13.5	13.3	12.1	13.2	15.6	17.0	25.4	14.82
1990	13.5	15.9	17.8	16.6	16.1	15.3	11.8	13.4	15.4	16.6	16.9	16.6	15.49

1991	14.6	17.1	16.7	17.5	15.0	14.6	12.5	12.3	16.8	17.1
	17.0	13.5	15.42							
1992	15.1	17.6	16.6	17.3	12.0	14.1	13.5	11.5	15.0	16.4
	16.0	15.5	16.22							
1993	15.2	17.6	17.1	17.4	16.0	15.0	13.7	14.8	16.6	18.3
	14.2	14.4	16.13							
1994	13.7	15.6	19.3	16.1	16.0	15.3	14.2	13.0	15.5	18.1
	189.2	15.9	15.91							
1995	14.5	18.0	18.9	16.9	16.3	15.0	13.8	13.2	16.3	18.0
	18.5	15.6	16.25							
1996	15.0	17.3	18.2	18.0	16.7	15.1	13.2	12.8	15.2	17.9
	18.1	12.8	15.86							
1997	15.8	18.3	17.5	18.7	17.7	15.7	13.3	14.9	14.9	17.3
	18.0	15.4	16.43							
1998	14.3	17.6	11.9	18.4	16.8	16.2	13.4	12.6	14.3	18.3
	18.1	15.6	15.58							
1999	14.8	17.5	19.8	18.4	16.3	14.0	14.0	11.8	13.6	16.8
	17.6	15.6	15.85							
2000	15.5	17.5	[illegible]	17.6	16.0	15.1	13.1	13.7	16.0	18.1
	17.7	16.0	16.33							
2001	16.0	18.0	19.9	17.4	17.0	16.1	13.2	13.0	16.0	19.4
	19.3	17.0	16.86							
2002	15.0	18.4	19.0	17.4	17.3	16.5	14.0	13.5	16.0	14.0
	18.0	17.0	16.59							
2003	15.3	17.3	18.0	16.4	16.0	15.0	14.0	13.2	14.5	18.0
	18.2	16.0	15.99							
2004	15.0	17.4	18.0	17.0	16.0	14.0	12.0	12.4	16.0	16.0
	18.0	15.0	15.57							
	MEANS	14.04	16.43	17.08	16.68	15.55	14.51	1281	12.51	14.61
	16.68	16.86	14.54							

SOURCE: N.C.R.I BADEGGI

APPENDIX B_1

TABLE B_1: BLANEY – MORIN – NIGERIA EVAPO – TRANSPIRATION MODEL AVERAGE VALUES OF 30 YEARS (1975 -2004) WAS USED

YEAR/MONTH 1975 -2004	MEAN TEMP ^0C	RELATIVE HUMIDITY %	RADIATION	RAINFALL	r_r	$r_r(0.45t + 8)$	$520 - R^{1.31}$	ET_0 mm/day $\frac{r_r(0.45T + 8)(520 - R^{1.31})}{100}$
November	27.15	73.00	16.87	3.86	0.092	1.860	243.97	4.538
December	25.19	63.20	14.54	00.00	0.080	1.547	291.47	4.509
January	24.83	59.00	14.04	0.62	0.077	1.476	311.16	4.593
February	44.58	50.97	16.43	2.16	0.090	2.525	347.58	8.776
March	39.46	59.60	17.08	17.77	0.094	2.421	308.37	7.466
April	30.31	70.80	16.69	54.48	0.092	1.991	254.82	5.073
May	29.02	77.83	15.61	138.83	0.086	1.811	219.81	3.980
June	27.65	82.70	14.51	180.44	0.080	1.635	194.96	3.118
July	26.89	83.07	12.81	207.72	0.070	1.467	193.06	2.716
August	26.85	86.03	12.51	234.38	0.069	1.386	177.71	2.463
September	26.71	85.37	14.66	239.28	0.080	1.602	181.15	2.902
October	27.68	82.17	16.65	83.88	0.091	1.861	197.69	3.679
			$\sum$182.40					

APPENDIX B_2

e crop co-efficient factors (Kc) value for rice production for different stages of growth is given low:-

able B_2 : Kc Values of rice

		Initial Stage	Crop development Stage	Mid-season stage	Maturity/Late season
Type of crop	Rice				
Days		40 days	55days	45 days	20days
Kc value		1.1	1.15	1.20	1.0

Kc

– Initial stage: 40 days – Nov 1 – Dec 10

1.15 – Crop development stage: 55 days – Dec 11 – Feb 5

1.20 – Mid-season stage: 45 days – Feb 6 – March 22

1.0 – Maturity/Late stage: 20 days – March 23 – April 11

Planting date is November 1

Kc

$$Nov = \frac{30}{30} \times 1.1 + \frac{10}{31} \times 1.15 = 1.1$$

$$Dec = \frac{11}{30} \times 1.15 + \frac{20}{31} \times 1.20 = 1.13$$

Jan = 1.20

$$Feb = \frac{5}{28} \times 1.15 + \frac{23}{28} \times 1.20 = 1.20$$

$$March = \frac{22}{31} \times 1.20 + \frac{9}{31} \times 1.0 = 1.14$$

April = 1.0

$ET_c = ET_0 \times K_c$

For November: -

ET_c = 4.538 x 1.1 = 4.992mm/day

= 4.992 x 30 = 49.76mm/month

December: -

ET_c = 4.509 x 1.13 = 5.095mm/day

= 5.095 x 31 = 157.95mm/month

January: -

ET_c = 4.593 x 1.20 = 5.512mm/day

= 4.992 x 30 = 49.76mm/month

February: -

ET_c = 8.77 x 1.20 = 10.531mm/day

= 10.531 x 28 = 294.87mm/month

March: -

ET_c = 7.446 x 1.14 = 8.511mm/day

= 8.511 x 31 = 263.84mm/month

April: -

ET_c = 5.073 x 1.0 = 5.073mm/day

= 5.073 x 30 = 152.19mm/month

The design depth for this project = the peak consumptive use (10.531mm/day) + required depth of ponding which give the best yield which is 30cm.

Design depth = 10.531mm + 300mm = 310.531mm = 31.0531cm water requirement for rice for the total area $\frac{1}{10^3}$ x 294.87 x 120 x 10,000

= 353,844m^3

APPENDIX C

MAIN CANAL DESIGN

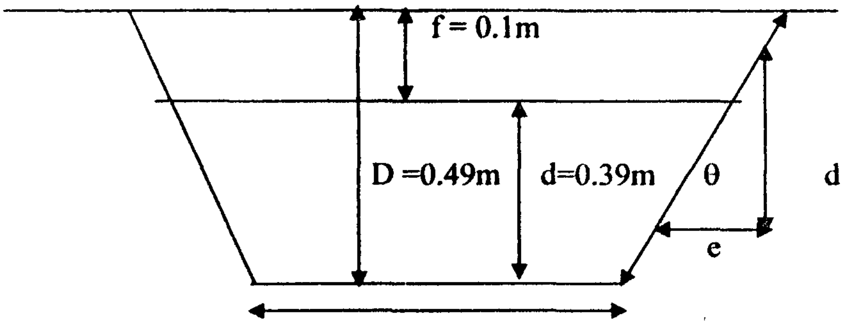


Fig. C_1

$Z = e/d = 1.5/1 = 1.5$

$Tan^{-1}\theta = d/z = 1/1.5 = 33.69^0$

$B = 2d \tan \theta/2 = 2d \tan 33.69^0$

$= 2d\ (0.30277)$

$b = 0.6055d$

$$Q = \frac{q.A}{100} \ldots\ldots\ldots\ldots (3.9)$$

Data: q = 5 L/s for flooded rice (Lambart 1983)

A = 100ha

$$\therefore\ Q = \frac{5 \times 100}{100} = 0.5m^3/sec$$

Adding 20% to cover losses, we have:

$Q = 0.5 + 0.5 \times 0.2 = 0.6m^3/sec$

$Q = 0.6m^3/sec$ discharge to be pumped into the main canal.

Using trial and error method: -

Data: - $Q = 0.63 m^3/sec$, $A = 0.023$, $S = 0.001$, side slope = 1.5:1

When d = 0.3m: -

$b = 0.6055 \times 0.3 = 0.1816m$

$A = 0.3\,(0.1816 + 2 \times 0.3) = 0.2345 m^2$

$P = 0.1816 + 2 \times 0.03 \sqrt{1.5^2 + 1} = 1.2633$

$$R = \frac{A}{P} = \frac{d}{2} = \frac{0.2345}{1.2633} = \frac{0.3}{2} = \frac{0.1856}{0.15} = 1.2373$$

$V = 1 \times (1.2373)^{2/3} \times (0.0001)^{1/2} = 43.478 \times 1.1525 \times 0.0316 = 1.5834 m/s$

$Q = A.V = 0.2345 \times 1.5834 = 0.371 = 0.4 m^3/sec$

<u>When d = 0.4m:-</u>

b = 0.2422m

$A = 0.4169 m^2$

P = 1.6844

R = 1.2375

V = 1.5837m/s

$Q = 0.66 m^3/sec$

<u>When d = 0.35m</u>

b = 0.2119m

$A = 0.3193 m^2$

P = 1.4738

R = 1.2377

V = 1.5838m/sec

$Q = 0.5055 m^3$

<u>When d = 0.38m</u>

b = 0.2301m

$A = 0.3762 m^2$

P = 0.6002

R = 1.2374

V = 1.5836m/s

$Q = 0.5957 m^3/sec$

<u>When d = 0.39m:- (o.k)</u>

b = 0.2361m

$A = 0.3963 m^2$

P = 1.6423

R = 1.2374

V = 1.5836m/sec

$Q = 0.6276 \approx 0.6 m^3/sec$

$D = 5/4.\,d = 1.25 \times 0.39 = 0.49m$

$F = (D - d) = 0.49 - 0.39 = 0.1m$

$T = 0.2361 + 2 \times 0.49 \times 1.5 = 0.71m$

FIELD CHANNEL N0. 1

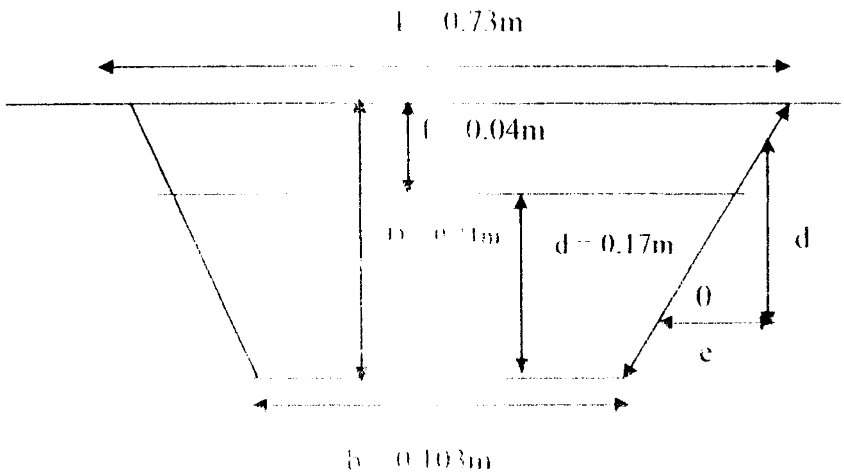


Fig. C

$Z = e/d = 1.5/1 = 1.5$

$\tan^{-1}\theta = d/z = 1/1.5 = 33.69^0$

$b = 2d \tan \theta/2 = 2d \tan 33.69^0$

$= 2d(0.30277)$

$b = 0.6055d$

$$Q = \frac{q.A}{100} \; ....(3.9)$$

Data:- q = 5L/s for flooded rice (Lambart 1983)

A = 16.7ha

$$\therefore Q = \frac{5 \times 16.7}{100} = 0.0835 m^3/sec$$

Adding 20% to cover losses, we have:-

$0.0835 + 0.0835 \times 0.2 = 0.1002 \approx 0.1 m^3/sec$

Using trial and error method:-

When d = 0.08m:-

b = 0.6055 x 0.08 = 0.0484m

A = 0.08 (0.0484 + 2 x 0.08) = 0.0167m^2

P = 0.0484 + 2 x 0.08 $\sqrt{1.5^2 + 1}$ = 0.3368

$R = \frac{A}{P} = \frac{d}{2} = \frac{0.0167}{0.3368} = \frac{0.08}{2} = \frac{0.0496}{0.04} = 1.24$

$V = \frac{1}{0.023} \times (1.24)^{2/3} \times (0.001)^{1/2}$ = 43.478 x 1.1542 x 0.0316 = 1.5858m/sec

Q = A.V = 0.0167 x 1.5858 = 0.0265m^3/sec

When d = 0.1m:-

b = 0.0606m

A = 0.-2606m^2

P = 1.4212

R = 1.238

V = 1.584m/s

Q = 0.04m^3/sec

When d = 0.09m:-

b = 0.0545m

A = 0.0211m^2

P = 0.3790

R = 1.2378

V 1.5838m/s

Q = 0.0334m^3/sec

When d = 0.099m:-

b = 0.0599m

A = 0.0255m^2

P = 0.4168

R = 1.2364

V = 1.5827m/s

Q = 0.04m^3/sec

When d = 0.0995m:-

b = 0.0602m
A = 0.0258m^2
P = 0.4190
R 1.2369
V = 1.5832m/sec
Q = 0.04m^3/sec

When d = 0.0995m:-

b = 0.1211m
A = 0.1042m^2
P = 0.8422
R 1.237
V = 1.5832m/sec
Q = 0.1650m^3/sec

When d = 0.0995m:-

b = 0.0908m
A = 0.0586m^2
P = 0.6316
R 1.2373
V = 1.5834m/sec
Q = 0.0928m^3/sec

When d = 0.17m:- (o.k)

b = 0.1029m
A = 0.0753m^2
P = 1.7158
R 1.2376
V = 1.5837m/sec

Q = 0.1193 = 0.1m^3/sec
D = 5/4. d = 1.25 x 0.17 = 0.21m
f = (D – d) = 0.21 – 0.17 = 0.04m
T = 0.1029 + 2 x 0.21 x 1.5 = 0.73m

FIELD CHANNEL N0. 2

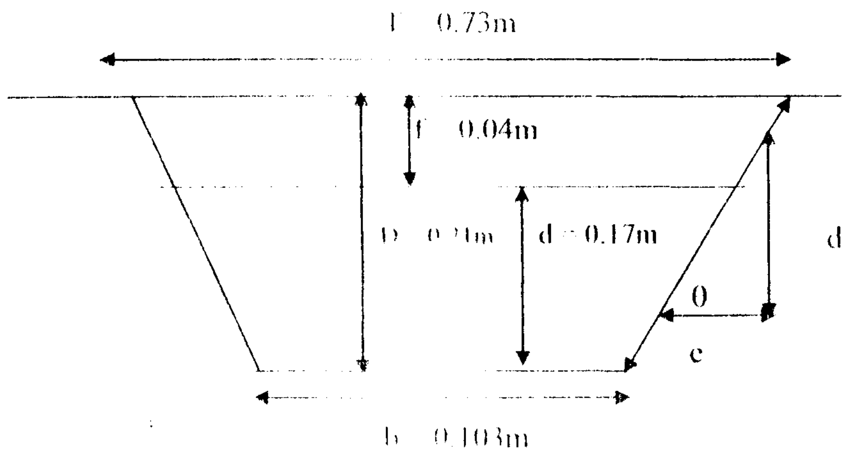


Fig. C2

$Z = e/d = 1.5/1 = 1.5$

$\tan^{-1}\theta = d/z = 1/1.5 = 33.69^0$

$b = 2d \tan \theta/2 = 2d \tan 33.69^0$

$= 2d\ (0.30277)$

$b = 0.6055d$

$$Q = \frac{q.A}{100} \ldots\ldots\ldots\ldots\ldots(3.9)$$

Data:- q = 5L/s for flooded rice (Lambart 1983)

A = 16.7ha

$$\therefore Q = \frac{5 \times 16.7}{100} = 0.0835 m^3/sec$$

Adding 20% to cover losses, we have:

$0.0835 + 0.0835 \times 0.2 = 0.1002 = 0.1 m^3/sec$

Jsing trial and error method:-

<u>When d = 0.08m:-</u>

b = 0.6055 x 0.08 = 0.0484m

A = 0.08 (0.0484 + 2 x 0.08) = 0.0167m^2

P = 0.0484 + 2 x 0.08 $\sqrt{1.5^2 + 1}$ = 0.3368

$R = \frac{A}{P} = \frac{d}{2} = \frac{0.0167}{1.3368} = \frac{0.08}{2} = \frac{0.0496}{0.04} = 1.24$

$V = \frac{1}{0.023} \times (1.24)^{2/3} \times (0.001)^{1/2} = 43.478 \times 1.1542 \times 0.0316 = 1.5858$ m/sec

$Q = A.V = 0.0167 \times 1.5858 = 0.0265$ m^3/sec

When d = 0.1m:-	When d = 0.09m:-	When d = 0.099m:-
b = 0.0606m	b = 0.0545m	b = 0.0599m
A = 0.-2606m^2	A = 0.0211m^2	A = 0.0255m^2
P = 1.4212	P = 0.3790	P = 0.4168
R = 1.238	R = 1.2378	R = 1.2364
V = 1.584m/s	V 1.5838m/s	V = 1.5827m/s
Q = 0.04m^3/sec	Q = 0.0334m^3/sec	Q = 0.04m^3/sec

When d = 0.0995m:-	When d = 0.0995m:-	When d = 0.0995m:-
b = 0.0602m	b = 0.1211m	b = 0.0908m
A = 0.0258m^2	A = 0.1042m^2	A = 0.0586m^2
P = 0.4190	P = 0.8422	P = 0.6316
R 1.2369	R 1.237	R 1.2373
V = 1.5832m/sec	V = 1.5832m/sec	V = 1.5834m/sec
Q = 0.04m^3/sec	Q = 0.1650m^3/sec	Q = 0.0928m^3/sec

<u>When d = 0.17m:- (o.k)</u>

b = 0.1029m	D = 5/4. d = 1.25 x 0.17 = 0.21m
A = 0.0753m^2	f = (D – d) = 0.21 – 0.17 = 0.04m
P = 1.7158	T = 0.1029 + 2 x 0.21 x 1.5 = 0.73m
R 1.237	V = 1.5837m/sec

FIELD CHANNEL N0. 3

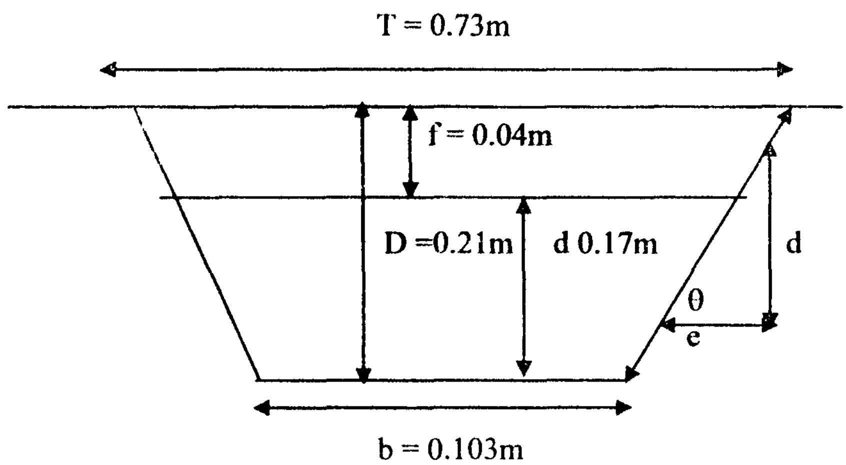


Fig. C_4

$Z = e/d = 1.5/1 = 1.5$

$\text{Tan}^{-1}\theta = d/z = 1/1.5 = 33.69^0$

$b = 2d \tan \theta/2 = 2d \tan 33.69^0$

$= 2d\ (0.30277)$

$b = 0.6055d$

$$Q = \frac{q.A}{100} \ldots\ldots\ldots\ldots\ldots(3.9)$$

Data:- q = 5L/s for flooded rice (Lambart 1983)

A = 16.7ha

$$\therefore Q = \frac{5 \times 16.7}{100} = 0.0835 m^3/sec$$

Adding 20% to cover losses, we have:-

$0.0835 + 0.0835 \times 0.2 = 0.1002 = 0.1 m^3/sec$

Using trial and error method:-

<u>When d = 0.08m:-</u>

b = 0.6055 x 0.08 = 0.0484m

A = 0.08 (0.0484 + 2 x 0.08) = $0.0167m^2$

P = $0.0484 + 2 \times 0.08 \sqrt{1.5^2 + 1} = 0.3368$

$$R = \frac{A}{P} = \frac{d}{2} = \frac{0.0167}{1.3368} = \frac{0.08}{2} = \frac{0.0496}{0.04} = 1.24$$

$$V = \frac{1}{0.023} \times (1.24)^{2/3} \times (0.001)^{1/2} = 43.478 \times 1.1542 \times 0.0316 = 1.5858\text{m/sec}$$

$Q = A.V = 0.0167 \times 1.5858 = 0.0265m^3/sec$

<u>When d = 0.1m:-</u>

b = 0.0606m

A = $0.-2606m^2$

P = 1.4212

R = 1.238

V = 1.584m/s

Q = $0.04m^3/sec$

<u>When d = 0.09m:-</u>

b = 0.0545m

A = $0.0211m^2$

P = 0.3790

R = 1.2378

V 1.5838m/s

Q = $0.0334m^3/sec$

<u>When d = 0.099m:-</u>

b = 0.0599m

A = $0.0255m^2$

P = 0.4168

R = 1.2364

V = 1.5827m/s

Q = $0.04m^3/sec$

<u>When d = 0.0995m:-</u>

b = 0.0602m

A = $0.0258m^2$

P = 0.4190

R 1.2369

V = 1.5832m/sec

Q = $0.04m^3/sec$

<u>When d = 0.0995m:-</u>

b = 0.1211m

A = $0.1042m^2$

P = 0.8422

R 1.237

V = 1.5832m/sec

Q = $0.1650m^3/sec$

<u>When d = 0.0995m:-</u>

b = 0.0908m

A = $0.0586m^2$

P = 0.6316

R 1.2373

V = 1.5834m/sec

Q = $0.0928m^3/sec$

<u>When d = 0.17m:- (o.k)</u>

b = 0.1029m

A = $0.0753m^2$

P = 1.7158

R 1.2376

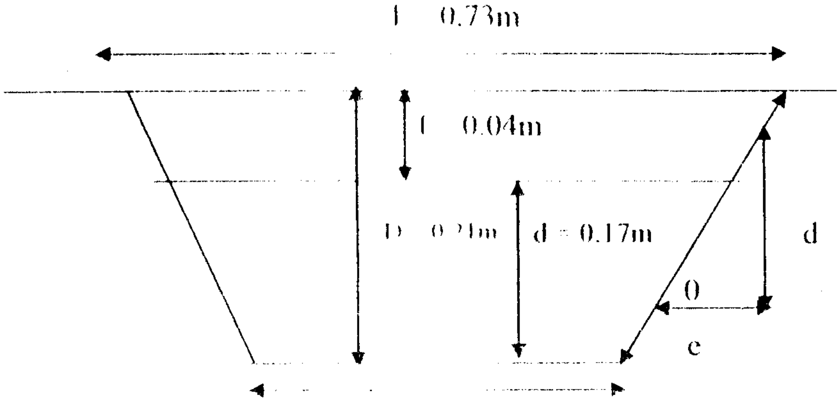
V = 1.58[illegible]7m/sec

D = 5/4 d = 1.25 x 0.17 = 0.21m

f = (D − d) = 0.21 − 0.17 = 0.04m

T = 0.1029 + 2 x 0.21 x 1.5 = 0.73m

FIELD CHANNEL NO. 4



b = 0.103m

Fig. C

$Z = e/d = 1.5/1 = 1.5$

$Tan^{-1}0 = d/z = 1/1.5 = 33.69^0$

$b = 2d \tan 0/2 = 2d \tan 33.69^0$

$= 2d\,(0.30277)$

$b = 0.6055d$

$$Q = \frac{q.A}{100} \qquad (3.9)$$

Data:- q = 5L/s for flooded rice (Lambart 1983)

A = 16.7ha

$$\therefore Q = \frac{5 \times 16.7}{100} = 0.0835m^3/sec$$

Adding 20% to cover losses, we have:-

$0.0835 + 0.0835 \times 0.2 = 0.1002 = 0.1m^3/sec$

Using trial and error method:-

<u>When d = 0.08m:-</u>

b = 0.6055 : 0.08 = 0.0484m

$A = 0.08\,(0.0484 + 2 \times 0.08) = 0.0167m^3$

$P = 0.0484 + 2 \times 0.08\,\sqrt{1.5^2 + 1} = 0.3368$

$R = \frac{A}{P} \div \frac{d}{2} = \frac{0.0167}{1.3368} \div \frac{0.08}{2} = \frac{0.0496}{0.04} = 1.24$

$V = \frac{1}{0.023} \times (1.24)^{2/3} \times (0.001)^{1/2} = 43.478 \times 1.1542 \times 0.0316 = 1.5858m/sec$

$Q = A.V = 0.0167 \times 1.5858 = 0.0265m^3/sec$

<u>When d = 0.1m:-</u>

b = 0.0606m

$A = 0.\text{-}2606m^2$

P = 1.4212

R = 1.238

V = 1.584m/s

$Q \;\; 0.04m^3/sec$

<u>When d 0.09m:-</u>

b = 0.0545m

$A = 0.0211m^3$

P = 0.3790

R = 1.2378

V 1.5838m/s

$Q \;\; 0.0334m^3/sec$

<u>When d = 0.099m:-</u>

b = 0.0599m

$A = 0.0255m^2$

P = 0.4168

R = 1.2364

V = 1.5827m/s

$Q \;\; 0.04m^3/sec$

<u>When d = 0.0995m:-</u>

b = 0.0602m

$A = 0.0258m^2$

P = 0.4190

R 1.2369

V = 1.5832m/sec

$Q \;\; 0.04m^3/sec$

<u>When d 0.0995m:-</u>

b = 0.1211m

$A = 0.1042m^2$

P = 0.8422

R 1.237

V = 1.5832m/sec

$Q \;\; 0.1650m^3/sec$

<u>When d = 0.0995m:-</u>

b = 0.0908m

$A = 0.0586m^2$

P = 0.6316

R 1.2373

V = 1.5834m/sec

$Q \;\; 0.0928m^3/sec$

<u>When d = 0.17m:- (o.k)</u>

b = 0.1029m

$A = 0.0753m^2$

P = 1.7158

R 1.2376

V = 1.5837m/sec

$D = 5/4.\,d = 1.25 \times 0.17 = 0.21m$

$f = (D - d) = 0.21 - 0.17 = 0.04m$

$T = 0.1029 + 2 \times 0.21 \times 1.5 = 0.73m$

FIELD CHANNEL N0. 5

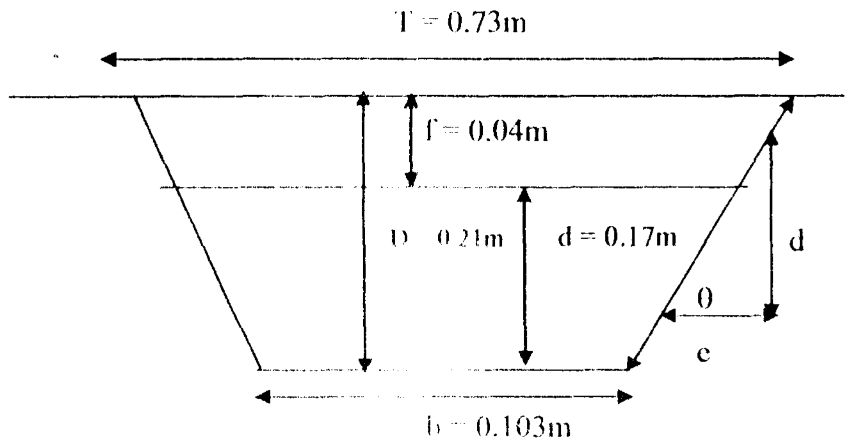


Fig. C_6

$Z = e/d = 1.5/1 = 1.5$

$\tan^{-1}\theta \quad d/z \quad 1/1.5 \quad 33.69^0$

$b = 2d \tan \theta/2 = 2d \tan 33.69^0$

$= 2d\ (0.30277)$

$b = 0.6055d$

$$Q = \frac{q.A}{100} \ldots\ldots\ldots\ldots\ldots(3.9)$$

Data:- q = 5L/s for flooded rice (Lambart 1983)

A = 16.7ha

$\therefore Q = \underline{5 \times 16.7} = 0.0835m^3/sec$

Adding 20% to cover losses, we have:-

$0.0835 + 0.0835 \times 0.2 \quad 0.1002 \quad 0.1m^3/sec$

Using trial and error method:-

<u>When d = 0.08m:-</u>

$b = 0.6055 \times 0.08 = 0.0484m$

$A = 0.08\ (0.0484 + 2 \times 0.08) = 0.0167m^2$

$P = 0.0484 + 2 \times 0.08 \sqrt{1.5^2 + 1} = 0.3368$

$R = \underline{A} \quad \underline{d} = \underline{0.0167} \quad \underline{0.08} = \underline{0.0496} \quad 1.24$

$P = 2 \times 1.3368 \times 2 \times 0.04$

$V = \frac{1}{0.023} \times (1.24)^{2/3} \times (0.001)^{1/2} = 43.478 \times 1.1542 \times 0.0316 = 1.5858$ m/sec

$Q = A.V = 0.0167 \times 1.5858 = 0.0265 m^3/sec$

<u>When d = 0.1m:-</u>

b = 0.0606m

A = 0.-2606m^2

P = 1.4212

R = 1.238

V = 1.584m/s

Q = 0.04m^3/sec

<u>When d = 0.09m:-</u>

b = 0.0545m

A = 0.0211m^2

P = 0.3790

R = 1.2378

V 1.5838m/s

Q = 0.0334m^3/sec

<u>When d = 0.099m:-</u>

b = 0.0599m

A = 0.0255m^2

P = 0.4168

R = 1.2364

V = 1.5827m/s

Q = 0.04m^3/sec

<u>When d = 0.0995m:-</u>

b = 0.0602m

A = 0.0258m^2

P = 0.4190

R 1.2369

V = 1.5832m/sec

Q = 0.04m^3/sec

<u>When d = 0.0995m:-</u>

b = 0.1211m

A = 0.1042m^2

P = 0.8422

R 1.237

V = 1.5832m/sec

Q = 0.1650m^3/sec

<u>When d = 0.0995m:-</u>

b = 0.0908m

A = 0.0586m^2

P = 0.6316

R 1.2373

V = 1.5834m/sec

Q = 0.0928m^3/sec

<u>When d = 0.17m:- (o.k)</u>

b = 0.1029m

A = 0.0753m^2

P = 1.7158

R 1.2376

V = 1.5837m/sec

D = 5/4 d = 1.25 x 0.17 = 0.21m

f = (D - d) = 0.21 - 0.17 = 0.04m

T = 0.1029 + 2 x 0.21 x 1.5 = 0.73m

FIELD CHANNEL NO 6

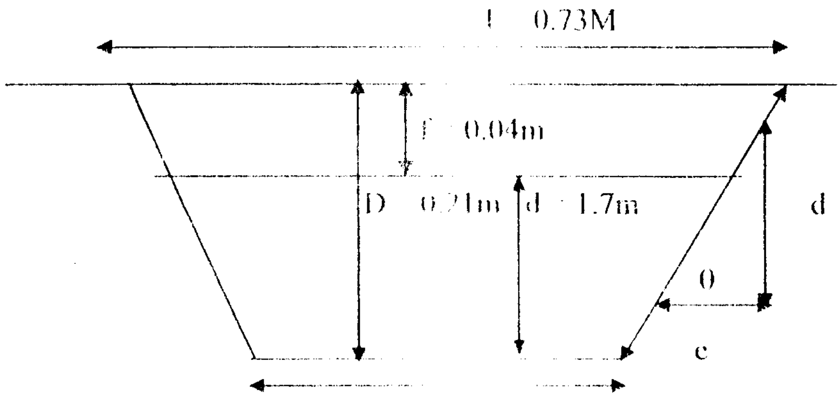


Fig. C

$Z = e/d = 1.5/1 = 1.5$

$\tan^{-1}\theta = d/z = 1/1.5 = 33.69^0$

$b = 2d \tan \theta/2 = 2d \tan 33.69^0$

$= 2d\ (0.30277)$

$b = 0.6055d$

$Q = \frac{q.A}{100}$ .... (3.9)

Data:- q = 5L/s for flooded rice (Lambart 198[illegible])

$A = 16.7ha$

$\therefore\ Q = \frac{5 \times 16.7}{1000} = 0.0835 m^3/sec$

Adding 20% to cover losses, we have :-

$0.0835 + 0.0835 \times 0.2 = 0.1002 \approx 0.1 m^3/sec$

Using trial and error method:-

<u>when d = 0.08m:-</u>

$b = 0.6055 \times 0.08 = 0.0484m$

$A = 0.08\ (0.0484 + 2 \times 0.08) = 0.0167m^2$

$P = 0.0484 + 2 \times 0.08\sqrt{1.5^2 + 1} = 0.3368$

$R = \frac{A}{P} = \frac{d}{2} = \frac{0.0167}{1.3368} = \frac{0.08}{2} = \frac{0..0496}{0.04} = 1.24$

$V = 1 \times (1.24)^{2/3} \times (0.001)^{1/2} = 43.478 \times 1.1542 \times 0.0316 = 1.5858 m/sec$

$Q = AV = 0.0167 \times 1.5858 = 0.0265 m^3/sec$

When d = 0.1m:-	When d = 0.09m	When d = 0.099m
b = 0.0602m	b= 0.1211m	b = 0.0908m
$A = 0.0258m^2$	$A = 0.1042m^2$	$A = 0.0586m^2$
P = 0.4190	P = 0.8422	P = 0.6316
R = 1.2369	R = 1.237	R = 1.2373
V = 1.5832m/s	V = 1.5832m/sec	V =1.5834m/s
$Q = 0.04m^3/sec$	$Q = 0.1650m^3$	$Q = 0.0928m^3/sec$

When d = 0.17m:- (o.k)

b = 0.1029m

$A = 0.0753m^2$

P = 1.7158

R 1.2376

V = 1.5837m/sec

$Q = 0.1193 \approx 0.1m^3/sec$

$D = 5/4.\ d = 1.25 \times 0.17 = 0.2m$

$F = (D - d) = 0.21 - 0.17 = 0.04m$

$T = 0.1029 + 2 \times 0.21 \times 1.5 = 0.73m$

FIELD DRAIN DESIGN NO. 1

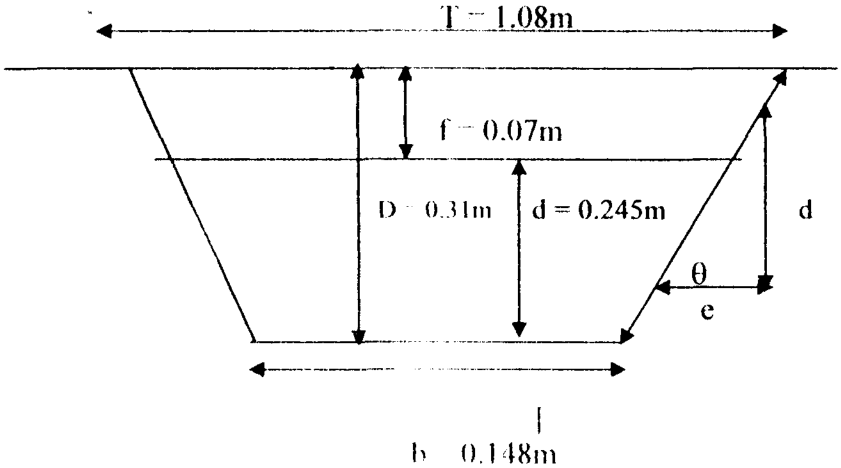


Fig. C_8

$Z = e/d = 1.5/1 = 1.5$

$Tan^{-1}\theta = d/z = 1/1.5 = 33.69^0$

$b = 2d \tan\theta/2 = 2d \tan 33.69^0$

$= 2d\,(0.30277)$

$b = 0.6055d$

$$Q = \frac{q.A}{100} \quad \ldots\ldots\ldots\ldots\ldots(3.9)$$

Data:- q = 5L/s, A = 50ha

$$\therefore Q = \frac{5 \times 50}{100} = 0.25m^3/sec$$

$Q = 0.25m^3/sec$

Using trial and error method:-

When d = 0.25m:-	When d = 0.23m:-	When d = 0.24m:-
b = 0.1514m	b = 0.1393m	b = 0.1453m
$A = 0.1629m^2$	$A = 0.1378m^2$	$A = 0.1501m^2$
P = 1.0528	P = 0.9686	P = 1.0106
R = 1.2376	R = 1.2374	R = 1.2375
V = 1.5837m/s	V 1.5836m/s	V = 1.5837m/s
$Q = 0.2580m^3/sec$	$Q = 0.2182m^3/sec$	$Q = 0.24m^3/sec$

When d = 0.245m:- (o.k)

b = 0.1483m

A = $0.1564m^2$

P = 1.7158

R 1.2376

V = 1.5837m/sec

Q = $0.25m^3/sec$

D = 5/4 d = 1.25 x 0.245 = 0.31m

F = (D - d) = 0.31 - 0.245 = 0.07m

T = 0.1483 + 2 x 0.31 x 1.5 = 1.08m

FIELD DRAIN DESIGN NO. 2

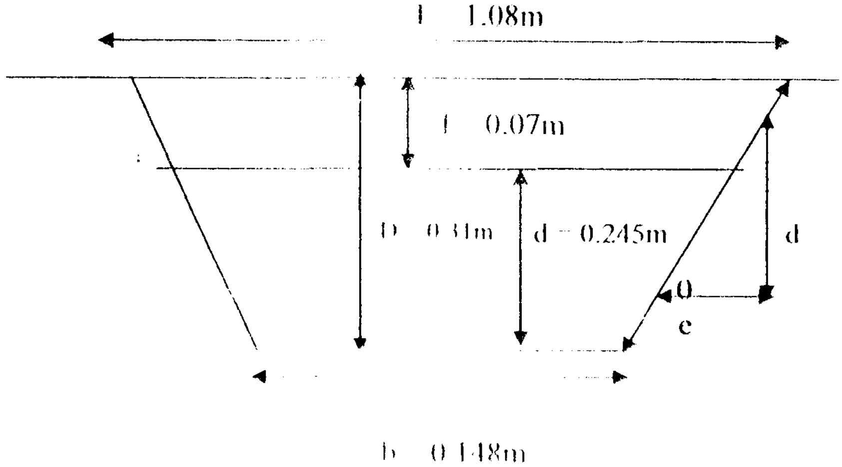


Fig. C6

$Z = e/d = 1.5/1 = 1.5$

$\tan^{-1}\theta = d/z = 1/1.5 = 33.69^0$

$b = 2d \tan \theta/2 = 2d \tan 33.69^0$

$= 2d\,(0.30277)$

$b = 0.6066d$

$$Q = \frac{q.A}{100} \quad \ldots\ldots\ldots\ldots\ldots(3.9)$$

Data:- q = 5L/s, A = 50ha

$\therefore Q = \frac{5 \times 50}{100} = 0.25m^3/sec$

$Q = 0.25m^3/sec$

Using trial and error method:-

When d = 0.25m:-

b = 0.1514m

A = $0.1629m^2$

When d = 0.23m:-

b = 0.1393m

A = $0.1378m^2$

When d = 0.24m:-

b = 0.1453m

A = $0.1501m^2$

P = 1.0528	P = 0.9686	P = 1,0106
R = 1.2376	R = 1.2374	R = 1.2375
V = 1.5837m/s	V 1.5836m/s	V = 1.5837m/s
Q = 0.2580m^3/sec	Q = 0.2182m^3/sec	Q = 0.24m^3/sec

<u>When d = 0.245m:- (o.k)</u>

b = 0.1483m

A = 0.1564m^2

P = 1.7158

R 1.2376

V = 1.5837m/sec

Q = 0.25m^3/sec

D = 5/4 d = 1.25 x 0.245 = 0.31m

F = (D − d) = 0.31 − 0.245 = 0.07m

T = 0.1483 + 2 x 0.31 x 1.5 = 1.08m

APPENDIX D

BASIN DESIGN CALCULATIONS

The selection of intake family is done by plotting the values of cumulative infiltration rate to time obtained from the infiltration rate experiment on Table 4.3.

The intake family is that of the curve closest to which the point fall. in this case, the points fall on the curve 1.5 from Fig. G

Therefore, the intake family = 1.5, values of constant a = 2.284, b = 0.7799 and c = 7.0 (from Table G). Desired depth (Net) application (F_n) 17 mm (See Appendix E for calculation)

Assumption:-

Manning's co-efficient (n) = 0.023

Efficiency (E) = 80%

Efficiency advance ratio from table D = 58

BASIN DESIGN FOR FIELD CHANNELS 1 – 6

Basin design will be same for all the 6 field channels since they are commanding equal area and will therefore have same unit inflow rate (Q_u).

Intake family = 1.5

Values of a = 2.284, b = 0.799, c 7.0 from the table

Efficiency (E) = 80%

Unit flow rate (Q_u) = 0.1m^3/sec/sec/16.7ha = 0.006m^3/sec

Desired depth (net) application (F_n) = 24.8mm

Manning co-efficient (n) = 0.023

Efficiency advance ratio = 0.58

Opportunity time (T):-

$$= \left(\frac{17 - 7.0}{2.248}\right)^{1/0.7799} = 7 \text{ min}$$

Advance Time (T_t):-

= (0.58 x 60 x 0.58) = 20 min

Basin length (L):-

$$= \frac{(6\times10^4)(0.006)(20)}{= \frac{(2.284 \times 20)^{0.799} + 7.0 + 1798 (0.023)^{3/8} (0.006)^{9/16} (20)^{3/16}}{1 + 0.7799} \text{ min}}$$

L = 118m

Inflow time (T_a):-

$$= \frac{17\times118}{600 \times 0.006 \times 80} = 7\text{min}$$

Maximum depth of flow (d):-

$$= 2250 (0.023)^{3/8} (0.006)^{9/16} (7)^{3.16} = 41.5\text{m.}$$

Table D:

Efficiency as a function of the efficienc[illegible] advance ration R(R = T_t/T_n)

Efficiency E	efficiency advance ratio R(R = T_t/T_n)
Percent (%)	-
95	0.16
90	0.28
85	0.40
80	0.58
75	0.80
70	1.08
65	1.45
60	1.90
55	2.45
50	3.20

Efficiency (E) ------- 80   0.58 ---------------Efficiency Advance Ratio

APPENDIX E

SIMPLE CALCULATION OF PARAMETERS

A. Determination of Soil Texture

P_1 0 15cm

Percentage sand:-

100 - [14.5 + 0.36] 27 – 20] - 2.0] 2 69.96%

Percentage clay: -

0.5 + 0.36 [27 – 20] - 2.0] 2 = 2.04

Percentage silt: -

100 – [69.96 + 2.04] = 28%

P 15 – 30cm

Percentage sand: -

100 - [15.0 + 0.36] 27 – 20] - 2.0] 2 = 68.96%

Percentage clay: -

3 + 0.36 [27 – 20] – 2.0] 2 = 7.04%

Percentage silt: -

100 – [68.96 + 7.04] = 24%

P_1 30 – 45cm

Percentage sand: -

00 - [15.5 + 0.36] 27 – 20] - 2.0] 2 67.96%

Percentage clay: -

5.5 + 0.36 [27 - 20] - 2.0] 2 = 12.04%

Percentage silt: -

100 – [67.96 + 12.04] = 20%

P_1 45 – 60cm

Percentage sand: -

100 - [16 + 0.36] 27 – 20] - 2.0] 2 = 66.96%

Percentage clay: -

8 + 0.36 [27 – 20] – 2.0] 2 = 17.04%

Percentage silt: -

100 – [66.96 + 17.04] = 16.0%

P_2 0 – 15cm

Percentage sand: -

100 - [4.5 + 0.36] 26 – 20] - 2.0] 2 = 90.68%

Percentage clay: -

0.0+ 0.36 [27 – 20] – 2.0] 2 = 1.04%

Percentage silt: -

100 – [90.68 + 1.04] = 8.2%

P_2 15 – 30cm

Percentage sand: -

100 - [12.5 + 0.36] 27 – 20] - 2.0] 2 = 73.96%

Percentage clay: -

1.5 + 0.36 [27 – 20] – 2.0] 2 = 4.04%

Percentage silt: -

100 – [73.96 + 4.04] = 22%

P_2 30 – 45cm

Percentage sand: -

100 - [20.5 + 0.36] 27 – 20] – 2.0] 2 = 57.96%

Percentage clay: -

3 + 0.36 [27 – 20] – 2.0] 2 = 7.04%

Percentage silt: -

100 – [57.96 + 04] = 35%

P_2 45 – 60cm

Percentage sand: -

100 - [28.5 + 0.36] 27 – 20] - 2.0] 2 = 41.96%

Percentage silt: -

4.5 + 0.36 [27 - 20] – 2.0] 2 = 10.04%

P_3 0 – 15cm

Percentage sand: -

100 - [10.5 + 0.36] 27 – 20] - 2.0] 2 = 77.96%

Percentage clay: -

1 + 0.36 [27 – 20] – 2.0] 2 = 3.04%

Percentage silt: -

100 - [77.96 + 3.04] = 19.4%

P_3 15 – 30cm

Percentage sand: -

100 - [12.0 + 0.36] 27 – 20] - 2.0] 2 = 74.96%

Percentage clay: -

1.5 + 0.36 [27 – 20] – 2.0] 2 = 4.04%

Percentage silt: -

100 - [74.96 + 4.04] = 21%

P_3 30 – 45cm

Percentage sand: -

100 - [13.5 + 0.36] 27 – 20] - 2.0] 2 = 71.96%

Percentage clay: -

2.0 + 0.36 [27 – 20] – 2.0] 2 = 5.04%

Percentage silt: -

100 - [71.96 + 5.04] = 23.0%

P_3 45 – 60cm

Percentage sand: -

100 - [15.0 + 0.36] 27 – 20] - 2.0] 2 = 69.96%

Percentage clay: -

$2.5 + 0.36\ [27 - 20] - 2.0]\ 2 = 6.04\%$

Percentage silt: -

$100 - [68.96 + 6.04] = 25\%$

From the U.S.D.A. textural triangle, the soil class is sandy loam.

B. Bulk Density (BD)

$$\frac{467.745}{288.67} = 1.62 gm/cm^3$$

C. Permanent Wilting Point (P.W.P)

$$\frac{145.75 - 142.98}{142.98} \times 100 = 1.94\%$$

D. Available Water (A.W)

$$\frac{60\ (7.49 - 1.94)}{100} = 3.33 cm$$

E. Net Water Application (F_n)

Moisture deficit (M.D) is taken as 50% Larry (1988)

$3.33 \times 0.5 = 1.7 cm$

F. Gross Application (F_g)

$$\frac{1.7}{0.80} = 2.13 cm$$

G. Basic Infiltration Rate (I)

$= 3.7 cm/hr$

H. Irrigation Application Time (T_n)

$$\frac{2.13 cm}{3.7 cm/hr} = 0.58 hr$$

I. Volume of Core Sampler

$3.142\ (3.5)^2\ (7.5) = 288.67 m^3$

APPENDIX F

MONTHLY IRRIGATION FREQUENCY (I.F) IN DAYS

November	-	17/4.538	= 4 days
December	-	17/4.509	= 4 days
January	-	17/4.593	= 4 days
February	-	17/8.776	= 2 days
March	-	17/7.446	= 2 days
April	-	17/5.073	= 3 days

The average irrigation frequency = 3 days

APPENDIX G

TABLE G INTAKE FAMILY AND FURROW / Basin ADVANCE COEFFICIENTS

Intake family	a	b	c	f	g
0.05	0.6334	0.618	7.0	7.16	1.088×10^{-4}
0.10	0.6198	0.661	7.0	7.25	1.251×10^{-4}
0.15	0.7110	0.683	7.0	7.34	1.414×10^{-4}
0.20	0.7772	0.699	7.0	7.43	1.578×10^{-4}
0.26	0.8534	0.711	7.0	7.52	1.741×10^{-4}
0.30	0.9246	0.720	7.0	7.61	1.904×10^{-4}
0.35	0.9957	0.729	7.0	7.70	2.067×10^{-4}
0.40	1.064	0.736	7.0	7.79	2.230×10^{-4}
0.45	1.130	0.742	7.0	7.88	2.393×10^{-4}
0.50	1.196	0.748	7.0	7.97	2.656×10^{-4}
0.60	1.321	0.757	7.0	8.15	2.883×10^{-4}
0.70	1.443	0.765	7.0	8.33	3.209×10^{-4}
0.80	1.560	0.773	7.0	8.50	3.535×10^{-4}
0.90	1.674	0.779	7.0	8.68	3.862×10^{-4}
1.00	1.786	0.785	7.0	8.86	4.188×10^{-4}
1.50	2.284	0.799	7.0	9.76	5.819×10^{-4}
2.00	2.753	0.808	7.0	10.66	7.451×10^{-4}

Intake (see equations [13.1] and [13.40]) Advance (see equation [13.35])

$F = (aT^{b} + c)\ P/W$, mm $\qquad T_T = \frac{x}{f}\, e^{(gx/QS^{1/2})}$, min

T = minutes $\qquad$ Q = furrow inflow

$\frac{P}{W} = \frac{\text{Wetted perimeter}}{\text{Furrow spacing}}$ $\qquad$ S = furrow slope

x = distance

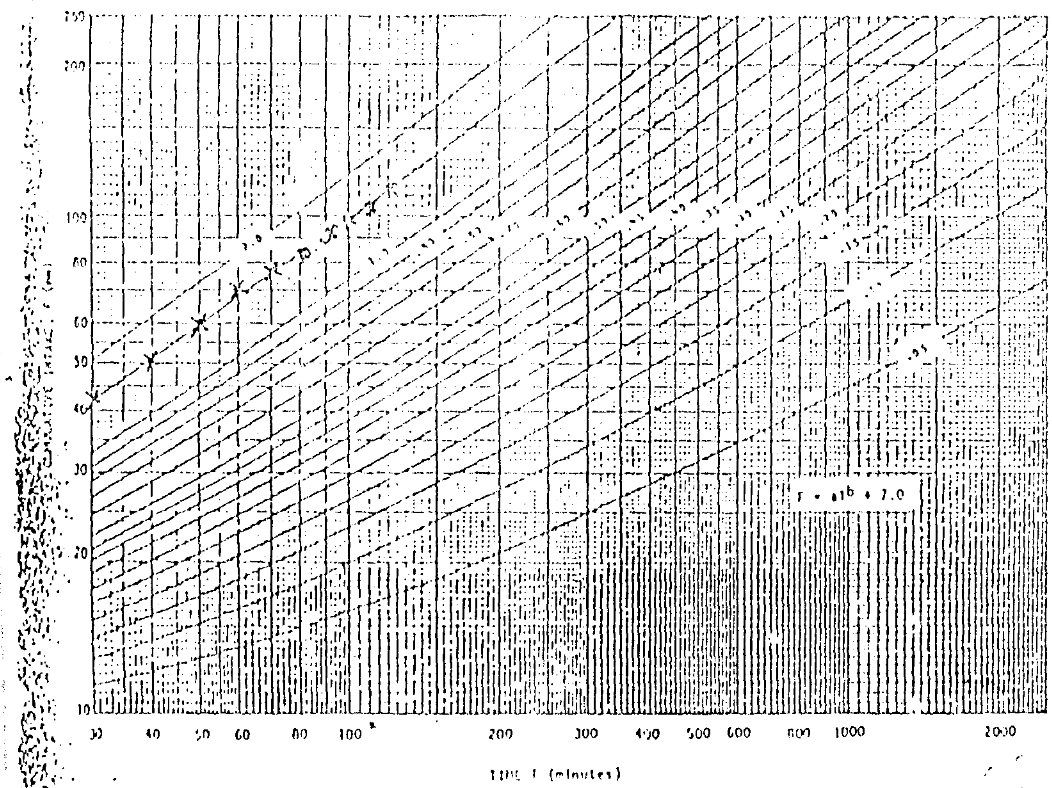
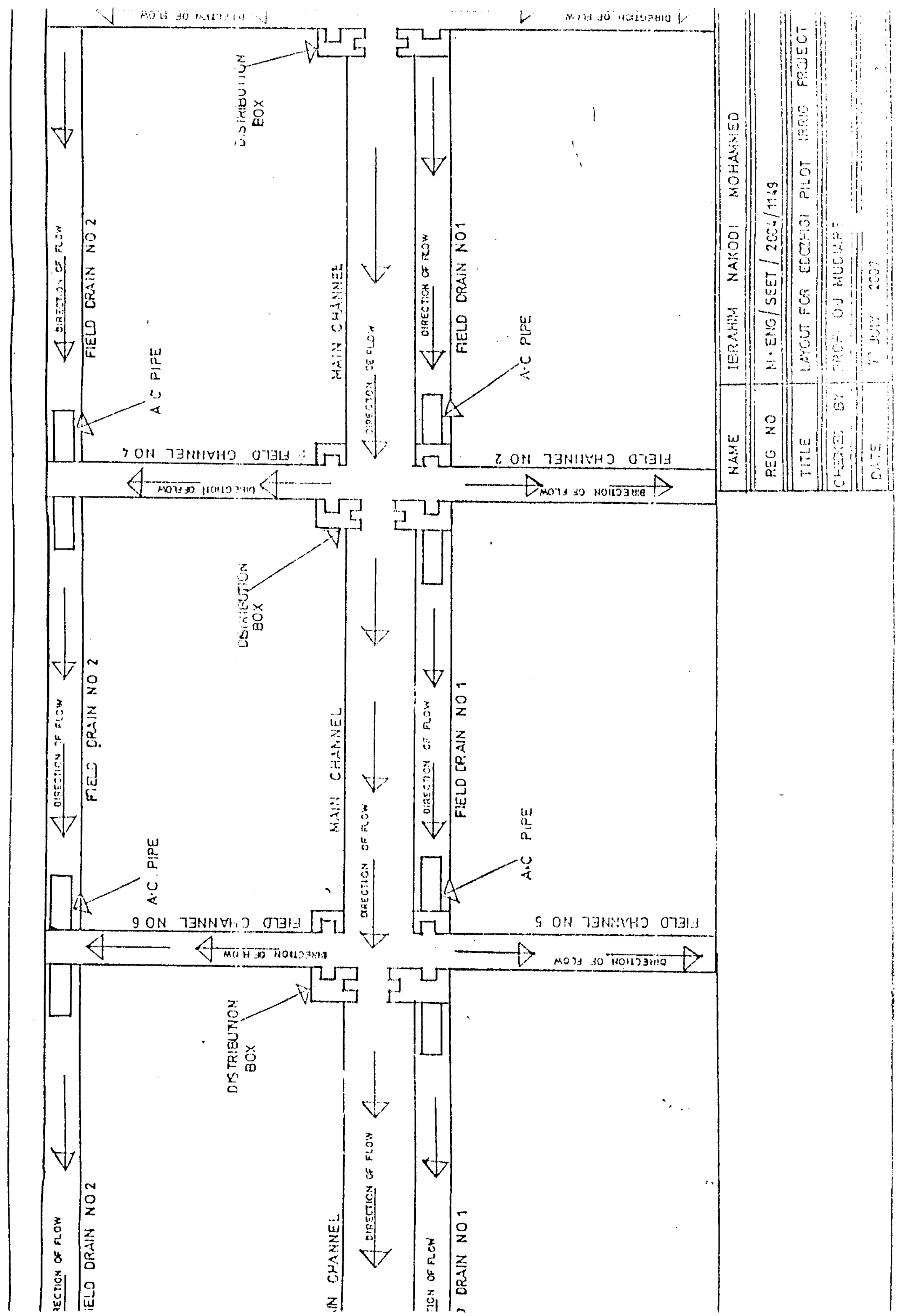
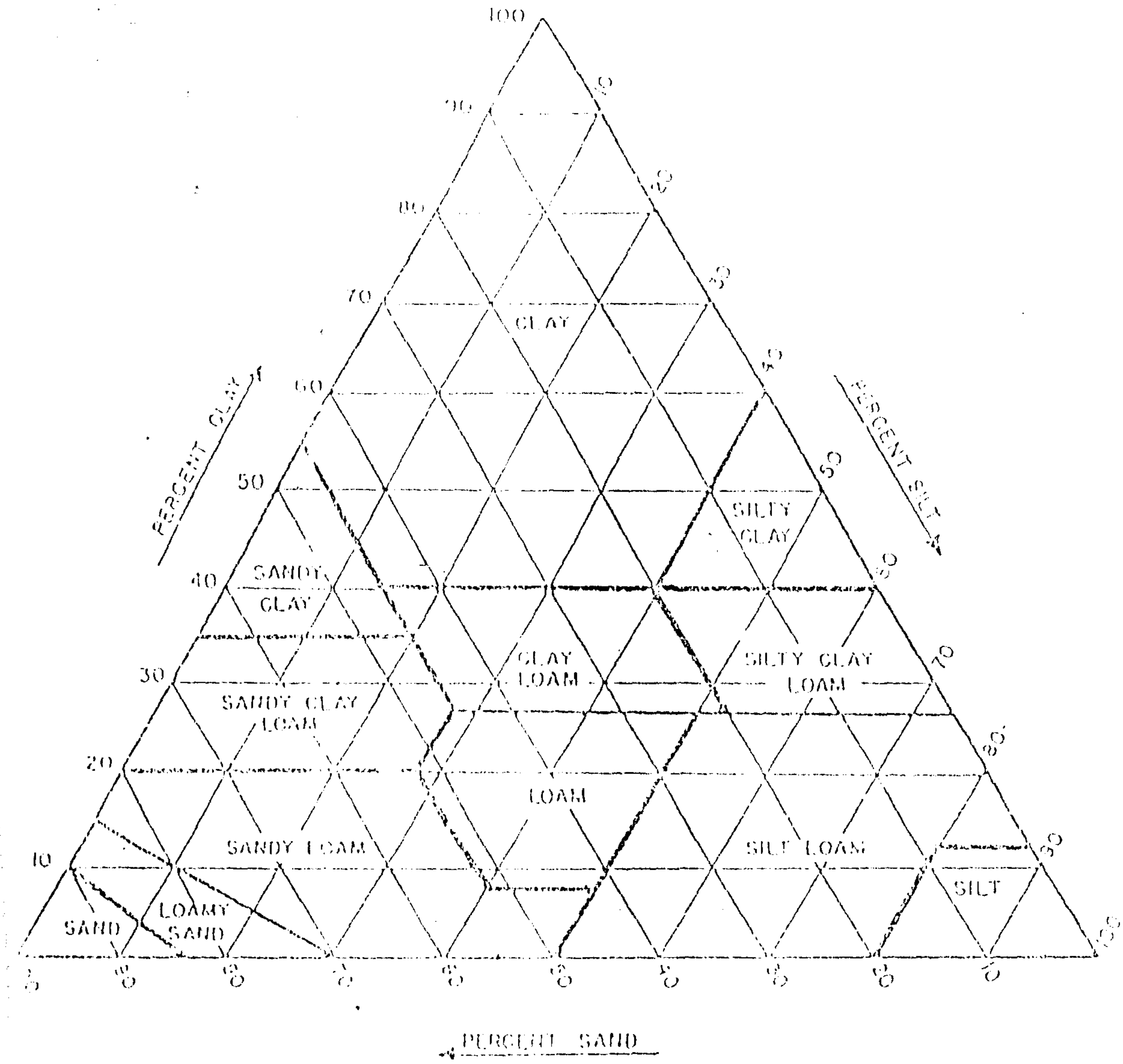


FIG. G Intake families (USDA, 1979).


NAME IBRAHIM NAKODI MOHAMMED
REG NO M·ENG/SEET/2004/1149
TITLE LAYOUT FOR EDOZHIGI PILOT IRRIG PROJECT
CHECKED BY PROF. O.J. MUDIARE
DATE JULY 2007
FIELD DRAIN NO 2
FIELD DRAIN NO 1
MAIN CHANNEL
DIRECTION OF FLOW
DISTRIBUTION BOX
A·C PIPE
FIELD CHANNEL NO 4
FIELD CHANNEL NO 2
FIELD CHANNEL NO 6
FIELD CHANNEL NO 5

APPENDIX K



Proportions of sand, silt, and clay in the basic soil-textural classes. (From U.S. Dept. Agr. Handb. 18. Soil Survey Manual. 503 pp., illus. 1951.)

1

Down stream face of the Dam supplying water to the project

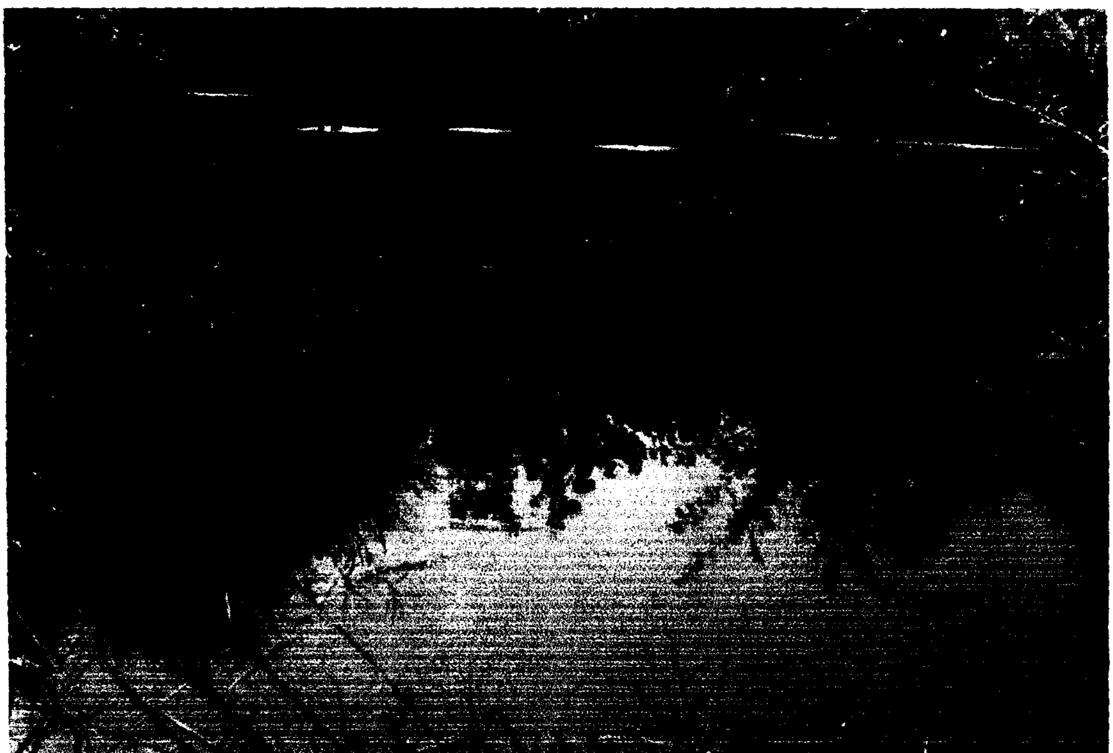
2

Down stream face of the Dam

3

Take off channel with control gate.

4

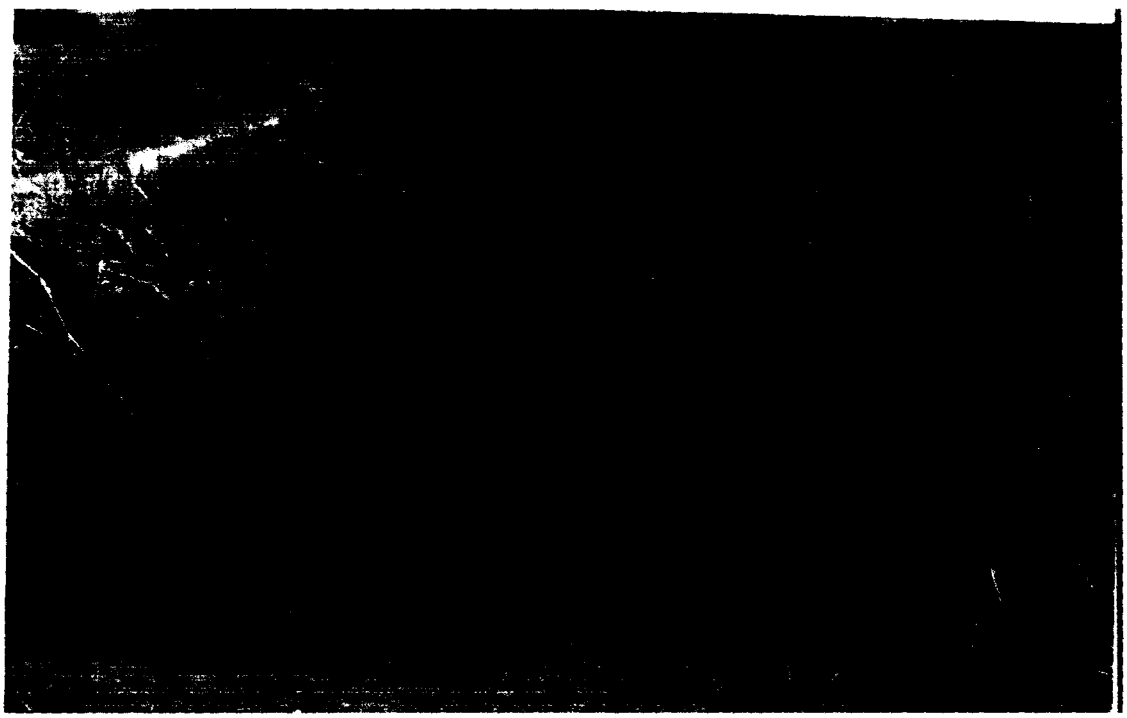


Upstream side of the Dam with arrow pointing
to the takoff channel to the field.

5

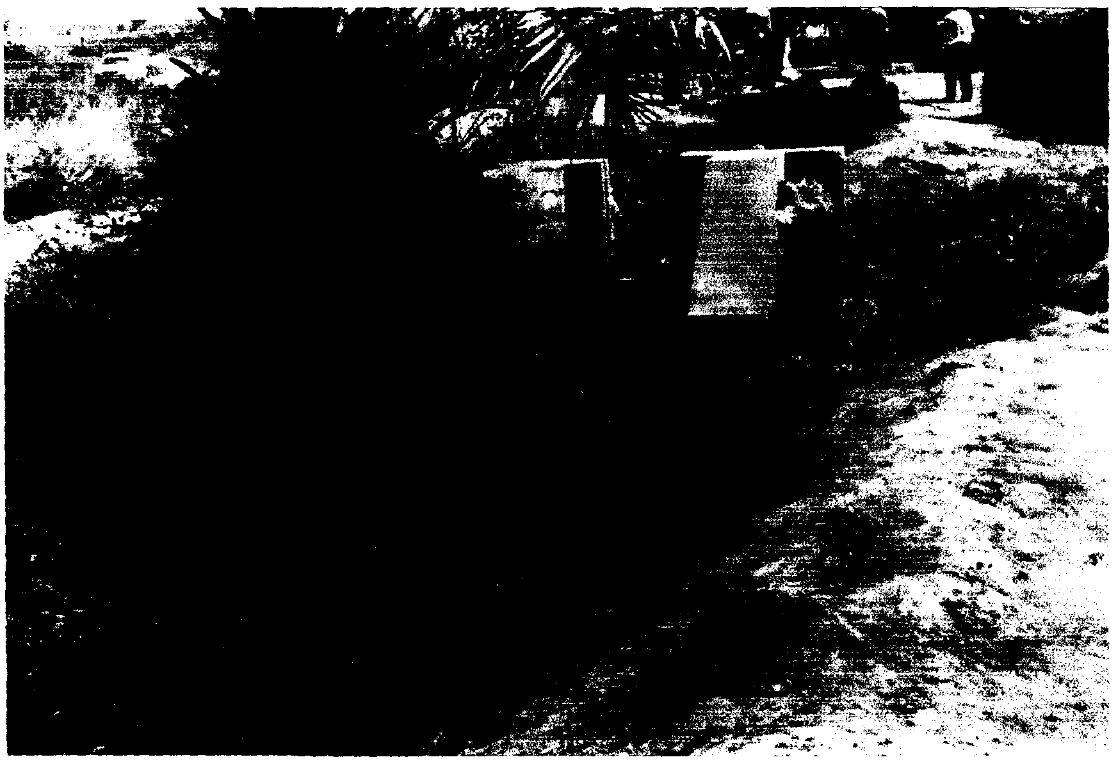
Diversion structure with washed away channel embankment

6



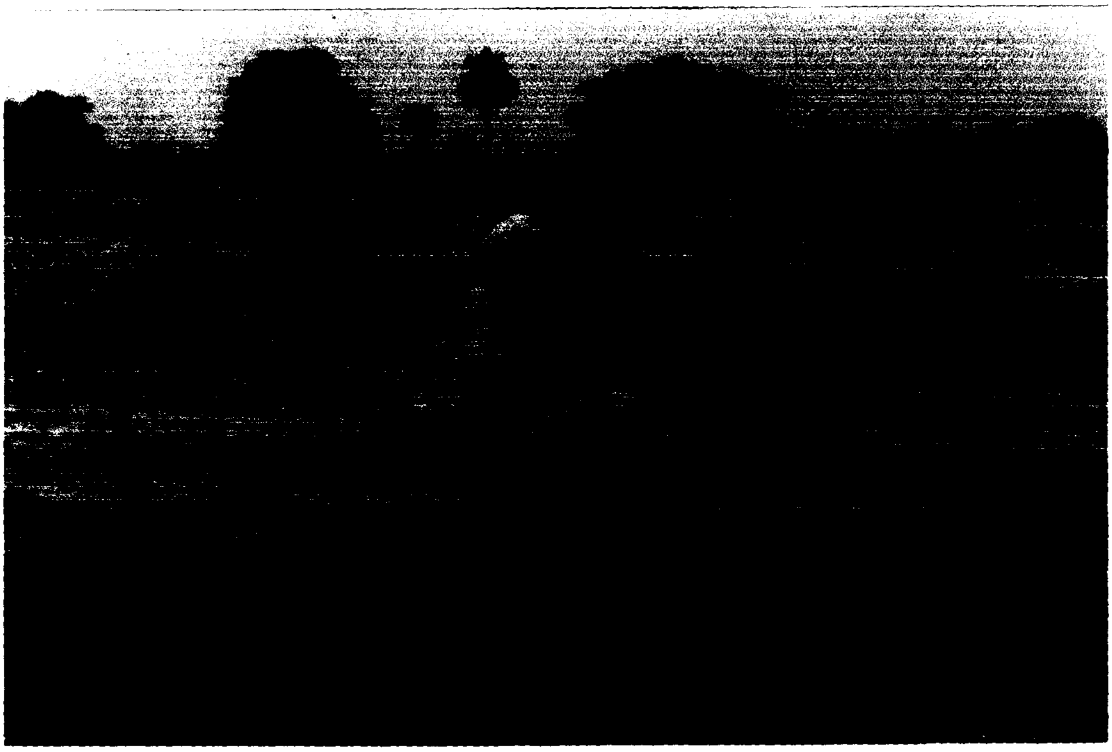
Take off channel overgrown with weeds.

A hanging Drop structure and takeoff of field channel with washed away channel embankments.



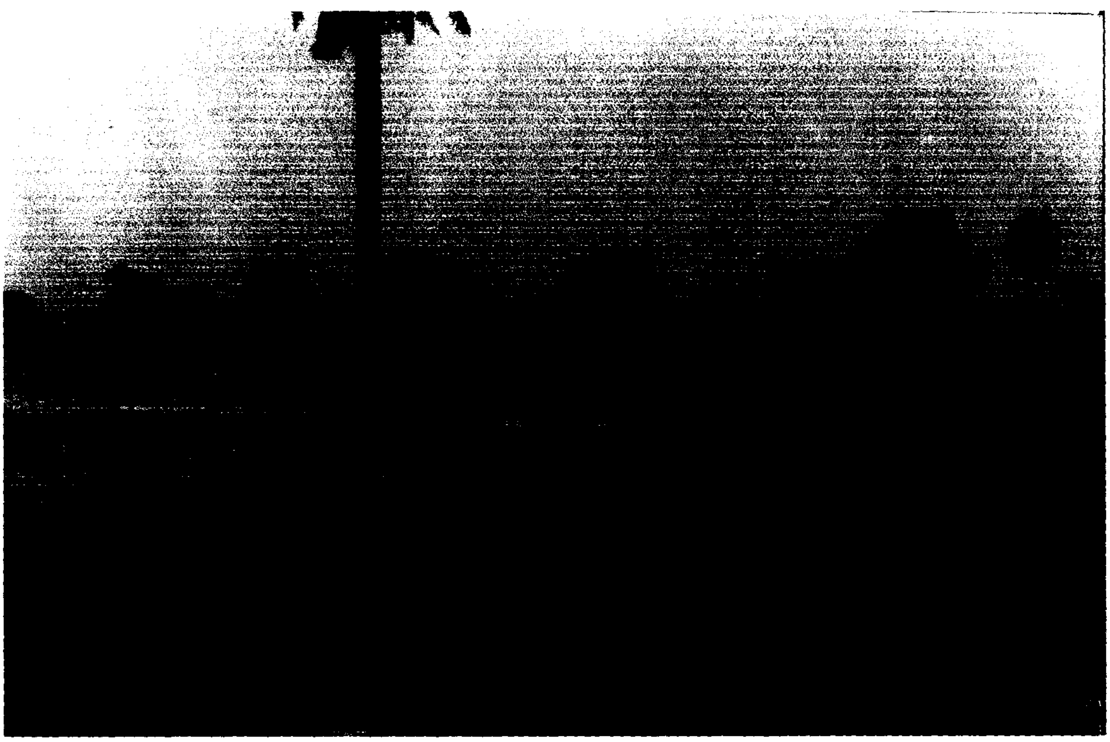
Hanging Drop structure with Palm Tree growing inside the conveyance channel.

11



Disappearing field channel with hanging control gate.

12



Rice field with no trace of field channel.

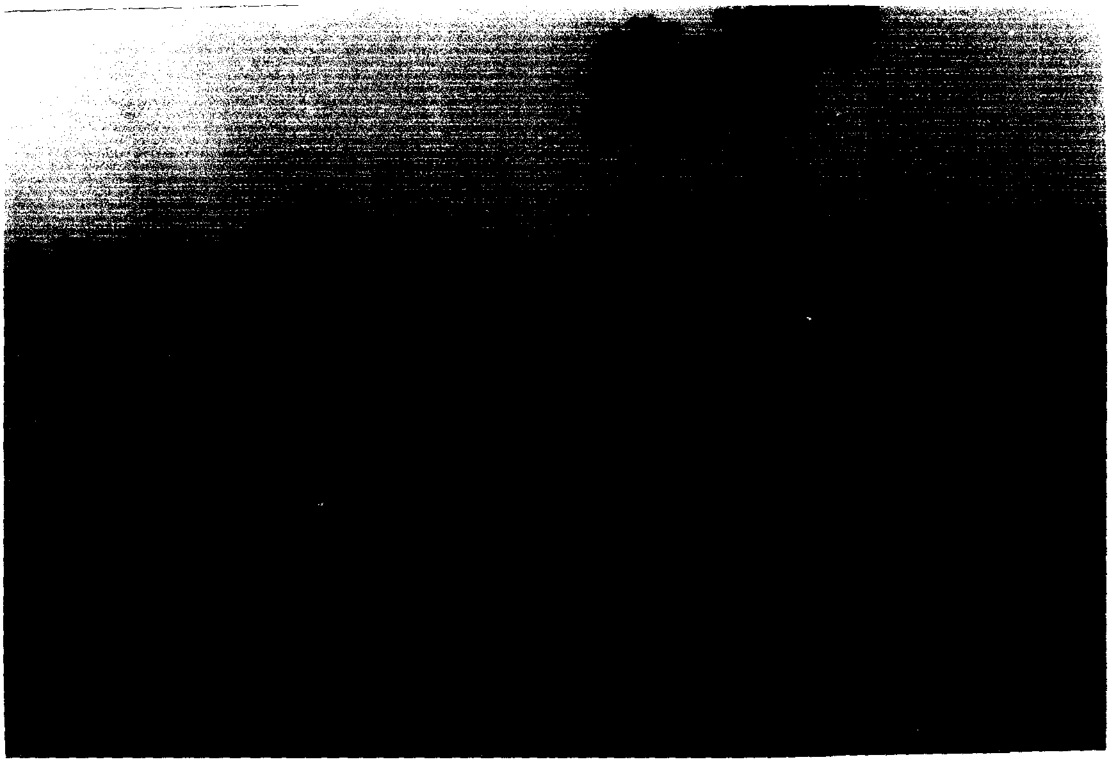
13

Re-constructed channel control gate with block resulting from communal effort.

14

Re-constructed channel control gate.

15



Rice field with arrow pointing to washed away field channel

16

Rice field with arrow indicating bare field without any sign of field channel.