REPUBLIC OF KENYA

MINISTRY OF WATER AND IRRIGATION

PRACTICE MANUAL

FOR

WATER SUPPLY SERVICES

IN

KENYA

OCTOBER 2005

KENYA – BELGIUM STUDY AND CONSULTANCY FUND

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PREFACE

The review and updating of the “Design Manual for Water Supply in Kenya, 1986” has involved a cross-section of the key stakeholders including participation from DONORS, NGOs, MENR, NWCPC, MOLG, LAs, UON, KBS, IEK, ERB and members of the private sector, in particular the Consulting Engineers. There was a high level consultative meeting for building consensus bringing together a wide spectrum of stakeholders on the sector, which culminated in the adoption of the updated “Practice Manual for Water Supply Services in Kenya, 2005”.

We firmly believe that this Practice Manual for Water Supply Services in Kenya 2005 will continue to provide useful guidelines in Waterworks facilities to meet the challenges in the water sector reforms anticipated through Water Act 2002, the amendment in the administration of the development and management of water resources, the enormous technical input in water facilities engineering works, the revolution of technological and environmental development, the socio-political changes, and the changes in the living standards. This is an essential handbook of technical know-how for Water Supply facilities of varying scale and uses. It gives the basics in practice in water supply facilities engineering in Kenya today.

Each water supply system is strongly linked to a specific local condition. The demand for water differs substantially according to availability, climate, and the structure of houses, life style and living standards. Therefore, the demand and source of water supply is to be determined by the natural and socio-economic conditions of each community, which means planning and design approaches applicable to one system are never the same on another different system. The mere copying of a successful system in one area does not necessary guarantee the same success in another area. We must always bear in mind that planning, design and subsequent implementation of a water supply system will never be served by mere imitations. Nevertheless, the fundamental concept and common techniques that form the basis of this water supply practice manual are the same.

The Development and Management of water facilities and the handling of their effluents are, so to speak, both sides of the same coin. Therefore, at the stage of planning, designing and construction of water supplies, the actual development and management of sanitation facilities must be fully conceptualized for their safe and efficient operation. The Planning, Design and implementation of water facilities should go along with thorough investigation of the functional and operational characteristics of the sanitation facilities, and aim to establish such complementary facilities that will best meet the actual conditions of sustainable water and sanitation systems.

The Director, Ministry of Water and Irrigation wishes to convey gratitude to all persons and institutions who have been involved in the review, updating and publication of “Practice Manual for Water Supply Services in Kenya, 2005” and in particular to Kenya – Belgium Study and Consultancy Fund.

Eng. R.N. Gakubia, HSC,
Director of Water,
Ministry of Water and Irrigation.

LIST OF ABBREVIATIONS

The following abbreviations are frequently used in the Manual and in the Water Sector in general.

AC	Asbestos Cement
AV	Air Valve
B/H	Borehole
BPT	Break Pressure tank
BS	British Standard
CI	Cast Iron
CWP	Communal Water Point
DI	Ductile Iron
DR	Design Report
FD	Final Design
FDR	Final Design Report
FS	Feasibility Study
GS	Galvanised Steel
IC	Individual Connection
ISO	International Organisation for Standardization
KBS	Kenya Bureau of Standards
KB	Kenya Standard
mhw	Metre Head of Water
MoWI	Ministry of Water and Irrigation
MUWS	Minor Urban Water Supply
NC	Non-individual Connection
NPSH	Net positive Suction Head
O & M	Operation and Maintenance
PD	Preliminary Design
PDR	Preliminary Design Report
RE	Resident Engineer
RSF	Rapid Sand Filter
RWS	Rural Water Supply
S/H	Self-Help
SI	System International
SSF	Slow Sand Filter
uPVC	Unplasticized Polyvinyl Chloride
UWS	Urban Water Supply
WO	Washout.

Other abbreviations that may be encountered in the Manual and have become a common feature in the water sector includes:

AM	Aide Memoir
ASAL	Arid and Semi-Arid Lands
BTC	Belgium Technical Cooperation
CB	Catchment Board
CBO	Community Based Organization

DCU	Dam Construction Unit
DRU	Dam Rehabilitation Unit
EIA	Environmental Impact Assessment
ENNIDA	Ewaso Nyiro North Development Authority
GIS	Geographical Information System
GOK	Government of Kenya
GDP	Gross Domestic Product
GTZ	German Agency for Technical Co-operation
IMC	Inter-ministerial Committee
PRSP	Poverty Reduction Strategy Pager
KENGEN	Kenya Electricity Generating Company
KEWI	Kenya Water Institute
KMD	Kenya Metereological Department
KWS	Kenya Wildlife Services
LAs	Local Authorities
LBDA	Lake Basin Development Authority
MENR	Ministry of Environmental and Natural Resources
MLS	Ministry of Lands and Settlement
MoA	Ministry of Agriculture
MoH	Ministry of Health
MoLG	Ministry of Local Government
MoR&DW	Ministry of Roads & Public Works
NIB	National Irrigation Board
NWCPC	National Water Conservation and Pipeline Corporation
NWMP	National Water Master Plan
NWP	National Water Policy
NGO	Non-Governmental Organisation
OP	Office of the President
PSP	Private Sector Participation
SHG	Self Help Groups
SIDA	Swedish International Development Agency
SSIPs	Small Scale Independent Providers
TARDA	Tana and Athi Rivers Development Authority
TNA	Training Needs Assessment
WAB	Water Apportionment Board
WAB	Water Appeals Board
WB	World Bank
WRM	Water Resources Management
WRMA	Water Resources Management Authority
WSB	Water Service Board
WSP	Water Service Providers
WSRB	Water Services Regulatory Board
WSRU	Water Sector Reform Unit
WSS	Water Supply and Sanitation
WSSB	Water Supply and Sanitation Services Board
WSSP	Water Supply and Sanitation Services Providers
WSTF	Water Service Trust Fund
UFW	Un-accounted for water

LIST OF SYMBOLS

The following symbols are frequently used in the Manual and should be used in design reports and drawings.

Symbols	Unit (Example)	
A	m^2	Area
C	m^3	Capacity
C	-	Coefficient of roughness (Manning formula)
C	m/s	Celerity
D	m	Diameter
E	N/mm^2	Elasticity Modulus
e	%	Efficiency
H	m	Height
I	%	Inclination
I	Kwh/m^2-day	Radiation
K	mm	Coefficience of roughness (Colebrook)
Q, q	1/s	Flow
R, r	m	Radios
R	mm	Rainfall per year
T	month	Time
t	mm	Thickness
V	m/s	Velocity
L	Kwh/day	Load demand
N	Kw	Power
P	Kw	Power

LIST OF GRAPHS

PART A; WATER SUPPLY

PART B: IRRIGATION

LIST OF TABLES

PART A: WATER SUPPLY

PART B: IRRIGATION

LIST OF MAPS

PRACTICE MANUAL

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WATER SUPPLY SERVICES

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KENYA

PART A

WATER SUPPLY

LIST OF CONTENTS

1. INTRODUCTION

1.1 BACKGROUND

First set of guidelines for development studies were produced within the water development in 1971, of which certain parts were incorporated in the reports from WHO "Sectoral Study and national programming for community and rural water supply, sewerage and water pollution control" of 1973. The WHO Study comprised guidelines that were used extensively by design engineers during the period 1973-76.

In 1976 the Water Department embarked on an ambitious scheme to produce a more comprehensive design manual that besides technical guidelines gave information on pure administrative and organizational matters. Planners and engineers in the Water Ministry used the work re-vetted in the Design Manual dated May 1978 extensively for the design of water supply projects.

In 1986, "Design Manual for Water Supply in Kenya, 1986" was published. It was partly based on the previously mentioned Design Manual, however numerous addition and changes were incorporated to include experience gained and technological changes.

The review publication "Practice in Water Supply Services in Kenya, 2005" is an update of the "Design Manual for Water Supply in Kenya, 1986" that incorporates the latest technological changes as well as water sector reforms to enhance harmony and standardization in planning, design, construction and management of water supply services countrywide.

1.2 PURPOSE

The Practice Manual has been compiled with the aim of providing guidelines and criteria for all those involved in Water Sector.

The intent is to streamline the water supply activities and guarantee high standards in water supply services.

By so doing it is believed that many past mistakes can be avoided and that resources can be used more effectively if the framework within which they are deployed is clearly defined.

1.3 WATER SUPPLY ADMINISTRATION

1.3.1 Legal Framework

The Water Act 2002 provides the legal – Institutional framework for the management and development of Kenya's water resources and the provision of water services.

1.3.2 Institutional Framework

Under the Water Act 2002, an institutional set up have been created with clear delineation of roles in the Water sector as follows; -

Table 1.1: Institutional Set up for Water Supply Services.

REF	ACTOR	ROLES
1	Ministry of Water & Irrigation	• Water Conservation • Establish independent National Water Boards /WSRB Regulatory Board • Policy formulation/Strategy development • Water quality/pollution control, standards (National, local) regulator • Research and Training • Register professionals, contractors etc. • Sector coordination and financing. • Establishing relevant institutions and legal framework.
2	WSRB Regulatory Board	• Regulate WSRBs • Determine applications from WSRBs and WSS Providers (licenses) • Approve operational plans, tariffs, rates, levies, charges • Monitor performance and quality • Setting and enforcing performance standards and codes of practice • Promote fair competition among service providers • Conduct studies related to economy and efficiency of WSS (e.g. viability studies) • Classify/maintain register of WSS Providers • Disseminate information to consumers • Advise WSSBs on handling consumer information complaints • Develop sector guidelines and procedures • Report to parliament through minister

		responsible for water • Promote water conservation and demand management • To perform such other functions as are incidental to the above
3	WSB	• Holding/lease assets • Prepare business plans/operational plans • Borrow and invest/pay • Contract out • Hold WSSBs license • Tariff setting • Expand network • Hold abstraction license • Submit financial reports • Submit quarterly reports on performance to WSS Regulatory Board
4	WSPs (LAs, PSP, NGOs, SHGs, others etc.)	• Bid for schemes operations • Obtain WSO license • Operate schemes on a cost recovery basis • Comply with quality and service levels • Obtain providers license from the WSS Regulatory Board
5	Ministry of Health	• Policy formulation on environmental Sanitation/Public Health Act
6	Water Appeals Board (WAB)	• Adjudicating disputes within the Water Sector
7	WRMA	• Implementation of Policies and strategies relating to management of water resources • Development of catchments level management strategies including appointments of Catchments Area Advisory Committees (CAAC) and their facilitation
8	WSTF	• Assisting financing of provision of water supplies in areas that are inadequately provided for

1.3.3 Water Supply Service Boards

The Country has been subdivided into seven Water Service Boards (WSB) as shown in Table 1.2 below and also graphical illustrations in Appendix A.1.

Table 1.2: Water Supply and Sanitation Service Boards.

Serial No.	Name of WSB	No. of Districts covered	Area (Km^2)	Population (1999 census)
1	Nairobi	6	40,130	5,617,000
2	Central	13	52,777	5,032000
3	Coast	7	82,816	2,487,000
4	Rift valley	8	113,771	2,999,000
5	Northern	9	244,864	1,703,000
6	Lake Victoria North	11	16,977	5,135,000
7	Lake Victoria South	16	20,340	5,730,000

1.4 WATER SUPPLY PLANNING

1.4.1 Basic Planning

Basic planning for a water supply project involves an orderly consideration of the project from the original statement of purpose through the evaluation of alternatives to the final decision on course of action. It includes all the work associated with the design of the water supply except the detailed engineering of the components, forming the basis of whether to proceed or to abandon the project. Water planning in Kenya usually passes through two phases: -

Phase I - Reconnaissance study that is intended to screen and eliminate those projects or actions that are clearly infeasible without extensive study, hence, identifying feasible alternatives requiring further study.

Phase II - Evaluation of the feasibility of the “feasible” activities for the selection of desirable actions.

1.4.2 Basic Policy

The basic plan for a water supply project should be formed on the basis of the following principles: -

(i) Capacity to satisfy demand to target year – Horizon of 20-25 years.
(ii) Provision of safe and sanitary water.
(iii) Wise, effective and efficient use of the water resource.
(iv) Safe and sound O & M of Water facilities with no negative environmental impacts.
(v) The system must be in conformity with Water Act 2002.
(vi) Enhancement of quality of living standards.
(vii) Appropriate Technology relevant to beneficiaries.

1.4.3 Basic Objectives

A feasible water supply project should effectively serve its objectives without negative environmental impacts. Such objectives for a feasible water supply project include: -

(i) Enhancement of national socio-economic development.
(ii) Enhancement of increase in national food production.
(iii) Enhancement of quality of environment.
(iv) Enhancement of reduction in poverty.
(v) Enhancement of increase in Industrialization.
(vi) Enhancement of quality of living standards.
(vii) Enhancement of quality and conservation of water resources.

1.4.4 Water Planning Coordination

The development and management of water resources in Kenya for specific purposes is the responsibility of several agencies. To provide some coordination between these agencies and establish common methodologies so that project studies are comparable, the Government of Kenya has enacted the Water Act 2002. Through the Water Act 2002, administrative centers such as Water Resources Management Authority (WRMA), Water Services Regulatory Board (WSRB), Water Services Trust Fund (WSTF) and Water Appeal Board (WAB) have been established to ensure harmony and form a bridge between the National planning effort and the water development and management agencies.

1.5 SELECTION CRITERIA PRINCIPLES

1.5.1 General

The function of selection criteria for a water supply service is to decide when particular areas should be provided with water services. The very serious consequences that arise from lack of water services in the urban areas makes the existence of urban population alone a sufficient criteria for providing water supply services. The selection for rural water services is based to a large extent on requests for Rural water Schemes. It is not obvious where the need is greatest and hence the criteria for lowest cost per head should be the guiding principle for selection.

1.5.2 Parameters for Selection

- The urban population is normally captive of the water supply services and has no alternative water source in case of a failure. An inadequate urban water supply service is a health hazard and could result in epidemics.

- The quantity and quality of the socio-economic benefits that arises or will be impacted by the water supply services.

- The likely failure, in the near future, for the water supply scheme to meet water demand for the current population.

1.6 INSTRUCTION TO USERS OF THE MANUAL

This manual should not replace the sound judgment of the user and does not relieve the Water Development and Management Personnel of his responsibility to Develop and Manage an economical and effective water supply.

However any deviation from the general guidelines or the detailed criteria shall be justified and brought to the attention of the Director of Water and Irrigation in the Ministry of Water and Irrigation. As the manual to a great extent is based on experience from the design, construction and operation of water supplies in Kenya and also on policy decisions in the Ministry the justification for any deviation has to be strong if it shall be accepted.

The manual contains guidelines for simple as well as more complicated technical solutions in order to give the Water Development and Management Personnel a range of options that will cover the needs of most water supplies. However simplicity and reliability should always take preference over sophistications.

No pumping, treatment or mechanical equipment should be proposed without convincing technical or economical justification.

By its very nature, the work on a manual should never be considered complete. Any suggestions on additions and improvements would be most appreciated.

1.7 CLASSIFICATIONS

1.7.1 Classification of Urban Service Centres

Ministry of Lands and Settlement has given the service centers classified as urban areas the following designations:

- Principle Towns
- Urban Centres
- Rural Centres
- Market Centres

- Local Centres

The long-term guidelines for the location of infrastructure facilities in the various levels of centers and the names and current status of the various centers are given in Appendix A.

Note that Nairobi as the capital of Kenya has been given special status and is not included in the list.

1.7.2 Urban Housing Classes

Housing is generally classified as high, medium or low class housing and is defined for planning purposes as follows: -

High-class housing: generally low-density development plots of 0.2-0.8 hectares. Houses furnished with internal pipes and hot water, electricity supply, refrigerator, electric cooker, bathrooms, WC and internal arrangement for cloth and dish washing.

Medium-class housing: generally low-density development plots of about 0.05 – 0.1 hectares. Houses furnished with at least internal piped cold water, gas or electric refrigerator and cooker, shower, WC and internal arrangement for dish washing, splash area outside for cloth washing.

Low-class housing: generally high-density development. Houses furnished with very simple piping for cold water or only external water points for cloth and dish-washing (splash area). Includes site and service scheme housing.

1.7.3 Classification of the Land Potential

The following method, which takes into account only the rainfall, may be used for a very rough classification.

High-potential area is normally considered to be an area with an annual rainfall more than 1000mm. However, local factors such as very concentrated annual rainfall, adverse topography, soil conditions, special attitudes, tourism, roads, irrigation etc., should be taken into account.

Medium-potential area is normally considered to be an area with an annual rainfall of between 500mm and 1000mm. However, local factors may be adjusted as above.

Low-potential area is normally considered to be an area with an annual rainfall of less than 500mm. However, local factors may be adjusted as above. In low-potential areas the schemes are invariably small, which serve trade centers, institutions and some people around the center.

A more accurate classification where also the soil type, location and other factors are being considered is shown in Appendix A.

1.7.4 Livestock Potential

The Livestock potential should be determined in liaison with the District Agricultural Officer of the area. If the local information is lacking the following tables may be used as approximate guide.

Rainfall (mm)	**Area per Livestock Unit (ha)**
600-800	1.2
800-1000	1.0
1000-1200	0.8
1200-1700	0.6
Over 1700	0.4

One livestock unit *is defined as high-grade stock cattle over 2 years.*

Conversion of other types of cattle for the purpose of calculating the livestock potential can be made using the following tables: -

Fraction of a Livestock Unit

Livestock	High Grade Stock	Local Stock
Cattle 2 years	1.00	0.67
Cattle 1-2 years	0.67	0.44
Cattle 2 years	0.33	0.22
Mature sheep/goats	0.20	0.13
Camel	1.00	0.67
Donkey	0.22	0.22

Note that this conversion table must not be used for the estimation of the water demand for different categories of cattle. For this purpose see chapter "Water Demand".

2 WATER DEMAND

2.1 DESIGN PERIOD

2.1.1 Projection Years

Water demand projections should normally, be made for the "initial" the "future" and the "ultimate" year. The "initial" year is the year when the supply is expected to be taken into operation that may be assumed to be 0-5 years from the date of the commencement of the preliminary design. The "future" is 10 years and the "ultimate" year 20 years from the initial year. Once the initial, future and ultimate years have been determined for a project they should not normally be changed during the design period.

2.1.2 Design Demand

A water supply should normally be designed for the ultimate demand. However phasing of the implementation will often become a financial necessity and the possibilities of phasing should therefore be examined using the initial and future demand projections.

Mechanical equipment is often designed for shorter periods. Further see chapter "pumps and power sources".

2.2 RESIDENTIAL DEMAND

2.2.1 Population Projections

The present population should be estimated based on the latest (1999) census. However, sometimes the figures are unreliable and should be crosschecked with information obtained form other sources e.g. Chiefs.

The population in each sub-location in the rural area should be projected separately. Rural, market and local centers are usually included in the population figures for respective sub-location. Population figures for these centers will therefore have to be obtained by counting of houses etc.

The population in principal towns and urban centers should be analysed for different areas and income categories separately. High, medium and low class housing areas should be forecast independently.

To forecast the future population is difficult and therefore all possible information should be collected and evaluated. The following sources of information should be used in determining the likely future growth rate in rural areas: -

- The growth, which has taken place in the area in the past. Compare figures of the 1962, 1969, 1979, 1989 and 1999 census. It should be noted that location and sub-location boundaries often have been changed between two censuses.

- The growth, which has taken place in the District as a whole.
- The National Master Water plan
- Regional physical development plans
- The District Development Plan
- Settlement Plan
- The forecast by Central Bureau of Statistics
- The market study if carried out
- Land carrying capacity if possible to determine
- Opinion of the district administration, especially that of the District Development Officer.

The forecast of the future growth rates of principle towns and urban centers should be based on same sources of information as above where applicable together with information from the town councils, Ministry of Local Government or other local administration. In particular the town plan should be considered.

The population figures for 1979, 1989 and 1999 are given in Appendix A.

2.2.2 Service Type

The distribution between individual connection users (IC) and non-individual connection users (NC) i.e. consumers using kiosks or communal water points or share connections for the purpose of the demand projection for new supplies should be assumed to be as shown in the table below.

However, local factors may warrant deviation from the figures in the table, which only shall be construed as indicative. When the designed supply is an extension or completion of an existing supply, then the distribution of IC and NC is estimated after the monitoring of the existing situation.

Table 2.1: Service Type

	IC %			NC %		
	Initial	**Future**	**Ultimate**	**Initial**	**Future**	**Ultimate**
<u>Urban Areas</u>						
High and Medium Class Housing	100	100	100	0	0	0
Low class Housing	10	30	50	90	70	50
<u>Rural Areas</u>						
High potential	20	40	80	80	60	20
Medium potential	10	20	40	90	80	60
Low potential	5	10	20	95	90	80

2.3 LIVESTOCK DEMAND

2.3.1 Population Projections

The present livestock population should be estimated based on the livestock census usually available from the District Live-stock officer.

The forecast of the livestock growth should be based on:

- The historical data from livestock census
- Regional physical development plans
- District Development Plans
- National Master Water Plan
- The livestock carrying capacity (see chapter 1 Introduction – Livestock potential, 1.7.4)
- The market study if carried out.

In livestock projections, grade cattle, local cattle, small stock and other livestock should be estimated separately, poultry need not normally to be considered.

For the purpose of estimating the water demand for livestock the following conversion factors apply:

1 Grade cow	equivalent to	1 Livestock Unit (LU)
3 Indigenous cow	,,	1 Livestock Unit (LU)
15 Sheep or goats	,,	1 Livestock Unit (LU)
5 Donkeys	,,	1 Livestock Unit (LU)
2 Camels	,,	1 Livestock Unit (LU)

2.3.2 Service Level

It should be assumed that consumers with individual connections water their cattle from the piped water supply except where reliable alternative sources of water are available on the farms.

Consumers without individual connections will be expected to retain the traditional sources for the watering of cattle except where these sources are seasonal or unreliable.

The livestock-watering situation shall be examined for all rural supplies and detailed proposals for any measures to be taken in this respect shall be included as an integral part of the water supply design. Such special measures may comprise special water holes, dams or water pans for the cattle.

2.4 INSTITUTIONAL DEMAND

2.4.1 Schools

The development in educational facilities should be based on the existing situation, the plans of Ministry of Education and the projected growth of the population. For rough calculations it may be assumed that 30% of the population attend primary and/or secondary school.

2.4.2 Health Facilities

The development of health facilities should be based on the existing situation, the plans of Ministry of Health and the anticipated growth of the population. In the long term, one-health center and two to four dispensaries will be planned to serve about 35-40,000 people. The number of hospital beds can be assumed to 0.8 beds per 1000 people. Regional and District hospitals should be studied separately.

2.5 COMMERCIAL AND INDUSTRIAL DEMAND

2.5.1 Small Shops, Workshops, Restaurants, Bars etc.

The development of small-scale enterprises should be based on the existing situation. It should be anticipated that the future increase in commercial activity would be directly related to the growth of population.

2.5.2 Large Enterprises, Tourist Hotels, Military Camps, etc.

The development of large establishments should be examined in detail by interviewing relevant bodies. Urban areas marked as industrial areas in the town plan but for which the exact nature of the industry s not known, may be allocated an amount of water per area unit as shown in section "Water consumption rates". However, a realistic time plan over the exploitation of such areas must be proposed.

2.6 OTHER DEMANDS

2.6.1 Irrigation

The water demand projections should not include any provision for irrigation besides for very limited garden watering which is included in the per capita consumption rates. Where this is a must, the irrigation section, part B of this manual should be used.

2.6.2 Fire Fighting

- In urban areas where fire authorities exist should the demands be examined in collaboration with these.

- For urban and rural centers it is recommended that the capacity for fire fighting should not be less than 10 1/s during 2 hours. Further see chapter "Transmission and distribution lines".

- No provision will normally be necessary in Market and Local centers or in rural areas.

2.6.3 Internal Demand in the Water Works

- It should be assumed that 5% of the water production is used for backwashing of rapid sand filters where these are part of the treatment.

- Other internal uses than for rapid filtration may be neglected for the purpose of estimating the total water demand.

2.7 WATER CONSUMPTION RATES

2.7.1 General

- The water consumption figures include about 20% allowance for water losses through leakage and wastage.

- The figures are the consumption rates for which the supply system shall be designed. No additional peak-factors shall be applied to calculate the design demand.

- The rates are proposed as a guide and may be adjusted if different rates are shown to be more appropriate in a particular case. The rates represent the consumption of the average consumer category. Within a consumer category there may be considerable variations.

2.7.2 Rates

Table 2.2: Consumption Rates

CONSUMER	UNIT	RURAL AREAS			URBAN AREAS		
		High potential	Medium potential	Low potential	High Class Housing	Medium Class Housing	Low Class Housing
People with individual connections	1/head/ day	60	50	40	250	150	75
People without connections	1/head/ day	20	15	10	-	-	20
Livestock unit	1/head/ day	50			-		
Boarding schools	1/head/ day	50					
Day schools with WC without WC	1/head/ day	 25 5					
Hospitals Regional District other	1/bed/ day	 400 200 100 } + 20 1 per outpatient and day (minimum 5000 1/day)					
Dispensary and Health Centre	1/day	5000					
Hotels High Class Medium Class Low Class	1/bed/ day	 600 300 50					
Administrative offices	1/head/ day	25					
Bars	1/day	500					
Shops	1/day	100					
Unspecified industry	1/ha/day				20,000		
Coffee pulping factories	1/kg coffee	25 (when re-circulation of water is used).					

2.8 CONSUMPTION PATTERNS

2.8.1 Rural Areas Inclusive Rural, Market and Local Centres

- It should be assumed that all water is drawn between 7a.m. and 7 p.m. The same pattern applies for NC (CWP, Kiosk), IC and for livestock consumption.

- When the number of water users exceeds 1000 it be should assumed that the draw-off is constant through the 12-hour consumption period.

- Large institutions, industry etc. may have their own balancing reservoirs, which may reduce the peak demand. Such balancing reservoirs should be encouraged and considered when determining the design flow.

2.8.2 Principal Towns and Urban Centres

- It should be assumed that all water is drawn within the whole day i.e. 24 hours.

- No additional peak factors should be applied to water consumption rates used in table 2.2.

- It should be assumed that most houses have individual roof tanks which will reduce the peak factors considerably.

3 WATER QUALITY

3.1 GENERAL

3.1.1 Basic Requirements

The basic requirements for drinking water are that it should be:

- Free from pathogenic (disease causing) organisms.
- Containing no compounds that have an adverse acute or long-term effect on human health.
- Fairly clear (i.e. low turbidity little colour).
- Not saline (salty).
- Containing no compounds that cause an offensive taste or smell.
- No causing corrosion or encrustation of the water supply system not staining clothes washed in it.

3.2 BACTERIOLOGICAL QUALITY

3.2.1 General

- The bacteriological quality is very essential and should be tested before the selection of the sources and during the operation of a supply. In this regard microbiological quality should not be confused with aesthetically pleasing water.
- A good bacteriological quality is best obtained by selecting a source without contamination (see chapter "Water Sources"), by protecting the intake (see chapter "Intake Structures" and by adequate treatment (see chapter "Water Treatment").

3.2.2 Guideline Values for Distributed and Bottled Water

- Given under table 3.1 are the Kenya drinking water quality standards, KS 150 – 1996 that conforms to WHO guideline limits.

Table 3.1: Microbiological limits for drinking water and Containerized water

Type of microorganism	Drinking Water	Containerized Water
Total viable counts at 37^0C per ml, max.	100	20
Coliforms in 250ml	Shall be absent	Shall be absent
E-coli in 250ml	Shall be absent	Shall be absent
Staphylococcus aureus in 250ml	Shall be absent	Shall be absent
Sulphite reducing anaerobes in 50ml	Shall be absent	Shall be absent
Pseudomonas aeruginosa fluorescence in 250ml	Shall be absent	Shall be absent
Streptococcus faecalis	Shall be absent	Shall be absent
Shigella in 250ml	Shall be absent	Shall be absent
Salmonella in 250ml	Shall be absent	Shall be absent

3.2.3 Remedial Action on Bacteriological Deficiencies

- Remedial action has to be taken if deficiencies of the quality are detected. Such actions may be temporary such as issuing recommendations to boil the water or/and long term such as localizing and eliminating the source of contamination and improving the treatment.

3.2.4 Guideline Values for Raw Water

Table 3.2 give indication as to the treatment required for raw water.

Table 3.2: Guideline Values for Raw Water

Coliform organisms 1)
(Number/100 ml)

0-50	Bacterial quality requiring disinfection only
50-5000	Bacterial quality requiring full treatment (coagulation, sedimentation, filtration and disinfection).
5000-50000	Heavy pollution requiring extensive treatment
Greater than 50000	Very heavy pollution unacceptable as source unless no alternative exists. Special treatment needed.

1) When more than 40% of the number of coliforms are found to be of the faecal coliform group, the water source should be considered to fall into the next higher category with respect to the treatment required.

3.3 CHEMICAL QUALITY

3.3.1 Constituents of Health Significance

The following constituents have some health significance and the guideline values given should generally not be exceeded in drinking water (KS 150 – 1996 and WHO guidelines).

Table 3.3: Limits for Inorganic Contaminants in Drinking Water and Containerized Water

Substance	**Limit of Concentration Mg/l, max.**
Arsenic as As	0.05
Cadmium as Cd	0.005
Lead as Pb	0.05
Mercury (total as Hg)	0.001
Selenium as Se	0.01
Chromium as Cr	0.05
Cyanide as CN	0.01
Phenolic substances	0.002
Barium as Ba	1.0
Nitrate as NO_3	10
Fluoride as F	1.5

Table 3.4: Limits for Organic Constituents of Health Significance in Drinking Water and Containerized Water

Substance	**Limit of Concentration Mg/l, max.**
Benzene	10
<u>Chlorinated alkanes and Alkenes</u>	
Carbon tetrachloride	3
1,2-Dichloroethylene	10
1,1-Dichloroethylene	0.3
Tetrachloroethylene	10
Trichlorophenol	30
<u>Chlorophenols</u>	
Pentachloraphenol	10
2,4,6 trichlorophenol	10
<u>Polynuclear aromatic hydrocarbon</u>	
Benzo (α) pyrene	0.01
<u>Trihalomethanes</u>	
Chloroform	30
<u>Pesticides</u>	
Aldrin/Dieldrin	0,03
Chlorodane (total)	0.3
2,4 D	100
DDT (total)	1
Heptachlor and Heptachlor	0.1
Eposide	0,01
Hexachlorobenzene	3
Lindane BHC	30
Mehtoxychlor	

<u>NOTE:</u>

- The Local and climatic conditions necessitate adaptation of Fluoride concentrations in excess of 1.5 mg/l.
- In exceptional cases, a Fluoride content of 3 mg/l can be acceptable in Kenya.

Table 3.5: Limits for Radioactive Materials in Drinking Water and Containerized Water

Radioactive Substance	Limit of Concentration Bq/l, max.
Gross Alpha activity	0.1
Gross Beta activity	1

3.3.2 Desirable Aesthetic Quality

Common constituents that do not affect health in concentration in which they normally are present in water may however affect the aesthetic quality of the water.

The following quality is desirable for water, which should be generally accepted for human consumption and for all usual domestic purposes (KS 150 and WHO Guidelines).

Table 3.6: Aesthetic Quality Requirements of Drinking Water and Containerized Water

Substance or Characteristic	Drinking water	Containerized Water
Colour in True colour units (TCU), max.	15	15
Taste and Colour	Shall not be offensive to consumers	Shall not be offensive to consumers
Suspended matter	Nil	Nil
Turbidity in Nephelometric Turbidity Units, max.	5	1
Total dissolved solids in mg/l, max.	1,500	1,500
Hardness as $CaCO_3$, mg/l max.	500	500
Aluminum as Al, mg/l	0.1	0.1
Chloride as Cl, mg/l, max.	250	250
Copper as Cu, mg/l max.	0.1	0.1

Iron as Fe, mg/l max	0.3	0.3
Managanese as Mn, mg/l, max.	0.1	0.1
Sodium as Na, mg/l, max.	200	200
Sulphate as SO_4^{-2}, mg/l, max.	400	400
Zinc as Zn, mg/l, max.	5	5
PH	6.5 to 8.5	6.5 to 8,5
Magnesium as Mg, mg/l, max.	100	100
Chlorine concentration as Cl, mg/l	0.2-0.5	Nil
Calcium as Ca, mg/l	250	250
Ammonia as (N), mg/l, max.	0.5	0.5

3.3.3 Permissible Aesthetic Quality

Under certain circumstances when it is not practicable to produce a water of the desirable aesthetic quality it may be permissible to raise certain guideline values as shown below. Further see chapter "Water Treatment".

Table 3.7: Permissible Aesthetic Quality

Parameter	Unit	Guideline value	Remark
Chloride	mg/1	600	
Colour	TCU	50	
Copper	mg/1	1.5	
Iron	mg/1	1.0	
Manganese	mg/1	0.5	
pH	-	6.5 – 9.2	
Solids	mg/1	1500	
Turbidity	NTU	25	
Zinc	mg/1	15	
Other constituents	-	As in table "Desirable aesthetic quality"	

3.4 SUBSTANCES AND CHARACTERISTICS AFFECTING BUILDING AND PIPE MATERIALS

3.4.1 General

The materials usually applied in water supply are cement products, steel, iron and plastic. The various factors, which affect the different materials, are described below. Plastic is generally unaffected by water.

Aggressive substances either have to be removed or materials chosen, which can best resist the aggressivity of water.

In this connection it should also be mentioned that attention should always be paid to the fact that the attack can also be from outside (groundwater, swampy areas, or just humid acid soil, especially peat and those soil containing calcium sulphate). Further see chapter "Transmission and distribution lines".

3.4.2 Cement Products

- **Acid Water** (pH value below the neutral line, Fig. 3.1 must be regarded as harmful to concrete. It becomes very harmful if the pH value is more than 1 to 2 points below the neutral line).

- As it can be seen from Fig.3.2, **soft water** (with low carbonate hardness) becomes always very aggressive if it contains free carbon dioxide. This **aggressive CO_2** dissolves the calcium salts of the concrete and mortar and it destroys gradually these cement products. Flowing water with such properties performs this very rapidly.

- **Moor water** is often very harmful

- Alkaline water (Fig.3.1, pH above the neutral line) can also cause damage to cement products if the **sulphate content** is above 300 mg/1 in standing or 100 mg/1 in flowing water. Calcium and magnesium sulphate and, to a small extent also the corresponding chlorides, destroy concrete.

- Harmful to concrete is also water containing sodium hydrogen sulphide and larger amounts ammonium salts (e.g. waste).

- Concrete is attacked by water containing sodium hydrogen carbonate (especially in coastal areas).

- AC-pipes contain calcium carbonate and show high internal and external resistance to concentrated salt solutions. Experiments have proved that asbestos pipes are not corroded by water containing 2000mg $CaSO_4$/liter and 5000 mg Na_2S0_4/liter and $MgS0_4$.

- AC-pipes are also resistant to electric currents.

- But larger amounts of **aggressive carbon dioxide** occurring with low carbonate hardness cause damage (see Fig.3.2) of AC-pipes internal linings of bitumen and external coal-tar coats improve the resistance of asbestos pipes to the limit “very aggressive” in Fig. 3.2.

3.4.3 Steel and Iron Products

- Standing water effects greater corrosion in the pipes than flowing water. Therefore aggressive water has a specially evil influence in the **terminal parts** of the piping system.

- Water of hardness above 35 mg/1 $CaCo_3$ and of an oxygen content of at least 6 mg/1 of v < 0.5 m/s or 2 mg/1 if v > 0.5 m/s (but without aggressive carbon dioxide) form a protective layer of calcium and magnesium compounds named **anti-rust layer** on the internal surface of the pipe.

- Water attacks the iron pipe if the oxygen content is insufficient, even if the other corrosion factors do not favour attack. The oxygen concentration should never be below 4.0 mg/1 (respectively 2.0 mg/1 in case of v > 0.5 m/s).

- Iron is always attached and dissolved by water containing **aggressive carbon dioxide, which** prevents the formatting of a protective layer against rust (see above and fig. 3.3).

- The pH-value should always be equal to or just below the equilibrium for unprotected iron pipes: - 0.5 points for galvanized steel pipes (see Fig. 3.3).

- Unprotected iron pipes are attacked by hydrogen sulphide (e.g. in **moor-soils**).

- Water with a high **chloride content** (e.g. blackish water) attacks iron pipes strongly. The limit for unprotected iron pipes is 150 mg/1 in soft water.

- Special attention has to be given to the **external attack.**

- Steel pipes are more susceptible to chemical attacks than cast iron pipes. **Cast iron pipes** are more resistant than steel pipes against soft water of high oxygen content and aggressive properties.

Fig. 3.1: pH Value for Neutral Water Depending on Calcium Content

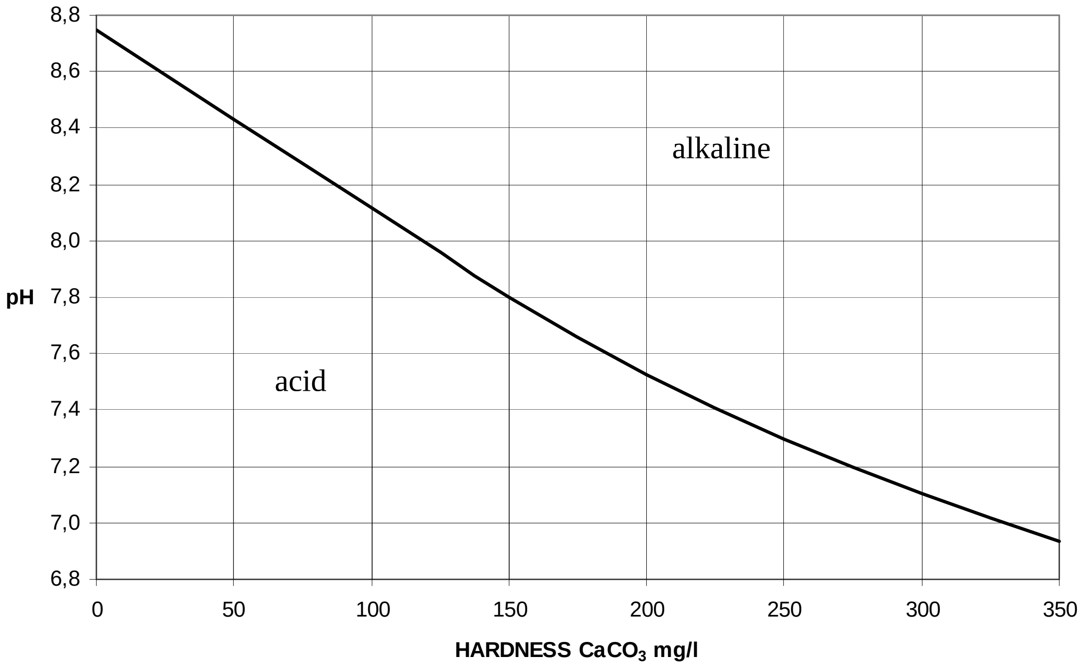


Figure 3.2 Aggressivity towards cement products (Concrete, mortar, AC pipes) depending on the hardness and the free CO2

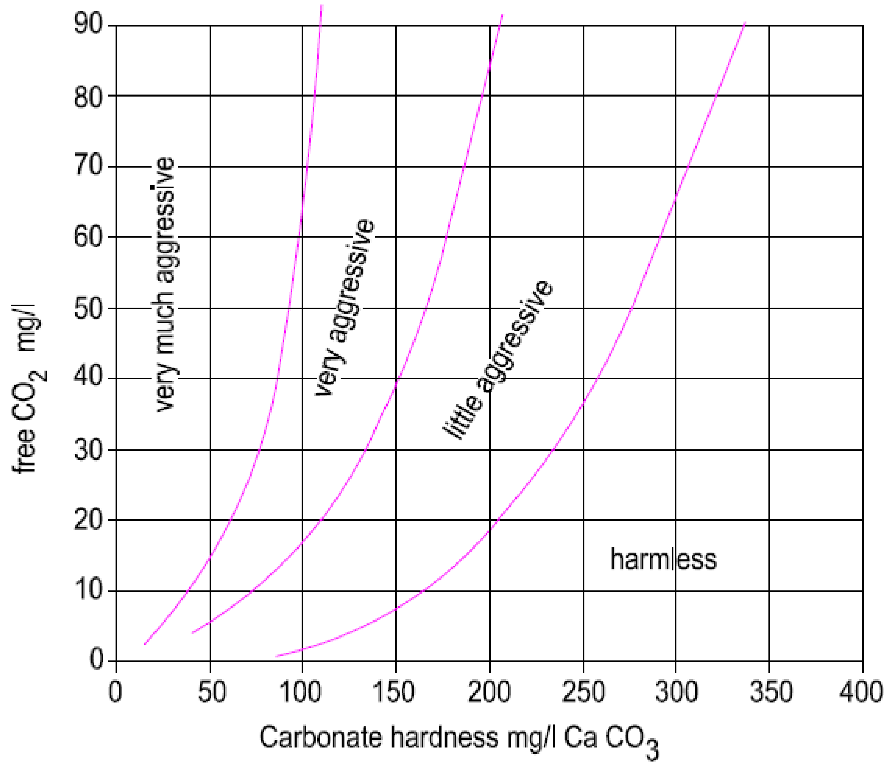
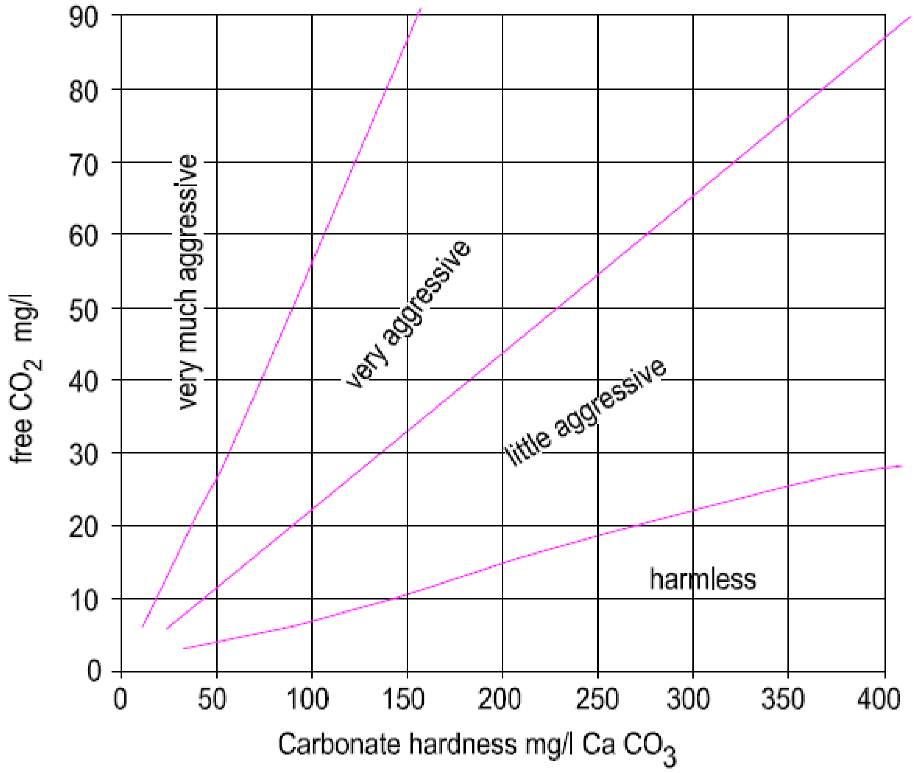


Figure 3.2 Aggressivity towards iron products (Steel pipes) depending on the hardness and the free CO2



3.5 WATER SAMPLES

3.5.1 Sampling for Selection of Source and Treatment

- The selection of source and treatment method will require the collection and analysis of water samples from the alternative sources, which may be considered for a supply.

- The samples shall cover all regimes of a river and be taken in a sufficient number, minimum 4 of which at least 3 shall be taken in the rainy seasons.

- Samples from new wells and boreholes should be taken after at least 24 h pumping.

- Both chemical and bacteriological analysis should generally be made unless it is clear that only one of the two is of interest in a particular case.

- Whenever the result of the analysis leaves doubt as to the selection of source or treatment method additional samples should be collected and analyzed.

4. WATER SOURCES

4.1 GENERAL

4.1.1 General Selection Considerations

In selecting a source of drinking water, there are a number of factors that must be considered e.g:-

- **Quantity:** Is the quantity of water available at the source sufficient to meet future development?

- **Quality:** Is the raw water quality such that, with appropriate treatment water can be supplied that meets or exceeds the quality specified in the "Water Quality Chapter"?

- **Protection:** Can the water, today and in the future, be protected from human excreta, from industrial discharges and from agricultural run-off? Can the catchment's area, e.g. a forest, be protected efficiently to ensure sustained quantity and quality of the raw water?

- **Feasibility:** Is the source available at reasonable cost considering both capital and O&M costs? Can the source be exploited using simple and reliable treatment and transmission technology?

4.1.2 Specific Selection Considerations

- Sources, which require little or no treatment of the water should be chosen in first instance provided the required quantity of water, can be obtained. Hence springs and ground water resources should always be exploited in the first hand.

- For household and small-scale community supplies rainwater harvesting may serve well in most medium and high potential areas in Kenya.

- Surface water from river streams and lakes will almost always require some treatment to render it safe for human consumption. However, for large supplies surface water will often still be the most economical alternative. Rivers, which have the bulk of their catchment in forest areas, should be preferred.

- Sub-surface water drawn from a riverbed or riverbank can sometimes be a viable alternative in dry areas with only seasonal flow in the river, or in rivers with a high silt load.

- It should be studied whether a combination of sources may give a more economical and reliable water supply than a system based on only one source. Mixing can also be used to reduce the content of certain constituents, e.g. Fluoride, to acceptable levels.

- Sources from which water can be supplied by a gravitational system are particularly favourable.

4.2 RIVERS AND STREAMS

4.2.1 Save Yield for Principle Towns and Urban Centers with a Population over 10,000

The 96% - probability **daily** low flow shall be regarded as the safe yield of a river. The flow – frequency analysis shall be made by using the lowest recorded daily flow of each calendar year for which records are available for the dry season.

4.2.2 Save Yield for Rural Areas including Urban Centers with a Population under 10,000 and Rural, Market and Local Centres

The 96% probability **monthly** low flow shall be regarded as the safe yield of a river. The flow-frequency analysis shall be made by using the recorded lowest average flow during one calendar month for each year for which records are available for the dry season.

4.2.3 Flood Flow

Small dams (in this context taken as dams with a height less than 4m), spillway and intake structures shall be designed for the 100 year flood unless an economic-statistical analysis is used to determine the optimal design flow.

4.2.4 Flow Analysis

- For rivers with no or few observation records, shall full use be made of flow records from adjacent rivers and of rainfall data to construct a probable flow-frequency curve.

- Rivers and streams which lack installations to measure the flow but which have been identified as potential sources of a water supply should be provided with permanent or temporary gauging stations as early as possible in the planning process.

- The draw-off for other water supplies from the same river should be considered in the flow analysis and when determining the available water.

4.3 SPRINGS

(a) Definition and Necessity of Spring Production

Real spring water is pure and usually can be used without treatment. Springs are found in areas of impervious substrata, and can give reliable supply and if properly maintained, protected and sufficiently distant from pit privies and soakpits, a supply free from pollution.

The water is naturally discharged from the ground where its flow is impeded by a less permeable strata. It is essential that careful control and protection is maintained of the land near the seat of the spring in order to prevent pollution. Quite often a source like this can be utilized to provide a supply to a community without pumping at minimum cost and maintenance.

However, be sure that spring water is really seeping from the ground, and is not a stream that has gone underground for a short distance. Real spring water is pure, but it can become polluted if it stands in an open pool, or flows over the ground. The spring should therefore be protected with brick, masonry or concrete, so that the water flows directly into a pipe without ever being open to pollution from outside. (see Appendix J)

(b) General Method of Spring Tapping and Protection.

To protect a spring you should dig back into the hillside to the waterbearing layer where the water is flowing from the "eye" of the spring, and build a collecting tank or "spring box" around the eye as shown in Appendix J. Be careful not to dig too far into the impervious layer, as that may let the water seep downwards so that the spring disappears or moves down the hill.

Before you build the back of the springbox, you should pile loose stones against the eye of the springbox, this is partly to make a foundation for the box, and also to prevent the spring water washing soil away from the eye. Remember that the spring may sometimes flow much faster after rains, than it is flowing while you work, so everything should be firmly in place. This may require quite big stones, gravel and even sand laid behind them to plug the spaces between. The outlet pipe should be at least 10cm above the bottom of the springbox, but below the eye of the spring if possible. If the waterlevel in the springbox is too high, silt may settle over the eye and block it up. The end of the outlet-pipe inside the box should be covered with a screen, to prevent stones, rubbish and frogs from blocking the pipe.

There should also be an overflow pipe which is big enough to carry the maximum flow of the spring in the wet season. This pipe should be below the eye of the spring if possible. The top of the springbox should be at least 30cm above the ground to prevent surface water running into it.

The box should be covered with a concrete slab and should preferably have an access hole so that it is possible to get inside and clean it. The hole should have a raised edge to prevent surface water running into the box.

The cover should be lockable or so heavy that it can be opened only with a lever or a manhole key. A third pipe for cleaning out silt form the bottom of the springbox is also recommended.

If it is possible to dig deep enough for the bottom of the spring box to be at least 10cm below the outlet pipe, then you could use an outlet pipe at least 5cm in diameter, and lead the water to another box, not more than 50m away, which is called a "silttrap". (see Appendix J).

The box also needs a manhole cover, a mosquito proof overflow and a strainer on the outlet pipe. If the spring has a yield of less than 5 litres per minute the springbox may be quite small, but it should at least have an accesshole and an overflow pipe. Water from several springs may be collected in one silttrap. (see Appendix J). One point to watch when piping water by gravity from several springs is the danger of the pressure from one spring blocking up the other.

The pipelines from separate springs should only come together as separate inlets above the water level. When the springbox is complete, the space behind it should be filled with soil. At the bottom, level with the eye the space should be filled with gravel or sand at least as coarse as the waterbearing layer. Further up, it should be made water tight to prevent surface water running down the outside wall and into the box. This can be done with cement or puddle clay. Now springboxes and silttraps should be sterilized by scrubbing on the inside with bleach solution, as described for handdug wells. Lastly, you should dig a ditch at least 8m up hill and around on each side of the springbox to take surface water away from it and prevent pollution of the spring water. The soil from the ditch should be piled up the downhill side of the ditch, to make a ridge or "bund", which will help to keep away surface water. If you put a fence or prickly hedge on top of the bund, this will help to keep people and animals away.

Springboxes and silttraps will gradually become filled with silt, they should be cleaned out once a year. For springs with very large flows, over 10 litre/second, a springbox would be difficult to build, and might be eventually washed away by the flow. In these cases an infiltration gallery may be built into the side of the hillside, running across the slope.

(c) Tapping of Gravity Springs

Gravity overflow springs in granular ground formations can be tapped with drains consisting of pipes with open joints placed in a gravel pack. To protect the spring, it is necessary to dig into the hillside, so that sufficient depth of aquifer is tapped even when the groundwater table is low. The design of drains follows common engineering practices. They must be laid so deep that the saturated ground above them will act as a storage reservoir compensating for fluctuations of the groundwater table. The water collected by the drain discharges into storage chamber or springbox.

For sanitary protection the top of the gravel pack should be at least 5m below ground level, which may be insured by locating the spring catchment works in the hillside or by raising the groundlevel with backfill from elsewhere. An area extending along the gallery over its full length plus 10m at each side, and in the other direction, to a distance of at least 50m upstream, should be protected against contamination from cesspools, manure or pits. This area should preferably be fenced in, to prevent trespassing by people and animals.

(d) Tapping of Artesian Springs

- **Artesian Depression Spring:**

In outward appearance, depression springs are quite similar to gravity depression springs, but their yield is greater and less fluctuating, as the water is forced out under pressure. To tap water from an artesian depression spring, the seepage areas should be surrounded by a wall extending a little above the maximum level in which the water rises under static conditions.

For artesian depression springs of Large Lateral extent, a system of drains will have to be used discharging the collected water into a storage chamber. To increase the infiltration rate and for protection of the water quality, the recharge area should be cleaned of all debris.

- **Artesian Fissure Springs**

Here the water rises from a single opening, so that the catchment works can be small. Some increase in capacity may be obtained by removing obstacles from the mouth of the spring or by enlarging the overflow opening.

- **Artesian Contact Springs**

The water flows out under pressure and is protected against contamination by the overlaying impervious layer. The discharge can be large and stable, with little or no seasonal fluctuations. For a large lateral spring a retaining wall should be constructed over its full length, with the abutment (borders) extending into the overlaying impervious layers and the base of the wall a gallery should be constructed, covered with a layer of protection clay. (see Appendix J)

4.3.1 Location and Reliability

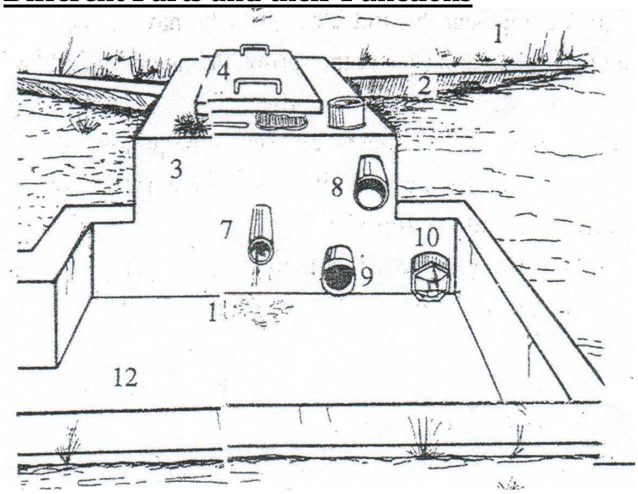
The best way to locate adequate springs and to get information about their reliability during dry spells is to interview people resident in the area.

4.3.2 Yield

- There are seldom records of the flow from springs. Simple over-flows weirs, V-notches etc. should be installed for gauging the flow as early as possible in the planning process.

- The flow from an artesian spring often fluctuates less than that of a gravity spring. As also the bacteriological quality of the water from an artesian spring is better, this type of spring is preferred.

<u>Different Parts and their Functions</u>



Names	**Functions**
1. Catchment Area	- Source of water or spring eye
2. Catchment wall	- Collects and directs all water from the catchment area in the box
3. Spring Box	- Storage tank/sedimentation tan,
4. Spring Box cover	- Prevent water from outside contamination
5. Manhole	- Provides access to the box during cleaning
6. Screen pipe/intake	- Allow and also fillers water from the catchment area into the box
7. Draw pipe	- Collection point for users
8. Overflow pipe	- Lets out excessive water from the box
9. Washout pipe	- Outlet for water/dirt from the box during cleaning
10. Catchment Drain pipe	- For draining the catchment area during cleaning
11. Splash pad	- Prevents violent splashing of running water from scour pipe.
12. Apron	- Concrete flow where all excess water pours collects and later led to drain.
13. Drain	- Collects all
14. Runoff water drain	- Prevents surface water from contaminating.

4.4 BOREHOLES AND WELLS

4.4.1 General

Except for shallow wells for one or a few families the safe yield should be determined by a hydro-geologist.

4.4.2 Fully Exploited Aquifers

- Test pumping should be done to determine both the hydraulic characteristics of the boreholes and that of the aquifer.

- The borehole characteristics should be determined by a step test pumping with 3-5 different pump capacities each step taking about one hour.
- The Characteristics of the aquifer should be determined through medium terms test pumping and observation of the water levels in adjacent production or special observation boreholes.

- The required test-pumping period will depend on the aquifer and should be decided by a hydro-geologist.

- Generally a pumping period of 1-3 days will be adequate for an artesian reservoir whereas a reservoir with a free water table will have to be pumped for a longer period.

- The pump capacity should be kept as constant as possible and also as big as possible during the test pumping.

4.4.3 Only Partly Exploited Aquifers

- When the draw-off from a borehole is going to be considerably lower than the expected yield, then a simplified test-pumping programme may be followed.

- In this case observation boreholes will not be required and the test-pumping period should be 1-3 days with a pump capacity, which exceeds the future draw-off with about 50%.

- Survey boreholes for shallow wells (0-20m) may be test-pumped for only 1 hour (50 min, with approximately the future draw-off 10 min. with twice this capacity). This test method is allowed only for wells to be equipped with hand pumps and which are to serve as point supply for maximum 500 people.

4.4.4 Manual Water Level Observations

The water levels in the test boreholes and the observation boreholes should be read with the following frequencies during the test-pumping and during the recovery after the pumping has been stopped.

Time after Start or Stop of Pumping	Gauging Interval
0-10 min	1 min
10-20 min	2 min
20-40 min	5 min
40-60 min	10 min
60-90 min	15 min
90-180 min	30 min
180-360 min	60 min
360-600 min	120 min
10-24 hours	4 hours
1-3 days	6 hours
Over 3 days	12 hours

4.4.5 Yield Records from Existing Boreholes

Information about the yield of existing boreholes shall be used with great care.

The yield may have changed after many years of use or what is reported as the yield may very well be the capacity of the pump once used for the test pumping.

Fresh test pumping is recommended unless the background of the reported yield is very well documented.

4.4.6 Borehole Spacing

To avoid interference between cone of depression of various boreholes, it is proposed that a borehole should not be drilled not less than 0.8 Km radius from existing one. However test pumping, modeling and property boundaries may be used to determine suitable spacing for economical and sustainable borehole system.

4.4.7 Criteria for Successful Rate of Boreholes

The criterion for successful rate of borehole is: -

(i) Borehole yield: 330 L/hour or more
(ii) Water Quality: Meeting KS 150 guideline standard. If it does not meet this standard, appropriate treatment should be undertaken.

4.5 SUB-SURFACE DAMS

4.5.1 General

The construction of sub-surface dams is an important means of solving water storage problems, particularly in arid and semi-arid land (ASAL) areas. This is primarily because it minimizes the construction and environmental costs; minimizes evaporation losses; minimizes sedimentation; improves water quality; and minimizes loss of valuable land. Any dry riverbed, seasonal stream or lagga, which receives some flow during the rainy season, is potentially suitable for the development of sub-surface dams.

The sand retains water for relatively long periods after the flow in the river has ceased. The volume of water stored varies depending on the grading of the sand and the gravel. Generally, the available volume of water is about 20% to 30% of the total volume of sand. This water is retained behind the dam structure, which forms a seal across the width of the river and down to foundation, built on an impermeable layer of rock or clay.

A pumping well or any other suitable outlet structure is located upstream for drawing water and delivering to the consumers to avoid damage caused by direct access to the dam.

The water quality in sub-surface dam is usually much better than water from open surface reservoir, since it is protected from contact with animals and humans and has undergone some form of filtration through the sand.

In rivers with only seasonal flow it is often possible to abstract water from the riverbed also in the dry season if a structure is built across the riverbed under the surface to retain the sub-surface flow. This method is particularly suitable in areas where the groundwater is generally saline or has high fluoride content. The water is generally withdrawn through infiltration drains up-stream the sub-surface dam.

4.5.2 Withdrawal of Stored Water

The riverbed sometimes can store considerable amounts of water, which can be drawn during dry seasons. The bed is then recharged during the rainy seasons. The available water can be estimated using the specific yields as given in Section 6.6.3. The depths of the material in the river bed should be investigated before any accurate estimate of the available water can be done. It is sometimes possible to find a natural barrier in form of a rock outcrop or impervious material in the riverbed.

Behind such barriers large amounts of water may be stored. This type of sub-surface dams has proven very successful in dry areas like Turkana District.

4.5.3 Withdrawal of Sub-surface Flow

The total sub-surface flow can be estimated from:-

$$Q_t = KAI$$

Where:

Q_t	=	total sub-surface flow, m^3/s
K	=	permeability, m/s (see table).
A	=	cross-sectional area, m^2
I	=	hydraulic gradient, dimensionless

Coefficient of permeability, m/s at unit hydraulic gradient

1 10^{-1} 10^{-2} 10^{-3} 10^{-4} 10^{-5} 10^{-6} 10^{-7} 10^{-8} 10^{-9} 10^{-10} 10^{-11}

Nature of soils	Clean gravel	Clean sands; mixtures of clean sands and gravel	Very fine sands, silts mixtures of sand, silt and clay glacial till, stratified clays, etc	Unweathered clays
Flow characteristics	Good aquifers		Poor aquifers	Impervious
Retention characteristics	Good drainage		Poor drainage	Non-drainage
Use in dams and dikes	Pervious parts of dams and dikes		Impervious parts of dams and dikes	

In practice seldom more than 60 to 70% of the total sub-surface flow can be intercepted.

4.5.4 Sand Dams Construction and Suitability

Sand dams are particularly appropriate in semi-arid areas where the floodwater often carry high silt load and the evaporation from a free water surface is high. The dam across the river should be built in stages to ensure that mainly sand and gravel are deposited. The first stage should be a dam about 2m high. Later the wall should be raised as the sand and gravel builds up until the full height, often 6-12 m is reached.

4.6 ARTIFICIAL RECHARGE

4.6.1 General

Ground water usually has the great advantages over surface water from rivers and lakes in that it is free from organisms and bacteria causing illness and also that the turbidity and the colour is usually not a problem.

Where the groundwater yield is inadequate it should therefore be investigated whether it can be supplemented by artificial recharge from an adjacent surface-water source.

4.6.2 Bank Infiltration

The horizontal distance between the river or lake and the recovery point should be a minimum of 20m and preferably 50m or more in order to guarantee the desirable bacteriological quality. However, often it will not be practicable to place the recovery point so far away from the stream or lake because of the ground conditions. In this case provision should be made for chlorination of the water.

4.6.3 Artificial Aquifer

An artificial aquifer should be designed for a retention time of the water underground of 60 days. If the retention time is shorter chlorination may be required. The specific yield, or storage capacity, of different materials is shown below. It must be understood that large variation can be expected and that soil investigations are required to determine the specific yield accurately.

Material	Specific yield (%)
Clay	3
Sand	25
Gravel	22
Gravel and sand	16

Thus a rough estimate of the required volume 'V' of a gravel and sand aquifer can be made by using the formula $V = 400\ D\ m^3$ which will give a retention period of approximately 60 days.

D is the water demand in m^3/day.

The intake of water to the infiltration basin should be arranged so that the flow can be stopped when the river is polluted or otherwise of poor quality.

4.7 RAINWATER HARVESTING

4.7.1 Rainfall Data

The 90% - probability annual rainfall should be regarded as the dependable rainfall for the purpose of rainwater harvesting for domestic use. Maps showing the 90% - probability annual rainfall and the average annual rainfall in Kenya can be seen in Appendix B. The maps can be used for rough estimates of available water for a certain location. However specific rainfall data for the location in question should be obtained for each individual case.

4.7.2 Run-off Coefficients

The following run-off coefficients should be used for calculating the fraction of the rainfall which can be harvested.

Surface Type	**Run-off Coefficient**
Roof tiles, corrugated sheets, concreted bitumen, plastic sheets	0.8
Brick pavement	0.6
Compacted soil	0.5
Uncovered surface, flat terrain	0.3
Uncovered surface, slop 0-5%	0.4
Uncovered surface, slope 5-10%	0.5
Uncovered surface, slope >10%	>0.5

Refer to Section 4.7.5 for details.

4.7.3 Roof Catchments

Using the following formula, a rough estimate of the required minimum roof area can be calculated as:-

$$A = \frac{450 \times D}{R}$$

Where:-

A = Minimum roof area in m^2
D = Total water demand in litres/day
R = The 90% - probability annual rainfall in mm

4.7.4 Selection of Tank Size

The required capacity of the collection tank should be calculated using available meteorological data showing the rainfall pattern of the area. owever, for rough calculations the tank capacity may be calculated by the formula:

$$C = 0.03 \times D \times (T^{+2})$$

Where:-

C = Tank capacity in m^3
D = Total water demand in litres/day
T = Longest dry spell in moths, average year

In this connection a **dry spell** may be defined as the period when the average monthly rainfall is less than 50mm. The length of the dry spell in different areas in Kenya can be found in Appendix B.

4.7.5 Harvesting of Rainwater

Harvesting of rainwater from roofs or ground catchments find applications in supplementing to other sources of water supply. The major problem with this apparently low-cost approach is the storage necessary to span periods of draught.

Rainfall records representative of the catchment are essential as a basis for reliable design of such a system. For a catchment of area A m^2 receiving rainfall run in a month, the yield Y is calculated as follows:-

$$Y = \frac{f \times A \times R}{1000} \ m^3/month \ \ldots\ldots\ldots\ldots..(1)$$

Where, f = catchment run off coefficient typical values of which are given in sec. 4.7.2.

For a constant daily water demand, a large catchment area is required if available storage is small, and vice versa, but a large volume of storage will be required irrespective of catchment area, where the draught period is long. If N people are supplied with drinking water entirely from a rainwater system, the quantity of water to be supplied per month, Q will be:-

$$A = \frac{N \times 30 \times C}{1000} \ m^3/month \ \ldots\ldots\ldots\ldots(2)$$

Where C = daily consumption per person L/p/d

With a large variation in rainfall distribution, the more critical parameter is the minimum storage volume required. Selecting the critical or design draught period, T months, from rainfall records, the minimum storage volume, V_{min} is given by:-

$$V_{min} = \frac{N \times 30 \times C \times T}{1000} \ m^3 \ \ldots\ldots\ldots\ldots\ldots.(3)$$

Hence, a family of 6 will require a storage volume of 10.8 m^3 to span a four month drought period.

There are three common methods of determining the size of tank as outlined here below.

(i) Balance Method

This method balances the yield, or supply of water with the user or demand at the end of each month and calculates the storage left in the tank. Assuming that the storage at the end of each month can never be less than zero, then this method can be used to determine the minimum tank size necessary to satisfy the water use by the family.

The result of using this method is presented in Table 4.1. The data used in this analysis is for Kivuti village, Kikumbulyu location in Kibwezi division.

Annual Rainfall	567 mm
Roof area	72m^2
Family size (persons)	6
Water use/ demand (litres/person/day)	14
Total water use (litres/day)	84

Table 4.1: Balance Method for roof tank sizing

Month	**80% Reliability Rainfall (mm)**	**Supply S (Roof Area x Rainfall x 90%) litres**	**Demand D (84L/day x No. f days) litres**	**S-D**	**Cumulative (S-D)**	**Actual storage at the end of the month**
October	22	1430	2604	-1174	-1174	0
November	365	23652	2520	22132	19958	17.000
December	87	5655	2604	3051	23009	17000
January	0	0	2604	-2604	20405	14396
February	35	2275	2352	-77	20328	14319
March	58	3770	2604	1166	21494	15485
April	0	0	2604	-2604	18890	12881
May	0	0	2604	-2604	16286	10277
June	0	0	2520	-2520	13766	7577
July	0	0	2604	-2604	11162	5153
August	0	0	2520	-2520	8642	2833
September	0	0	2520	-2520	6122	113
	567	36782	30660	6122		

The basic formulae (balance equation) is:-

$$S = S(I) + I - D$$

Where:-

S = storage at the end of the month
S(I) = the amount stored at the end of previous month
I = product of monthly rainfall x roof area x loss factor
D = amount of water used by a family in a given period.

(ii) Cumulative Supply and Demand

In this method the supply and demand are calculated and the cumulative supply and demand of each month is also calculated. Either graphically or by calculation the maximum difference between the cumulative supply and demand is determined. This difference is the optimum tank size (see sketch for the optimum tank size as shown in figure 4.1). In this case annual demand equals annual supply.

Thus monthly supply = 36,782/12 = 3,065 liters/month.

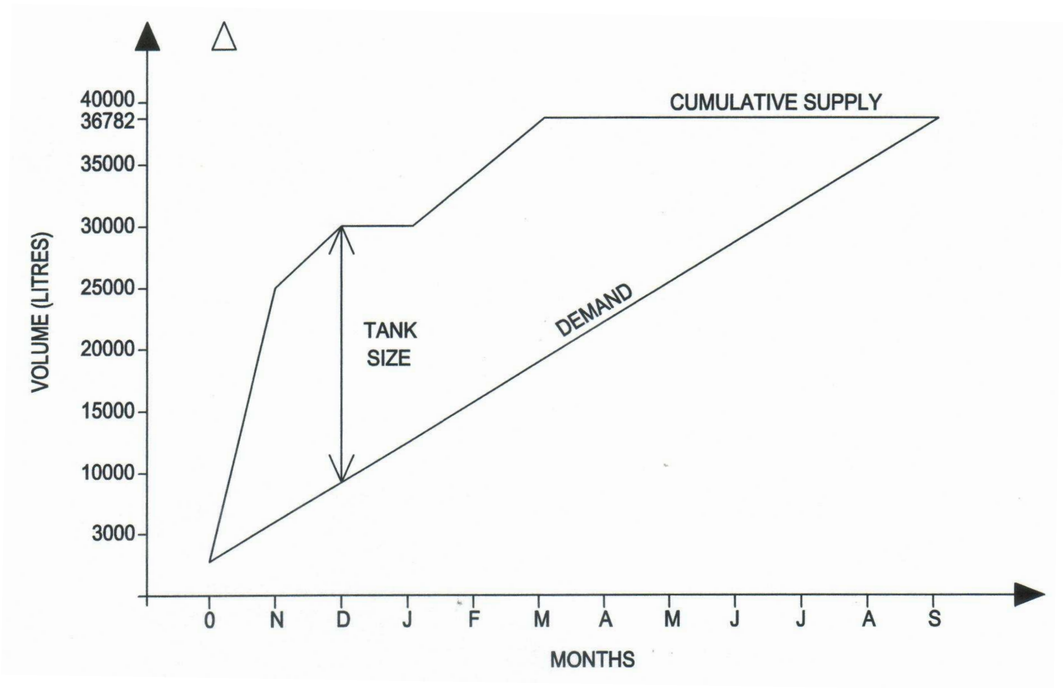


Figure 4.1: Graph of cumulative supply and demand

The tank size is the optimum to collect the rainfall from a given roof area and satisfy water use by the family. For comparison, when using similar input data as for the balance method, the optimum tank size is 23,009 litres. This can be seen in the column labeled "cumulative (S-D)" in Table 4.1 and on the graph above.

(iii) Dry Season Storage Method

This is perhaps the simplest method for determining a roof catchment tank size.

First estimate the longest period during the year without rain. For example: November, December, January and February constitute the dry period in some parts of Kenya. This is about 120 days without rain.

Secondly, estimate the daily water use. For example, for a family of 8 people using 200 litres per day (one drum per day is an average use), the size of storage tank can be determined as follows:-

Tank size = No. of dry days x daily water use
= 120 days x 200 litres
= 24,000 litres (24 m3)

If the annual yield is less than the dry season storage tank size, then the tank will have to be reduced to the value of the annual rainfall yield. For example, if the rainfall is 500 mm per annum and the effective roof area of $40m^2$, a maximum yield of $18m^3$ (500mm x $40m^2$ x 0.9) is obtained. This is less than the ideal tank size, so the tank size would be reduced to $18m^3$.

Using the input the data Table 1, the tank size for this method would be:

Dry days = 6 months
= 183 days

Tank size = 183 days x 84 litres/day
= 15,372 litres

iv) Collecting all the rainfall and storing

This involves collecting all the rainfall from the roof and storing it until when there is an acute shortage. This implies that the tank size will be equal to the total supply for the year.

Tank size = Annual rainfall x roof area x 90%
= 36,782 litres (refer to Table 1 on the column labeled "supply, S")

This method is common in ASAL areas where it may rain only once per year.

v) Summary

The accuracy of the first two methods depends on the accuracy of the rainfall data. Usually mean monthly data is used. However there is wide variation in rainfall (mostly in ASAL area) both geographically and in time, so mean monthly data may not reflect the actual rainfall distribution. In this example the 80% reliability rainfall has been used.

The balance method allows for a minimum tank size to be determined yet satisfy daily need and will be of great use to situation where money for building tanks is limited. It is suitable for the majority of rural people looking at rainwater as clean, safe and may be the only source of water.

The cumulative supply/demand method enables the ideal tank size to be determined, especially where funds are not limiting.

The dry days method is quick and easy but cannot reflect rainfall patterns accurately. Any method only works well if the following key points for the management of a successful rainwater catchment system are adhered to:

1. Have gutters on the maximum roof area
2. Maintain the gutters to collect the MAXIMUM amount of rainfall
3. Use the water carefully (economically) especially towards the end of the rainy season when the tank is full, to conserve water for the dry season.
4. Clean the gutters and the tank at regular interval and maintain proper standard of sanitation. If possible, install a foul flush system or self cleaning gutters (inlets)

4.7.6 Length of Dry Spell in Kenya

(i) Definition

A dry spell may be defined as the period when the average monthly rainfall is less than 50mm.

(ii) Length of dry spell

The length of the dry spell in different areas in Kenya can be found in Appendix B.

4.7.7 Rock Catchments

(i) General

Rock catchments system consists of two components, a catchment area formed by a bore rock surface and a pond normally formed by a concrete weir. Using cement guttering may extend the catchment area. The rock catchment is an extremely low cost method of community water supply, which lends itself well to community participation. This makes it extremely attractive water supply alternative with a suitable geology.

Another attractive aspect of Rock catchments is that they are almost invariably suited to conversion to a gravity supply system.

(ii) Storage capacity

Storage capacity of the reservoir may vary from $20m^3$ to $10{,}000m^3$, depending on the size of the Rock outcrop, and its area extent, elevation and gradient. The storage capacity can be estimated using equation D, sec. 4.7.5.

5. INTAKE STRUCTURES

5.1 RIVER INTAKES

5.1.1 Location

Whenever practicable an intake should be positioned:

- Intake spot and its neighbourhood area shall be of good geological formation and safe from landslides and flood caused collapses.
- Such a spot shall be selected as is free from change of stream center, rise or fall of riverbed with a calm and soft flow.
- Intake facilities shall be built at such a place as is safe from inflow of polluted or salt water and as promises to give us water of good quality
- On a river whose main catchment area is in the forest.
- On a level that allows the water to be gravitated to the consumers. Further see "Transmission and distribution lines".
- Up-stream populated areas and farming areas
- Up-stream of bridges, cattle watering, laundry washing and sewerage outlet points
- At a location where the area immediately up-stream the intake is not easily accessible to people and cattle. If it is, fencing should be provided.
- Where the ground is rocky (firm) and does not get flooded.
- At the outside of a river bed.
- Where the flow is adequate to cater for the ultimate water demand. Further see chapter "Water Sources".

5.1.2 Structures

- Small streams and rivers may require that a dam is built across the watercourse so that the water depth remains sufficient also during low flows
- The dam should be designed with a stepped weir with the lowest part of the weir next to the actual draw-off point in order to prevent siltation. The velocity at this point should be at least 1 m/s for all regimes of the river. The area immediately upstream and downstream the dam should be protected against erosion.
- The draw-off should be perpendicular to the direction of the flow of the river. The bottom of the intake should be positioned some distance above the river,

if possible at least 1m. The water should flow towards the intake at a velocity less than 0.1 m/s.

Once the water has passed the screens the velocity should be at least 0.5 m/s. This minimum velocity must be upheld in intake chambers, canals and intake pipes also during the initial phases of a water supply when the water demand is low. There should be a possibility to close the intake with stop logs or similar.

- A floating intake may be a viable alternative in relatively large rivers with variable water levels.

5.1.3 Screens

The intake should be equipped with a course screen and a fine screen, both removable.

- The course should have an open spacing of 30 – 50mm between the bars
- The fine screen should have a spacing of 5 – 10mm
- The screens should be possible to clean with a rake, which should be supplied together with the screens. Thus, the mesh type of screen is not permitted.
- The screens should be designed with a maximum velocity, V_s, in the opening between the bars of 0.7m/s even if the screen is clogged to 50%.
- The head loss through the screen can be calculated with the following formula.

$$h = 1.1 \times \frac{V_a^2}{2g} \left[\left(\frac{L_2}{0.8L_1} \right)^2 - 1 \right] \text{ m}$$

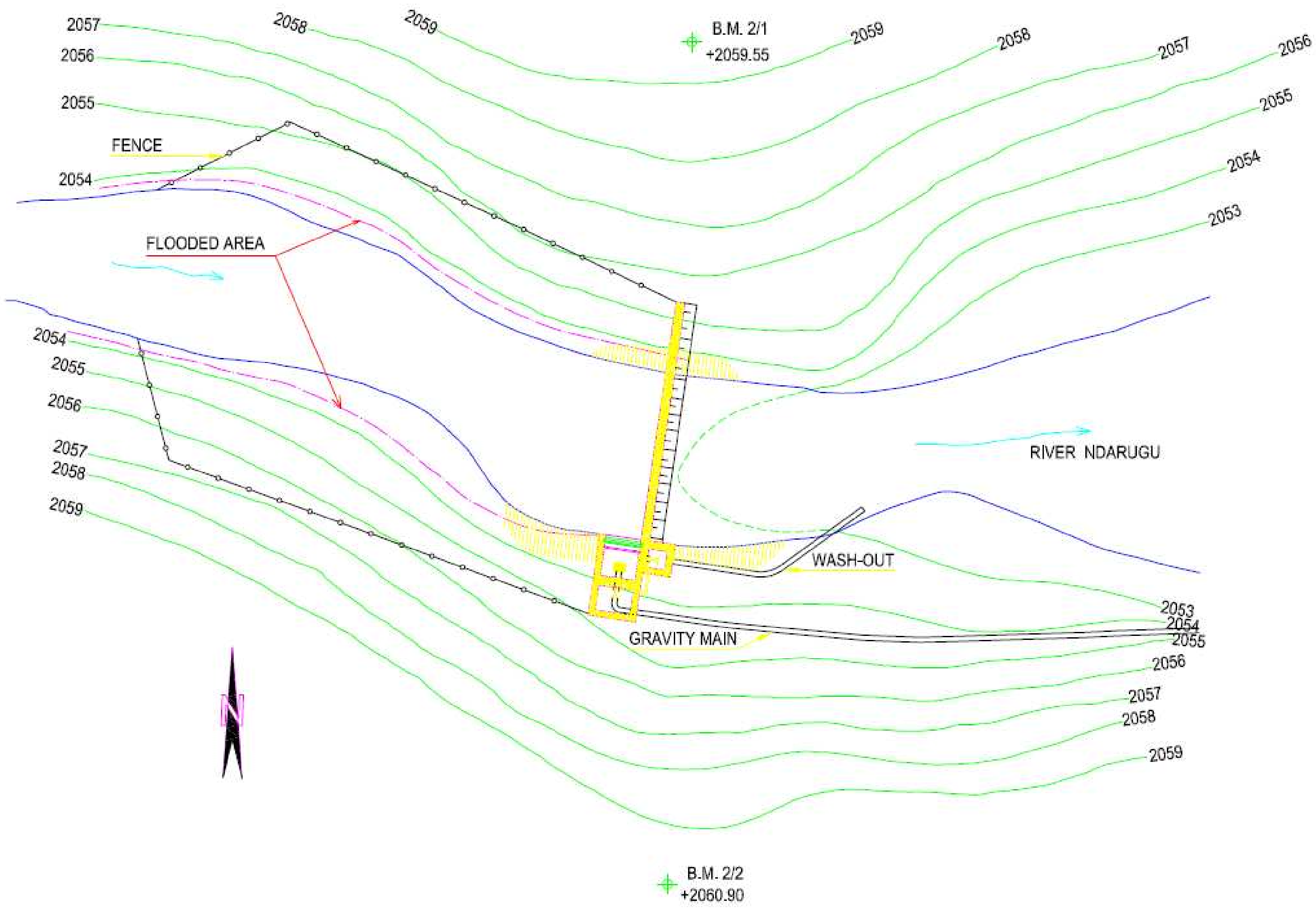
in which

h = loss of head in the screen, m
V_a = Actual approach velocity, m/s
L_2 = Total width of the screen, m
L_1 = Total width between the bars, m

Example

$Q = 0.1\ m^3/s$. Screen area $0.5m^2$, screen width 0.5m. Distance between bars 15mm (center). Thickness of each bar 5mm.

Hence, the velocity between bars when the screen is clogged to 50% will be:-



SOURCE : NDARUGU WATER SUPPLY

$$V_s = \frac{Q}{A} = \left(\frac{0.1}{0.5 \times \frac{10}{15} \times 0.5} \right) = 0.6 \text{ m/s} \quad ; \quad V_S < 0.7 \text{ m/s}$$

The head loss will be

$$h = 1.1 \times \frac{\left(\frac{0.1 \times 2}{0.5} \right)^2}{2g} \times \left[\left(\frac{0.5}{0.8 \times \left(\frac{10}{15} \times 0.5 \right)} \right)^2 - 1 \right] = 0.023\text{m}$$

The intake should generally be equipped with a platform and handrails. The platform should cover at least one meter round the screens. Equipment to lift the screens should be provided when the weight of each screen exceeds 30kg.

5.2 INTAKE IN LAKES AND MARSH

5.2.1 Location

- The intake should preferably be 3-5m below the surface but at least 1m above the lake bottom.

- In lakes with bilharzias the intake point should be a minimum of 100m from the shore.

5.2.2 Design Details

- The underwater pipe should be laid with an even slope without any peaks where air pockets can form. If peaks cannot be avoided the pipe should be punctured at these points.

- Lifting of the underwater pipe when empty should be prevented through adequate anchorage.

- The underwater pipeline should usually be flexible either the pipe material itself or the joints should give the required flexibility.

- The cleaning of the intake screen should be considered in the design. When feasible a connection should be from the discharge pipe to the underwater pipe to make back washing of the intake pipe possible.

- The intake level should be adjustable in lakes with widely fluctuating water levels (e.g. Lake Victoria).

- The water should flow towards the intake at a velocity less than 0.1 m/s.

5.3 BOREHOLE INTAKES

5.3.1 Setting of the Intakes

- The intake of a borehole pump should be set at least 2m above the bottom of the borehole.

- The exact setting of the borehole intake will be controlled by the characteristics of the borehole and the ground-water reservoir as obtained through long term test pumping. Further see chapter “Water Sources”.

- If only results from short time test pumping are available then the intake should be placed between 2, and 5m above the borehole bottom unless the draw-off from the borehole is shown to be much lower than the potential yield.

5.3.2 Design Details

- Screen, pump and pipe material which can sustain the aggressiveness of the water has to be selected further see chapter “Water Quality”.

- For pump selection, see chapter “Pumps and Power Sources”.

- All boreholes should be equipped with level indicator of simple design eg. The air type with foot pump and manometer.

5.4 SPRING INTAKES

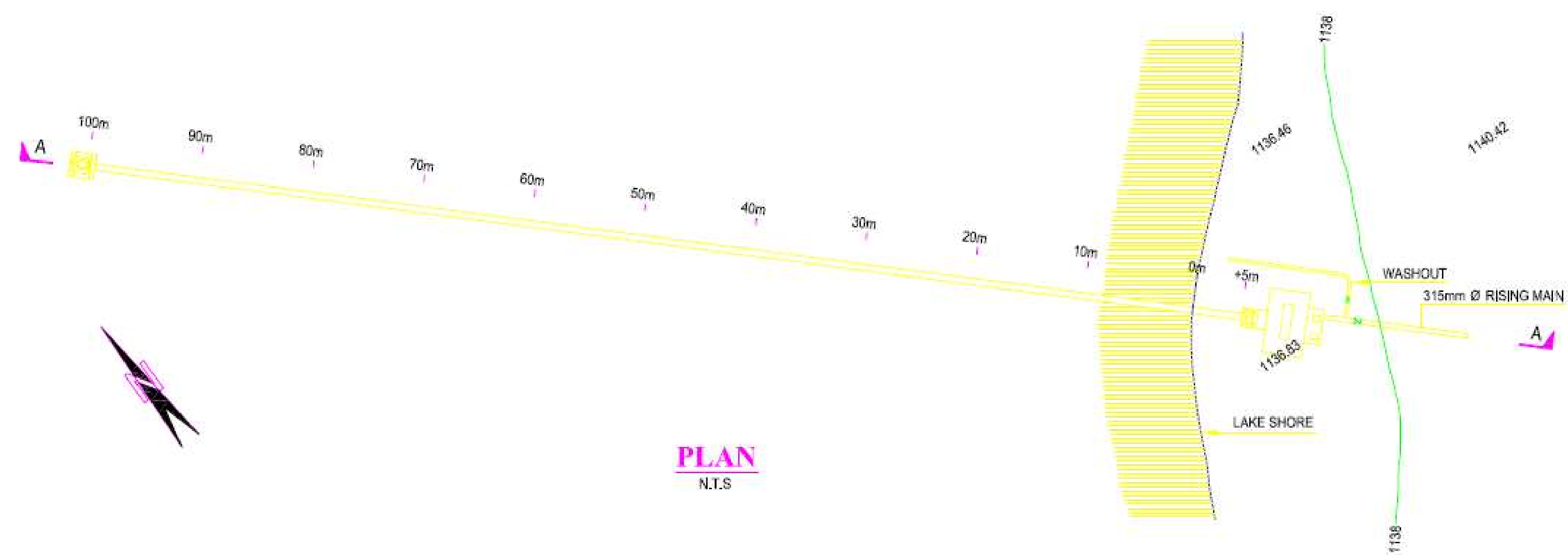
5.4.1 Protection

Where the spring is tapped with drains placed in a gravel pack the top of the gravel pack should be at least 3m below the ground surface. An area extended along the drain gallery over its full length plus 10m at each side and in the other direction to a distance of at least 50m upstream should be protected against contamination from cesspools, manure and pits and should be fenced.

Above the fenced-in spring site there should be a drainage ditch to divert any surface water / runoff.

5.4.2 Storage Chamber

The drains should discharge into a storage chamber (spring box) which should be equipped with a lockable manhole cover. Any air vents, overflow pipes and scour pipes must have screened openings.



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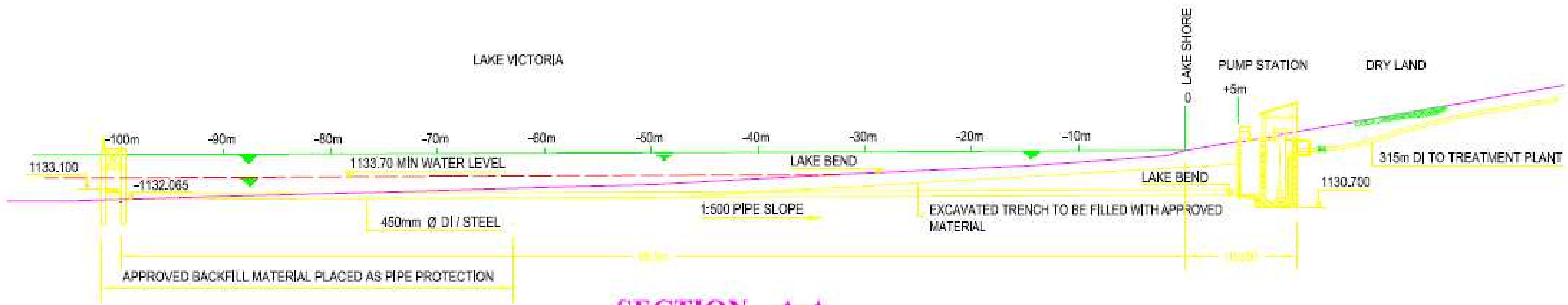


SECTION A-A
N.T.S

TYPICAL LAKE INTAKE
SOURCE: MBITA WATER SUPPLY

5.5 ROOF CATCHMENTS

5.5.1 General

Regarding available rainwater, roof area and capacity of the storage tank see chapter "Water sources".

5.5.2 Roof

Water can be collected from house roofs made of tiles, slates (corrugated) galvanized iron or aluminum. Thatched roofs are not suitable because of health hazards.

Plastic sheeting is often not durable. Painting the roof for water-proofing may impart taste and colour and should be avoided.

5.5.3 Roof Guttering

The roof guttering should slope evenly towards the down pipe to prevent the formation of breeding pools for mosquitoes. New houses should be carefully planned so that the length of guttering and pipes will be as short as possible and so that the water can be tapped by gravity.

5.5.4 Foul Flush

There should be arrangements to prevent the first water from each shower from being collected in the clear water container in order to prevent pollution by dust, leaves and bird dropping which accumulate during the dry periods.

This can be accomplished simply by arranging the down pipe so that the foul flush can be diverted or by having a small vessel which collects the foul flush before the water overflows to the clear water tank. A foul flush vessel with a capacity of 100 – 200 liters should be adequate for an ordinary roof.

5.5.5 Storage Tank

- Regarding tank capacity see section 4.7.3.
- The inlet pipe should be equipped with a sieve or net to trap any foreign materials.
- The tank shall be covered to reduce evaporation and contamination.
- The outlet pipe should be placed 0.2m above the floor of the tank.
- The tank should have a scour or be constructed in a manner that facilitates

removal of sediments and cleaning.

- The tap area should be drained and have a concrete apron to keep it dry and clean.

- The tank should be well raised to allow easy tapping.

- Corrugated steel tanks should be laid on wooden supports placed on raised concrete platforms to ensure that the outer bottom surface is kept dry to reduce corrosion.

5.6 DUG WELLS

5.6.1 Diameter and Depth

- The diameter of a dug well should be at least 1.2m to allow two men to work together during the digging. Slightly smaller diameter may be used if the digging is to be done by one man only.

- Well for a large community should usually have a diameter of 2-3m.

- The well should be dug at least 3m below the expected lowest water level.

5.6.2 Lining

- Most dug wells need an inner lining of materials such as brick, stone masonry, concrete rings cast insitu or precise concrete rings. Back filled dug wells with a 100 – 150mm tube can also be a good solution.

- Sinking a dug well by excavation from the inside is very often a good and safe technique, however, in very loose soil (fine or medium sand) in thick layers (over 3m) other methods e.g. Hand-drilling should be used.

- In consolidated ground (e.g. Rock) the well may stand unlined but the upper part should always have a lining.

- The section of the well penetrating the aquifer requires a lining with openings or perforations to allow the groundwater to enter. Any backfilling at the same level as the aquifer should be made with gravel.

- However in fine sand aquifers the lining should be without perforations and the groundwater should enter only through the bottom of the well. The bottom should be covered with graded gravel e.g. three layers each 150mm thick with grain sizes 102mm for the deepest layer, then 4-8mm, and 20-30mm effective size at the top.

5.6.3 Protection

- The space between the walls of the dug hole and the lining of materials such as brick, stone masonry, concrete rings cast insitu or precast concrete rings. Back filled dug wells with a 100 – 150mm tube can also be a good solution.

- The wall lining should be extended approximately 0.5m above the ground to form a wall round the well.

- A concrete apron should be constructed on the ground surface extending about 2m all round the well.

- The well top should be sealed with a watertight slab. A manhole that can be tightly and securely locked should be provided for inspection and disinfection.

5.7 DRILLED WELLS

5.7.1 Hand-drilled Wells

- Hand drilling of 150-300mm diameter wells down to a depth of 15-20m is particularly feasible in clay and sandy soils. If gravel and small stones are found and in semi-cemented layers such as soft sandstone, weathered granite and weathered laterite, hand drilling is still possible, though rather time consuming.

- A filter pipe of at least 6, length and 100-150,, diameter and a sand filter should be put in the well.

- Protection should be made as described under "Dug Wells".

5.7.2 Mechanical Well Drilling

- Mechanical well drilling (digging) has to be used in layers with big stones and boulders and in heavily cemented soils.

- Regarding filter and protection see above.

5.8 INFILTRATION GALLERIES

5.8.1 General

Infiltration gallery is piping equipment laid under the bed of a river in actual use or defunct, through which the many perforated holes on the pipe surface, subsoil water or free ground water is taken in. The gallery is laid in an aquifer of good permeability where flow conditions are good and stable intake can be assured.

5.8.2 Gallery in slowly permeable material with minimum depth of water above streambed, channel or lakebed.

The length of the screen should be at least:

$$L = \left(\frac{Q\,d}{K\,H\,B} \right)$$

Where:

L	=	length of screen required, m
Q	=	desired discharge, m^3/s
d	=	vertical distance between riverbed and center of screen, m
K	=	permeability of gravel backfill, m/s (see 6.5.3)
H	=	head acting on center of the pipe (distance between minimum water surface elevation and center of the pipe), m
B	=	average width of the trench backfill with gravel.

The screen should be placed at a depth as great as physically possible and economically practical beneath the minimum water surface.

5.8.3 Gallery in permeable riverbed or lakebed with minimum depth of water above the bed. See figure

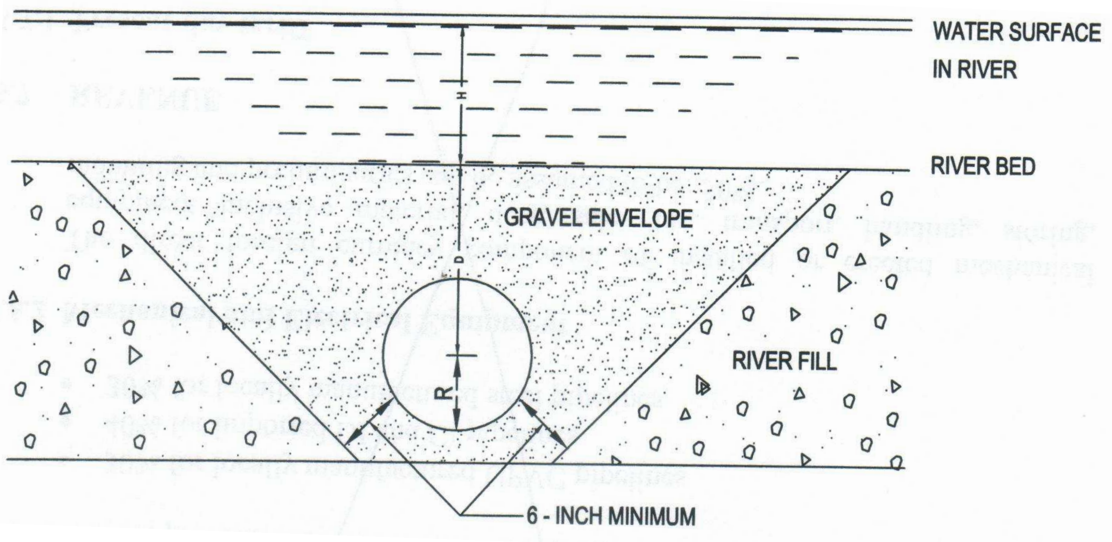
The length of the screen should be at least:

$$L = \frac{Q \;\; \text{In}\,\frac{2d}{r}}{2\,\pi\;KH}$$

Whereas:

L, Q, K and H are as in 5.8.2 above

1n	=	natural log
r	=	radius of pipe, m



5.8.4 Gallery in an Ephemeral or Intermittent stream channel filled with Permeable Material through which a Perennial Sub-surface Flow is moving.

The calculations for this case is complicated. That is why special manuals e.g. Groundwater Manual (1977) should be consulted.

5.8.5 Collection Pipes (screen)

- The pipes should be designed for a velocity of 0.5 to 1.0 m/s in order to be self-cleaning.
- The average entrance velocity through the holes or slots of the screen should not exceed 0.03 m/s.
- The diameter of the holes or width of the slots should normally be 3-4 times d_{60} of the gravel pack. (d_{60} is the sieve diameter through which 60% of the soil material can pass).

However the holes must never be bigger than d_{85} of the gravel pack.

6. WATER TREATMENT

6.1 GENERAL

Before a treatment works is designed it should be investigated whether it would be feasible to use an alternative source with a better rawwater quality.

6.1.1 Required Water Quality and Water Samples

See chapter “Water Quality” for definitions and general guidelines:

6.1.2 Principle Towns and Urban Centers with a population over 10,000

The treatment shall be effective enough to:-

- Achieve the bacteriological standards as set out in the chapter “Water Quality”.

- Achieve the chemical standard as set out in section “Constituents of health significance” and section “Desirable aesthetic quality” in chapter “Water quality”.

- Produce water that complies with the requirements set out in section “Substances and characteristics affecting Building and Pipe Materials “in chapter “Water Quality”.

6.1.3 Rural areas including Urban Centres with a population under 10,000 and Rural, Market and Local Centres.

The treatment shall be effective enough to:

- Achieve bacteriological standard as set out in the chapter “Water Quality”.

- Achieve the chemical standard as set out in section “Constituents of health significance” and section “Permissible aesthetic quality” in chapter “Water Quality” except for certain supplies in low and medium potential areas. See below.

- Produce water that complies with the requirement set out in section “Substances and characteristics affecting Building and Pipe Materials” in chapter “Water Quality”.

In low and medium potential areas where people and livestock use water from traditional sources which contain higher levels of aesthetic constituents than “Permissible” and where the costs for constructing or operating a treatment works would be out of proportion then the aesthetic quality could be reduced by the minimum margin achievable.

The criteria in such cases would be that the quality of water should not be lower than in the traditional source and that other improvements of the supply such as increase of the quantity and decrease of the walking distance to the water collection point compensate for the lack of improvement of the water quality.

However, treatment shall always be provided to guarantee the technical function of the distribution by removing debris and silt that may cause blockage or clogging of storage tanks and pipelines.

In this context the concept "cost out of proportion" may be considered as a cost per m^3 delivered water which is higher than what would be the cost of fully treated water (coagulation flocculation, sedimentation, rapid filtration, disinfection) from a treatment works designed to provide $250m^3$ clear water per day.

6.1.4 General Design Considerations

The following general principles shall be adhered to in the planning and design of treatment works:

- The works shall normally be designed for continuous operation i.e. 24 hours a day.

- All major units (chemical feeders, flocculation, sedimentation, filters) which require frequent servicing or cleaning should be provided at least in a number of two. Sedimentation tanks and filters should preferably be provided in a number of three in order to limit the temporary overloading to 50% when one unit is cleaned.

  However, in treatment works designed for less than $500m^3$/day it will not always be possible to have more than one unit as they then will be too small.
- Besides the above criteria the optimal number of units shall be determined through an economic analysis.

- As a general principle all kinds of mechanical and automatic equipment shall be kept to an absolute minimum.

6.1.5 Phasing of Treatment Works

- Even when the water analyse do not indicate immediate or future need for treatment provision shall always be made to incorporate full treatment at a later stage in case of deteriorating raw water quality or raised demand for high-quality clean water. A suitable site for the treatment works should be identified and adequate gradients provided to allow for the additional head losses through the works.

- When the design and construction of the treatment works is phased it shall be made sure that there is enough space for future extensions. Other preparations should not generally be made.

- Units whose performance depends on the velocity of the water have to be designed so that they will function also for flows smaller than the ultimate design flows. This applies in particular to the inlet pipe and the flocculation tanks.

6.2 PROLONGED STORAGE

6.2.1 General

A storage basin may serve a threefold purpose, it reduces turbidity by sedimentation, it improves the quality of the water because of substantial reduction of pathogenic bacteria through the activity of algae, other organisms and the ultra-violet rays in sunlight and finally it improves the reliability of water. The oxygen content of the stored water will usually be reduced after long storage and aeration may be required before distribution.

6.2.2 Design Details

The storage basin may be constructed by erecting a simple earth dam up to a height of about 6m. The dead storage should be about 2m and the detention time may be in the order of several weeks to a few months. Allowance should be made for losses due to evaporation and seepage, which often can amount 15-25mm/day.

For killing the schistosome larva (Bilharzia) a storage period of 48 hours is adequate.

6.3 INFILTRATION

6.3.1 General

Artificial infiltration of surface water improves both chemical and bacteriological quality of the water.

6.3.2 Design Details

See Chapter “Water Sources”

6.4 PLAIN SEDIMENTATION

6.4.1 General

A plain sedimentation basin mainly serves the purpose of reduction of turbidity and removal of suspended matter, and may serve as pre-treatment or the only treatment where the demand on the water quality is low. A sedimentation tank may have a batchwise or continuous operation and be made of concrete or masonry, or simple dug basin with protected sloping walls. Plain sedimentation may be applied as pre-treatment to slow sand filters when the average annual turbidity is 20 – 100 NTU.

6.4.2 Design Details

The surface load should be in the range of 0.1 – 0.5m^3/m^2h. The exact surface load shall be determined after settling tests on samples of raw water typical of all regimes of the river. The settling properties of a water depend on the soil and vegetation condition of the catchment's area and will vary considerably between different locations and regimes of the river. For design details see section "Sedimentation" below.

6.5 ROUGHING FILTRATION

6.5.1 General

Roughing filtration can be used as pre-treatment before slow sand filtration or as the only treatment in order to reduce turbidity.

6.5.2 Upflow Filters

- This kind of filter is often suitable as roughing filters as they provide for a coarse-to-fine filtration process and is not liable to clog rapidly. Used as roughing filters the surface load can be high, up to 20m^3/m^2h.

- The filter should have three layers of filter media, at the bottom 0.75m of grain sizes 10-15mm them 0.5m of size 7-10mm and at the top 0.5m of size 4-7mm.

- The wash rate can be relatively low as no expansion of the bed is needed but the washing takes long time about 20-30 minutes. Raw water can be used for back washing.

6.5.3 Horizontal Gravel Filters

The filters should have a depth of 1-2m, subdivided into three zones each about 5m long composed of gravel with sizes 20-30mm at the inlet side, then 15-20, and

10-15mm at the outlet side. The horizontal water flow rate computed over the full depth should be 0.5-1.0 m/h. Cleaning of the gravel will be needed only after a period of years. Turbidity removal of 60-70% can be expected for waters having up to 150 units of turbidity content.

6.5.4 Coconut Fibre Filters

These kinds of fibres have proved very effective to reduce turbidity. The filters should be designed similar to down-flow sand filters but the height of the filters bed should be only 0.3-0.5m and the depth of the supernatant water about 1m. The filters should operate at rates of 0.5-1.0 m^3/m^2h which should give a length of filter run of several weeks.

There should be a drain to let out all water in the filter before the coconut fibres are taken out and discarded. New fibres should be soaked in water for 24hrs before being placed in the filters. A reduction at the turbidity of 60-80% can be expected.

6.5.5 Rapid Roughing Sand Filters

These are built as conventional vertical – flow rapid sand filers, see section later in the chapter, only that the sand should be coarser, effective size 0.8–1.2mm. The backwashing may be done with raw water. The rapid roughing sand filter can be used as pre-treatment before slow sand filters when the turbidity of the raw water is 20-100 units. The surface load should be in the range of 5-15m^3/m^2h.

6.6 AERATION

6.6.1 Surface Water

There is generally no need for aeration of surface water with the intention of reducing high contents of iron or manganese. The high contents of these compounds, which are often found in Kenya surface waters are usually in a form, that is easily removable by coagulation, settling and filtration. Aeration may be required to raise the oxygen content in water from a large storage reservoir.

6.6.2 Ground Water

Aeration of groundwater may be necessary to increase the oxygen content, reduce the carbon dioxide content, removing hydrogen sulphite, methane and various volatile organic compounds responsible for taste and odour and often to reduce high iron or manganese. Sedimentation or filtration must follow the aeration if iron or manganese contents are to be reduced.

6.6.3 Cascade Aerator

The cascade aerator is one of the simplest types of aerators consisting of a flight of 4-6 steps each about 300mm high. The capacity should be about $35m^3/h$ per meter of width.

6.6.4 Multiple Tray Aerator and Diffuser

This type of aerator should be 4-8 trays with perforated bottoms at intervals of 300-500mm. The surface load should be about $70m^3/m^2h$. A simple diffuser consisting of a perforated pipe may be adequate for very small supplies.

6.7 COAGULAION

6.7.1 Rapid Mixing

The entire dose of chemicals has to be dispersed throughout the mass of raw water within a few seconds. Detention time before entering the flocculation chamber should be 30-120 seconds in mechanical and pneumatic mixers. Gravitational mixers will not require detention time.

6.7.2 Gravitational Mixing

Gravitational mixing can be achieved by feeding the chemical into a baffle chamber, an overflow weir or a hydraulic jump.

6.7.3 Mechanical Mixing

Mechanical or pneumatic mixers should be used only in very exceptional cases.

6.7.4 Coagulants

(a) Introduction

Quite a number of Water Treatment chemicals have been used as coagulants. They include inorganic primary coagulants such as Aluminum Sulphate (this is the most widely used coagulant), aluminum chloride, Ferrous Sulpate, Ferric chloride, Sodium Aluminate etc. These materials have highly charged metal irons which react with alkalinity in the water to form insoluble precipitates. These materials also chemically precipitate organic colour bodies in the water.

Other coagulants are activated silica, natural and synthetic polyelectrolytes.

(b) Polyelectrolytes

(i) General

Polyelectrolytes are linear or branched chains of small identical sub-units sometimes of two or three different kinds of subunits. The sub-units contain ionizable – COOH, -OH, $-POP_3\ H_Z$, $-NH_2^+$, and $R_1NR_2^+$ groups.

They are generally organic and inorganic polymers with ionizable groups.

They are:-

- Soluble in water
- Conduct electricity
- Used in the water treatment process as primary coagulants, coagulant aids, filtration aids and sludge conditioners.

Synthetic polyelectrotes are formed by polymerization of simple substances, known as monomers. Long chain molecules of this kind may be designed to contain from 2 to 3 to nearly 10^6 sub-units.

Polyelectrolytes used for water treatment are usually divided into three classes according to the electric charge of the macro-ions dissociated in water:-
Carbonic polyelectrolyes; positively charged anionic polyelectrolytes. Negatively charged Nonionic Polyelectrolytes, have no formal charge.

The electric charge determines the mode of reaction of the electrolytes when in contact with water. Polyelectrolytes differ in molecular weight, charge, density, charge chemistry and physical configuration.

They are supplied as liquid solution, granule and liquid emulsion polymers with specified active ingredient. These materials fall into two major categories.

Natural Polyelectrolytes
These are used as coagulant aids with Alum as the primary coagulant. Examples of natural polyelectrolytes include sodium alginate, hydroxyethyl cellulose and derivatives of potato starch

Natural polyelectrolytes are advantageous in that since they are also used in food preparation hence not toxic.

Synthetic Polyelectrolytes
There are many of these materials which vary in composition, for instance polydially dimethylamine Ammonium chloride – Poly (DADMAC) and polyamine Epichlorohydrin – diethyllamine (epi-DMA)

Owing to the varying composition of synthetic polyelectrolytes, the merits of any particular one have to be tried with the water it is proposed to treat before details of its use can be decided on.

(ii) Use of Polyelectrolytes

- Synthetic polyelectrolytes are increasingly being used in water supplies as primary coagulants aids, filtration aids and sludge conditioners. They are used in such countries as the U.S.A., Britain Egypt and Ethiopia while they have been banned in others such as Japan and Switzerland.

- Though natural polyelectrolytes are suitable for water treatment they are not readily available.

- The Application of synthetic polyelectrolytes in water requires very careful control particularly while dosing. For this reason, well trained Technical personnel (water operators) are needed at water works to manage their use.

(iii) Equipments and apparatus

The use of polyelectrolytes does not necessitate major changes of the equipment and apparatus normally used in our Water Treatment Works. But it is mandatory that those equipment and apparatus be availed as and when needed.

(iv) Specifications

The United States Environment Protection Agency (USEPA) and the Department of Environment of the United Kingdom (DOE) have carried out research on various polyelectrolytes used in water treatment and set specifications. The specifications have been adopted world wide.

(v) Availability of polyelectrolytes

The Polyelectrolytes currently available in the Local market for Water Treatment are (Epichlorohydin dimethylcemine (Epi-DMA) whose trade name is superfloc C-573, (supplied by cynamid) and Polydially dimenthylamine ammonium chloride (Poly (DADMAC)) whose trade name is Cat-floc TL (supplied by CALGON CORPO) popular package drums of 204 kgs. The current cost of a kilogram of the products Approx. KShs.700/= exclusive of VAT.

(vi) Performance

- Polyelectrolytes do not affect the chemistry of the water
- Different types of polyelectrolytes perform differently with the same water. Therefore a particular polyelectrolyte cannot be said to be suitable for treating raw water having different characteristics.
- They are fully soluble in water and give better overall water clarity.
- Polyelectrolytes give better compacted sludge hence less volume of sludge.
- Simple water works methods for determination of residual polyelectrolytes in water are not currently available.

(vii) History and usage

Ngethu Treatment works has been using polyelectrolytes since 1974. Activated silica polyelectrolytes (ASP_6) was used for about 10 years ie upto about 1983 when they could not get any more.

The Government chemist stipulated a maximum dose of 20mg/ℓ but due to availability of Analytical methods for measuring residual polyelectrolytes in the treated water in the weir, the City Council set a limit does of 5mg/ℓ.

It should be noted that the City Council in case of Ngethu Water Works has used Polyelectrolytes only as coagulant Aid especially during the rainy season when the turbidities and colour are high.

(viii) Health Aspects

Synthetic Polyelectrolytes may contain contaminants in the form of residual monomers, other reactants and reaction by products that can affect human health. However, when used within the recommended specifications, the concentrations in water are quite low to cause any harmful effects.

(c) Selection criteria for a coagulants

(i) Selection of coagulants must be made in consideration of the amount, quality and turbidity (max, and changes), filtration method and drainage disposal system, etc.

(ii) Dosage must be determined according to the following notes:-

- Dosage shall be decided on the basis of experiments conducted according to the quality of raw water, the results of experiments obtained in other water works, etc. must also be referred to in advance.

- Concentration to which the coagulant is diluted or dissolved for actual dosing must be adequate as viewed from feeding and handing.

- Dosage must be calculated on the basis of water amount treated and feeding rate.

(iii) Mixing must be performed in the mixing basin.

(d) Aluminum Sulphate (Alum)

Aluminum Sulphate is briefly called Alum and produced in either solid or liquid state. This agent proves effective against most of the turbids, and will be still more so when, at the time of high turbidity, used conjointly with coagulant Aid

Chemicals. Among the chemicals in use at present, Aluminium salts are predominant.

- Solid Aluminium Sulphate will be better to apply in solution of 5-10% concentration and must not be weaker than 1.5%.

- There should be at least two solution tanks one for mixing and one for dosage in between the holding 45 hours supply. The solution tanks should be equipped with hand agitators.

- If the alkality of the water is low, soda ash may have to be added in a proportion determined after jar test. The strength of the soda ash solution is usually 1-10%. The solution tanks should hold the same supply as for Aluminum.

The Chemical solutions shall be fed into the raw water by means of gravity dosers, floating balls or similar simple devices. Dosage pumps may be used only in very exceptional cases.

6.8 FLOCCULATION

6.8.1 Gravitational Flocculation

Baffle chambers should be designed for a channel velocity of 0.1 – 0.3m/s and a slot velocity of 0.5 – 0.6 m/s in the first 2/3 and 0.3 – 0.4 m/s in the last 1/3 of the chamber.

6.8.2 Mechanical Flocculation

This type of flocculator should be used only in very exceptional cases. There should be a minimum of 2 chambers. The peripheral velocity should be 0.6 m/s in the first tank and 0.2 m/s in the second tank. If more than two chambers are used the peripheral velocity can vary form 0.9 m/s to 0.2 m/s.

6.8.3 Power Input and Detention Period

The following criteria apply to both flocculation methods. The detention period should be 15-20 min. and the velocity gradient G 30 to 60 5^1.The product G.t. (where t is the detention period in seconds) should be 30,000 – 100,000.

The formula for computing the velocity gradient is:

$$G = \sqrt{\frac{P}{\mu C}}$$

in which

G = Velocity gradients (S^{-1})
P = Power transmitted to the water (kw)
C = volume of the flocculation tank (m^3)
μ = Kinematic viscosity of water (m^2/s)
($0.9 \times 10^{-6} m^2/s$ at 25^0c)

The power, P, can be calculate as

$$P = \left(\frac{q \times h}{102}\right) kW$$

in which
q = flow project in flocculation tank (l/s)
h = head loss in the flocculation tank (m)

It may be assumed that all the velocity head is lost when the water passes through the baffle slot and that also all the velocity head is lost at the changes of direction of 90^0 and more. Thus, the losses can be calculated as:

h = head loss in the flocculation tank (m)

It may be assumed that all the velocity head is lost when the water passes through the baffle slot and that also all the velocity head is lost at all changed or direction of 90^0 and more. Thus, the losses can be calculated as:

$$h = \left(\frac{nV_1^2 + mV_2^2}{2g}\right) + \text{ normal channel losses}$$

in which
n is the number of direction changes
m is the number of baffles
V_1 is the velocity in the channel
V_2 is the velocity in the baffle slot

Unless the floor or the flocculation channel is sloping with the same gradient as the hydraulic gradient of the water, V_1 and V_2 will change along the channel due to the decreasing water depth. The total head losses in the flocculation tank usually amount to 150-600mm.

6.9 SEDIMENTATION

6.9.1 Inlets

The inlets must distribute the flow evenly among the sedimentation tanks and within the individual tank itself. For uniform distribution of the influent the openings should be spaced close to each other less than 0.5m apart, and their diameter should be at least 50mm. The inlet velocity should not exceed 0.2m/s. The inlet channel should be generously sized with a cross sectional area of at least twice the combined area of the openings. The inlet to each tank should be possible to close for cleaning of a tank while the parallel tanks are operating.

6.9.2 Outlets

The draw-off should be made over at least 25% of the length of a horizontal tank. The effluent water should leave the tanks over a weir whose level is adjustable about 30mm or through a submerged perforated pipe. The total length of the weir in meters should be at least 0.1 x A where A is the surface area of the sedimentation tank in square meters. When filtration succeeds flocculation the outlet velocity from the sedimentation tank should not exceed 0.4 m/s in order not to destroy residual flocs.

6.9.3 Horizontal Flow Settling Tanks

The ratio length/width should be between 3:1 and 6:1 and the effective water depth should be at least 2m. There should be an additional volume of 25% to allow for sludge accumulation. The tank floor should slope gently (2-3%) towards a pocket placed at 1/4 to 1/3 of the length of the tank from the inlet side.

The surface load in settling tanks succeeding chemical coagulation and flocculation should be $1.0m^3/m^2h$.

6.9.4 Vertical Flow Settling Tanks of Sludge Blanket Type

The operational requirements of vertical flow sludge blanket-type tanks are so strict that they are generally not suitable for rural or urban water supplies except under very exceptional circumstances.

6.10 RAPID SAND FILTRATION

6.10.1 Design Details for Final Filters Succeeding Sedimentation

- Each filter unit should have individual inlet that can be closed for servicing and backwashing. The inlet should be designed in such a way that flushing and velocities 0.4 m/s do not occur.

- The surface load shall be $5m^3/m^2h$.

- The filter structure shall be designed with a minimum free height between the top of the filter media and the bottom of the wash-water troughs of 40% of the height of the filter media.

- There should be a first-filtrate connection on the outlet pipe through which filtered water can be let to waste the first 10 minutes after backwashing.

- The underdrain system will be reasonably well balanced if the following rules of thumb are applied.

  i) Ratio of area of orifice to area of bed served [(1.5 to 5) x (10^{-3})] : 1

  ii) Ratio of area of lateral to area of orifices served (2 to 4) : 1

  iii) Ratio of area of main laterals served (1.5 to 3) : 1

  iv) Diameter of orifices 5-20mm

  v) Spacing of orifices 100 – 300mm on centers

  vi) Spacing of laterals = closely approximating of orifices.

- Air binding of the filter media must be prevented by raising the outlet pipe to at least 50mm above the top level of the filter media.

- The underdrain system should normally be perforated pipes. Imported nozzles will only be accepted in very exceptional cases.

6.10.2 Filter Media

The filter bed should be 0.7 – 1.0m thick and preferably consist of quartz sand with an effective size of 0.5–1.0mm, normally 0.6-0.8mm, and a uniformity coefficient not exceeding 1.5.

6.10.3 Supporting Bed

The filter bed should be supported by a layer of course material generally gravel, that will not be dislodged by the backwash water. For example, filter sand of 0.7-1mm effective size would require 4 gravel layers from top to bottom.

15cm	2 - 2.8 mm
10cm	5.6 - 8.0mm
10cm	16 - 23mm
20cm	38 - 54mm

6.10.4 Backwashing

The backwash rates depend on the grain size of the filter media and should be in accordance with the table below for backwashing with water only. The table is valid for water temperatures of 30^0C, and for filter bet expansion of about 20%.

Effective grain size of the filter sand mm	Backwash Rate m^3/m^2h
0.5	25
0.6	30
0.7	40
0.8	50
0.9	60
1.0	70

- The backwash outlet system shall be designed for a minimum backwash rate of $50m^3/m^2h$ even if the calculated rate is lower.

- For calculation the amount of water used for backwashing it should be assumed that the period of backwashing is 8min.

- Backwashing with air in addition to water may only be used in very exceptional cases.

- The required pressure of the backwash water has to be determined through calculation of friction losses in the piping system and in the filter. The total head loss at backwashing often amounts to around 6 meters from the tank to the backwash water troughs.

- The distance the wash water will have to travel horizontally to the washwater trough should not exceed 1.5m.

- The carrying capacity of rectangular wash-water troughs is approximately as shown in the table below.

Depth of wash water flow in trough m	Width of trough m		
	0.25	0.35	0.45
0.25	30	40	52
0.35	53 1/s	75 1/s	96 1/s
0.45	82	115	148

- The backwash water tank should normally be designed for the backwashing of only one filter unit at a time.

6.11 SLOW SAND FILTRATION

6.11.1 Limitations

Slow sand filters can be expected to perform well if the turbidity of the raw water is less than 10 NTU. A turbidity of more than 50 NTU is acceptable only for few weeks and turbidities of 100-200 NTU can be tolerated for a few days only.

A too high content of algae in the raw water may disrupt the proper functioning of the filter. Some kind of pre-treatment to reduce the turbidity of river water at a peak flow will often be necessary. Possible pre-treatment methods are prolonged storage, plain sedimentation and roughing filtration as described earlier in this chapter.

6.11.2 Design Details

- The inlet structure should be designed in such a way that the raw water is equally distributed over the filter bed area. To achieve this the inlet velocity should be around 0.1 m/s and the width of the inlet structure should be at least 0.05 Q meters where Q is the design flow in m^3/h.
- Short-circuiting near walls should be prevented by the obstruction of flow along the same.
- The minimum size of one filter unit should be 15-12m^2.
- The surface load should be 0.1-0.2 m^3/m^2h.
- The height of supernatant water should be 1-15m.
- An oxygen content of more than 3 mg/1 in the clear water after the filter should be achieved.

6.11.3 Filter Media

- The full height of the filter bed of sand should be 1.0-1.4m. It should never be allowed to be less than 0.5m after the periodical cleaning.
- The effective grain size should preferably be 0.15-0.35mm and the uniformity coefficient 2-5. Locally available upgraded sand (builder grade sand) can however serve very well as the filter medium.
- There should be provisions for the washing of the filter media at least in a simple sand-washing platform where the sand can be flushed from a water hose. The washing area should be as close to the filters as possible.

6.12 DISINFECTION

6.12.1 Introduction

Many of the water-borne diseases like Cholera, Dysentery and Typhoid are transmitted by use of polluted water for drinking and food preparation purposes.

Bacteria, viruses, protozoa and cysts are among the various types of disease causing organisms (pathogens) found in polluted water. These disease causing organisms originate from the intestine of warm blooded animals. These are the organisms in water which have to be destroyed by disinfection.

The relation between Health and Water quality has been established for centuries. Already in 400 B.C., Hypocrates emphasized the importance of boiling and straining water. However, not until the 1800 was significant progress made in improving the quality of water. This was largely accomplished by adoption of filtration and chlorination.

The wide use of slow sand and rapid sand filters improves the microbiological quality. Bacterial and other organisms have to be removed by use of disinfection. Thus the hygienical safety of the water supply depends on disinfection, which can be achieved at the final stage of Water Treatment.

In practice, chlorine has been the most widely used chemical disinfectant.

The principal reasons for disinfecting drinking water are:

- To ensure the destruction of pathogens
- To maintain a protective barrier against pathogens entering the distribution system and
- To suppress bacterial regrowth in the pipe.

In practice, all water supplies require disinfection. From experience, it has been shown that adequate chlorine produces zero coliforms/100ml of test sample.

6.12.2 Mode and Methods of Disinfection

(a) Mode of disinfection

General

The presence of feacal micro-organisms in drinking water sources is a constant danger of water borne diseases carried through the water distribution system to consumers. These disease causative agents could be identified as viral, bacterial, protozoa and helminthic.

The common diseases caused by ingestion of contaminated water includes gastro-enteritis, bacillary, dysentery, typhoid, cholera, virus hepatitis and amoebiasis.

The table below shows the common water-borne diseases and their health effect.

Table showing water-borne diseases due to microbes.

Water-borne disease	Organism	Health effect
Gastro - enteritis	Various pathogens	Acute diarrhea and vomiting
Typhoid	Salmonella Typhosa (bacteria)	Inflamed intestine, enlarges spleen, high temperatures- fatal
Bacillary dysentery cholera	Shigella (Bacterial) vibrio comma (bacteria)	Diarrhea, rarely fatal. Vomiting, severe diarrhea rapid dehydration, mineral loss- high mortality.
Infections hepatitis	virus	Yellow skin, enlarged liver, abdominal pain – low mortality – lasts upto four months
Amoebic dysentery	Entameoba histolytica (protozoa)	Mild diarrhea, chronic dysentery
Giardiasis	Giardia lambilia (protozoa)	Diarrhea, cramps, nausea and general weaknesses – lasts 1-30 weeks – not fatal.

These diseases could be prevented through safe water supply, proper disposal of sewage and improved standards of living.

Mode of Disinfection

The different treatment processes practiced in water purification, particularly disinfection, play a great role in removing and destroying the disease causing organisms. The inactivation mechanisms of viruses by chlorine and other disinfectants has never been well understood. It is thought that the oxidant penetrates the cell membrane and attacks the nucleic acid.

Basically, a virus is composed of genetic material encased in protein cell or capsule. The genetic material is either deoxibonucleic acid (DNA) or ribonucleic acid (RNA). The capsule protects the virus from unfavourable environmental conditions and promotes development of virus.

The penetration of oxidants through the cell membranes will impair the action of enzymes; as a result the organism dies.

It is generally agreed that the relative efficiency of various disinfectants used in water treatment being powerful oxidizing agents, destroy the semi-permeability and enzymic activity of cell membrane, impair protein synthesis and react with nucleic acid. Similarly, the ultra violet radiation results in activation of micro-organisms DNA at 254μm.

Small dose of free chlorine is capable of inactivating 90 – 99% of Escherichia coli

(E. Coli) and other enteric Bacteria, Faecal coliforms and Faecal streptococci, and 50% or more of different types of virus and 100% of corcaria of Bilharzia.

6.12.3 Different Methods of Disinfection

Disinfection of portable water can be accomplished by physical or chemical means. The chemical method has been extensively used for over a century. The various disinfectants tried in full or in laboratory scale include chlorine in its various forms, ozone, bromine, iodine, chlorine dioxide, ultra violet radiation, silver, chloramines and potassium permanganate.

The criteria for appraising the suitability of various disinfectants for water supply are:-

- Disinfectants ability to destroy the organisms within the contact time available and the range of water temperatures encountered.

- Ready and dependable availability of the disinfectants at reasonable cost and in a form conveniently, safely and accurately applied.

- Disinfectants ability to accomplish the desired objectives without rendering the water toxic or objectionable aesthetically or otherwise.

- Disinfectants ability to persist in residual concentrations as a safeguard against recontamination.
- Adaptability of practical duplicable, quick and accurate assay techniques for determining the disinfection concentration for operating control of the treatment process, and as a measure of disinfecting efficiency.

Currently, there is no single disinfectant which satisfies all the above mentioned criteria. Even chlorine once considered to be free of any adverse health effect has shown its limitation.

Without forgetting the final objective of producing hygienically safe water, each factor has to be weighted carefully and proper selection of disinfectant has to be done.

(a) Chlorination

The addition of chlorine and chlorine compounds in water is called chlorination. Chlorine is commercially available in three forms; liquid, powder and liquefied compressed gas.

Among the available disinfectants chlorine in its various forms is widely used to disinfect drinking water. This is attributed to cheap cost, ease to apply and readily adaptability to different circumstances.

The main advantages of chlorination are the residual chlorine in the distribution network and storage, which provides protection against recontamination and microbial aftergrowth.

When chlorine is added to pure water, i.e. free form ammonia and organic matter, it produces hypochlorous acid (HOCL) and hypochlorite ion (OCL^-), which are the actual disinfecting agents. The reaction depends on PH and temperature of the water. The disinfecting capacity is retained in HOCL, and it is regarded to be greater than OCL^-. It has been shown that HOCL kills 80 – 100 times more E,Coli than OCL^-.

(b) Ozonation

Ozonation is the method of diffusing ozone, an allotropic form of oxygen into the water for the purpose of disinfection.

The inherent instability of ozone to decompose back to oxygen necessitates its production on site for immediate use. It is produced by passing an electric discharge of high voltage alternating current through the dry air. In the generation process, a voltage of 4-20kv is applied to dielectric plates about 6mm apart or to concentric tubes through which clean, dry oxygen rich gas is blown. The amount produced may be upto 30 mg/m^3 of the blown air. The generation process and the need of skilled manpower for operation and maintenance leads to higher cost. These disadvantages hinder its adoption in developing countries.

Ozone is also used for taste and odour control, colour removal and oxidation of iron and manganese and organic removal. When using ozone as a disinfectant the recommended dosage is 0.2 – 1.5 mg/L.

If a residual of 0.2 – 0.4 mg/h is maintained for 4 minutes, ozone has shown to be an effective viral disinfectant. Even though it is a very potent viricidal agent it lacks provision of lasting residual disinfecting power due to its decomposition back to oxygen. Thus, addition of chlorine, chloramines or chlorine dioxide to obtain residual in the network is unavoidable.

The use of ozone as a disinfectant has the following advantages:

- It is a highly effective disinfectant in destroying pathogenic bacteria and viruses.
- It is more effective in disinfecting water containing ammonia than chlorine.

Ozone's ability to decompose organic matter may cause problems in the network due to increasing volume of bioassimilable compounds, unless the degradation products are removed or an additional disinfectant with a long term effect is used.

Its high cost and the inability to maintain residual might be the main reason why it has not been widely used for water disinfection.

The following table summarises the disadvantages and advantages.

Advantages	Disadvantages
Wide Spectrum Disinfectant	High capital cost for equipment
Avoids taste and odour problems	Must be generated at site
Removes colour	High reactivity results in low selectivity
• Adds oxygen to water • Disinfection is rapid • Has high oxidation potential • Lowers BOD and COD values • Low concentrations are adequate • Does not form toxious compounds in treated water • Avoids problems associated with transportation of potentially harmful chemicals	• Low solubility under normal conditions • Operation and maintenance may be a headache • Residual 0_3 cannot be maintained in H_20 for long periods • More expensive than chlorine.

(c) Ultra violet radiation

The UV rays are part of the electromagnetic spectrum between x-rays and visible light.

The most common source of UV radiation is the coated mercury low pressure lamp. This type of lamp is mostly used in existing UV plants for disinfection purposes.

To achieve the maximum efficiency the lamps must be kept clean so that the organisms are exposed to the full intensity of the UV light.

Disinfection is accomplished when the water is exposed to the UV light source. For effective disinfection, the water must be without turbidity or colour and free from organic colloids that might form deposits on the tubes or obstruct the passage of light. Also the depth of water must be shallow.

Because the UV radiation leaves no residual, the disinfection has to be complimented either with chlorine or chlorine dioxide. The shortcomings coupled with the hardware problems make UV unfeasible in developing countries.

Table showing advantages and disadvantages of UV for disinfection of portable water

• Foreign matter is not introduced into water and physical or chemical character is not significantly affected	• Spares, cysts, and viruses are less susceptible than vegetative bacteria.
• Constituents of the water in solution, such as ammonia, exert on effect on disinfecting capacity	• Thorough water preconditioning is required because UV is absorbed by many constituents normal even to pretreated waters.
• Tastes or odours are not produced (but UV has no effect on removal of odour or colour).	• Residual disinfecting capacity apparently is not provided.
• Short contact time is effective	• Extensive amounts of electric energy and expensive equipment are required.
• Overdosing produces no detrimental effects.	• Frequent and expensive apparatus maintenance is necessary to ensure stable energy application and essentially uniform density throughout effective radiation area. Treatment efficiency is not readily determinable (lack of rapid field test).

(d) Comparison of different disinfectants

It seems that there is no competitive alternative to chlorine for the water supplies.

(e) Chlorine and its compounds

(i) Chlorine gas
Chlorine gas is generally used for disinfection in large Water Treatment plants. It is supplied as a liquid gas under pressure in steel containers or cylinders. The capacity of the cylinders range 45-100kg. The chlorine content is 100%.

(ii) Calcium hypochlorite (TCL)
The powered form of chlorine is known as calcium hypochlorite, $Ca(ocl)_2$ and it is used as loose powder or compressed tablet forms.

It is commercially available as bleaching powder containing about 30% available chlorine and calcium hypochlorite granules. When mixed with water it produces suspension which needs precipitation, which is one of the disadvantages of using $ca(ocl)_2$.

(iii) Calcium Hypochlorite granules (HTH)

This is 70% the bleaching powder, calcium oxy chloride – except the available chlorine content which is 65-70% by weight. The chemical is supplied in 50kg drums of plastic lined containers containing 45kg of ca(ocl)$_2$.

For making the standard strength solution of ca(ocl)2, it is preferable to use water containing hardness (ca co3) less than 75-100 mg/l. Use of hard water results in precipitation of its hardness

6.12.4 Feeders

The solution should be fed into the water by means of gravity dosers or displacement doser. Dosage pumps may be used in very rare circumstances.

6.12.5 Contact Period and Residual Chlorine

- A contact period of at least 30 minutes in the clear water tank and the transmission main should be allowed before the water reaches the first consumer.

- The residual should be 0.3-0.5 ppm free chlorine after half an hour contact period. A tap for sample should be provided after the clear water tank.

- Important: The gauging rod must be in a distance of at least 3 h from the weir. The zero point of the rod must be on the same level as the crest of the weir.

6.12.6 Pot Chlorination

It is desirable to chlorinate open dug wells also when they serve only one or a few families.

This can be done using an earthen pot of 7-10 litres capacity with 5-10 holes with a diameter of 6 to 8mm in the bottom. The pot should be half filled with pebbles, 20-40mm size, TCL and sand in a 1 to 2 mixture should be placed on top of the pebbles and the pot should be filled up to the neck with pebbles. The pot should be lowered into the well with its mouth open.

For a well from which water is taken at a rate of 1000-1200 litres/day a pot containing about 1.5 kg of TCL should provide adequate chlorination for about a week.

6.13 FLOW MEASUREMENTS

6.13.1 General

The raw water should be measured in a V-notch weir or a flume which also should serve as the feeding spot for precipitation chemicals when applicable.

There should be a master meter measuring the amount of clear water distributed to the consumers.

6.13.2 Thompson weir

This method is suitable for quantities up to about 10 1/s
The following arrangements have to be made.

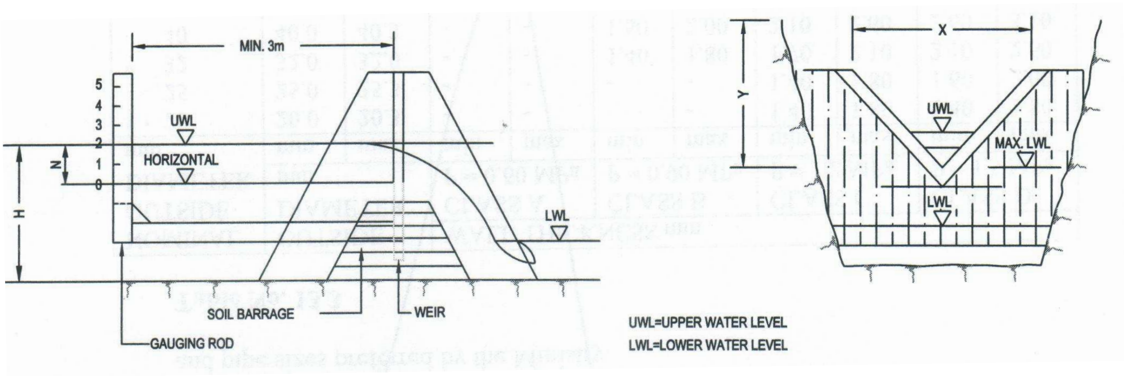


Fig No. 6.1:_Thompson Weir

- Minimum H = 2h
- Maximum velocity of water at the gauging rod = 1 m/s
- Normally a 90 ^{0} weir is used $\left(\frac{x}{V}\right) = 2$

Important: The gauging rod must be in a distance of at least 3 h from the weir. The zero point of the rod must be on the same level as the crest of the weir.

Fig 6.2: Discharge over Thomson weir for h = 0 – 10 cm

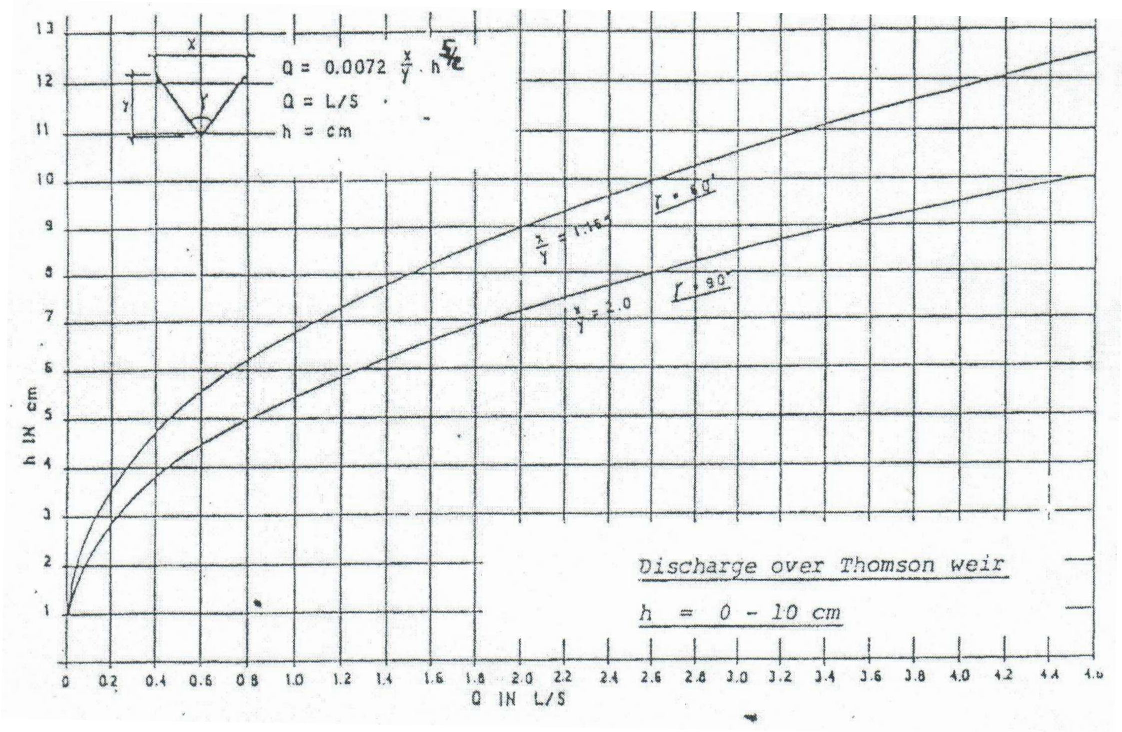
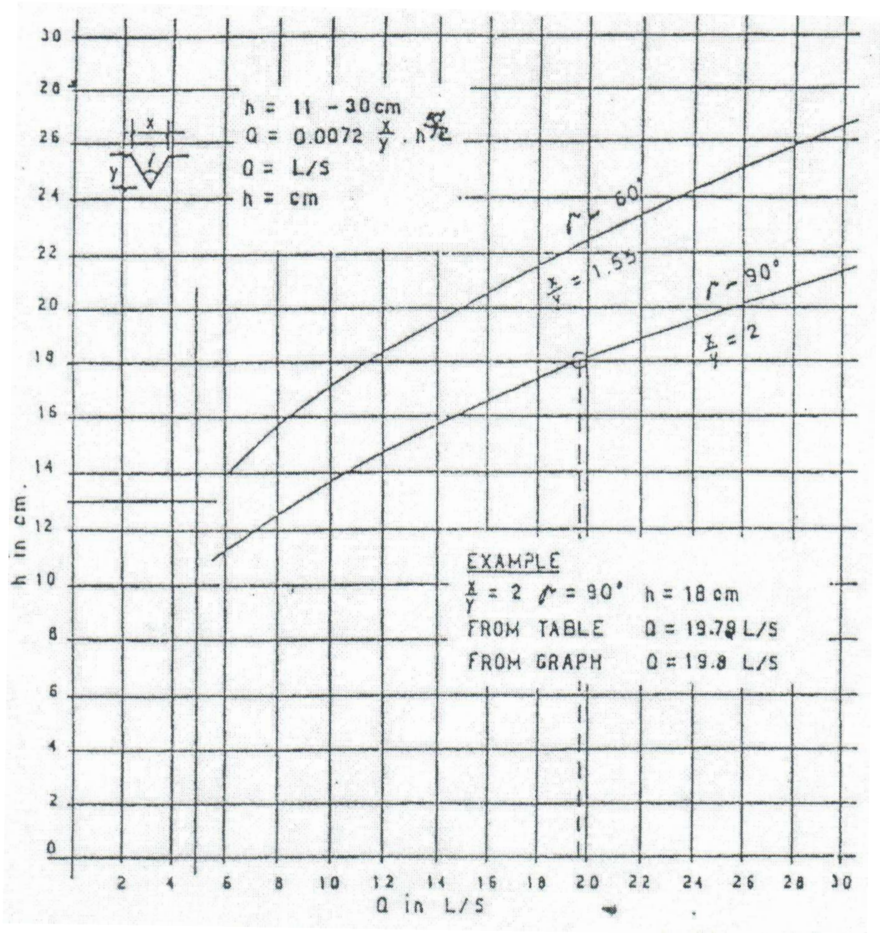


Fig 6.3: Discharge over Thomson weir for h = 11 – 20 cm



6.13.3 Rectangular Weir

This method is applicable for quantities above 10 l/s.
The following arrangements have to be made:

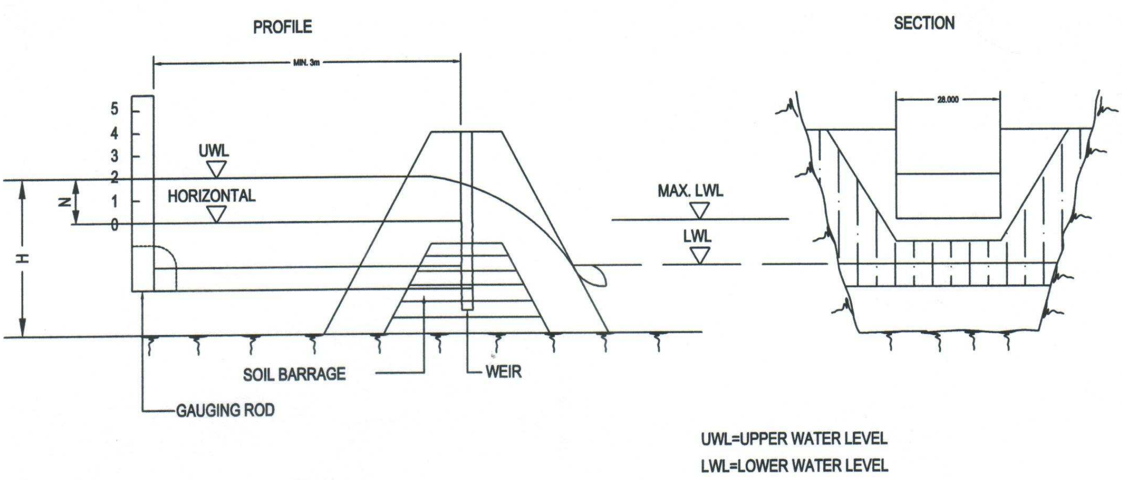
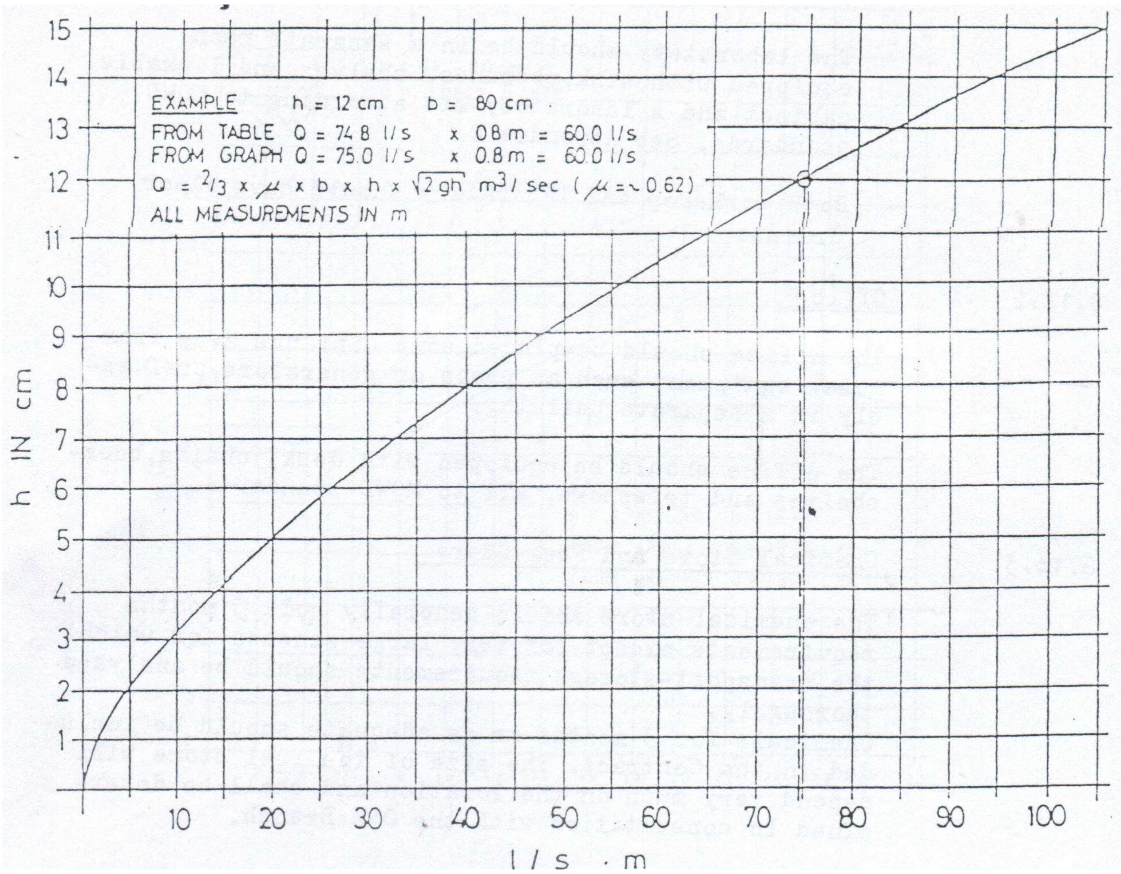


Fig. No. 6.4: Rectangular Weir

- Minimum H = 4 h
- Maximum velocity of water at the gauging rod = 1m/s
- **Important**: The gauging rod must be in a distance of at least 3h from the weir, the zero point and the rod must be on the same level as the crest of the weir.
- Normally the minimum width for a weir should be 50cm, better 1.00m.
- The crest of the weir must have a sharp edge.

Fig 6.5: Discharge over Rectangular weir



6.14 BUILDING

6.14.1 Workshop and Laboratory

- A workshop should generally be provided with working bench, shelves, a lockable cabinet and tools according to MOWD standards. See Appendix C.

- Both workshop and laboratory should have floor drains.

6.14.2 Office

The office should be placed some distance away from noisy equipment such as pumps or generators, preferably in a separate building.

The office should be equipped with desk, chairs, book-shelves and telephone, all according to the MOWD standards.

6.14.3 Chemical Store and Fuel Store

The chemical store should generally hold 3 months requirements except for very large schemes for which the transport-storage requirements should be analysed thoroughly. Chemicals for 3 months or as adequate should be included in the contract. The size of the fuel store will depend very much on the location and shall be determined in consultation with the O & M Branch.

6.14.4 Pipe Store

All schemes should have a lockable pipe store or shelter containing ten lengths of each size and class of pipes used in the scheme. The store should be equipped with simple racks for all the spare pipes included in the contract.

6.14.5 Staff Houses

The number of required staff houses should be determined after consultation with the Operation and Maintenance Branch of the Ministry. Standard drawings should normally be used. The house should be equipped with hard furniture. One of the staff houses will be used as the Resident Engineer's residence during the construction period. The staff houses should be placed at some distance away from the treatment works and pumping stations or other noisy equipment. There should be a fence between the treatment works and the staff houses.

6.15 MISCELLANEOUS

6.15.1 Access Roads

There should be access roads up to all buildings to which chemicals are delivered frequently or which contain heavy equipment e.g. pumps and generators.

6.15.2 Safety Measures and Devices

Safety devices such as lighting, fencing, handrails, fire extinguisher and first aid kit shall be provided. All valves within the treatment works area should be possible to operate from the ground level and not only from inside a covered manhole.

6.15.3 Flush Water

Connection for flushing water adjacent to flocculation tanks, sedimentation tanks and sand filters should be provided with the highest available pressure.

6.15.4 Marking

Visible pipes in the treatment area should be pained or marked in accordance with the following colour scheme.

Raw water	-	Grey
Clear water	-	Light blue
Sewage and Drain water	-	Brown
Chlorination solution	-	Green
Alum Solution	-	Red
Soda Ash	-	Yellow

6.15.5 Fencing

The treatment work site should be fenced and separated from the staff house site.

6.15.6 Transport Facilities

Bicycles or motorcycles should normally be included in the contract and at least one vehicle for a large scheme.

7. TRANSMISSION AND DISTRIBUTION LINES

7.1 GENERAL DESIGN ASPECTS

7.1.1 System Design

A given supply area may be best served by one large scheme or by several independent smaller schemes. Alternative systems have to be studied and compared from technical as well as economical viewpoint.

The designer should in particular, study peripheral or isolated high areas with few consumers within the supply area to determine whether these would be most economically served by a separate supply, e.g. a spring, booster pump, rain harvesting or by the main supply system.

The construction cost of the transmission lines, expressed as Shs per m^3 water, will generally increase with the size of the supply area. However the increase will often be off-set by decreasing costs for water treatment and pumping with larger supply areas. The most economical scheme area will have to be decided for each project, as it is not possible to give general indication as to the optimal scheme size except that simple gravity schemes without treatment should be smaller than complicated schemes with treatment ad pumping.

7.1.2 Method of Supply

A gravity fed supply should be preferred whenever technically, economically and financially feasible.

The economic analysis should comprise both the total cost and the foreign exchange component of the total capital and operation and maintenance costs over the design period. A gravity system with higher total costs than a pumped system may still be preferred if the foreign exchange component is higher for the pumped system.

The financial feasibility depends on the possibilities of obtaining funds from the Government or a donor and changes from time to time.

7.1.3 Pipeline Design

- The cost of the transmission and distribution system constitutes the bulk usually 80-90% of the construction costs of a large water supply. It is therefore important that the efficiency of the system is maximized through careful alignment of the pipe routes.

The design shall aim at supplying the maximum number of consumers at lowest possible cost. In practice this can be achieved by strict adhering to the following design rules.

- Balancing tanks shall be incorporated in the system in order to cut down peak flows. The position of and the capacity of each tank should be determined after economic analysis aiming at minimizing the system cost.

- A pipe transversing a supply area shall be designed for the peak flow of that area plus the mean flow of succeeding areas as shown on Figure No. 7.1. Regarding peak flows see chapter “Water Demand” and tanks chapter “Water Storage”.

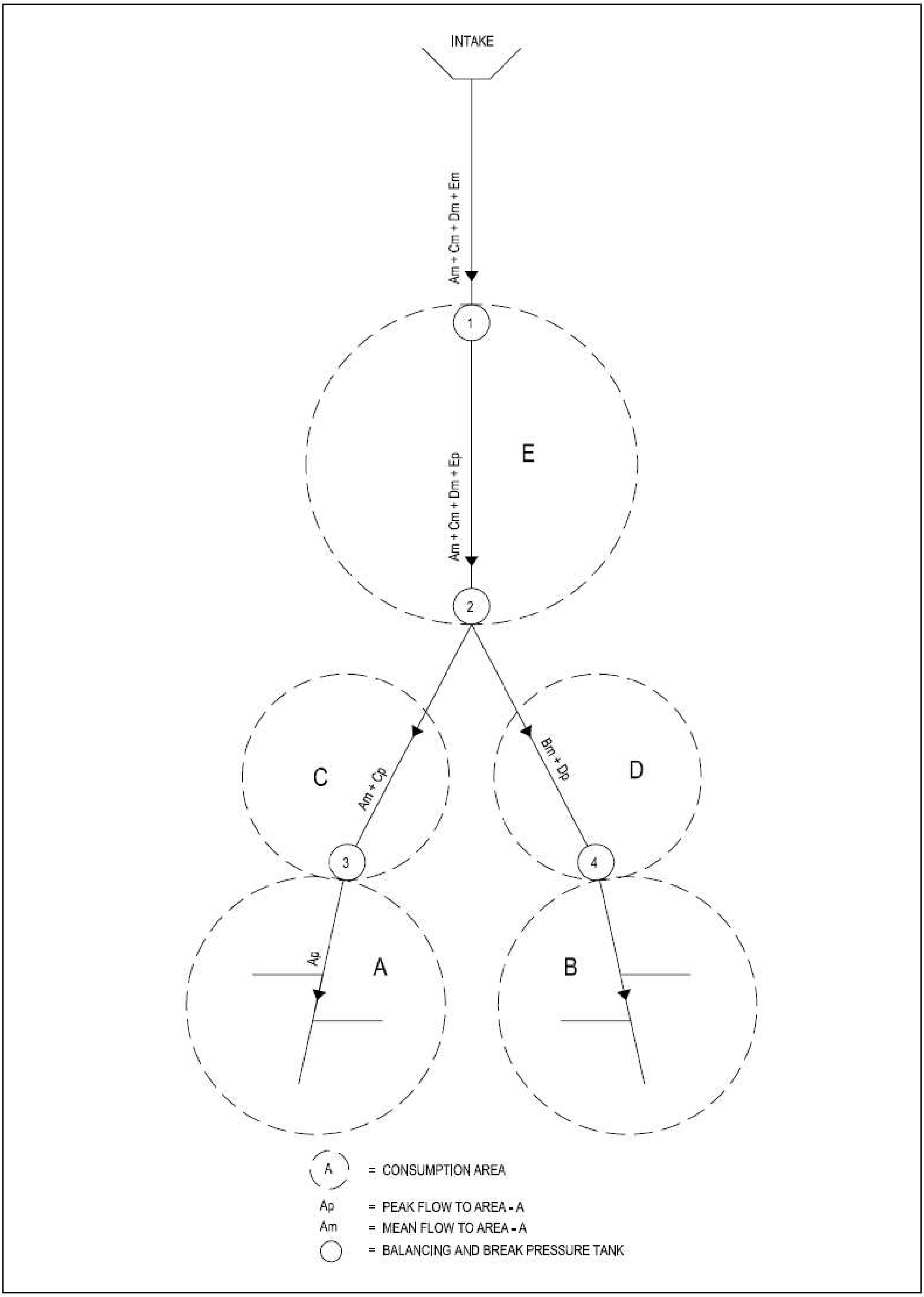
- The static pressure should be kept low by breaking the pressure preferably in the balancing tanks or in separate break-pressure tanks.

- The number of major high points and low points should be kept to a minimum where possible by trying to follow the contour lines of the terrain rather than only roads and tracks. This calls for active participation by the design engineer in the survey of the pipeline routes. For large size pipes (150mm) alternative routes may have to be surveyed for an economic analysis in order to find the optimal alignment.

- The excavation depth should be varied to avoid local high and low points in order to minimize the number of air-valves and washouts. Further see chapter “Drawings”.

- The pipeline should be set out by the Resident Engineer who also should check that the pipe levels are strictly in accordance with the drawings.

Fig 7.1 Gravity Flow calculation typical procedure



7.1.4 Phasing

The construction of large projects will generally have to be phased. This should be considered at the preliminary design stage so that the projects can be divided into technically functioning sub-schemes of financially manageable size.

7.2 SPACING OF PRIMARY AND SECONDARY PIPELINES

7.2.1 High and Medium Potential Rural Areas

Generally, the distance from 90% of the residential houses to the nearest primary or secondary pipeline should not exceed 1km. However, the features of the terrain, the spacing of roads and tracks etc. may make it necessary to deviate slightly from the general recommendation.

7.2.2 Low Potential Rural Areas

The distance from 90% of the permanent residential houses to the nearest primary or secondary pipeline should normally not exceed 2.5km. However, the local conditions will in many cases necessitate adjustments in order to serve the maximum number of people best.

7.2.3 Urban Areas

The pipelines should follow roads and streets as shown on the Town Plan.

7.3 PIPES

7.3.1 Material Selection – General

Preference should be given to pipes, which are manufactured in Kenya when there is no major difference between their performance and prices as compared to imported pipes.

At present (1983) polyvinyl chloride (UPVC) and steel pipes are manufactured in Kenya. Regarding standards see under the chapter “Standards” which also contain pipe dimensions, pressure classes etc. for the most commonly used pipe types.

7.3.2 Material Selection - Corrosion aspects

- Regarding internal corrosion see section “Substances and Characteristics Affecting Building and Pipe Materials” in chapter “Water Quality”.

- External corrosion may well be a greater problem than internal in particular for steel and iron pipes. The same constituents that affect the pipe from inside will also attack the outside of the pipe. Corrosion is often a complex process. Four general types are recognized: galvanic, electrolytic, stress and biochemical. The soil resistivity and corrosion often have a close relationship as shown in the table below:

Soil resistivity 1/ Ohm-m	Corrosivity of steel (Soil with pH>6)
<1	Very much corrosive
1-10	Very corrosive
10-100	Little corrosive
>100	Non-corrosive

1/ Determined as resistivity in 100g soil + 100g water.

Cast iron and ductile iron are more resistant to corrosion than steel. The table below can be used to determine whether the soil is corrosive to iron and ductile iron. If the total risk points exceed 10 the soil is likely to be corrosive and some protective measures should be taken.

Parameter	Value	Risk point
Resistivity Ohm-m	<7	
	7-10	
	10-12	
	12-15	
	15-20	
	>20	
pH	0-2	5
	2-4	3
	4-6.5	0
	6.5 – 7.5	0 2/
Redoxpotential mV	>100	0
	50-100	3.5
	0-50	4
	Negative	5
Sulphides	+	3/5
	trace	2
	-	0
Moisture	Poor drainage Always wet	2
	Fair drainage Mostly wet	1
	Good drainage Mostly dry	0

2/ If the redoxpotential is low or negative if also there are sulphides present the risk point will be 5.

Protective measures e.g. lining and coating of pipes with cement mortar, coal tar etc. or cathodic protection should always be determined in consultation with the pipe manufactures.

7.3.3 Design Dimensions of UPVC Pipes

The design of pipelines shall be made for pipe dimensions in accordance with KS 06-149 Part 2, 1992 (Metric Series) See section 13.5

The PVC-pipe shall be designated by its nominal outside diameter and the pressure class e.g. 110 A

If pipes manufactured to KS 06 – 149 Part 3 (Inch Series) are used for the construction of the pipeline the comparable minimum dimensions as shown in the table "Equivalent Dimensions" in the chapter "Standards" shall be used.

7.3.4 Friction Losses

The friction losses in pipes should be calculated with the Colebrook – white (Universal) formula. The roughness, K, to be as follows. This includes normal bends and fittings along a pipeline:
UPVC K = 0.1mm
Steel, GS, CI and DI K = 1.0mm
Diagrams over the losses in pipelines based on the above K-factors can be found in Figs N.7.2 and 7.3. Losses in bends and fittings are given in chapter "Pumps and Power Sources" for separate calculations of these.

FRICTION LOSS ‰$_0$ (m/1000m)

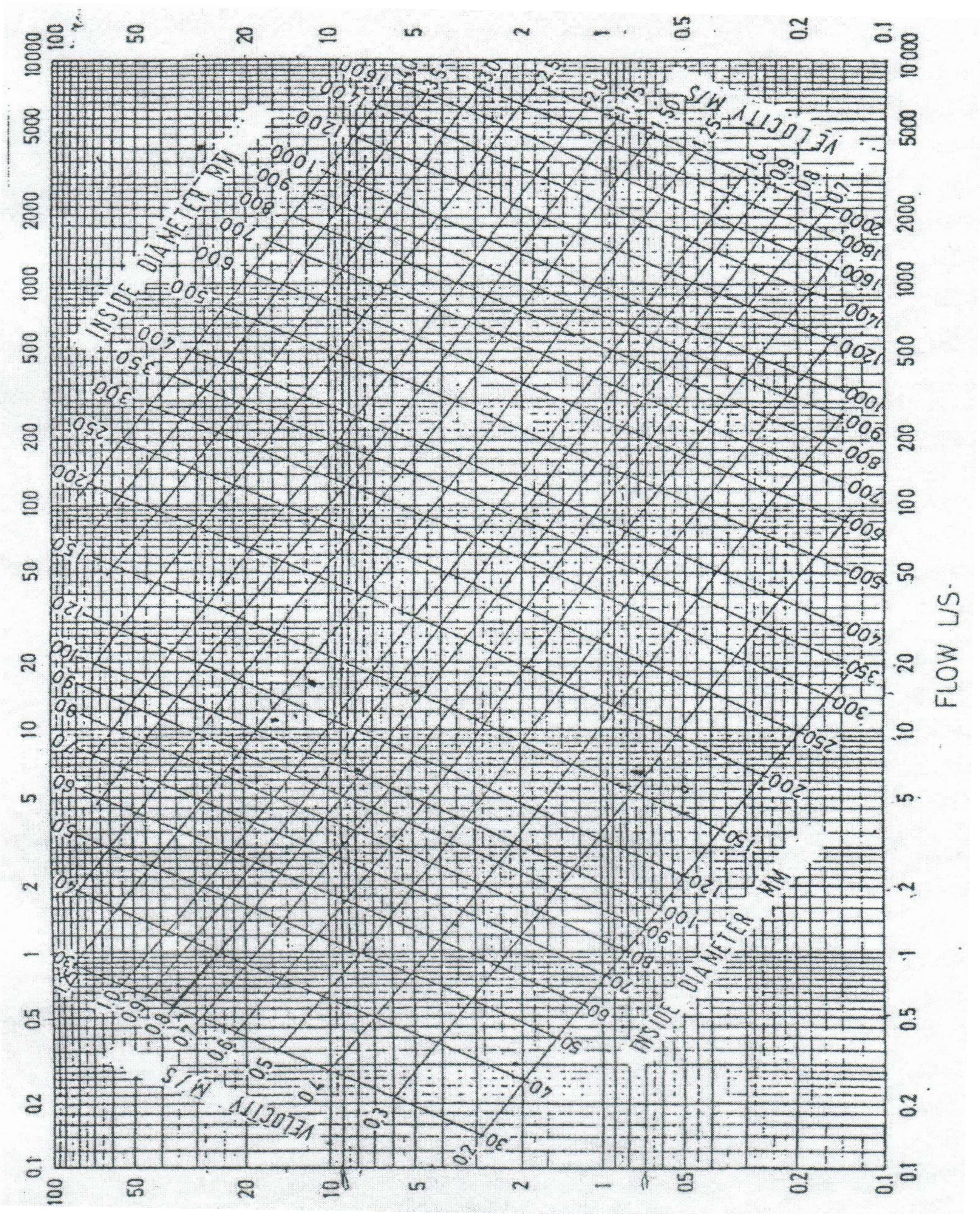


Fig 7.2: Loss of head for the flow of water in straight pipes, k = 0.1mm

FRICTION LOSS $‰_0$ (m/1000m)

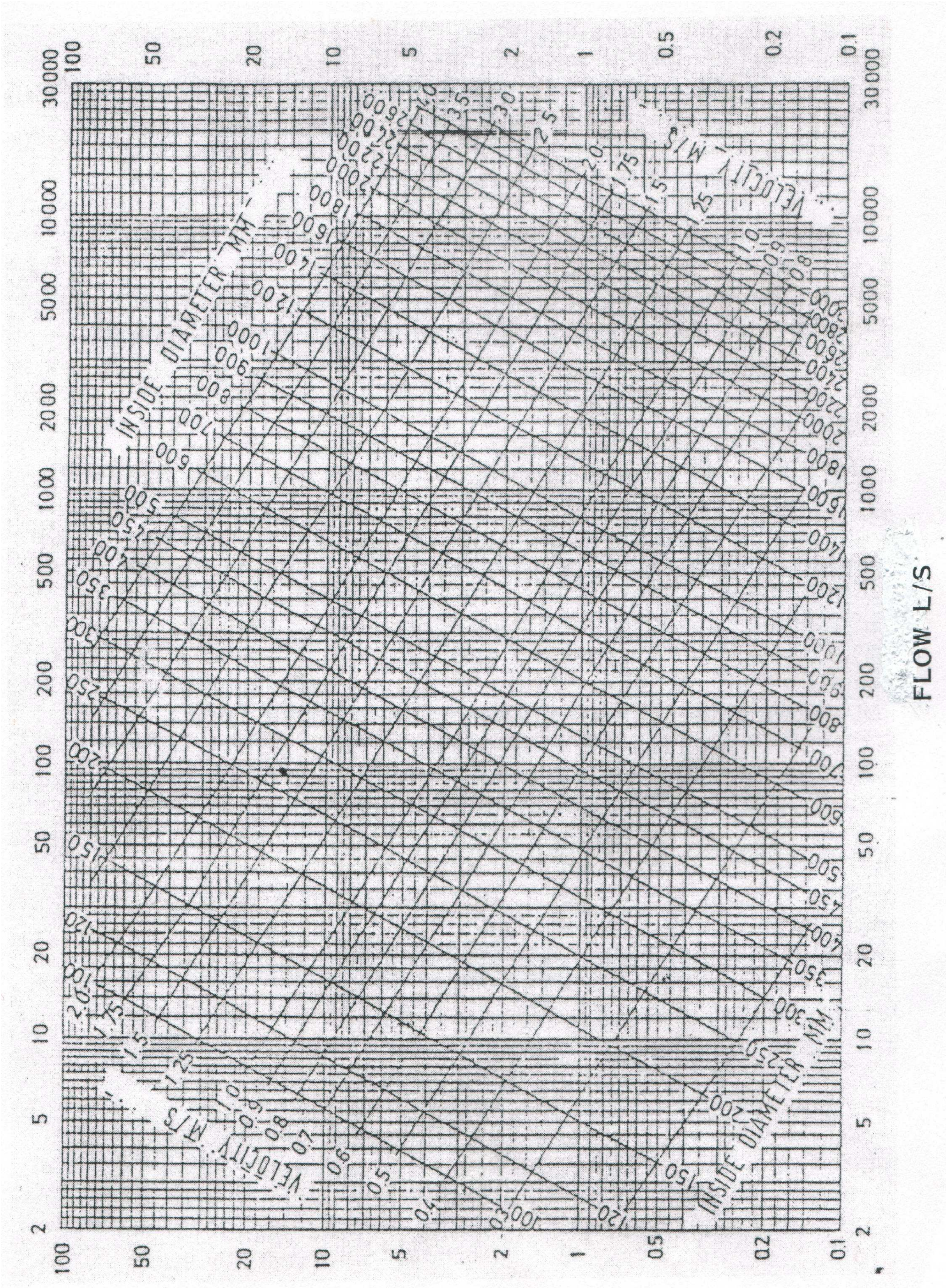


Fig 7.3: Loss of head for the flow of water in straight pipes, k = 1mm

7.3.5 Cover and Slope of Pipes

- The pipelines shall be put in straight lines between changes in gradient. The slopes shall at no place be less than 0.5% for diameters of 200mm and less and 0.2% for bigger pipes.

- To minimize the number of changes in grade the pipes shall be laid with a cover varying from a normal minimum of 0.6m to a normal maximum of 3m.

- The pipeline must not be designed having local high points where air pockets may develop without having any chance of being released.

- The minimum cover over unprotected pipes in areas where motor traffic may occur shall be 0.9m. Pipelines in road reserves should be located, whenever possible 1.5m from the edge of the road reserve.

- Pipelines below road surfaces should be laid as instructed by Ministry of Transport and communications.

7.3.6 Pumping Mains

- Water hammer and surge shall be taken into consideration when designing pumping mains. For UPVC-pipes the total pressure variations from minimum to maximum should be limited to 50% of the nominal working pressure of the pipe class.

- The peak pressure, inclusive the water hammer, should not exceed the nominal working pressure of the pipe class. UPVC-pipes class 0.6 Mpa must not be used in pumping mains.

- The most economical pipe diameter should be selected through an economic analysis. For a tentative estimate it may be assumed that the most economical size of long pumping mains can be found by using a velocity of flow of 0.8 m/s in pumping mains.

- Design of pipe installation in the pump house and of short discharge pipes should be done as described in chapter "Pumps and Power Sources".

7.3.7 Pressure

- The minimum pressure at design flow should be 0.1 Mpa (10 metre water head) in pipe sections to which there may be made consumer connections and 0.04 Mpa (4m) in other cases. The levels of the surrounding areas to be served form the pipeline must be considered when determining the minimum pressure.

- The static pressure in pipes with consumer connections should be not more than 0.6 Mpa (60m) unless the terrain makes higher pressures unavoidable. Higher pressure than 0.6 MPa may require special fittings, ball valves, stop valves etc. for the consumer connections.

- In urban areas with provision for fire-fighting, the minimum pressure of 0.15 Mpa (15m) should be up-held at a withdrawal of 10 1/s. There should be an isolating valve downstream of each fire hydrant in a non-loop system.

- The following is the design pressures of various materials: -

Pipe material	Maximum working pressure	Sizes
UPVC (Polyvinyl Chloride)	1.5 MPa	25-300 mm
PEH (Polyethylene High Density)	1.2 MPa	15-50 mm
GS (Galvanised Steel)	Depending on grade and size	ALL
CI (Cast Iron)	Depending on grade and size	80-1200 mm

7.3.8 Water Hammer

Water hammer is a phenomenon which may be caused by closing or opening a valve, or start or stop of pumps etc. Generally the maximum water hammer can be calculated with the following formula:

$$WH = \frac{\pm CV}{g} \qquad (1)$$

Where WH is the pressure rise (or drop) in m of water, C is the velocity of the pressure wave. V is the initial minus the final velocity of water when flowing in the pipe (m/s), g is the acceleration of gravity. For a circular pipe:

$$C = \frac{C_W}{\sqrt{1 + \frac{E_w}{E_p} \times \frac{D_m}{t}}} \qquad (2)$$

Where:-

C_W = Celerity of the pressure wave in water = 1425 m/s
E_w = Elasticity modulus of water (N/mm^2)
E_p = Elasticity modulus of the pipe material (N/mm^2)
D_m = Mean diameter of the pipe D_m = Di + t (mm)
t = wall thickness (mm)

If the pipe is fixed in the longitudinal direction, then Ep must be substituted by Ep/ $(1-r^2)$, where

r = Poisson's ratio

Knowing D_m and t, E_w and E_p formulae (1) and (2) can be simplified to:

WH = ± ℓV where also C = ℓg

(The above expressions are based on the assumption that a valve is opened/closed suddenly, or the time taken to close the valve, T<2L/C(s) where L is the length of pipe).

Elasticity modulus and Poisson's ratio

MATERIAL	**ELSTICITY MODULES Ep N/mm^2**	**POISSONS RATIO**
Polyvinylchloride (UPVC)	3×10^3	0.5
Polyethylene (Low Density) (PEL)	0.15×10^3	0.5
Polyethylene (High Density) (PEH)	0.8×10^3	0.5
Galvanised steel, me. Grade (GS.MG)	210×10^3	0.3
Cast Iron (CI)	100×10^3	0.3
Ductile Iron (DI)	170×10^3	0.3
Water	207×10^3	-

Value of ℓ can be found for UPVC pipes and GS pipes in the tales below:

UPVC pipes to KS 06 – 149 Metric Series			
Pressure class Mpa	Nominal outside Diameter mm	Celebrity of press. Wave, C m/s	Factor ℓ
0.6	≤ 160	295	30
	> 160	173	28
0.9	≤160	355	36
	> 160	331	34
1.2	≤ 160	399	41
	> 160	378	39
1.5	≤ 160	444	45
	> 160	419	43

STEEL pipes to ISO 65				
Nominal inside diameter mm	**Heavy Series**		**Light Series 2**	
	Celerity, C m/s	Factor ℓ	Celerity m/s	Factor ℓ
50	1345	137	1303	133
65	1324	135	1287	131
80	1320	134	1267	129
100	1301	133	1248	127
125	1276	130	-	-
150	1252	128	-	-

Example 1: In a UPVC 225/10.5 – 1.2 pumping main, 1km long, the flow Q=20 1/s gives V=0.6 m/s. The water hammer pressure is:

WH = ± 39 X 0.6 = ± 23.4 m if the sudden closure/opening of the valve or sudden start/stop of a pump is quicker than :

T = 2 x 1000/378 = 5.3 s

Example 2: Which class is required for a UPVC pumping main of diameter 160mm if the velocity is 0.8 m/s, friction losses 15m, static head 60m?

Assume pressure class 1.2 MPa

WH = ± 41 x 0.8 = ± 32.8 m

Hence, total head inclusive water hammer

H = 60 + 32.8 = 92.8m which is allowed.

However the total amplitude 2 x 32.8 = 65.6m is more than 50% of the nominal working pressure for the class 0.5 x 120 = 60m.

Thus, class 1.5 MPa has to be selected.

7.4 CORROSION PROTECTION

7.4.1 General Corrosion Protection

Generally, the internal surface of pipes is protected with a centrifugally applied cement mortar lining. The cement should be Portland cement, fly ash cement, or sulphate resisting Portland cement. The standard thickness of the lining is as shown in the table below:

The seal coat, if applied, should be bitumen, acrylic emulsion, or PVC solution.

Nominal Diameter DN (mm)	**Lining Thickness (mm)**	
	Nominal	**Minimum**
80 to 250	4	3
300 to 600	6	5
700 to 900	8	6
1000 to 1200	10	7
1350 to 1500	12	8
1600 to 2600	15	11

The internal surface of fittings is protected with hand applied cement mortar lining or with a 0.1mm thick tar-epoxy coating material.

7.4.2 Special Protection

When aggressive fluid is passed through the pipe, special care should be given to the internal protection of the pipe.
Aggressive fluids:

- High content of free carbon dioxide (CO_2)
- Raw sewage
- Acid water

(a) High free carbon dioxide (CO_2) content
Water containing free carbon dioxide above 20 ppm is considered to be aggressive to cement mortar lining.

(b) Raw sewage
When sewage is at high temperature, when flow velocity is extremely low, or when sewage flows in a partly filled pipe, sulphate in raw sewage will be reduced to hydrogen sulphide gas (H_2S) and finally from sulphuric acid (H_2SO_4) which damages pipes and other facilities.

(c) Acid water

In the situations mentioned above, special protection such as cement mortar lining seal-coated with tar-epoxy, fusion bonded epoxy coating or other coatings should be applied.

External Corrosion Protection

(a) General protection

Since ductile iron has excellent corrosion resistance, heavy-duty corrosion protection is generally not required. Usually, the external surface of the pipe and fittings is protected with tar-epoxy coating material having a minimum dry film thickness of 80 microns.

(b) Polyethylene sleeving method

Under normal conditions, standard coating provides sufficient protection against corrosion. However, when pipes are laid in exceptionally corrosive soil areas, it is recommended that the polyethylene sleeve corrosion protection method be employed in addition to the standard coating. The method of estimating the corrosiveness of soil is specified in the American National Standard (ANSI A 21.5).

(c) Special coating

(i) Above ground (exposed)

Pipeline, which is installed above ground, may be coated externally with 40 microns thick aluminum-pigmented bituminous paint on top of the 80 microns thick (bituminous) tar-epoxy coating. However, pipelines, which are installed inside or immediately outside of water treatment facilities and pumping stations, may be coated as indicated in Table 7.1 below. In this situation, pipeline can be distinguished by color by painting with several different, colored synthetic resin paints.

Table 7.1: Coating process

Coating process	Coating material	Coating thickness	Coating place
1st	Lead – type Anticorrosion paint	35 (microns)	Works
2nd	Lead – type Anticorrosion paint	30 (microns)	Works
3rd	Synthetic Resin paint	25 (microns)	Site
4th	Synthetic Resin paint	25(microns)	Site

Note: In lead-type anticorrosion paint, red-lead paint, lead sub oxide paint and lead cyanamide paint is available.

(ii) Immersed in water

Pipeline, which is installed under water, may be coated externally as follows. In this case, pipelines will not be able to be distinguished by color.

Coating process	**Coating material**	**Coating thickness**	**Coating place**
1st	Tar-epoxy paint	70 (microns)	Works
2nd	Tar-epoxy paint (M.I.O.)	50 (microns)	Works
3rd	Tar-epoxy paint	70 (microns)	Site
4th	Tar-epoxy paint	70 (microns)	Site

Note: 1: In the above table, M.I.O. means Micacious Iron Oxide
2: When pipeline is coated with colored paint, epoxy paint should be applied instead of tar-epoxy paint.

(iii) Underground in a corrosive soil

Special coatings as mentioned below are available for pipeline which is buried underground in a corrosive soil;

- 300 microns or 500 microns thick tar epoxy coating
- Zinc rich pain or metallic zinc coating seal-coated with tar-epoxy
- Polyethylene coating

(iv) Electrolytic corrosion protection

Ductile iron pipeline has inherent electrolytic corrosion resistance because the pipeline is almost insulated at the joint portion of each pipe by a rubber gasket. However, pipeline may form an electrically conductive line under certain conditions. In this situation, application of a polyethylene sleeve shows a high degree of insulation effect and protects the pipeline form electrolytic corrosion.

NOTE: The American National Standard for Polyethylene Encasement for Gray and Ductile Cast Iron piping for Water and other Liquids, ANSI A 21.5 (AWWA C 105), specifies that five properties of the soil should be investigated. These are:

- Earth receptivity
- pH value
- Redox potential
- Moisture content
- Sulphide content

Points are assigned according to the measured values of these properties, and corrosion protection by the polyethylene sleeve method is recommended if the total of these points is 10 or more.

Soil-test evaluation according to ANSI A 21.5 (AWWA C105)

Property	Measured value	Points
Resistivity (based on single probe at pipe Depth or water saturated miller soil box)	Less than 700 ohm-cm	20
	700 to 1,000 ohm-cm	8
	1,000 to 1,200 ohm-cm	5
	1,200 to 1,500 ohm-cm	2
	1,500 to 2,000 ohm-cm	1
	More than 2,000 ohm-cm	0
pH value	0 to 2 2 to 4 4 to 6.5 6.5 to 7.5 7.5 to 8.5 More than 8.5	5 3 0 0* 0 3
Redox potential	More than 100mv 50 to 100 mv 0 to 50 mv Less than 0 mv	0 3.5 4 5
Moisture content	Poor drainage and continuously wet: Fair drainage and generally moist: Good drainage and generally dry:	2 1 0
Sulphide content	Positive Trace Negative	3.5 2 0

*If sulphides are present and low or negative Redox potential results are obtained, three points shall be given to this range.

Cathodic protection:

The use of cathodic protection systems for ductile iron pipeline is not recommended for the following reasons:

- Ductile iron pipeline has a high degree of electrical resistance because of the jointing system using rubber gaskets.
- There are more effective and economical protection systems, e.g., polyethylene sleeving.

7.5 OPEN CONDUITS

7.5.1 Application

Open canals may be used for conveying raw water when economical, but should never be used for treated water.

7.5.2 Hydraulic Design

The Manning formula $V = C \times R^{2/3} \times I^{½}$
(V = Velocity, m/s ; C = Coefficient of roughness ; R = Hydraulic radius, m
I = Hydraulic gradient m/m) should be used with the following values of C.

• Planned timber, joints flush	C	= 80
• Sawn timber, joints uneven		= 70
• Concrete, trowel finished		= 80
• Masonry, neat cement plaster		= 70
• Masonry, brickwork; good finish		= 65
• Masonry, brickwork; rough		= 60
• Rock, cut smooth		= 30
• Rock, jagged		= 25

7.6 AIR-RELEASE VALVES

7.6.1 General

The number of peaks and hence the number of air valves on a pipeline should be kept to a minimum following the design rules earlier outlined in this chapter. Preventing air form entering the pipeline should reduce the amount of air. See e.g. chapter "Water Storage".

Air-release valves serve mainly thee purposes, namely

- To release air form the pipeline during the filling process (large orifice valves)
- To release air from the pipeline during the normal operation of the water supply (small orifice valves)
- To allow air to enter into the pipeline in order to prevent vacuum to occur (large orifice valves).

7.6.2 Small-orifice Air Valves

- This type of air valves should be placed at all high points relative to the horizontal on pipes with inside diameter of 80mm or larger.

- On smaller pipes air valves should be placed only at accentuate high points and then if air cannot be released through consumer connections.

- In this context it may be considered that a high point is accentuate if it is situated 10m higher than the low points preceding or succeeding it.

- The minimum orifice size with a diameter of approximately 2mm is normally adequate up to a pipe diameter of 300mm.

7.6.3 Large-orifice Air Valves

- Large orifice valves should be positioned at accentuate high points on pipelines of diameter 80mm or larger at t distance of about 1 km.

- Large-orifice valves should be placed on UPVC pipes class 0.6Mpa at points where vacuum may occur.

- Inlet diameters of 50mm are usually adequate for pipe diameters up to 400mm.

- At locations where a small-orifice and large-orifice valve coincides these should be combined to a double orifice valve.

7.6.4 Alternative Air Release

The air valves may be replaced with connections to SWPs or Kiosks or with rising branch lines. At the filling of the pipeline system washouts may serve as air release points. Manual air-release valves may replace automatic ones in special cases.

7.6.5 Isolating Valves

All air-release valves should be equipped with isolating valves for easy removal and repair of the air valves.

7.7 WASHOUTS

7.7.1 General

The number of low points and hence the number of washouts should be kept to a minimum following the design rules earlier outlined in this chapter.

7.7.2 Location

Washouts should be placed only at accentuate low points on raw water and clear water mains of inside diameter 80mm or larger.

In this context it may be considered that a low point is accentuate if the succeeding major high point is situated on a 10m higher level.

7.7.3 Washout Size

Assuming a shear stress of $10N/m^2$ on the walls of the main pipe and an available pressure of 0.1-.0.2 MPa the diameter, d, of the washout should be:

d = 0.6 D	if the upstream and the downstream sides of the main are washed simultaneously.
d = 0.4 D	if only one side is washed at a time

Where:

d is the diameter at the washout in mm

D is the diameter of the main pipe in mm

7.7.4 Washout Valves

There shall be a valve only on the washout pipe and **not** on the main pipeline unless the valve can be combined with a section valve (see below).

7.7.5 Drain

There shall be an open drain leading the water from the washout to a suitable steam or discharge point nearby.

7.8 FIRE-FIGHTING

7.8.1 General

Concerning provision for fire fighting see chapter "Water Demand".

7.8.2 Pipes

Urban areas with fire tenders do not require high main pressure and the only requirement is that there is adequate supply of water at the hydrant. Providing for piping not less than 100mm or in very high value commercialized areas 150mm normally ensures this. Further see "Section Valves".

7.8.3 Hydrants

In areas with high fire risk, town centers, industrial areas, etc., the distance between fire hydrants shall be 65-100m, and in residential areas 150-200m.

7.9 SECTION VALVES

7.9.1 Location

- Section valves on gravity mains (>80mm) should be located a distance of between 2 and 3km for rural areas and about 0.5km for urban areas. All branch lines should have valves at the connection. Pumping mains must not have any section valves outside the pump house.

- The valves should be placed in such a way that rationing of water can be done by closing suitable parts of the supply for certain periods.

- Whenever possible the section valves should be placed in a joint valve chamber with air valves or washouts and **upstream** of these valves.

- In urban areas where fire-fighting is provided for there should be an isolating valve downstream each fire hydrant.

7.10 BREAK PRESSURE TANKS

7.10.1 General

Break-pressure tanks should be used to keep the pressures within the limits stated under section "Pressure" of this chapter and to make it possible to use lower pipe classes and hence minimize the cost of the pipeline.

Break-pressure tanks should be combined with the balancing tanks whenever feasible.

7.10.2 Size

The volume of the break-pressure tank should be large enough to give a retention period of minimum 2 minutes.

7.10.3 Design Details

Break-pressure tanks should:

- Be covered and have lockable manhole cover.

- Have an inlet pipe which ends near the floor to prevent air entrainment by falling jet.

- Have an overflow placed at least 50mnm above the normal top water level and which allows the overflowing water to be seen when in operation.

- Be designed so that the ball valve is easily accessible from the manhole but does not block the same.

- Have a valve on the inlet pipe.

7.11 MARKER POSTS

7.11.1 Location

Marker posts shall be provided along pipelines at every 200m, except where they follow permanent roads.

Markers should be placed at all bends, river and road crossings which cannot be easily found otherwise.

7.11.2 Type

The marker should be square 100 x 100mm, height 700mm lettered MAJI. The post should be blue with white lettering.

7.12 VALVE CHAMBERS

7.12.1 Dimensions and Design

Valve chambers should be at lease 1000 x 1000mm internally. There must not be UPVC-pipes within the chamber.

The cover should be lockable. The chamber should be drained through the floor or through a drain pipe.

7.13 ANCHOR AND THRUST BLOCKS

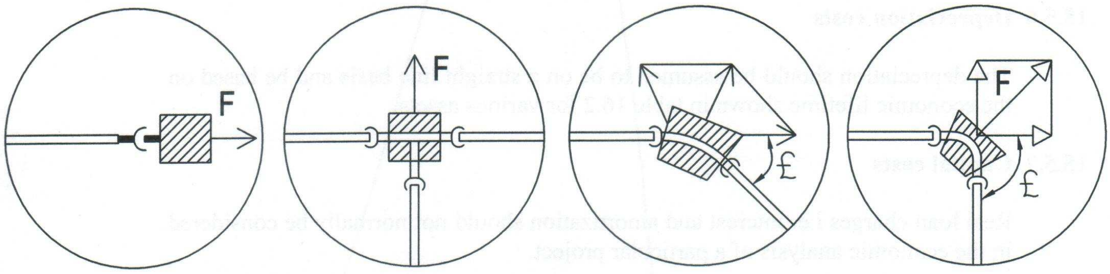
7.13.1 Location

Anchor or thrust blocks shall be provided for horizontal and vertical bends, capped ends, change of size and tees and for pipes laid in steep slopes (>1:6).

7.13.2 Thrust Forces on Pipes

The following table is a guide to be used when designing thrust blocks for pipe systems. The table has been calculated for PVC pipe dimensions, but can also be used for other pipes. The table shows thrust forces for pipelines with an internal pressure of 1.0 MPa.

Fig. 7.4 Thrust Blocks



The anchor and thrust blocks should be designed for the highest pressure that may occur in the pipeline. The highest pressure usually occurs during the pressure test when e.g. UPVC pipes are tested 1.5 times the nominal working pressure. Anchor or thrust blocks are generally not required when F is lower than 0.05 x d kN where d is the outside diameter in mm.

Table showing axial and resultant forces F kN at 1.0 MPa internal pressure.							
Outside Nominal Diameter mm	F = Axial Force	F = Resultant Force on bends and angles £ = kN					
		$11\ ^1/_4{}^0$	$22\ ^1/_2{}^0$	30^0	45^0	60^0	90^0
20 32	0.31 0.80	0.06 0.16	0.12 0.31	0.16 0.41	0.24 0.61	0.31 0.80	0.44 1.13
50 63	1.96 3.12	0.38 0.61	0.76 1.22	1.01 1.62	1.50 2.39	1.96 3.12	2.77 4.41
90 110	6.36 9.50	1.25 1.86	2.48 3.71	3.29 4.92	4.87 7.27	6.36 9.50	8.99 13.44
160 225	20.11 39.76	3.94 7.79	7.85 15.51	10.41 20.58	15.39 30.43	20.11 39.76	28.44 56.23
280 315	61.58 77.93	12.07 15.28	24.03 30.41	31.88 40.34	47.13 59.65	61.58 77.93	87.09 110.21

Example: Calculate the thrust force on a 45^0 bend of UPVC 160/6.3 – 0.9
Highest pressure 0.9 x 1.5 = 1.35 MPa occurs during the testing of the pipeline.
Hence, thrust force = 1.35 x 15.39 = 20.78 kN.

8 DISTRIBUTION POINTS

8.1 INDIVIDUAL CONNECTION (IC)

8.1.1 General

Individual connection lines to schools, hospitals, health centers and dispensaries should be included in the main design. Other connections will be designed and laid by the local water administration as the need arises.

8.1.2 Meters

It shall be assumed that all individual connections will be metered. Meters for the first 3 years after the commissioning of a supply shall be included in the bills of quantities. "It is highly recommended that all distribution main lines are fitted with zonal meters to monitor eventual losses."

8.2 COMMUNAL WATER POINTS (CWP) AND KIOSKS

8.2.1 Siting in Rural Areas

- The water points should be sited so that the maximum walking distance for 90% of the water users will be approximately 0.5km, 1km and 1.5km in high, medium and low potential areas respectively. However the number of water users per water point should be in the range of 200-500 which should be achieved by adjusting the walking distances if necessary.

- The water points should be placed on high ground to facilitate the drainage of spilt water and to make the point serve as an air-outlet from peaks in the distribution pipe.

- The positioning of the water points should be made in co-operation with the beneficiaries and the chiefs from the areas.

8.2.2 Siting in Urban Areas

The maximum walking distance in low class housing areas should be approximately 100m and the number of users per water point should be between 100 and 480. However the local water collection habits, the number of IC in the area etc. should always be considered before siting the water points.

8.2.3 General Design Details

- Standard and type drawing of CWP and Kiosks should be used, when available in the Ministry.

- Each water point should be installed with a stopcock in a valve chamber near the water point. There should also be preparations for the installation of the water meter in the valve chamber. The piping within the valve chamber and between the chamber and the taps should be made of galvanized steel.

- There should be proper drainage from the water point. If the terrain is too flat to allow natural drainage away form the point, a soak-pit or a soak-away trench should be made.

8.2.4 Number of Taps

- A ½ inch tap typically delivers 800 1/h and a ¾ inch tap 1500 1/h at a pressure of 0.1 MPa (10 mhw). For other pressures the rate of delivery can be calculated with the formula:

$$q\ act \;=\; q\ nom \sqrt{(10H_{tap})}$$

Where

q act = the actual delivery rate
q nom = the nominal delivery rate at a pressure just before the tap at 0.1 MPa (10 mhw)
H_{tap} = the available pressure just before the tap in MPa.

- The headloss through water meter can be calculated with the following formula:

$$h_{wm} = 0.1 \left(\frac{q\ act}{q\ nom} \right)^2 \quad MPa$$

Where:-

h_{wm} is the head losses through the meter in MPa
q act = the actual flow in m^3/h.
q nom = the nominal rating of the water meter in m^3/h at 0.1 MPa (ratings e.g. 3 and 5m^3/h).

The calculated discharge capacity of the water point should preferably lie in the range of 50 – 80% of the nominal capacity of the water meter.

- For available pressures at the tap of 0.05 – 0.5 MPa (5 – 50mhw) will the maximum delivery capacity in 12 hours be 5-17m^3 per ½ inch tap. (If an efficiency factor of 0.8 is assumed).

8.3 WELLS (POINT DISTRIBUTION)

8.3.1 General

Regarding the design of the well and the pumping arrangements see the chapter “Water Sources”, “Intake Structures” and “Pumps and power sources”.

8.3.2 Siting

- Besides the siting criteria, which consider the geohydrological factors the number of people to be served, and the walking distance should be similar to the criteria given for communal water points.

- For rough estimates it may be assumed that the sustained discharge rate from a shallow well is about 20 1/min. for one-hand pump.

8.4 CATTLE TROUGHS

8.4.1 General

Cattle troughs are not generally to be provided as it is assumed that water normally is carried from the IC, CWP or kiosks to the cattle. However in areas where there are no or inadequate separate water sources for cattle owned by NC users then water should be provided.

8.4.2 Siting

The siting should be determined in cooperation with the district water officer, the district livestock officer and the local chiefs.

8.4.3 Design

The type drawings of the Ministry should be used

8.5 FIRE HYDRANTS

8.5.1 General

See chapters “Water demand” and “Transmission and Distribution lines”.

9. WATER STORAGE

9.1 GENERAL

9.1.1 Purpose

The purpose of storing water is mainly fourfold, namely:

i) Raw water storage to ensure adequate supply of water during dry periods. This is dealt with in the chapters “Water Sources” and “Intake Structures”.

ii) Treatment e.g. sedimentation and disinfection. This is described in the chapter “Water Treatment”.

iii) Balancing of the variation in the water consumption during the day.

iv) Emergency storing to ensure the supply of water during break-downs or for fire-fighting.

Point 3 and 4 will be dealt with in this chapter.

9.2 RURAL AREAS

9.2.1 Balancing Storage

Balancing tanks shall be provided in order to reduce the peak flows in the transmission and distribution lines as shown in figure No. 9.1 in the chapter “Transmission and Distribution Lines”. The number and location of the tanks’ should be decided after an economic analysis aiming at minimizing the cost of the whole system of tanks, pipelines and pumping stations. The required storage capacity shall be calculated using the water demand pattern as given in the chapter “Water Demand”.

Generally the tank for the balancing of the daily peak demands will have a capacity of 50% of the daily water demand of the area served by the tank. See Figure 9.1. Hence tank 1 shall have a capacity of 50% of the daily water demand of area E, tank 2 shall have a capacity of 50% of the water demand of area C+ area D etc. Where water is pumped for less than 24h a day, the capacity of the receiving tank will have to be calculated by means of a mass diagram.

Example: Gravity system with 4 tanks. See Figure 9.1.

Area	Daily water demand m^3	Tank	Capacity m^3
A	200	3	100
B	200	4	100
C	150 }	2	150
D	150 }		
E	400	1	200
	TOTAL 1100		**TOTAL 550**

It is often economical to phase the balancing tanks as the required balancing capacity is generally low during the first few years after a supply has been taken into operation. The need for balancing shall therefore be analyzed for different years.

9.2.2 Emergency Storage

Generally not required in rural areas inclusive rural, market and local centers except for institutions or industry which may provide their own emergency storage to safeguard against interruption of the supply.   However, wind-powered supplies should have 3 days storage to provide for calm periods and supplies based on a simple borehole should have a storage capacity of one day's supply in addition to what is required for balancing.

9.3 URBAN AREAS

9.3.1 Balancing Storage

The same general principles as for rural areas supply.

9.3.2 Emergency Storage

Principle Towns and Urban Centres should have the following storage capacities in addition to the requirements for balancing.

Reservoir served by:	**Number of hours supply for breakdown and emergencies**
Gravity	12
Pumping	18
More than one independent system	8

The emergency storage should be placed as near the consumers as possible and without pumping between those and the reservoir.

Large consumers i.e. certain types of industry and consumers which depend very much on a steady water supply e.g. hospitals and schools should be encouraged to provide their own emergency storage.

9.4 RAW WATER RESERVOIRS

9.4.1 Raw Water Balancing Tanks

Except when storage is required as pre-treatment, raw water balancing tanks before a treatment works should not be provided. The intake of raw water to the works should be controlled through careful selection of pumps and by adjusting the number of pumping hours a day to fit the needs.

9.5 TANK DESIGN

9.5.1 Capacities

The standard capacities of the Ministry should be used. They are 10, 25, 50, 100, 150, 200, 300, 500, 800 and 1200 m^3. Larger tanks may have capacities as required.

9.5.2 Design Details

Tanks should:

- be covered and have a lockable manhole cover, universal type.
- be equipped with internal and external ladder or steps.
- have a level indicator which can be read from outside.
- have inlet pipe which ends not more than 0.5m above the floor to prevent air entrainment.
- have an outlet at a level at least 0.2m above the floor.
- have a scour pipe which allows complete emptying.
- have an overflow placed at least 50mm above the normal top water level which allows the overflowing water to be seen when in operation.
- be designed so that the ball valve (if any) is above the highest water level and is easily accessible from the manhole.
- have ventilation pipes covered with nylon nets.
- have outside walkway and handrail (only elevated steel tanks).
- not usually have any partitioning.
- not have a ball valve on the inlet pipe when a pumping main feeds it.

10. PUMPS AND POWER SOURCES

10.1 PUMP SELECTION

10.1.1 Pumps Commonly Used

	TYPE OF PUMP	CHARACTERISTICS AND APPLICABILITY
1.	RECIPROCATING (Plunger) a. Suction (Shallow well) b. Lift (deep well)	Low speed of operation; hand wind or motor powered; efficiency low (range 25-60%) Capacity range: 10-50 1/min. suitable to pump against variable heads; valves and cup seal require maintenance attention.
2.	ROTARY Helical rotor (mono)	Low speed of operation; hand animal, wind powered; Capacity range: 5-30 1/min. discharge constant under variable heads. Using gearing: head, wind or motor powered good efficiency; best suited to low capacity-high lift pumping.
3.	AXIAL-FLOW	High capacity-low lift pumping; can pump water containing sand or silt.
4.	CENTRIFUGAL	High speed of operation-smooth, even discharge; efficiency (range 30-85%) depends on operating speed and pumping head). Require skilled maintenance; not suitable for hand operation; powered by engine or electric motor 25-10,000 1/min.
5.	HYDRAULIC RAM	No external source of power required' utilizes head difference between source and pump; very little maintenance required; water is wasted as it is also used for driving. Delivery head up to 125m 1-200 1/min.
6.	JET PUMP	10-800 l / min relatively low efficiency increases the suction depth of a centrifugal pump up to 75m thus allowing the pump to be set on the ground suitable for sandy water as sand can be removed before entering the pump.

Table 10.1: Pumps commonly used.

10.2 POWER REQUIREMENT

The power required, N, for driving a pumping unit can be calculated with the following formula:

$$N = \left(\frac{Q \times H}{102 \times e}\right) \text{ kW}$$

Where:

Q = Flow in 1/s
H = Pumping head in metre (static head + losses)
e = pumping efficiency (will have a value between 0 and 1)

The energy demand can be calculated with the following formula:

$$E = \left(\frac{Q \times H}{e}\right) \text{ kWh per year}$$

Where:

Q = pumped quantity of water per day, m^3/day
H and e = as above,

In practice the efficiency of small-capacity pumps in particular is low. It can be assumed that the efficiency is in the range of 30% for a 0.4 kW pump and 60% for a 4 kW or bigger pump.

10.3 SUCTION HEAD

10.3.1 General

Negative head, i.e. when the pump is placed above the water level at the intake should be avoided where it is possible to place the pump beneath the water level without excessive additional building costs.

10.3.2 Practical Suction Head

The maximal practical suction head depends on mainly the altitude, the temperature, the intake arrangement and the pump design. For preliminary design the following maximum suction heads should not be exceeded.

Altitude above mean sea level m	Practical Suction Head m
0	5
500	4.5
1000	4
1500	3.5
2000	3
2500	2.5
3000	2.0

10.3.3 Net Positive Suction Head (NPSH)

The final design of the pump installation and the selection of the pump should be based on the concept of NPSH. See ISO 2548 for detailed information.

For each pump the required net positive suction head (NPSH req) to allow proper functioning of the pump can be determined. The NPSH required depends on the pump design and the flow through the pump. The **lower** the NPSH required the better the suction ability of the pump. The NPSH required increases usually rapidly with increased flow.

The NPSH required curve should be obtained from the pump manufacturer.

The require net positive suction head can be calculated with the following simplified formula.

$$NPSH_{req} \leq B + H_{sta} - H_f$$

Where

H_{sta} is the static height difference in metre between the center line of the pump cylinder and the water level in the intake chamber.

H_{sta} has a negative value if the pump is placed above the water level of the intake and a positive value if the pump is placed under the water level.

H_f is the head losses in metre in the foot valve, suction pipe etc. between the intake point and the pump.

B depends on the altitude as shown in the table below:

Altitude above mean seal level m	B m
0	9.4
500	8.9
1000	8.4
1500	7.9
2000	7.3
2500	6.8
3000	6.3

The approximate formula is applicable for Kenyan conditions with a water temperature up to 30^0C. The formula takes into consideration that the performance of a pump deteriorates somewhat because of wear. An increase of NPSH of the pump of 0.5m has thus been allowed for. Further it has been assumed that the water is pumped from an open chamber and that the velocity in the intake chamber is negligible.

When the NPSH of the pump is known the maximum static head can be calculated with the following formula:

$$H_{sta} \geq NPSH_{pump} + H_f - B$$

A negative value of H_{sta} indicates that the pump can be placed above the water level while a positive value shows that the pump must be placed under the water level.

Practical Examples.

A pump placed 3m above the water level in the river shall pump 40 1/s. The suction pipe of diameter 175mm is 8m long and has one 90^0 bend. There is a strainer with footvalve at the end of the pipe. The altitude is 1500m.

Question: What should be the NPSH of the pump?

Solution: H_{sta} = -3m

H_f =	Strainer with footvalve loss	0.47m
	1 bend 90^0	0.08m
	8m pipeline	0.20m
		0.75m

B = 7.9m

$NPSH_{req} \leq B+H_{sta} - H_f = 7.9 - 3 - 0.75 = 4.15$m

Hence, select a pump, which have a NPSH of 4.15m or **less** for the capacity of 40 1/s.

Question: What would the required NPSH be if the pump in the example above instead were placed 3m below the water level in the river?

Solution: $NPSH_{req} \leq 7.9 + 3 - 0.75 = 10.10$m

Hence, select a pump, which has a NPSH of 10.15m or **less** for the capacity of 40 1/s.

Question: What is the maximum height a pump with a NPSH of 5m can be placed above the water level assuming the same pipe installation as above but an altitude of 2000m?

Solution: H_f = 0.75m

B = 7.3m

$H_{sta} \geq NPSH_{pump} + H_f - B = 5+0.75 - 7.3 = -1.55$

Hence, the pump can be placed a maximum of 1.55m above the water level.

10.4 PUMP PARTICULARS

10.4.1 Hand Pumps

The maximum head for comfortable operation of a deep well hand pump of plunger type is shown in the table below. It has been assumed that the maximum handle force is approx. 20kg and the mechanical advantage 4 to 1.

Cylinder Diameter mm	Head (Lift) m
50	Up to 25
65	Up to 20
75	Up to 15
100	Up to 10

The power available from human muscle depends on the individual, the environment and the duration of the task. The long term (8h) power is often estimated to 60 to 75 watts. The short term (5-10min) power is around 200 watts. These figures are for a healthy young man. Many users operate most hand pumps used for domestic water each pumping only a few minutes. The operators are often women and children rather than men. Assuming a power output of 75 watts the **maximum** flow can be roughly estimated with the formula:

$$Q = \left(\frac{460 \text{ x } e}{H} \right) 1/\text{min}$$

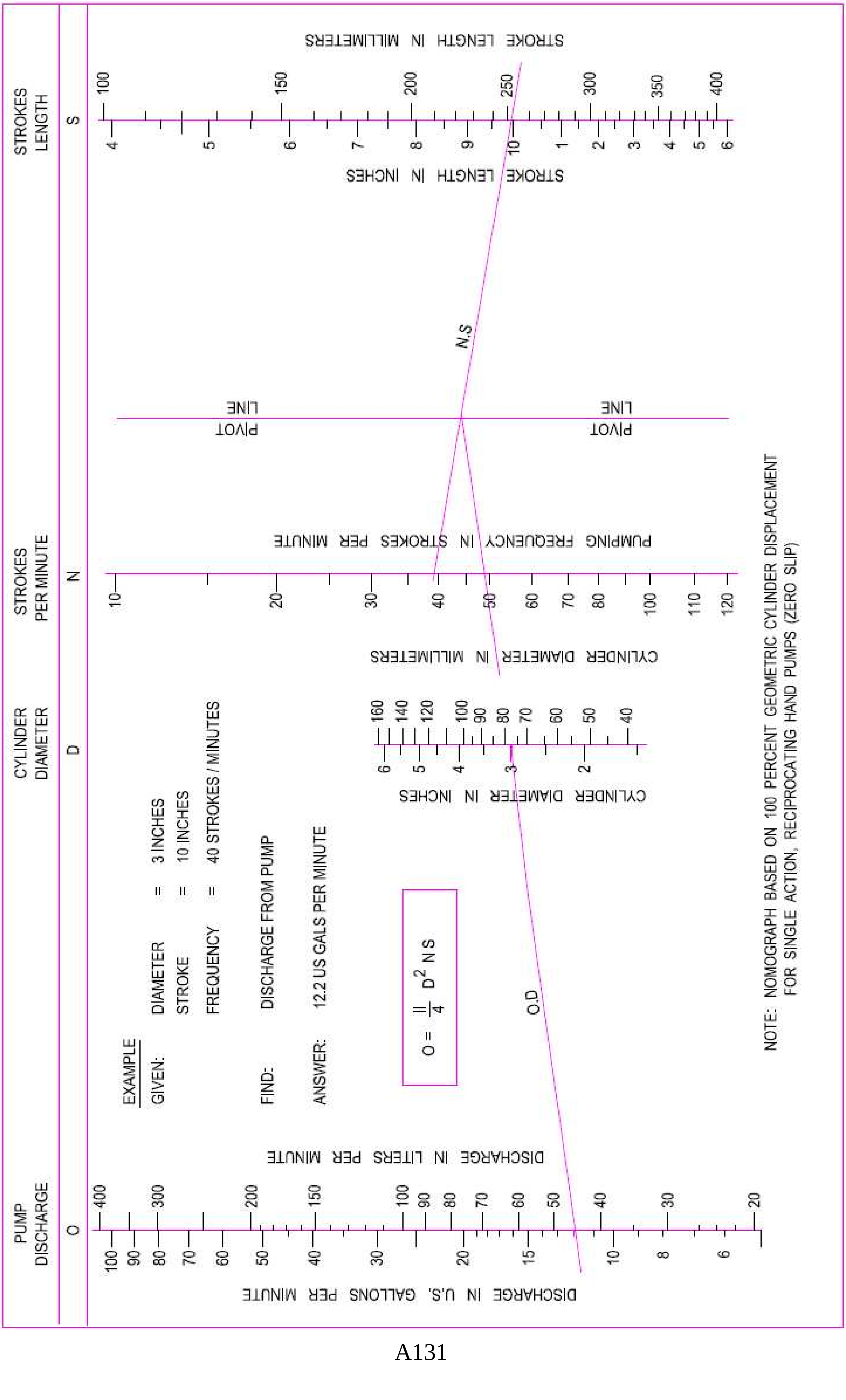
Where
H = head of water, metres
e = pumping efficiency (will have a value between 0 and 1)

The dependence of flow on the stroke length, stroke frequency and cylinder diameter can be seen in Figure No. 10.1.

As can be seen a pump capacity of about 30 1/min can be achieved with normal stroke frequency and length with a 65mm cylinder.

The stroke length and the practical stroke frequency will often limit the practical capacity of the pump as compared to the theoretical capacity if only the power input is considered.

Fig. 10.1 Nomograph for hand pump Discharge



10.4.2 Centrifugal Pumps

- Low speed centrifugal pumps can be expected to wear less and last longer than high speed pumps. Speeds of up to 1500 rpm should be chosen for raw water pumps. However, for raw water pumps with opening exceeding 20mm speeds up to 2900 rpm may be considered if economically justified. Clear water pumps are less likely than raw water pumps to wear fast at high speeds. However, as low speed as is economically feasible should be chosen.

- The efficiency should always be maximized by choosing a pump which will operate near the maximum of the efficiency curve.

- In order to determine the operating point the head losses should be calculated as realistically as possible without the safety margin which is normally built into the calculations for the purpose of selecting the pipe dimensions. Further, see under "Transmission and Distribution Lines". The less steep the pump characteristic is the more will the actual capacity of the pump deviate from the wanted capacity if the head is wrongly calculated. For this reason a pump with a steep characteristic is preferred.

  The friction losses must never exceed the static head as the operating point then will be very difficult to determine correctly.

- Belt driven pumps should be chosen whenever possible. These allow easier operation and maintenance than direct-driven pumps and allow easy modifications of the pump capacity by changing the drive wheel.
  The capacity of a centrifugal pump is directly proportional to the speed but the pumping head varies with the square of the speed.

- The pump manufacturer should be consulted before the final choice of pump is made and the size of the engine or motor chosen.

10.4.3 Submersible Pumps

- Submersible pumps in boreholes shall be equipped with a device to prevent the pump from running dry.

- The pumps should have a steel-wire safety line connected to the top of the borehole in order to make recovery possible in case of breakage of the discharge pipe or faulty handling.

- The pump diameter must be chosen so that the water velocity between the pump and the borehole casing does not exceed 5 m/s.

- Shaft driven pumps down to about 80m and jet pumps to approximately the same depth can often be viable alternative.

10.4.4 Hydraulic Rams

- The hydraulic ram usually has best efficiency when the delivery head is 3-4 times the drive head. The efficiency is normally 40-60% and maximum 70-80% for carefully manufactured pumps.

- The ram should normally be placed so that the delivery head is 3-10 times the drive head. However pumping heads up to 40 times the drive head are possible.

- The drive pipe should be made of steel and have a length of at least 4 times the working fall. The ratio pipe diameter/length of drive pipe should be 1:150 – 1:1000, ideally 1:5000.

- The drive water intake must be protected to prevent debris and sand from entering the pipe. If the water carries silt and sand the water should be drawn from a feed tank which allows settling of the coarse particles.

- The amount of drive water, can be roughly calculated with following formula:

$$Q_{drive} = \left(\frac{2 x h_{del}}{h_{drive}} \right) Q_{del}$$

where

Q_{drive} and Q_{del} are the drive and delivery flows respectively

h_{drive} and h_{del} are the driven and deliver heads respectively

10.5 POWER SOURCES

10.5.1 Diesel Engines

- The effect of altitude and temperature on the power output of the engine must be considered. The decrease in power can be assumed to be 1% for every 100m rise of altitude above mean sea level. The power will also decrease with about 2% for every 5^0C that the air inlet temperature rises above 30^0C. The humidity may also effect the power output however only slightly.

- Diesel-driven pumps should be equipped with diesel engines which can give 25-30% more power than is required under normal conditions.

- Diesel-driven generators should be equipped with engines which can give 100% more power than is required under normal conditions if the generator is used for running only one motor-driven pump. If more than one pump then 25-30% extra power is adequate.

- The diesel engine should run at 1500-2000 rpm under normal operation.

- The engines should be able to start by hand whenever possible.

- Water cooling is preferred for engines over approximately 20 kW.

- The selection of a diesel engine should always be made after consultations with the manufacturers.

- The fuel consumption can be estimated to 0.25-0.35 litre per kWh output for a load of over 50% of the rated engine size.

10.5.2 Generators

- Where there is no electrical power supply in the water supply area it may become necessary to install a generator, in particular for large and complicated treatment works. However direct diesel driven pumps are generally preferred because of easier operation and maintenance.

- Regarding engines see above under Diesel Engines.

- Frequency indicator should always be installed so as to control the frequency of the AC power.

- The selection of a generator should always be made after consultations with the manufacturers.

10.5.3 Electrical Motors

- The efficiency of three-phase motors under full load are:
  Approximately 70% for a 1kW motor
  Approximately 89% for a 2kW motor
  Approximately 85% for a 10kW motor
  Approximately 90% for a 50kW and larger motor

- It is not advisable to choose a motor smaller than 0.25 kW even if the power requirement is smaller.

- In order to avoid overloading of the electric motor the rated effect of the motor should exceed the effect calculated for the pump by the following percentages:

  Approximately 50% for pumps requiring up to 1.5 kW
  Approximately 30% for pumps requiring between 1.5 and 4 kW
  Approximately 20% for pumps requiring between 4 and 8 kW
  Approximately 15% for pumps requiring between 8 and 15 kW
  Approximately 10% for pumps requiring over 15 kW

10.5.4 Wind Power

- The use of wind power should be feasible if:-
  Winds of at least 2.5 – 3 m/s are present 60% or more of the time.
  The water source can be pumped continuously without excessive drawdown
  Storage is provided, typically for at least 3 days demand, to provide for calm periods.
  A clear sweep of wind to the windmill is secured, i.e. the windmill is placed above surrounding obstructions, such as trees or buildings within 125m. Preferably the windmill should be set on a tower 4.5 – 6m high.

  Windmill equipment is available that can operate relatively unattended for long periods of time, six months or more, is available. The driving mechanism should be covered and provided with an automatic lubrication system. Vanes and sail assemblies should be protected against weathering.

- The windmill should be equipped with a pull-out system to automatically turn wheel out of excessive wind stronger than 13-15 m/s.

- Direct pumping of water by a windmill requires matching of the characteristics of (1) the local wind regime, (2) the windmill, and (3) the pump. The manufacturers should always be consulted regarding the selection of equipment.

- The discharge Q can be estimated with the following formula:-

  $$Q = \left(\frac{2.8 \times D^2 \times V^3 \times e}{H} \right) \text{ 1/min}$$

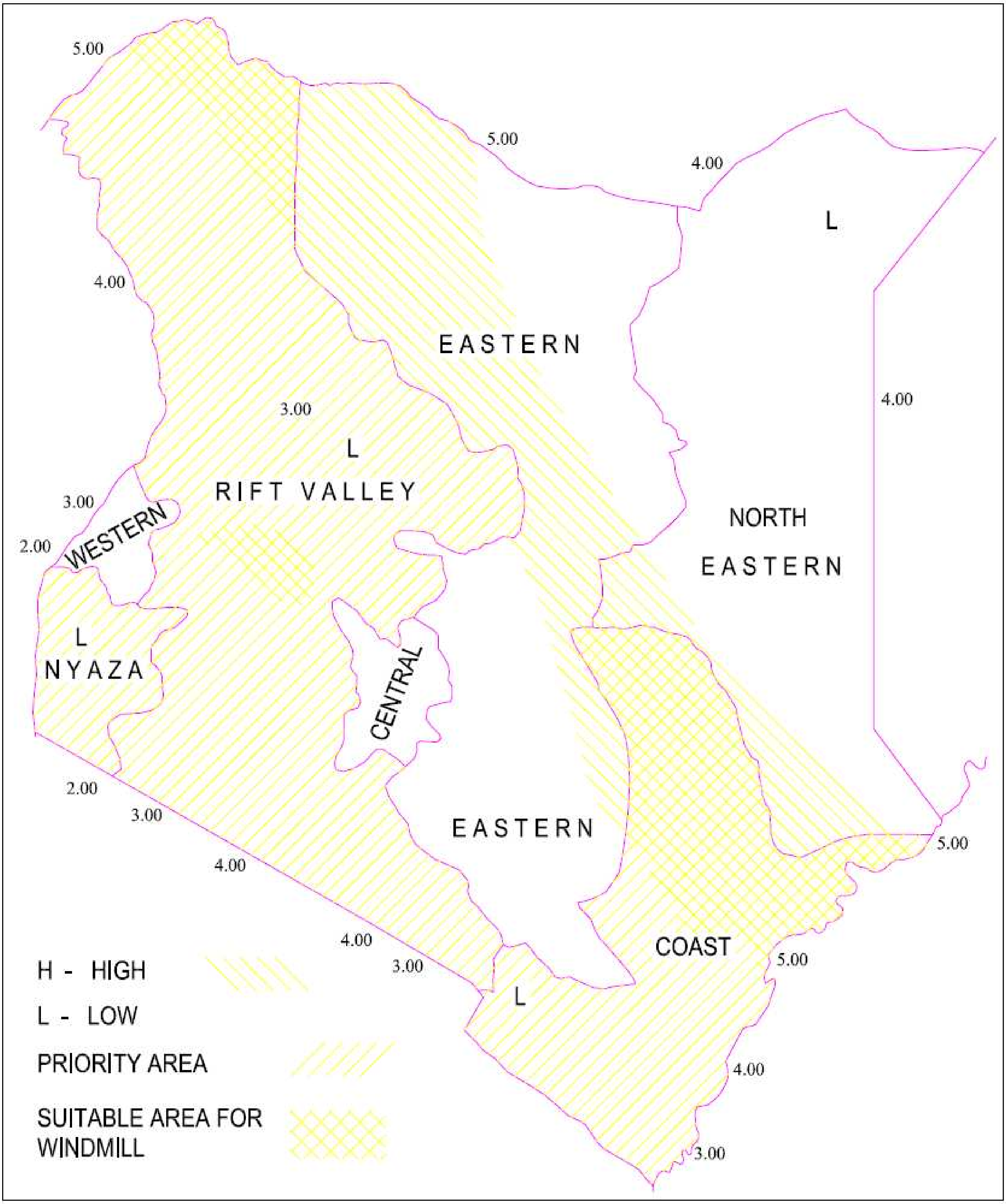
  Where
  D is wind rotor diameter in metres
  V is wind velocity in metre per second
  H is pumping head in metre
  e is the wind to water mechanical efficiency, (value 0-1)

  Windmills with rotor diameter between approximately 2m to 6m are usually available. The efficiency, e, will rarely exceed 30%.

- A map showing the average wind velocities in different areas in Kenya can be found in Fig. No. 10.2. Although the average velocities, are not the same as the 60% duration velocities, which is the criterion, the map can still given some indications about the feasibility of windmills in different areas of Kenya.

- A windmill intended for the driving of a pump is not easily converted to driving an electrical generator as the required torque and revolutions are different.

Figure No. 10.2: Annual wind speed in metres per second



10.5.5 Solar Power

- The available solar radiation in Kenya can be seen in Fig. No. 10.3 and Fig. No. 10.4 which show the average radiation in a year and the radiation during the worst month which is July for almost all parts of Kenya.

  Note that the radiation is given in W/m^2 and that the available energy in a day can be calculated by multiplication with 24h.

Example: Average available energy in July in Mombasa is 24 x 186 = 3.464 kWh/m^2 per day.

Figure No. 10.3 and Fig. No. 10.4 below give the average radiation per year and month respectively. However, in order to design a system we need to know the radiation during a shorter period e.g. a day. The 90% probability radiation, I_D, as a function of the average radiation in the worst month, I_{WM}, and the period C_L, can be obtained form Fig. No. 10.5.

Example: The radiation in Mombasa in the worst month as calculated above is I_{WM} = 3.464 kWh/m^2 – day. The probable radiation for 1 day, i.e. C_L = I, will then be approximately I_D = 2.0 kWh/m^2 – day. I_D for C_L = 7 days will be approximately 2.9.

A photovoltaic module is rated in peak watts, Wp, or peak kilo watts, kWp, which is the highest effect the module can give under special conditions under laboratory testing. Wp must not be confused with the effect the module is able to give under field conditions. The required power, rated in peak watts, can be calculated with the following formula:

$$P_T = \left(\frac{1.16 \text{ x } L}{I_D} \right) \text{ kWp}$$

Where P_T is the total power of the module at the operating temperature which in this case has been assumed to 60^0C.

L is the average daily load demand (energy demand) during the month under consideration in units of kWh/day.

I_D is the 90% probability average radiation during the period in question,
C_L, in units of kWh/m^2 – day and can be found in Fib. No. 10.5

Example: Assume that the energy **demand** (Not output) for a water pump is calculated to 4 kWh/day in Mombasa. The pump is for DC and there is enough storage to balance the variations of solar power during a period of 7 days. The previous example gave I_D = 2.9 kWh/m^2 – day.

$$P_T = \left(\frac{1.16 \times 4}{2.9} \right) = 1.6 \text{ kWp}$$

Hence, the photovoltaic array (an array consists of several modules) should have a rating of 1.6 kWp.

- The area of the solar array can be calculated with the formula:

  $$A = \left(\frac{L}{I_D \times e} \right) m^2$$

  Where L and I_D are as defined above and e is the efficiency (see below).

- There must be enough storage to balance the variations of the pumping capacity (due to variations of the solar radiation) within a day and also between different days. The variations within the day require a storage capacity equal to 24h supply, The balancing volume necessary to take care of the variation between different days should be estimated by means of a mass diagram taking into account the probable power output (Fig. No. 10.5) and the water demand.

- The practical efficiency of an array with cells of crystal silicon is about 8% with contemporary technology. The efficiency of the single cell is higher.

- The following points should be kept in mind when designing a solar pump system:

  The solar panels are prone to vandalism and should be protected or placed in such a way that the risk is minimized.

  The cheaper way to store energy from the solar system is by water storing. If batteries are used, there will be additional losses besides the system being more complicated and thus less reliable.

  Conversion of DC to AC power entails energy losses

  The efficiency of the pump and the motor is usually very low for the small units which are used in conjunction with solar power, often in the range of 25-50%. The total efficiency of the system (solar module, motor and pump) hence will be in the range of 2-4%.

Figure No. 10.3: Mean total 24 hour distribution of radiation (W/m^2) over Kenya per year (averaged over ten years or as shown in brackets)

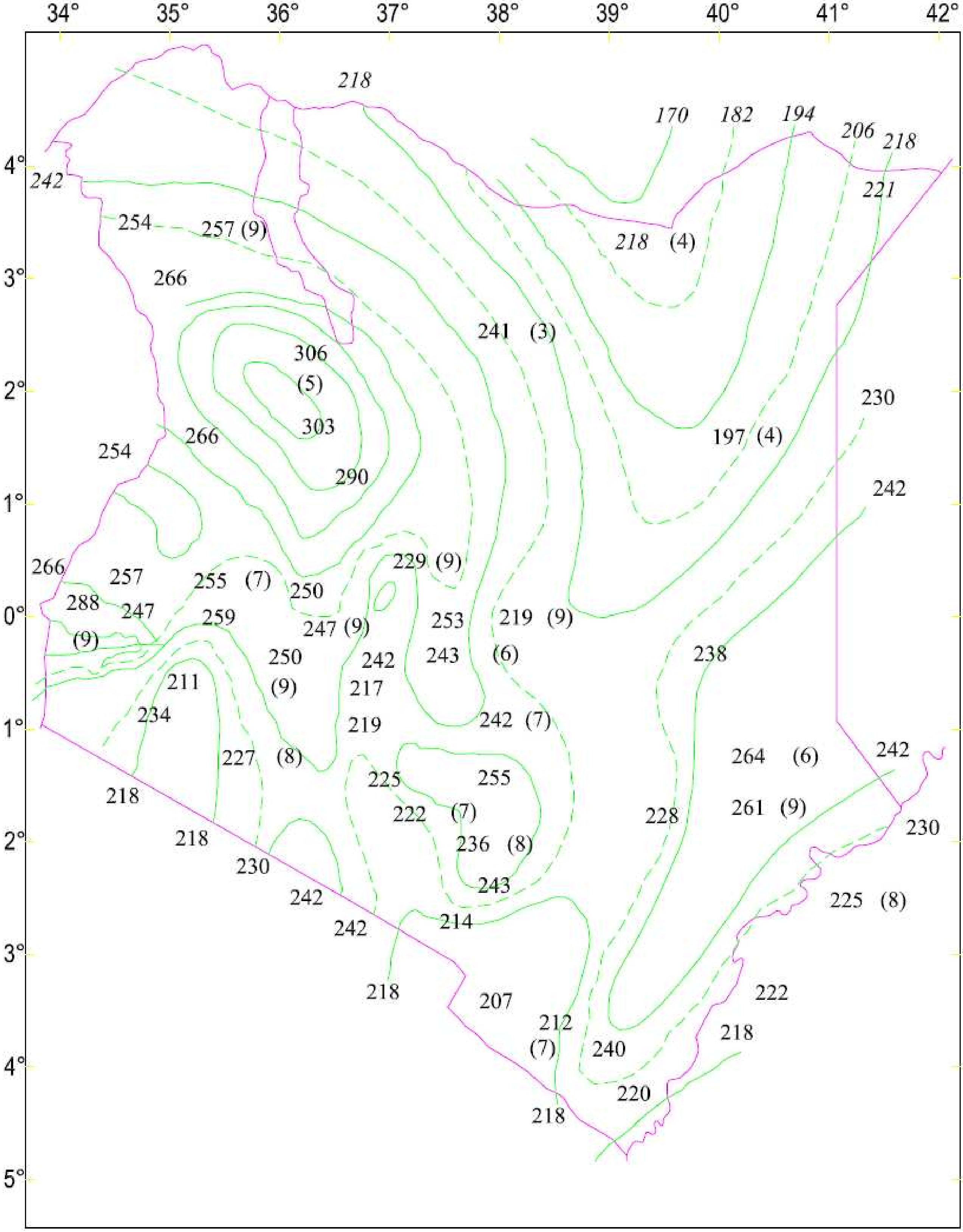


Figure No. 10.4: Mean total 24-hour distribution of radiation (W/m²) over Kenya in July (averaged over ten years or as shown in brackets)

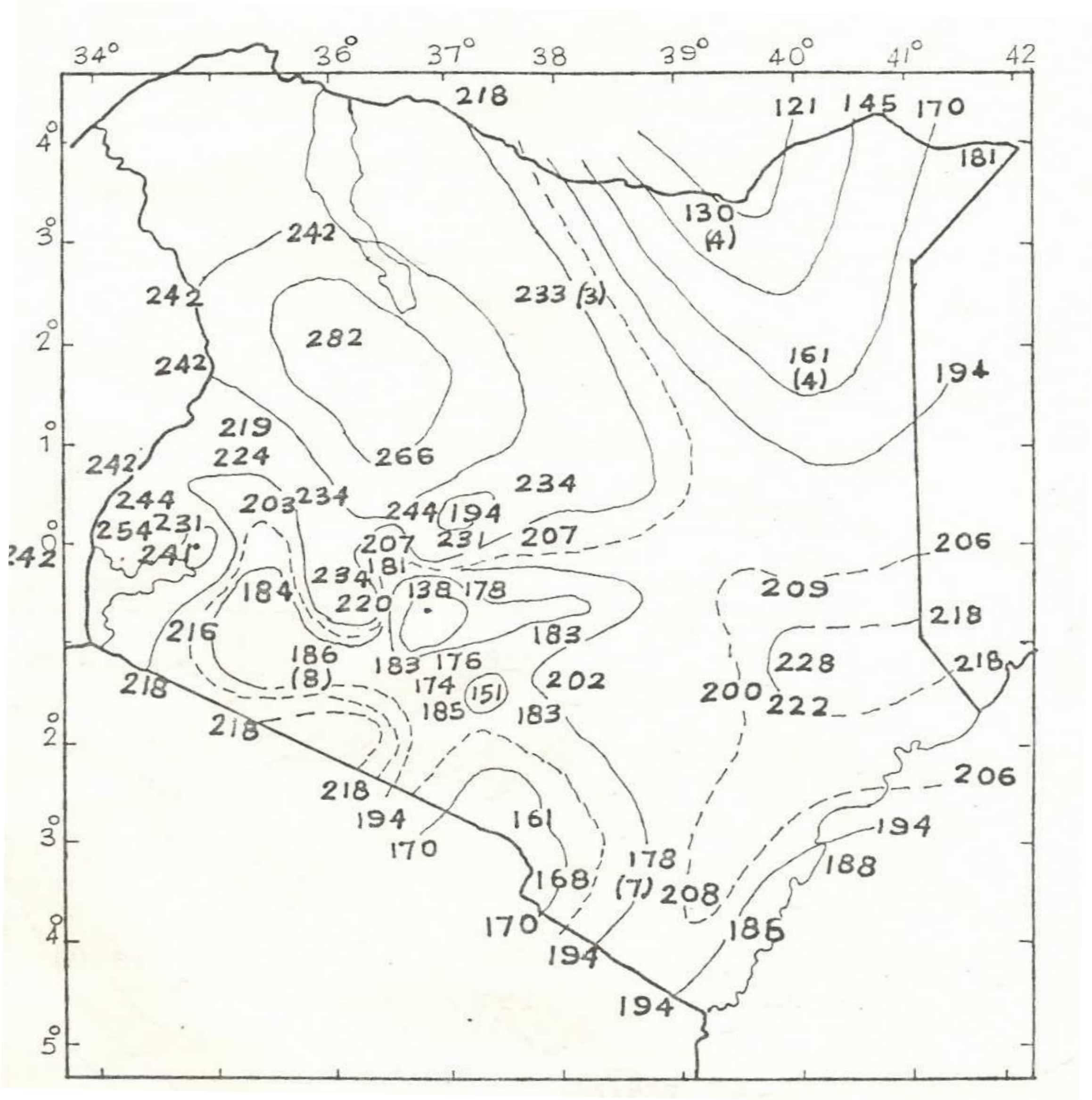
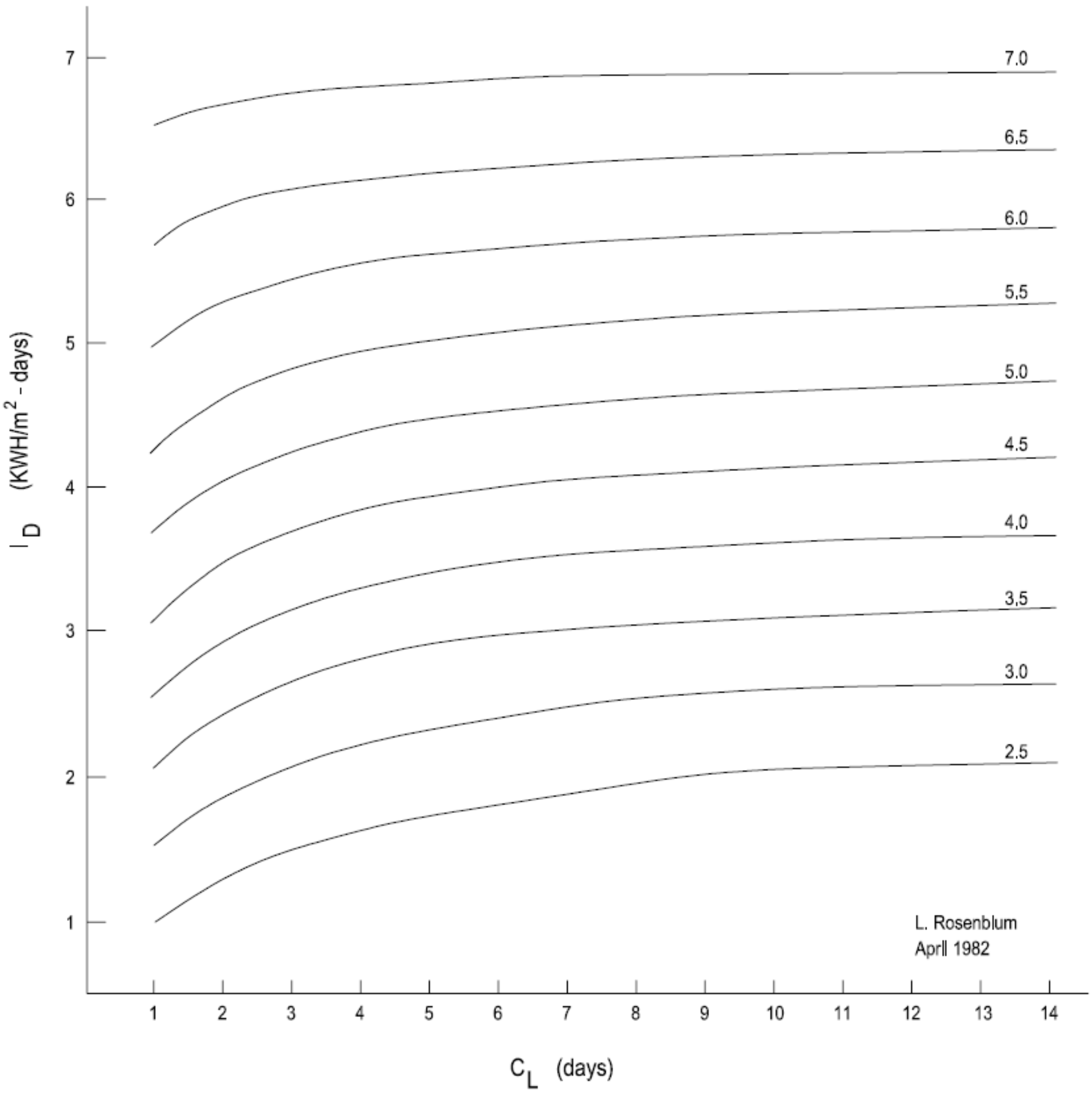


Figure No. 10.5: The 90% probability radiation, I_D, as a function of the average radiation in the worst month, I_{WM}, and the period C_L.



10.6 PUMP AND PIPE INSTALLATIONS IN THE PUMP HOUSE

10.6.1 Pump Installation and Pump House

- Pumping stations with delivery heads exceeding 150m should be avoided. Higher pressures put a considerable strain on pumps and pipe installations and are likely to cause operational and maintenance problems.

- Electrical motor driven pumps as well as diesel driven pumps and generators generally should be put on a concrete slab separated from the floor. On loose soil, e.g. black cotton soil, the slab should not be too heavy as this may result in sinking of the installation.

- The need for adequate ventilation should be considered when diesel engines are used. Floor drain should be provided in pump houses to drain water from the piping system e.g. when bursts or leaks occur.

- All motors, pumps and engines shall be labeled.

- Fuel storage shall be provided.

10.6.2 Pipe Installations, General Details

- Separate suction pipes shall be installed for each pump when the static suction head is negative.

- Suction pipes should always be equipped with a strainer and a non-return foot valve when the static suction head is negative.

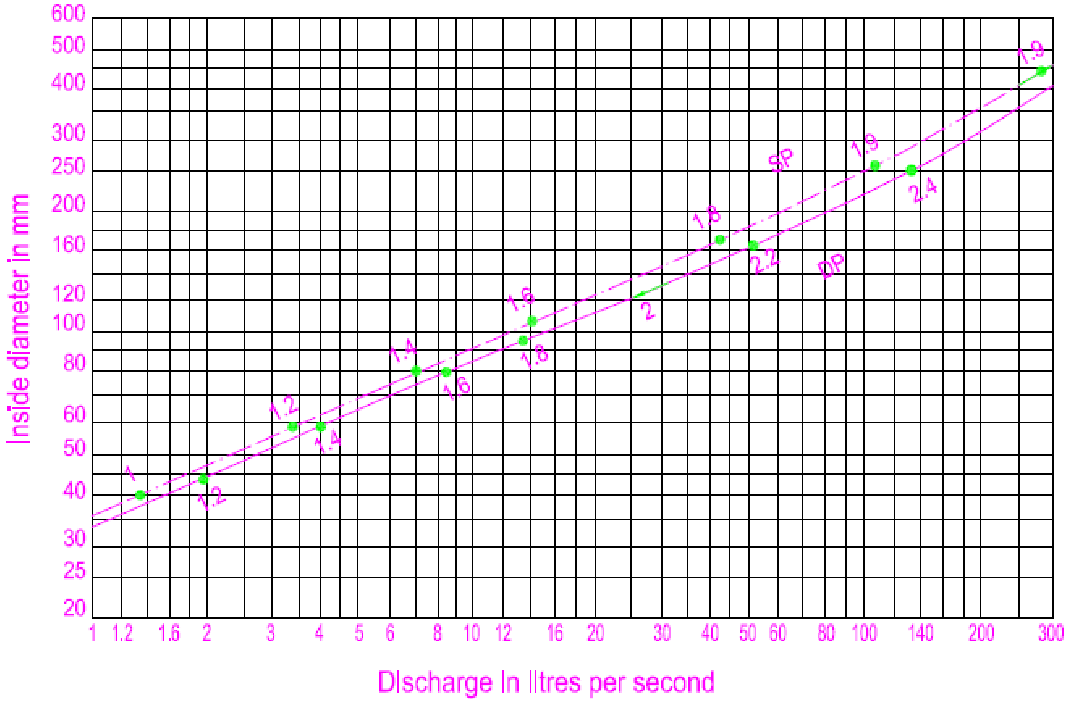
- There should always be a sluice valve and a check valve on the discharge side of the pump.

- All pumps shall be provided with both pressure and suction gauges.

- There should be a by-pass between the discharge and the suction pipe to allow cleaning of the strainer by flushing.

- The suction pipe and the discharge pipe in the pump house or whose length is approximately equal to the delivery head should be selected in accordance with Fig No. 10.6. Long discharge pipes should be designed as described in chapter “Transmission and Distribution Lines”.

Figure No. 10.6 : Diameter of Suction and Discharge pipes



Diameter of suction and discharge pipes

(The figure shown along the curves indicate the velocity of the liquid)

V_s = velocity in m/sec. in the suction pipe

V_d = velocity in m/sec. in the suction pipe

10.6.3 Head Losses in the Piping System

- For losses in straight pipes, see chapter Transmission and Distribution Lines.
- Losses in the strainer and foot valve are shown in Figure No. 10.7 below.
- Losses in 90 ^{o} bends can be determined in Figure No. 10.8.
- Losses in non-return (check) valves, gate valves etc. from Fig. 10.9.
- Approximate head losses due to friction in pipe fittings from Fig 10.10

Figure No. 10.7: Head losses due to strainers with foot valves

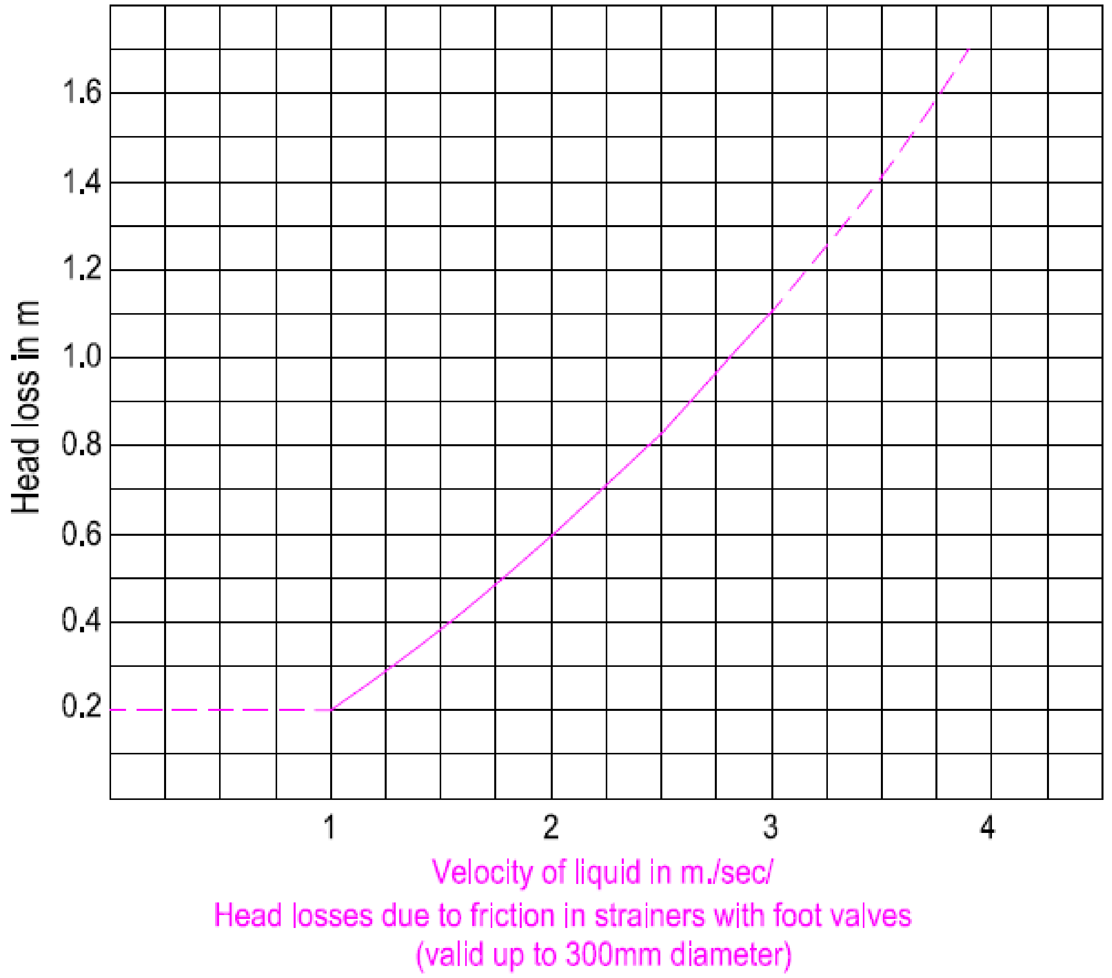
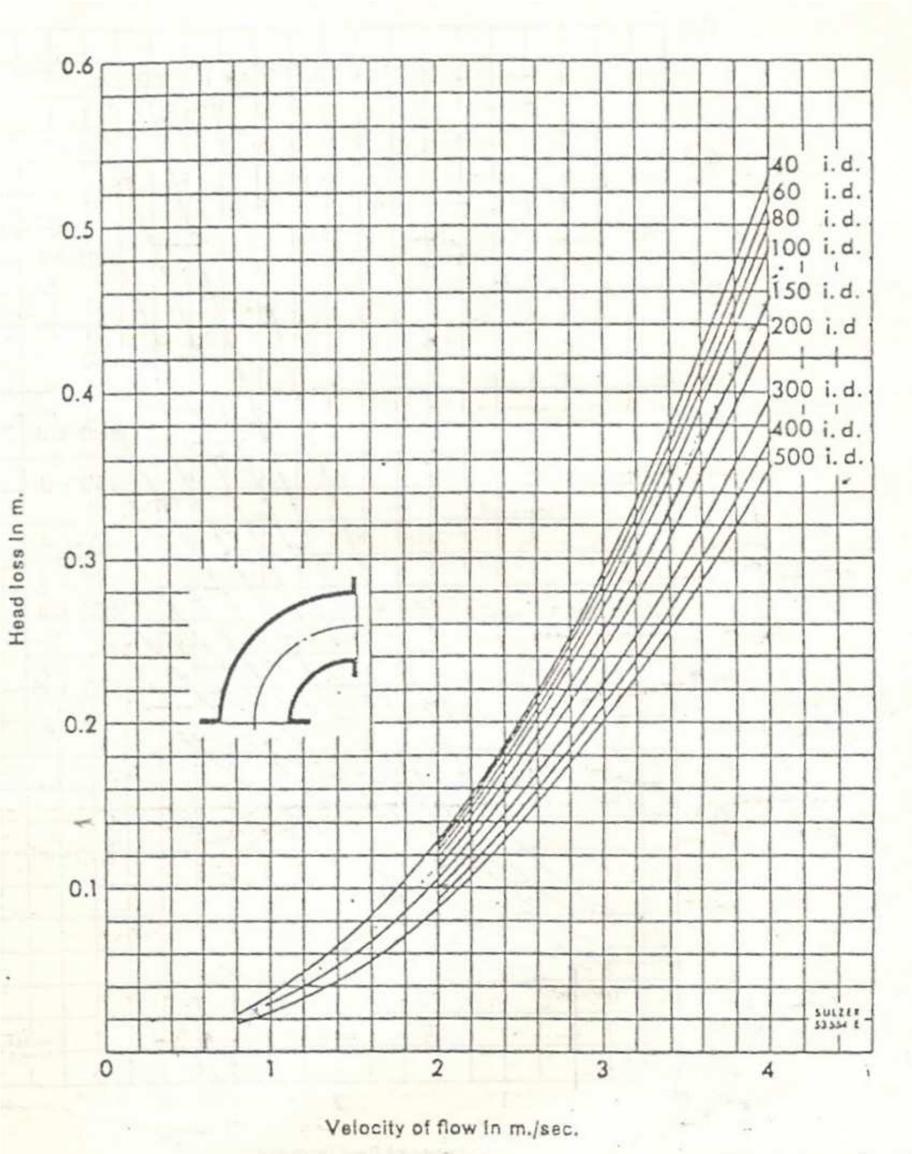
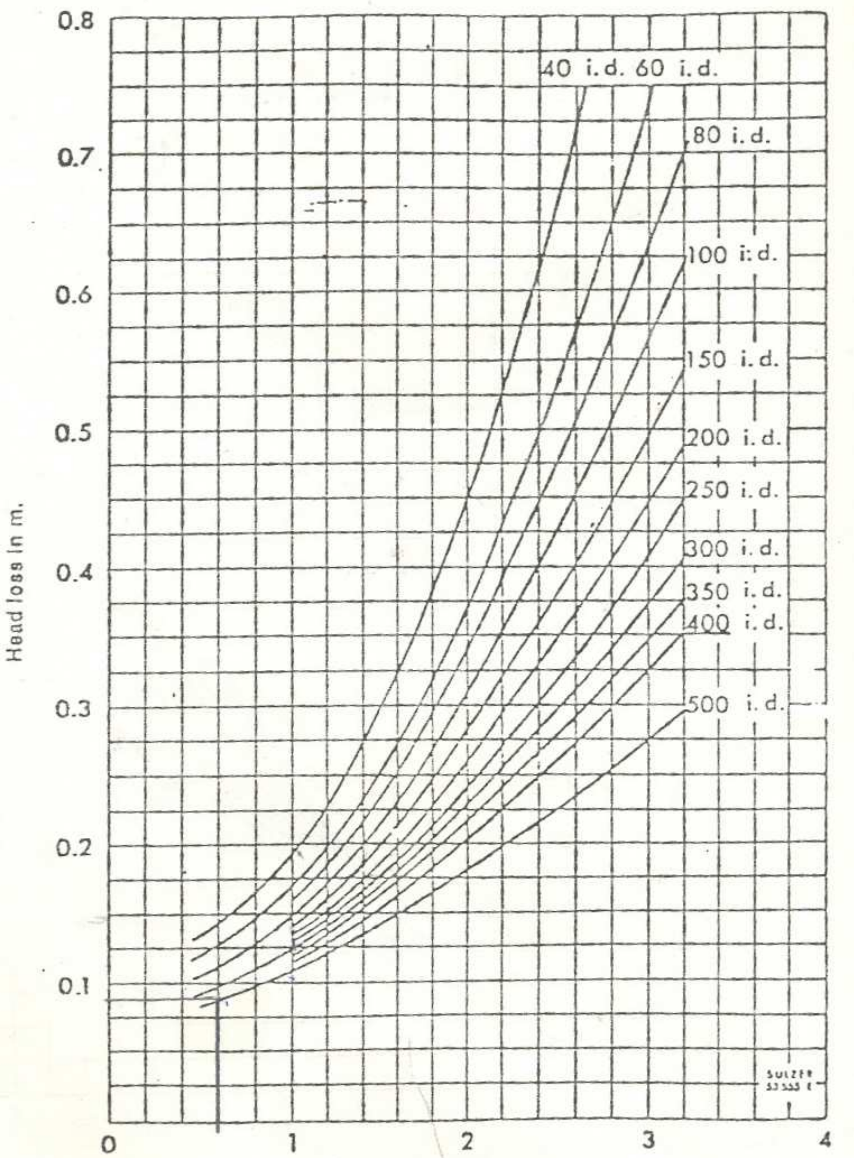


Figure No. 10.8: Head losses in 90 ^{o} standard bends



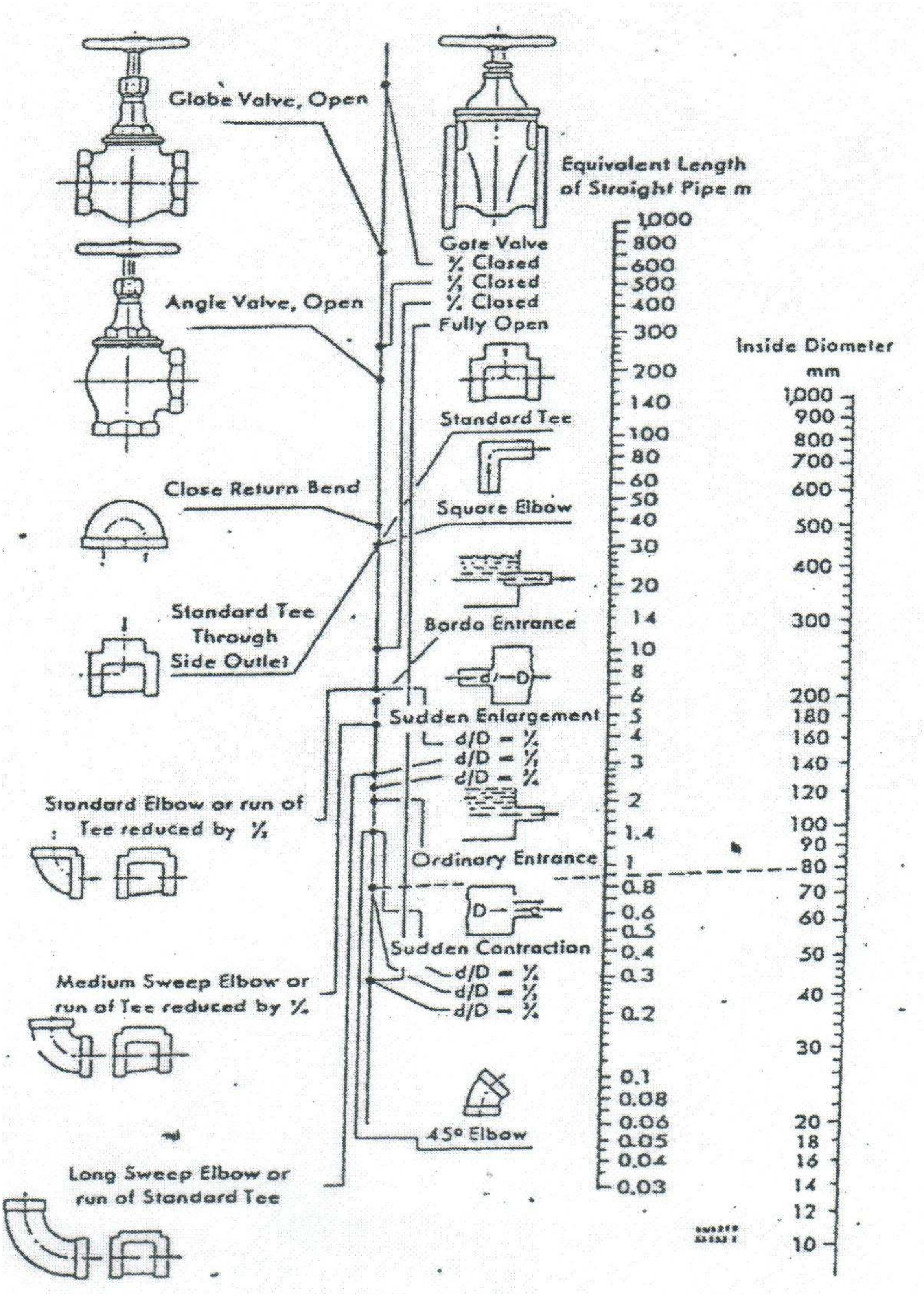
. Head losses in used 90° standard bends

Figure No. 10.9: Head losses in straight non-return valves



Head losses in straight non-return valves

Figure No. 10.10: Approximate head losses due to friction in pipe fittings



10.6.4 Spare Parts

Spare parts to the electrical and mechanical equipment for at least three years, as recommended by the supplier, should be included together with necessary tools.

10.7 OPERATION HOURS

10.7.1 General

Generally, the optimal number of working hours per day should be determined after an economic analysis taking into consideration cost of the pumps, pipeline and balancing reservoirs. It should be remembered that electrical power usually is paid both per kWh and per peak KVA. The latter charge depends on the installed peak demand and often contributes to a large position of the total energy costs. Thus by keeping the peak demands low the energy costs will be reduced.

10.7.2 Raw Water Pumping Station

A raw water pumping station should normally be designed for 24h operation to match the treatment works which are designed for 24 hour operation. It should be noted that the fact that the pumping station is designed for 24h does not necessarily mean that individual pump will run constantly as the available pumps should alternate.

10.7.3 Borehole Pumps

A borehole pump should normally work 24 hours a day as this gives maximum utilization of the borehole. However, when the water requirements are lower than the available yield of the borehole then the pumping hours may be reduced.

10.8 STANDBY UNITS

10.8.1 Raw Water and Clear Water Pumps

Pumping stations should have one stand-by pump with the same capacity as the pumps which are normally in operation. Hence if the station is designed for one pump, then two similar pumps should be installed. If the station is designed for two pumps running in parallel, then three similar pumps should be installed etc.

10.8.2 Electricity Supply

In rural areas where there exists an electric supply no reserve supply should be provided for motor driven pumps. Hence neither generators nor diesel engines should be provided. In principal towns and urban centers there should be either diesel driven stand-by pumps or generators to guarantee at least 50% of the normal water deliver in case of electricity breakdown.

10.8.3 Borehole Pumps

Boreholes equipped with motor or diesel driven pumps should normally not have any stand-by pumps.

10.8.4 Wind-powered Pumps

Windmill driven pumps should normally not have any stand-by. Instead ample storage capacity of 3 days should be provided.

10.8.5 System Reliability

The stand-by system as recommended above should be regarded together with other measures affecting the total reliability of the water delivery system such as emergency storage. See the chapter “Water Storage”.

10.9 PHASING

10.9.1 Design Period

Pump house and pipe installations should be designed for the same period as the supply in general i.e. usually 20 years. However pumps and engines should normally be designed for not more than 10 years.

10.9.2 Staging of the Pump Installation

The pumping requirements during the initial duty period of a pumping station are usually considerably smaller than at the end of the design period. This should be considered e.g. by adding pump units in pace with the increase of the demand or by changing the speed of a belt driven pump.

Due regard should be paid to the fact that the operation point of the pump will shift if the flow in the pipeline changes or the speed is different to the design assumptions for the ultimate stage. The pump and pipeline characteristics should therefore be studied for all phases to make sure that the pump efficiency remains high throughout the duty period.

10.10 HANDPUMPS INSTALLATIONS

10.10.1 Capacities of Hand Pumps

- The capacity/performance depends on the capacity of the hand pump and not the well/borehole

- Pumps with pistons of 7.5 – 10 cm diameter have a capacity of 1200 – 2,000 liters/hour, with a stroke frequency of one per second.

10.10.2 Comfortable Operation of Hand Pumps

For a comfortable operation of the Hand pump there exists a relation between the cylinder diameter and the head or lift:-

Cylinder Dia. (mm)	Head or lift (m)
51mm	Upto 25m
63	25
76	20
102	15

- The pump handle height for comfortable operation based on field survey is 100m, exclusive of the height of the foundation which should be limited to 0.10m above platform level.

10.10.3 Cost Considerations on the Selection of Hand Pump

The cost of the hand pump must take into consideration the following factors: -
The cost of well development, the cost of the hand pump should be related to the cost, yield and reliability of the well

Conditions of service: -
Stress and wear on the hand pump are directly proportional to the number of people it serves and to the depth from which water must be raised. Many people and deep water tables wear greater stresses and justify greater costs per hand pump, for example brass rather than iron cylinders.

Reliability: -
When the population is solely dependent on Hand pumps for water,

10.10.4 Protection of Health

The pump base and/or apron provides a means of supporting the pump on the well and protects the well opening against the entrance of objectionable material.

Wells should be sealed against contamination from surface water. An apron with a minimum diameter of 2.50m and a minimum thickness of 0.10m or more to be provided. Drainage for waste or spill including soakways or other means for prevention of puddles and pools, conducive to breeding of mosquitos.

Maintenance of suction (foot) valves is essential to protection of health. The valves when working properly eliminate the need for pumping the pumps from the top, a frequent source of contamination.

10.10.5 Types of Hand Pumps being used in Kenya

There are 4 types of hand pumps used in Kenya. Afridev, India Mark I, Indian Mark II Extra deep and Duba. Afridev, India Mark II and Duba are applied for a static groundwater up to 90m according to their specifications. To allow for hand pumps to be operated by women and children the maximum depth of hand pump is set at 55m.

10.11 WIND POWER INSTALLATIONS

10.11.1 Introduction

- A windmill pump provides a means of raising water from a well to an elevated tank without operational costs. Through its ability to pump 24 hours a day, water can be stored, and supply water at peak draw-off periods to several public standpipes at the same time and at higher rates of discharge than a single hand pump drawing water from the same source.

- Incase water is being pumped by a windmill from shallow wells, hand dug wells have distinct advantages over small diameter wells, namely the large volume of water stored, and the more rapid inflow.

- The power in the wind is proportional to the wind speed cubed:

$$P = ½ d AV^3$$

Where:

P = Power (KU)
d = Density of Air = 1.2kg/m^3 (ASL)
A = Cross section of windmill rotor (m^2)
V = Instantaneous free steam wind velocity (m/s)

or $P = 0.6 V^3$ (U/m^3 of the Area)

Because of this relationship, the power availability is extremely sensitive to wind speed, doubling wind speed increases the power by eight times more.

- Windmill sizing for a particular pumping application is stated from the wind data records. Ideally, about three years of recording are required to obtain reasonably representative averages, as monthly wind speeds can vary by 10-20% or so from one year to the next.

10.11.2 Size of Windmill

Windmills are usually sized by the diameter of their wind wheel, and the larger this is the greater the elevation to which water can be pumped. The following table shows desirable minimum heights of towers for average conditions as function of the windmill diameter.

Wind wheel diameter m	Windmill tower height m
1.80	7.50
2.40	9.00
3.00	9.00
3.60	12.00
4.20	12.00
5.10	13.50
6.30	13.50
7.50	16.50

If the windmill is located on a hilltop, the tower height given can be reduced.

10.11.3 Technical Data Required from the Field.

(i) max, min and mean wind velocities on a month by month basis for one year
(ii) water demand
(iii) static water level, seasonal variations
(iv) Details of drawdown at various rates of abstraction at above and below desired daily water output.
(v) Top water level at which water is to be pumped above ground level.
(vi) Height of tower
(vii) Tank capacity
(viii) Well internal diameter
(ix) Distance form nearest trees or obstructions, and height of these.

10.11.4 Maximum Pumping Head

The maximum pumping head of a wind pump is around 200m.

10.11.5 Application of Wind Pump

The relationships of rotator diameter, pumping head, and pumping capacity provided by the manufacturer are presented below.

Dia. of Rotator	**3.7m**			**4.9m**			**6.1m**			**7.4m**		
Wind velocity (m/s)	2-3	3-4	4-5	2-3	3-4	4-5	2-3	3-4	4-5	2-3	3-4	4-5
Head 10m	10	28	59	21	71	150	39	10 7	227	61	167	354
20m	5	14	29	10	35	75	19	53	113	30	83	177
40m		7	15	5	18	37	10	27	57	15	42	89
80m		3	7	3	9	19	5	13	28	8	21	44
120m			5		6	12	3	9	19	5	14	29
160m			4		4	9		7	14	4	10	22
200m			3			7		5	11		8	17
240m						6		5	9		7	14

To operate the wind pump requires an average wind velocity of least 2 to 3 m/s.

The following table gives pumping capacity at head of 60 meters for wind velocity of 2.5 to 3.5 m/s.

Average wind velocity	**Dia. of Rotator**			
	3.7m	4.9m	6.1m	7.4m
2.5 m/s	0 m^3/day	4 m^3/day	8 m^3/day	12 m^3/day
3.0 m/s	2 m^3/day	8 m^3/day	12 m^3/day	20 m^3/day
3.4 m/s	4 m^3/day	12 m^3/day	20 m^3/day	32 m^3/day

Wind pump with rotator more than 4.9m diameter provides pumping volume equivalent to that of handpump, but it costs around five times that of India I.

Mark II Extra Deep type.

Wind pump is, therefore, regarded as an alternative motorized pump but not to handpump.

Under average wind velocity of 3 m/s and 60 meters pumping head, a wind pump with rotator diameter of 3.7m has a cost per unit pumping volume equivalent to 60% that of a motorized pump for daily water demand of 4m^3/day. Whereas, the ratio becomes 80% for wind pump with rotator diameter of 7.4 meters operated to lift water or daily demand of 14m3/day under the same wind and head conditions.

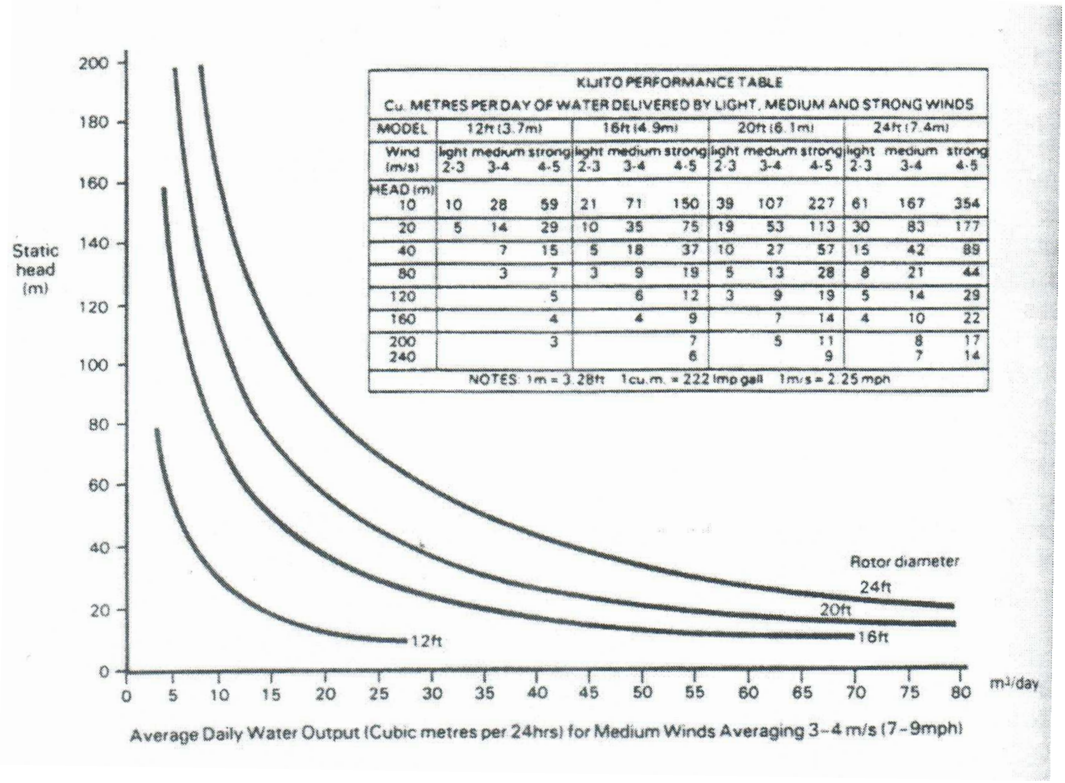
However, the previous reports indicate that cost effectiveness of wind pump becomes lower than that of motorized pump for average wind velocity less than 3 m/s. For the cost effectiveness of a wind pump is mostly dependent on wind velocity. Precise wind records are, therefore, required for proper design of a wind pump.

10.11.6 Wind Potential in Kenya.

One of the prerequisites for a windmill pump installation is a wind of not less than 2 to 3 m/sec. In the Table below are indicated wind potentials in certain stations. Especially to be noted are the stations with a wind potential greater than 5m/second practically all the year round, they are:- Eldoret, Equator, Isiolo, Kitui, Lamu, Lokitaung and Serani.

Two stations with wind potentials of 4-5m/second are:- Garissa, Machakos, Malindi, Marsabit, Wajir. The map below shows the points of the country where equipping of wind energy is possible.

Figure 10.11 shows a table and performance curves for the Kajito range of windpumps based on IT Windpump and made in Kenya. The table indicates the average daily output to be expected at different pumping heads from the four sizes of Kajito pump in three different average speeds defined as light (2-3 m/s), medium (3-4 m/s), and strong (4-5 m/s) while the curves reproduce these results just for the medium wind speeds.



KIJITO PERFORMANCE TABLE

Cu. METRES PER DAY OF WATER DELIVERED BY LIGHT, MEDIUM AND STRONG WINDS

MODEL	12ft (3.7m)			16ft (4.9m)			20ft (6.1m)			24ft (7.4m)		
Wind (m/s)	light 2-3	medium 3-4	strong 4-5	light 2-3	medium 3-4	strong 4-5	light 2-3	medium 3-4	strong 4-5	light 2-3	medium 3-4	strong 4-5
HEAD (m) 10	10	28	59	21	71	150	39	107	227	61	167	354
20	5	14	29	10	35	75	19	53	113	30	83	177
40		7	15	5	18	37	10	27	57	15	42	89
80		3	7	3	9	19	5	13	28	8	21	44
120			5		6	12	3	9	19	5	14	29
160			4		4	9		7	14	4	10	22
200			3			7		5	11		8	17
240						6			9		7	14

NOTES: 1m = 3.28ft 1cu.m. = 222 Imp gall 1m/s = 2.25 mph

Fig. 10.11: Performance data for Kajito wind pump range based on the IT wind pumps (Fraenkel 1986)

It can be seen clearly how sensitive wind pumps are to wind speed. For example the smallest wind pump with a 3.7 rotor will perform with 7.3m rotor in a 3 m/s wind. This is because there is 4.6 times as much energy per unit cross suction of a 5 m/s wind as in a 3 m/s wind as a result of the cube law.

10.12 SOLAR POWER INSTALLATIONS.

10.12.1 Advantages

A very important fact that distinguishes the solar water pump from the other types of equipment (Hand pump and windmill) is that the solar pump gives a maximum yield between 12.00 and 13.00 hours every day.

The peak demands are early in the morning and late in the afternoon.

Hence the equipment of shallow wells with solar water pump requires storage capacity, to provide the necessary quantities of water during the above mentioned peak hours, when the solar water pump is not functioning at full capacity. The solar water pump requires only a minimum of maintenance and is guaranteed to last 20 years after which the capacity of the solar panels is reduced. They however demand a very high capital investment.

10.12.2 Available Solar Radiation & Design.

Refer to 10.5.5 above.

11. OPERATION AND MAINTENANCE

11.1 Operation

11.1.1 Objective

The objective of the operational organization is to ensure the provisions of a continued and satisfactory service to the user of the water or sanitation system at minimum cost.

11.1.2 Institutional Arrangement

The Water Act 2002 proposes the following institutions for various water and sanitation services:-

Rural water supply - Non Governmental organisaiton (NGO) which includes church organization etc and Self Help Groups (SHG)

Urban Water Supply - Water companies fully owned by Local Authorities and Private Services Providers.

11.1.3 Staffing

(a) Staffing Norms

The Design of both processes and plant must be related to the level of Local staff capability if service is to be satisfactory. Depending on the water supply, the staffing is divided into management, which provide directions and control, the operators provide product quality and matching rate of working to requirement. Maintenance workers will be concerned with the replacement of worn or defective items so as to ensure continuous serviceability. These duties may overlap in the interests of economy.

Appendix F has a criteria for staff required for operation of water supplies.

(b) Composition and duties of staff

The managerial staff of a utility is likely to include engineers (civil, mechanical and electrical) and chemists, supported by engineering and Laboratory Technicians and Technical Assistants, Accounting, Clerical and Secretarial staff. Staff numbers and tasks will depend upon the system size and its complexity.

(c) Staff motivation

Job enrichment aims at increasing the levels of all satisfiers and removing any reason for discontent with the levels of the dissatisfied.

The effective operation depends upon trained, interested and motivated staff and it must be a primary task of management to create and maintain such staff.

11.1.4 Records

Records may relate to permanent construction, to operation or to maintenance and repair.

Records of permanent construction show what has been done and where it is located. They are used to locate the components of the system on the ground, to aid understanding of the Design and hence how the system is intended to operate, and to facilitate alteration or extension of the system.

Operational records may provide guidance for the operation of the system, so they will very often incorporate records of permanent construction or record operational performance to aid future design and to serve administrative purposes.

Records of maintenance and repair serve to allow critical evaluation of performance, and to facilitate planned maintenance.

11.1.5 Records of Permanent Construction.

The easy and economic operation of a scheme is particularly dependent upon an understanding of the layout and ready location of the component parts by persons having varying degrees of familiarity with it. The basic requirement therefore is a series of drawings showing increasing degrees of detail starting with a layout of the whole scheme and ending with intricate details of components.

11.1.6 Records Required for Operation and Maintenance

Additional drawings are likely to be required for operation, some only modifications of those already mentioned, these will include plans which show wastewater meter areas. Zoning may be altered over the years so zones should not be permanently depicted on the basic records except by erasable lines. Organization division for which branch offices are responsible will also be shown on plans.

As-built drawings and others will be incorporated in plant operating manuals wherever this assists the operator.

Card files, notebooks, or drawings may be used for other records relating to the distribution system.

Cards can also be usefully employed as part of the plant maintenance programme. For example, there should be records on every item of mechanical and electrical.

System curves for pipelines and performances curves for pumps should be included in operating manual.

Treatment works, pump stations and similar installations must provide operating records for control, costing and future design. The attendant, who should record the results of routine checks at prescribed intervals, will log basic data. The superintendent will add other information and general comments.

At waterworks, reports of this type are likely to show daily figures for the following:-

- Raw water intake
- Pure water output
- Peak day output for period of report.
- Clarifiers scoured: fine since last scour
- Filters back: washed time since last wash
- Water losses
- Chemical usage: Type of chemical
  Mass used
  Dosage added to raw water
  Deliveries
  Residual stock
- Mechanical/electrical plant (for each unit)
  Hour meter readings, hours run
  Power/fuel used
  Fuel stocks remaining
  Service done
  Maintenance done or needed
- Raw water storage level
- Distribution reservoir level
- General comments on materials received or removed, equipment (breakdowns, down time), expenditure and staff.
- The results of works laboratory testing.

The operational report from the sewage treatment works will relate to the type of treatment. For pond treatment, the operating report is unlikely to cover more than daily inflow, color of ponds, maintenance work done, and general remarks.

Where treatment is provided by settlement and biological filter, daily records should cover:-

Inflow – average, peak rate
Screenings volume
Volume
Raw sludge volume
Humus sludge volume

Volume of sludge withdrawn from digesters.

11.1.7 Updating Records

Special arrangement is necessary to ensure effective and continuous updating and reissue. The appropriate processes must be incorporated in the administrative system and time and staff must be provided for the work.

11.1.8 Treatment works – Operational Procedures

The operator at a water or wastewater treatment plant will be concerned with:- Distributing inflow among the various units to suit their ratings,

- Preparing and adding chemicals at certain stages in proportions selected to provide an outgoing flow of the correct quality at least overall cost.
- Periodic attention to treatment units, e.g. clarifier scour, water filter backwash or cleaning and removal of vegetable growths.
- Operation and adjustment of a variety of mechanical equipment e.g. screens, mixers, stirrers, chemical dosing equipment, compressors, pump sets, aerators and conveyors.
- Quality control of the effluent leaving the plant.
- Disposal of plant wastes (e.g., filter wash water, sewage sludge) by methods environmentally acceptable.
- Minor maintenance procedures
- General cleanliness and appearance of plant and surrounds and
- Record keeping

The waterworks superintendent in particular will be concerned to evaluate demand for an operating period immediately ahead, generally the next day and with matching to it the output of treated water from the plant as efficiently as this can be done. To assess demand we need to know the quantity of water supplied by his works during the previous day, and the distribution reservoirs. Then applying factors to allow for the weather, the day of week and so on, he will formulate his forecast.

11.1.9 Public Relations

As a corollary, the utility should have defined procedures for dealing with complaints, which should be tactfully received and investigated. A utility should have a public relation officer who records complaints and channels them to appropriate person for action simultaneously serves as a record. When appropriate action has been taken, the complaint is advised. Even where he is unreasonable, every attempt should be made to satisfy the complainant.

11.1.10 Laboratories

Objective:

The Laboratory has two chief objectives, firstly, that the result of treatment is a water or wastewater, which complies with prescribed standards and secondly that treatment is efficient.

Sampling:

To ensure the integrity of samples, they should be taken by persons disinterested in the results. Satisfactory results are obtained by engaging both operational and laboratory staff in sampling in a random manner.

11.2 MAINTENANCE

11.2.1 Purpose

Maintenance is a key element in the efficient operation of water supply system and hence the necessity of a good preventive and correction or breakdown maintenance program. Preventive maintenance is planned or scheduled maintenance, performed to eliminate or minimize breakdown or corrective maintenance and to extend the useful life of a water supply system. Breakdown or corrective maintenance refers to unplanned or unscheduled maintenance or repair caused by failure and requires immediate action.

11.2.2 Maintenance Programs

Key features of maintenance programs should include:

- Responsibility for maintenance clearly defined and vested in competent personnel.
- Management should state its maintenance objectives and make its position clear with proper support, morally and financially.
- Proper tools, parts, instruments and maintenance facilities must be provided.
- Preventive maintenance must be planned for scheduled and accomplished.
- An adequate system of written records and reports must be used and readily available to control and monitor the program.

11.2.3 Maintenance Systems

The creation of an efficient maintenance service will be facilitated if:-

- Management states its maintenance policy, objective and attitude clearly,
- Responsibilities are clearly defined and are vested in competent persons
- Adequate equipment and materials are scheduled, provided, and themselves maintained, and
- Records and reports facilitate control

11.2.4 Types of Maintenance

There are 3 classes of maintenance viz: -

(i) Operational, maintenance, carried out on a day to day basis by the operator and including cleaning minor adjustment, and lubricating

(ii) Corrective or breakdown undertaken only after fault or breakdown,

(iii) Planned or preventive maintenance. Regular maintenance and parts replacement in accordance with a programme based on calendar time or condition monitoring has superseded operating hours, which seeks to do the work just in time to avert breakdown or serious deterioration in performance.

11.2.5 Preventive (Planned) Maintenance

The intention behind a planned programme is to eliminate breakdown, thus ensuring performance at an acceptable level of efficiency without failure.

- Helps to assure continuous supply of water

- Can be scheduled at times of the year when customer service is not affected or when its adverse effects are minimized.

- The frequency of a planned maintenance program will vary from one utility to another and even among smaller types of equipment. Each equipment item must be studied individually, as similar pieces of equipment may have different maintenance requirements because of location and service. Table 11.1 provides a summary of recommended minimum inspection and test frequencies for preventive maintenance in Kenya.

Table 11.1 Recommended Summary of Inspection, Test and Maintenance Frequencies in Kenya

S/NO	Inspection, Test or maintenance performance		Frequency
A	S/NO	Underground water system	
	1	Visual inspection for leaks	Daily
	2	Flushing • Recommended • Minimum	 Semi-annually Annually
	3	Inspect/operate non-cutical valves (< 250 mm ∅)	Annually
	4	Inspect/operate large valves (>250mm ∅) cuticle valves or valves with closed gear boxes.	Semi-annually
	5	Fire hydrant inspection • Recommended • Minimum	 Semi-annually Annually
	6	Listening survey for leaks	Every 2-3 years
	7	Complete leak detection survey including flow measurements, 24-hour consumption and trunk main gauging.	Every 5 years.
	8	Fire flow tests	Annually
	9	Loss of head tests	Every 5 years
	10	Pressure testing of pipes • Recommended • Minimum	 Every 10 years Every 15-20 years
	11	Meter accuracy test >150 mm 75 – 150 25 – 75 < 25mm	 Annually Every 2 years Every 5 years Every 10 years
	12	Inspection/testing of backflow parameters (min)	Annually

B		Water plant	
	1.	Housekeeping – General condition and appearance of buildings, grounds and equipment.	Daily
	2	Valve inspection/ operation	Semi annually
	3.	Water storage tanks • Exterior visual observations • Check water levels • Inspect general condition • Complete inspection including tank drainage and checking interior condition and for sedimentation build-up. <5,000m^3 5000-50,000m^3 Over 50,1000m^3	 Weekly Weekly Annually Every 5 years Every 10 years Every 20 years.
	4	Pumps • Check operation of routinely operating pumps • Check operation of standby operation pumps • Check operation of standby generation equipment • Pump effectiveness and performance testing	 Daily Weekly Weekly Annually
<u>C</u>		Water Quality	
	1.	Chlorine residual	Daily
	2.	Turbidity • Surface water • Ground water	 Daily Every 2 years
	3	Bacteriological	Monthly
	4	Primary drinking water standards to KBS Standards • Surface water • Ground water	 Annually Every 2 years
	5	Redionuclides	Annually
	6	Trilialomethanes	Annually
	7	Secondary drinking water standards	Every 3 years

NOTES

1. Table is presented as a guide which should be modified based on site specific conditions

2. Table does not include a complete schedule of all types of equipment considered to be part of the water plant. Manufacturer's recommendation should be consulted before establishing preventive maintenance schedules on any specific equipment type.

3. Water quality frequency testing would be recommended according to the requirements of the state in which the utility is located. The testing listed is considered the minimum to be accomplished and should be supplemented by other tests needed to monitor and control specific water quality problems in a particular system.

4. Frequency and number of samples of C5 and C6 could be reduced pending at least one year's satisfactory results, but would be subject to the specific requirements of the National Standards.

11.2.6 Other Assets

The principle of planned maintenance, though most significant for mechanical and electrical equipment, should be applied in principle to every part of the system.

11.2.7 Organizing for Breakdown and Emergency

Any utility providing services to the public should prepare itself to deal with breakdowns and emergencies of varying severity.

If a local emergency system is adopted, a member of staff should be made responsible for the area in which he resides, and with tools and transport readily available, he should quickly go to the scene of any problem and attend to it or summon assistance.

11.2.8 Workshop

Workshop provides civil, mechanical or electrical services

11.2.9 Purchases

The system of stock control should produce the information needed for the timing of purchases and should record the consumption upon which the buyers depend for fixing the size of any order.

11.2.10 Receipt, Issue and Control

Operational efficiency depends upon procedures which make the receipt and issue of goods quick and easy. The accompanying accounting methods must determine the charge out rate for materials based on purchase price and stores overhead and allocate it to the user project.

11.3 UNACCOUNTED FOR WATER (UFW)

11.3.1 Definition

There are many ways in which it is defined. The most practical and acceptable method defines ufw as the difference between the measured amount of water entering the system and the total measured amount of water leaving the system.

UFW is represented in many ways the most common being:-

(i) UFW as a percentage of supply where:- % UFW = UFW/measured water entering the system.

(ii) UFW as a ratio of supply pipe length where:-
UFW/m = the UFW (litres) per km length of pipe

(iii) UFW as a ratio of consumers where:-
UFW/No. – UFW (Litres) per consumer (number) supplied

UFW includes leakage and other losses

11.3.2 Components of UFW

a. Bulk meters

Uncalibrated bulk meters lead to inaccuracies in flow measurements.

b. Consumption meters

(i) Meter inaccuracy
This constitutes about 2.5% of the UFW

(ii) Broken meters
When meters are broken, the consumption of the consumer is just estimated.

(iii) Malfunctioning meters
Meters tend to slow down with time. This is due to dirt or rust accumulation on their moving parts. The solution to this problem lies

in having a meter replacing programme, based on engineering recommendation and not on a time frame.

(iv) Oversized meters
In many cases the water undertaker is misled by the customer or exercises wrong judgment and installs oversized connection. Oversized meters fail to record lower flows.

c. Unmetered connections

Consumers who are unmetered consume much water and have more wastages than metered consumers. Un metered connections usually belong to one of the following categories:-

(i) Known monitored connections (Flat rate connections)

(ii) Un monitored connections
Often connection exist (with or without meters) that were at some time in the past legally provided. However, due to some reasons, records of existence of such connections have been lost.

(iii) Illegal connections
These are quite common and they refer to non-authorised connections.

d. Unmonitored usage

Water is used from the system without being measured.

e. Mistakes in billing

(i) This is due to inaccessibility of meters

(ii) Meter reading incompetency arising from human laziness or incompetence and system confidence where the credibility of the organization to produce correct bills is being questioned.

f. Methods of measurement

The water balance equation is often distorted when measurements are compared on a short term basis. To calculate UFW for a metered area (through bulk or zonal meters), one must relate consumption meters to zonal meters (in isolated zones) only if measurement are taken at the same time. But if the readings are taken over a long period of say 1 year, then there will be insignificant discrepancies.

g. Network operation related

The state of the Network plays an important role in UFW. Some of the aspects that control UFW are:-

- High pressures
- Wrong zoning
- Overflows
- Incorrect pipe sizing
- Partly crossed isolating valves

h. Leakages

(i) Definition

Leakage is that part of UFW, which escapes or is lost other than by deliberate or controllable action from a water supply or network. It comprises of the physical losses from pipes, joints and fittings, and also from over flowing services reservoirs. Larger losses are usually from burst pipes, or from sudden rapture of a joint, lower level losses are from leaking or "weeping" joints, fittings service pipes and connection.

(ii) Components

The components are badly corroded pipes, visible bursts and non visible leaks.

12. MANAGEMENT MODELS

12.1 LEGAL FRAMEWORK

12.1.1 Registration

Water Service Provider (WSP) can register under companies act, the Co-operative Societies Act, the Non Governmental Organization Act and/or the Societies Act.

12.2 Registration Laws

The choice of an appropriate registration law is a difficult one but largely depends on whether a group or an association is intended to be commercial or welfare – oriented. Other factors relate to size or membership of the group of association and how the group intends to relate to its own members, non-members and other person groups or association.

12.2.1 The Companies Act Cap 486

Under the Companies Act, a group or association of persons may incorporate a private or public company. A private company consists of at least 2 shareholders and a maximum of 50. Beyond fifty, a public company is formed.

The public company has been a popular mode of registering groups and associations. The legal requirements include forwarding to the registrar of companies the Articles of Association as well as the Memorandum of Association.

Articles of Association regulate the internal relationships, organization and management of the group, while the Memorandum of Association regulates the relationship between the group and non-members.

Many welfare groups, charities, trusts and non-governmental Organizations (NGOs) have hitherto been registered as public companies limited by guarantee of the directors.

Once all the formalities of incorporation are completed, the registrar of companies issues a certificate of Incorporation. This transforms the members into a body corporate. The company may thus in its own name own property, enter into contracts and sue and be sued in its own name.

12.2.2 Co-operative Societies Act, Cap. 490

The Co-operative movement has become a very important feature of Kenya's economic, social and political landscape since the 1980's.

Co-operative societies fall under three categories; primary co-operative societies, co-operative unions, and apex co-operative societies. Primary co-operative societies consist of individual members; co-operative unions are open to primary co-operative societies, and apex co-operative societies are restricted to co-operative union membership.

The most important type of co-operative for groups or associations envisaged is the primary one. The requirements for registration include objects which underscore the promotion of the members economic interests, applicants must be at least 10 in number; applicants must be aged 18 years and above; applicants must be residents of the society's geographical area of operation or they must occupy land within the said area.

12.2.3 Societies Act, Cap. 108

The societies Act, Cap 108 defines a society to include "any club, company, partnership or other association of ten or more persons, whatever it's nature or object, established in Kenya or having its headquarters or chief place of business in Kenya, and any branch of a society. But does not include a company defined by the companies Act; any corporation incorporated under any written law; a registered trade union; a company, firm, association or partnership consisting of not more than 20 persons, formed and maintained with a view to carrying on business for profit; a co-operative society registered as such under any written law, a school; a building society; a bank and inter-government organization of which Kenya is a member; combination or association which the Minister may, by order, declare not to be a society for the purpose of this Act.

The legal requirements for registration as a society are submission of the necessary application with sufficient details of the proposed society. Some of the required information is the name of the society, the current number of members; the titles of the office bearers; the names, occupations and addresses of the present or proposed officers and the land premises (if any) owned by the society and the manner in which such property is held or vested.

A large number of national and district-based Non-Governmental Organizations as well as community groups were originally (and still are) registered as Societies. Some have, however, since changed regulatory regimes since the Non-Governmental Organization Co-ordination Act (Act No.19 of 1990) came into force.

12.2.4 The Non-Governmental Organization Co-ordination Act, Act No.19 of 1990

One of the major reasons for enacting the NGO Co-ordination Act was to ensure that only political parties, and religious and welfare organizations remained under the purview of the societies Act, Cap. 108.

A number of organizations, groups and associations which had earlier been registered as companies limited by guarantee of directors or under the societies Act were given a grace period to shift into the regime of the NGO Co-ordination Act. A number have not changed, or are still unregistered under any of the foregoing regimes. This is mainly the case with small, local community based projects who are either unaware of the legal changes or cannot access funds and/or expertise to effect necessary adjustment.

The NGO C0-ordination Act makes it illegal to operate an NGO which has not been registered. There are striking similarities between the requirements for registration under the Societies Act and those under the NGO Co-ordination Act.

12.2.5 Ministry of Culture and Social Services

There is a category of income generating, welfare, self help, philanthropic, charitable, and other groups which utilize the regulatory under the Ministry of Culture and Social Services. In many instances, the groups are already registered in some parts of the country or outside the country.

Such groups approach the District Social development Officer under the Ministry of Culture and Social services for certificate of recognition to facilitate their operations in the district in question. The certificate does not constitute a certificate of registration and does not confer any new legal personality in the manner already discussed in relation to the four laws above. The main importance of the certificate is the social and political legitimacy which the project attains.

It is also notable that in a number of cases, such certificates have enabled community-based groups to access funds from donors. In many instances, unless such groups are registered under recognized systems, they can only utilize the funds through accounts to which the project attains.

It is also notable that in a number of cases, such groups are registered under recognized systems, they can only utilize the funds through accounts to which the District Social Development Officer is a signatory, or an account operated by an independently registered organization.

The main advantage of this system is that it is not technical hence it is relatively easy to obtain a certificate. It is possible for members to understand and participate in the process. The costs are also affordable.

12.3 COMPARATIVE ADVANTAGES AND DISADVANTAGES OF REGISTRATION REGIMES

The various registration regimes discussed above pose benefits and constraints for the efficient and effective management and operation of WIS projects. These advantages and disadvantages are summarized below.

12.3.1 Company's Act

Companies operate under rigid management systems. These include legal requirements such as the processing and filing of financial returns. Registration formalities of a public company are also very complicated and time consuming. The management systems and financial procedures for corporate governance require a measure of literacy and/or familiarity with technical information and processes. Incorporation of companies would thus exclude the majority of the local people from decision making processes. Infact partly because of the foregoing reasons, most decisions are likely to be taken by directors and other corporate managers.

Companies are largely or principally run in the spirit of capitalism. The profit motives may make a company insensitive to rural problems. The objectives of companies, societies and Non-Governmental Organizations tend to be diverse and diffuse. This gives rise to the problem of lack of focus and question of determining and monitoring the association's adherence to it's core concerns. In comparison the objects of co-operatives tend to be focused and specific.

Another management problem in Companies arises from the fact that some members may be only nominal shareholders or they may have minimal shares hence limited say. Stories on corporate governance in Kenya in the recent past are replete with instances or allegations of the exclusion or oppression of minority shareholders.

12.3.2 Cooperatives Societies Act, Cap-490

Inspite of progressive liberalization in a number of social-economic sectors, co-operative societies have increasingly suffered intrusive regulation by commissioner and/or the minister for Co-operative Development. This is mainly due to the wide regulatory powers governing entry into and operation of co-operative business. The Act is however under review to give co-operatives more autonomy. It is not known when the Act will be reviewed and it can only be analysed when it becomes law.

As compared to NGO's, stakes in co-operatives are easier to determine. And as compared to companies, co-operatives tend to have more acceptable face of capitalism, as their management systems tend to infuse welfare approaches into business scheme. It is thus said that co-operatives are unions of persons while companies are unions of capital.

Infact partly because of the foregoing scenario analysts have opined that cases of co-operative failure in Kenya may be due to the fact that many co-operatives are run as welfare associations rather than income generating business entities.

Part of the problem why co-operatives may not deliver community development is because of lack of exposure to or a capacity in financial management and technical skills, which are important in running a modern social-economic unit.

Like companies and NGO's Co-operatives have corporate status, legal personality, and perpetual succession. These attributes facilitate long-term planning and the negotiation and conclusion of contracts in the name of the entity.

12.3.3 The Non-Governmental Organization Coordinator Act, Act No. 19 of 1990

The registration of Societies and NGO's largely delay because of the screening and vetting processes (NGO's have for a long time been regulated under the internal security portfolio in the president's office).

In terms of the black letter law, however, Societies and NGO's may operate as soon as the applications have been filed notwithstanding the delays. But the problems arise from political quarters and through the provincial administration.

12.3.4 Societies Act, Cap-108

The registration regime under the Societies Act is however not clear cut, it encompasses all manner of business, welfare, religion, political and developmental entities although the intention in the recent past has been to remove NGO-like bodies from its remit. Bit it has used large community groups as a means of getting recognition. Societies do not have clear cut legal personality, hence they have to act through appointed trustees.

12.4 CONCLUSION

The legal regime governing the registration and operation of organized community groups and similar associations is quite murky. As the foregoing research and analysis has indicated the appropriate law is available in at least four statutes; the companies Act, the co-operative societies Act, the societies Act and the Non-Governmental Organizations Co-ordination Act. Perhaps as privatization and liberalized measures take place, these community groups will need to be addressed and agitate for change just as Non-Governmental Organization Co-ordination Act was found necessary.

13. INFORMATION TECHNOLOGY

13.1 INTRODUCTION

13.1.1 Information Technology

Information technology is the technology, which supports activities involving the creation, storage, manipulation and communication of information together with their related methods, management and application.

13.1.2 Computer

A computer is a device that works electronically under the control of stored programs, automatically accepting, storing and processing data to produce information that is the result of processing.

The forms in which data is accepted or produced by the computer vary enormously from simple words or numbers to signals sent from or received by other items of technology. The functions of the computer include: -

- **Input ----------** accepts data from outside for processing.
- **Storage--------** holds data internally before, during and after processing.
- **Processing----** performs operations on the data it holds within.
- **Output---------** produces data or information from within for external use.

13.1.3 Hardware

Hardware is the name given to all the physical devices found in a computer, including its associated facilities, the peripherals.

All electronic elements found in the computer e.g. the transistors, diodes, electronic paths/channels etc, and the input/output and storage devices are collectively described as the hardware.

Computer peripherals are the term used to describe all the elements connected to the computer apart from the computer itself. These are facilities connected to the computer to assist the computer in satisfying its users and include the input/output units that interface the computer from its users and the storage units that supplement the computer internal memory.

13.1.4 Software

Software is the general term used to describe all the various programs that may be used on a computer system together with associated documentation.

There are two classes of software: -

(i) **Applications software** – Software that is designed for specific use, for example, designed to solve sets of mathematical equations or controlling organizations budgetary allocations.

Many applications packages are designed in such a way that they can be used for a variety of similar problems. For example, engineering design packages are produced in forms that enable them to be set up and used by different disciplines, each having slightly different ways in which they need to produce their designs.

There is abundant selection of engineering packages that can be used on personal computers in the market.

(ii) **Systems software** – These are programs with associated documentation, that control the way the computer operates or provide facilities that extend the general capabilities of the system.

Within systems software is operating system composed of a specific program or suite of programs that controls the performance of the computer by doing a variety of jobs to ensure the proper, orderly and efficient use of hardware and application programs. Examples of operating systems will include MICROSOFT OFFICE XP and AUTO CAD 2000.

NOTE: Applications programs only work when used in conjunction with the operating system.

13.2 THE INTERNET

13.2.1 General

Internet is the world's largest computer network, consisting of over 2 million computers supporting over 20 million users in almost 200 countries. Its growth is phenomenal – between 20 to 30% per month and hence any estimate is quickly out of date.

The value of the Internet lies precisely in its ability, easily and inexpensively, to connect so many diverse people from so many places all over the world. Anyone who has an Internet address can log on to a computer and peach virtually every other computer on the network, regardless of location, computer type, or operating system.

13.2.2 Functions

Internet carries many kinds of traffic and provides users with several functions that include:

- Communication through e-mail, Usenet, newsgroups, chatting and telnet.
- Information retrieval through Gopher, Archie, Wide Area Information Servers (WAIS), and the World Wide Web (The Web).

13.2.3 Using Internet

Most characteristics of Internet is how data if taken from one computer to the other. Data transfer can be summarized as follows: -

(i) Piece of data is broken into small pieces called PCK

(ii) A header is added. This header explains
  (a) Where the PCK comes from
  (b) Where it should end up
  (c) How it is fixed with the rest of the PCK

(iii) A PCK is sent from computer to computer until it finds its way to its destination
NB: A computer along the way decides where next to send the PCK depending on how busy other computers are at the time of PCK was RX.

(iv) At destination the PCK are examined. If a PCK is missing or damaged a message is sent asking for this PCK to be resent and this is procedure is repeated until all PCK have been received intact.

(v) Packets are re-assembled into their original form load and at computers must have TCP/IP responsible for receiving sending and checking Packets.

The Internet also consists of a number of regional exchanges, which are large computers called **servers**. These servers are connected by a large cable (of copper wire), which is quite separate to the telephone system. Undersea cables connect the Australian system to the American system, and thence to the rest of the world. Universities and government departments own these servers, and are thus able to pass information back and forth. They are thus permanently connected to the Internet.

Recently, private companies called **Internet Service Providers (ISPs)** have established servers on the Internet cable. Individual users from their home or office connect to the server of an ISP, and thereby gain access to the entire Internet system. This connection between a user and their ISP is made over a normal telephone line. The ISP has a number of incoming lines to their office (like an SP bookie), which enable a number of users to connect to their server.

13.2.4 Data & Information Transfer and Retrieval

Internet is a vast computer database with variety of computer resources and information. The only easy way of finding information on internet is by using URL which is specified on location or address bar.

Some of the problems encountered when searching information on the internet

(i) Incorrect URL or the URL has been changed
(ii) What if we do not know the URL
(iii) Solution to the above is use of search engines

Searching engine is a computer that goes around the internet to find documents that contain the word that you type. The search engine also incites or takes information provide web designer and put it in a database.

Search engines works in different ways:-

a) Some rely on people to maintain a catalogue of the web log
b) Some use S/W to search key information on sides across the internet

E.g. of search engines
-Google search
-Yahoo search
-Excite
-NL search

Services available on the Internet

Internet provides a variety of services ranging from accessing remote computers and using then for transferring files to and from the computers on the Internet. These services are:

13.2.5 The Web

- The web is an information retrieval tool similar to Groper, Archie and WAIS. However, it has had an enormous impact on the commercial use of the net owing to its being attractive and easy to use.
- While other information retrieval methods are text-based, the web combines text, hyper media, graphics and sounds, making information more appealing more informative and easier to grass.

13.2.6 Benefits of Internet Use

- Reducing communication costs
- Enhancing communication and Coordination

- Accelerating distribution of knowledge
- Improving customer service and satisfaction
- Facilitating marketing and sales.

13.2.7 Problems of Internet Use

- Lack of security
- Technology problems
- Legal issues
- Anti commercial culture.

13.3 SPREADSHEET

13.3.1 Description

- A spreadsheet or worksheet is an application program that displays a working sheet consisting of rows and columns of cells; the rows are usually identified by numbers, and the columns by letters.
- Each cell can hold a numeric value, a text label or a formula that processes values contained in other cells.
- Many spreadsheets contain extensive libraries of built-in formulas and can even maintain links to other spreadsheets so that data entered in one spreadsheet updates entries in another.
- Extensive data presentation featured and standard in most spreadsheets, which include reports generation and powerful graphing capabilities.

13.4 GIS DATABASE

13.4.1 Introduction

At the frontier of the information age, geographic information management technology is emerging as a powerful means to manage voluminous geographic data, to help cope with the information explosion, and to provide a foundation for solving the problems that beset planet earth and its inhabitants.

Societies in general are becoming keenly aware of the need to manage information from a geographic perspective. This awareness has been brought about by the twentieth –century trends toward a global community and economy. At the same time, the often negative impact of advancing technology has shown the need for wise management of the earth’s resources. Geographic information systems (GIS) provide the tools to help meet these challenges.

Geographic information systems (GIS) rely on the integration of three distinct aspects of computer technology:

a. Database management (of graphic and non-graphic data);

b. Routines for manipulating, displaying and plotting graphic representations of the data; and
c. Algorithms and techniques that facilitate spatial analysis.

13.4.2 Definitions

Data: A set of facts or figures, which have been gathered systematically, and form which conclusions may be drawn e.g. coordinates, pollution data etc.

Information: Data that have been processed to have meaning to a user.

Geographic Information Systems (GIS):
Francis Hanigan (1988), defines a GIS as "any information management system which can:

(a) Collect, store and retrieve information based on its spatial location;
(b) Identify locations within a targeted environment which meet specific criteria;
(c) Explore relationships among data sets within that environment;
(d) Analyze the related data spatially as an aid to making decisions about that environment.
(e) Facilitate selecting and passing data to application – specific analytical models capable of assessing the impact of alternatives on the chosen environment.
(f) Display the selected environment both graphically and numerically either before or after analysis".

GIS database: A pool of integrated and structured graphic and non-graphic data from which relevant facts may be retrieved and processed to provide information to users.

13.4.3 Digital Data for GIS

A G.I.S database can be divided into two basic types of data: graphic and non-graphic (attribute) data. Each of these types has specific characteristics and requirements for data storage, processing and display. In G.I.S, data consists of features or entities, which are represented by four geometric concepts i.e., points, lines, polygons and surfaces.

(a) Points: Points describe individual objects depicted by cartographic symbols e.g. Schools, hospitals, spot heights.
(b) Lines: Are used to describe networks e.g. roads, railways rivers canals, contours etc. Normally networks are assumed to have no area because their widths on maps are assumed to be zero.
(c) Polygons: Are used to describe area features such as zones of population density, administrative districts, lakes etc.

(d) Surfaces: Are a three-dimensional representation of features e.g. a Digital Terrain Model (DTM) of a hill.

13.4.4 Feature Definition in GIS

In the GIS database, any feature is defined by 3 parameters:

(a) Its position
(b) Its attributes; and
(c) Its relationships to other features.

Feature Position: Feature position consists of locational data defining a feature's spatial extent. It may be in either vector or raster formats. In vector format, locational data consists of Cartesian coordinates in some frame of reference e.g. UTM projection.

(a) Point: represented by a pair of coordinates
(b) Line: represented by series of coordinates
(c) Polygon: represented by a series of coordinates closing onto themselves
(d) Surface: represented by 3D coordinates.

In raster format:

(a) Point: represented by a single pixel whose value is different from the value of neighbouring pixels
(b) Line: represented by a series of contiguous pixels of the same value surrounded by pixels of different values.
(c) Polygon: represented by a 2-D extent of pixels of equal value
(d) Surface: represented y a 2-D extent of pixels of equal value, plus a height value at each pixel.

13.4.5 Feature Attributes

Feature attributes is data that specify the non-geometric characteristics of a feature. Attributes may be numeric (eg. size, slope, height etc) or semantic (eg. name, type, etc). Feature attributes in GIS are stored by means of feature codes. A feature code is a concise alphanumeric code which describes the type of feature represented by given coordinates according to some chosen coding scheme. A commonly used coding technique is that of generalization and specialization which arranges features hierarchically in a tree structure.

13.4.6 Feature Relationships

Spatial relationships i.e. neighborhood, connectivity and inclusively are not as apparent in digital data as in analogue data. Digital data cannot adequately represent the real world without relationships. There are three kinds of relationships:

(a) **Proximal-** Describe the closeness of non-adjacent features i.e. nearness or farness.
(b) **Hierarchical –** Describe the relationship of feature subclasses to their super classes. It is encoded by a tree structure.
(c) **Topological –** Describe the neighborhood, connectivity and inclusion properties of features. It is encoded via a topological data structure in a process called topology building.

13.4.7 Sources of Data for GIS Database

(a) Analogue maps and plans
(b) Digital remotely sensed imagery
(c) Surveying field notes
(d) Total Stations
(e) Photogrammetric work stations
(f) Aerial photographs
(g) GPS
(h) Tabular data e.g. Census, rainfall, soils etc. from various collecting agencies.
(i) Direct import from other GIS's

13.5 COMPUTER AIDED DESIGN (CAD)

13.5.1 Introduction

- The use of a computer to produce designs is referred to "computer Aided Designs (CAD)" sometimes known as "Computer Aided Drafting and Design (CADD)".
- CAD programs present objects as wire frame outlines or more complex three-dimensional (3D) images.
- CAD reduces the time needed to create, edit, store and transmit drawings by using high performance computers and monitors, with inputs devices like scanners and graphics tablets, sending their output mostly to laser printers or to pen plotters.

13.5.2 CAD capabilities

The capabilities of modern CAD systems include:

- Reuse of design components
- Ease of design modification and versioning
- Automatic generation of standard components of the design
- Validation/verification of designs against specifications and design rules
- Simulation of designs without building a physical prototype
- Automated design of assemblies, which are collections of parts and/or other assemblies

- Output of engineering documentation, such as manufacturing drawings, and Bill of Materials
- Output of design directly to manufacturing facilities
- Output directly to or Rapid Manufacture Machine for industrial prototypes

13.5.3 Tools and Methods

Development in CAD has resulted in the following tools and methods:

- Wire frames
- Solid modeling
- Intelligent wiring diagrams and production linked database systems
- Graphically represented system or plant diagrams and databases
- Parametric design models
- Real-time process simulation
- Computer Numerically Controlled (CNC)) load files (tool path instructions)
- Finite Element Analysis (FEA)
- Rapid prototyping

Many CAD drawings are created from scratch using the application software using design sketches and other inputs. Other CAD drawings are created from pre-existing electronic CAD files by copying all or part of another CAD file, making changes, then saving it as a new file. Drawings that only exist in physical form (blueprints, plots of lost files, etc.) can be converted into CAD files using a procedure called “Paper-to-CAD conversion”, drawing conversion, digitization, or vectorization.

13.5.4 CAD application in Water Supply services

All personnel involved in the water supply need to have knowledge of data processing through computer. In more recent years, the technological developments associated with the creation, manipulation, storage and communication of data and information, principally computing, telecommunications and electronics is now carried out through computers. These developments have given rise not only to the rapid evolution of data processing techniques but also to a greater integration of the data processing techniques with other activities in an organization. This calls for technical personnel in the water supply industry to have knowledge of data processing and its related information technology.

In a water supply system, the computer-aided designs are based on computer model network analysis that is widely used to identify the causes of deficiencies in a proposed system, thereby developing the most effective components. The network analysis of water systems will include a number of interdependent variables: -

- Water sources (Quality and quantity)

- Water demand (Consumption)
- Intakes (Capacity and type)
- Pipelines (Diameters, material and pressures)
- Treatment (Quality, quantity and methodology)
- Storage (Capacity, duration and type)

System efficiency and effectiveness, and hence system performance can be affected by altering various combinations of these variables. Comprehensive steady state analysis of the water supply systems should perform for a wide range of water demand conditions. However, if the water supply system can operate satisfactorily under the most severe demand conditions, it will operate satisfactorily for all conditions. For this reason, the demand condition most limiting to the performance of the water supply system variables will be established, and the computer model runs will represent the system operations at these most limiting demand conditions.

13.6 HYDRAULICS MATHEMATICAL MODELS APPLICATIONS

13.6.1 Models and Modeling Systems

The Use or numerical models as tools in engine{e_ring practice has increased rapidly over the Last decades. In fact, the major hydraulics laboratories of Western Europe now do more than 80% of their modeling work with mathematical models.

Modeling has nearly always been a part of engineering practice. Solving a particular river problem mainly requires construction of a model, this model could be a physical (scale) model or mathematical (numerical) model.

13.6.2 Applications of mathematical models

With the introduction of the digital computer it became possible to analyze and solve whole sets of governing equations using solution methods which, because of their large number and long Complex Calculations had been previously considered too difficult and unreliable due to the possibility or making (simple) mistakes in hand calculations.

- The application of these solution methods to the analysis and design of engineering problems is referred to as *numerical (mathematical) modeling.*
- Models canbe applied to an extremely wide range of problems. However, in order to keep the models as Mathematical simple and efficient as possible, particular fields of application can be identified for which different types of models should be used; e.g.- free surface flows
- Pressurized flows
- Transport and dispersion of pollutants or heat
- Groundwater flows

- Morphological computations – Short waves, etc...

13.6.3 Development of Mathematical Model

1. **Objective**

The choice of the objectives of the model determines the level of complexity of the system and the main parameters that are involved.

2. **Schematization**

The schematization selected depends upon the complexity of the processes involved and the availability of models, which can be used at expenses that can be justified by the value of the answers given. Consideration must be given to the variations in time (t) and space (x, y, z). The variables t, x, y, and z are referred to as independent variables. All real processes vary with variations in one or more of the independent variables. However, by carefully examining the particular process, assumptions can be made about the dimensionality of the problem.

As far as the number of dimensions in space is concerned a distinction can be made into: . ID (one dimensional model) . 2D (two dimensional model) . 3D (three dimensional model)

Furthermore models are distinguished into steady models and time dependent model

1 D models: *(variation in t and x) This type of models is often used for e.g. water flow, sediment transport and morphology, flows in tidal estuaries, control of irrigation systems, Simulation of dam break waves, flood propagation through channels and reservoirs.*

2D Models: *(variations in t, x, and y) This type of models is often used for e.g. sectional models of structures, models to study salt intrusion and water quality, tidal flows, detailed flows in rivers (e.g. circulations), short-wave modeling.*

3D Models: *(variations in t, x, y and z) This type of models is often used for e.g. study of discharge from power station of hot water plume into ocean, physically-based, distributed hydrological model.*

The next step in Schematisation is the choice of dependant variables. These variables are the unknown quantities which vary in time and space and whose values we wish to calculate by modeling e.g. flow (Q), depth (h), stage (H), Concentration(I), etc.

3. **Equations**

The variations in the dependent variables can be described by equations, which relate all the dependent and independent variables. The equations are generally derived from balances in control elements, e.g. mass, momentum, energy, heat, population balances. This leads, in general, to one or more ordinary or partial differential equations.

4. Solution Methods

There are many solution methods for differential equations. In hydraulic engineering numerical methods are most commonly used. These include: - finite difference, - finite element, boundary element methods,

5. Computer program

Once a solution method has been selected, a computer program has to be written. In this program the computer is instructed to execute all the steps or the solution method in the correct sequence and to realize input and Output of a wide variety of data. Tile data must be collected from prototype or experiment to help determine the values or model parameters and to determine the boundary conditions to be specified during the model simulations.

6. Calibration

A series of simulations with the model for different values of parameters leads to choice of parameter values, which give the best comparison between, measured and computed results.

7. Verification

After the model has been calibrated satisfactorily it should then be run once more with a completely new set of data for a final verification of the suitability of the model. This run should accurately reproduce the measured data without having to adjust any of the parameter values.

8. Simulation

Finally the model is ready for application to the objectives of the study, which was defined at the initiation of the model development.

13.7 CANDES (CANAL DESIGN)

13.7.1 Objectives

CANDES is a straight forward spreadsheet computational method used to solve canal X-sec. Design for a given flow, roughness and bed slope, through iterative computations.

This program is based on manning / strickler equations, used to solve earth canals.

Q = *A_R3S2*
Q=AkR3S2

Manning Formula

Strickler Formula

13.7.2 Data Inputs:

Q_des = design flow (m3/sec)

b/y – Corr = factor for tuning the results by adjusting the computed bid ratio
S_des = bed slope (m/km)

m_des = side slope (m) Ks = strickler coeff. = Un

Flag i = given values 1,2 or 3 according to maintenance
condition (see notes)

b_des = bed width (m)

13.7.3 Outputs:

V _avo = average velocity (m/sec) y = water depth (n1)
b/y = ratio between depth and width

Criteria for conveyance of suslJended sediment:

Pw gvav S

= *constant or none decreasing*

Criteria for conveyance of bed load tranSlJort:

I
d"2 S = constant or non decreasing

Boundary- shear stress:

T_CpgyS

Note: c = bid correction factor

13.8 PROFILE

13.8.1 Objectives

Profile is a computer program to calculate the dimensions and bed Slope of trapezoidal canals assuming uniform steady flow conditions, using strickler / Manning resistance formula, it computes any of the following variables when another variables are given by users;

13.8.2 Data Inputs:

Unknown variable(s)	Known variables
Q (discharge)	H,b,ks,m,S,b
h (water depth)	Q,b,ks,m,S,b
b (bed width)	Q,h,ks,m,S,b
Ks (roughness coefficient)	Q,h,b,m,S"b
m(side slope)	Q"h, b, Ks,S, b
S (bed slope)	Q,h,b, Ks, m, S, b
band h	Q,Ks, m,S, n*=(b/h)

13.8.3 Outputs:

In addition the program computes the following values;

Vav. = average flow velocity

T = Tractive force $T = P$ g h s
N/m2

E = energy dissipation in canal $E = P$ g v s

w/m3

13.9 FLOP (FLOW PROFILES)

13.9.1 Objectives

Flop is a windows computer program used to calculate the gradually varied flow profiles (positive and negative) in open semi prismatic channels, for the following conditions;

- Channel with one or more prismatic reaches,
- Trapezoidal and rectangular cross-sections,

- Steady sub-critical flow; for super-critical flow FLOP will suspend the calculation.
- Only friction losses are considered; local losses due to changes in Geometry, lateral flows and control sections are not included.

13.9.2 Data Inputs:

Schematisation

To model a channel reach, parameters related dimensions of that reach are required and they Include:

the
./ Length L in m;
,/ Bottom width b in m;
,/ Side slope m (I vertical: m horizontal);
,/ Bed slope S, in mlm;
,/ Bottom elevation z in m. above a reference level. ,/ Roughness coefficient can be defined either by Chezy or as defined by Manning.! Strickler, (k for Chezy, n or ks' Manning and Strickler respectively).

13.10 EPANET 2

13.10.1 Objectives

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behavior within Pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs.

EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of chemical species throughout the network during a simulation period comprised of multiple time steps, in addition water age and source tracing can be simulated.

13.10.2 Modeling Capabilities:

EPANET Hydraulic Modeling capabilities:

- No limit on the size of the network that can be analyzed -Computes friction head loss using the Hazen-Williams, Darcy Weisbach, or Chezy-Manning formulas.
- Includes minor head losses for bend fittings, etc.
- Models constant or variable speed pumps
- Computes pumping energy and cost
- Models various types of valves
- Models any shape of storage tanks (varying diameter with height). Considers multiple demand categories at nodes, each with its own pattern

of time variation
- Can base system operation on both simple tank level or timer controls and on complex rule-based controls

13.11 SEWER (CIVIL DESIGNER)

13.11.1 Objectives

Sewer is a foul water network analysis and design module, which combines interactive drawing functions with powerful design functions.
The DBase file format allows rapid data access — finds an entry in less than 0.25 seconds in a dataset of 20,000 records — and an un-limited model size. It also ensures compatibility with most geographic information systems and other programs.
The graphical functions allow you to easily insert nodes and links, make changes to an existing layout by moving a node, change the flow direction or query a node or link with a click of the mouse.
Manholes and culverts are displayed in WYSIWYG mode with user-defined line styles, colours and annotations such as manhole name, inlet level, depth, link type, link size, flow, velocity and capacity.
Combine new and existing culverts, consisting of multiple types and sizes, in one model. You can easily simulate the complex system of flows and overflows that are often found in modern towns and cities.
The design will take into account the hydraulic and geometric relationship between the different branches in the network. Attenuation is calculated at every manhole resulting in a solution, which is 30% to 100% more cost-effective than simply adding the inflow peaks together.
The dynamic time simulation (unit inflow hydrographs) allows you to view the flow hydrographs of single or multiple culverts in order to gain an insight into the dynamics of the system.
Once you have completed the analysis the network can be displayed according to specific criteria such as flow or calculated size much as a GIS system would.
The flexible pumps option allows you to simulate pump stations with up to 4 pumps.
Thereafter you can generate construction drawings such as layout plans and long sections or calculate quantities. The ground profile is automatically extracted from the supporting digital terrain model.
The graphical functions make Sewer ideal for the data capture of foul water GIS models. You can transfer the data between Sewer and our MAP module at any time.

13.11.2 Sewer Functions

Design Parameters
- Minimum and maximum velocity
- Proportional flow depth
- Minimum/maximum culvert slopes
- Drops, defined steps, inverts common or soffits common in manholes

- Supports pipes, box culverts, portal culverts, and custom culverts such as egg-shaped sewers
- Define culvert types, sizes and flow characteristics
- Fix culvert inlet level, slope or size
- Limit the flow in any culvert and direct the excess flow to another manhole
- Stormwater infiltration as a percentage of inflow

Unit Flow inflow method

- 10 inflow classes with average daily flow and equivalent population per unit
- Attenuation curve
- Up to 5 inflow classes per manhole plus point source inflow

Harmon Formula inflow method

- Design unit flow
- Contributors per unit

Unit Inflow Hydrograph

- Dynamic time simulation spans up to seven days
- Up to 5 unit inflow hydrographs per manhole
- Continuity or Time Shift routing
- View the flow hydrographs of single or multiple culverts

Pumps

- Pump station with up to 4 pumps with different characteristics and control levels
- Specify pipe diameter, roughness and minor loss coefficients

Quick Long Sections

- Scroll through branch sections
- Display up to 3 ground lines and selected design data
- Graphically move manholes up or down

Analysis and Design

- Comprehensive network analysis
- Auto numbering of branches and manholes
- Analyze the entire network, a sub-network or just a branch
- Flows attenuated according to inflow method
- Flow calculations use internal culvert dimensions for hydraulic accuracy
- Calculates culvert size, slope, inlet and outlet level, flow, capacity, velocity, flow depth and hydraulic grade

Quantities (SABS 1200)

- User-defined trench widths and depth increments
- Excavation length and volume
- Bedding, backfill and selected backfill volumes
- Manhole depths
- Culvert lengths
- Sorted by culvert type and size

Drawings

- Long sections of single or multiple culverts with crossing pipes on single or multiple sheets
- Plan layout with manhole and culvert annotations, schedule, manhole coordinate and

connectivity tables

- Annotations such as manhole name, cover level, invert level, depth, culvert type, size, length, slope, flow, capacity, velocity and flow depth

13.12 CAD ADVANTAGES

(i) Operation speed – The timing aspect of the information is very important, because late information is not better than no information. Automatic processing as provided by the computer quickens the operations on the input data to produce timely information.

(ii) Accuracy – The accuracy and preciseness of the information is increased by the use of the computer, which remove use of illegible hand written entries.

(iii) Volume – Computer can handle and process large volumes of data in very short periods of time, usually measured in microseconds.

(iv) Convenience – Repeated operations that may be boring and tedious normally, can be conveniently processed by the computer.

(v) Linkage and Flexibility – The versatile nature of the computer allows it to operate with a common data pool that supports several applications as long as the relevant programs are available. The software used on a given computer is relatively easy to change and it is that capability which gives computers their flexibility of purpose.

13.13 CAD DISADVANTAGES

(i) The user does not have direct control over the package in the same way, as would be the case if the software were produced in-house for local conditions.

(ii) The package will have been produced to meet general needs and may not be ideal for particular customer.

(iii) No matter how good the CAD system within an organization are, if the other aspects of the organizations information system are poor, the information produced may be under-utilized to the detriment of the organization.

(iv) It is unfortunately not uncommon for programs to be supplied, which may be faulty in some way. Sometimes these faults, called bugs, do not come to light until some time after the software has been delivered and put to use.

14 DRAWINGS

14.1 SCALES

14.1.1 General

The following reduced scales may be used (after ISO 5455 –1979 (E). Further see section 15.6 and 15.7.

1 : 2	1 : 5	1 : 10
1 : 20	1 : 50	1 : 100
1 : 200	1 : 500	1 : 1000
1 : 2000	1 : 5000	1 : 10000
etc	etc	etc

14.2 FORMAT

14.2.1 Construction, Standard and Type Drawings

A1 (840 x 594 mm) is preferred for all drawings, which are to be used on site for construction.

A3 (450 x 297 mm) may also be used for type drawings.

Building and Civil Engineering drawings – symbols for concrete reinforcement ISO 3766 – 1977 (E) Appendix E

14.2.2 Title Block

The title block of drawings made by departmental design and consultants should be as shown in Appendix E.

14.3 BAR SCHEDULING

14.3.1 Building and Civil Engineering Drawings – Bar Scheduling

ISO 4066 – 1977 (E) Appendix E.

14.4 WATER SUPPLY DRAWINGS

14.4.1 Water Supply General

As shown in appendix E

14.4.2 Geotechnical Drawings

As shown in Appendix E

14.4.3 Installation Drawings

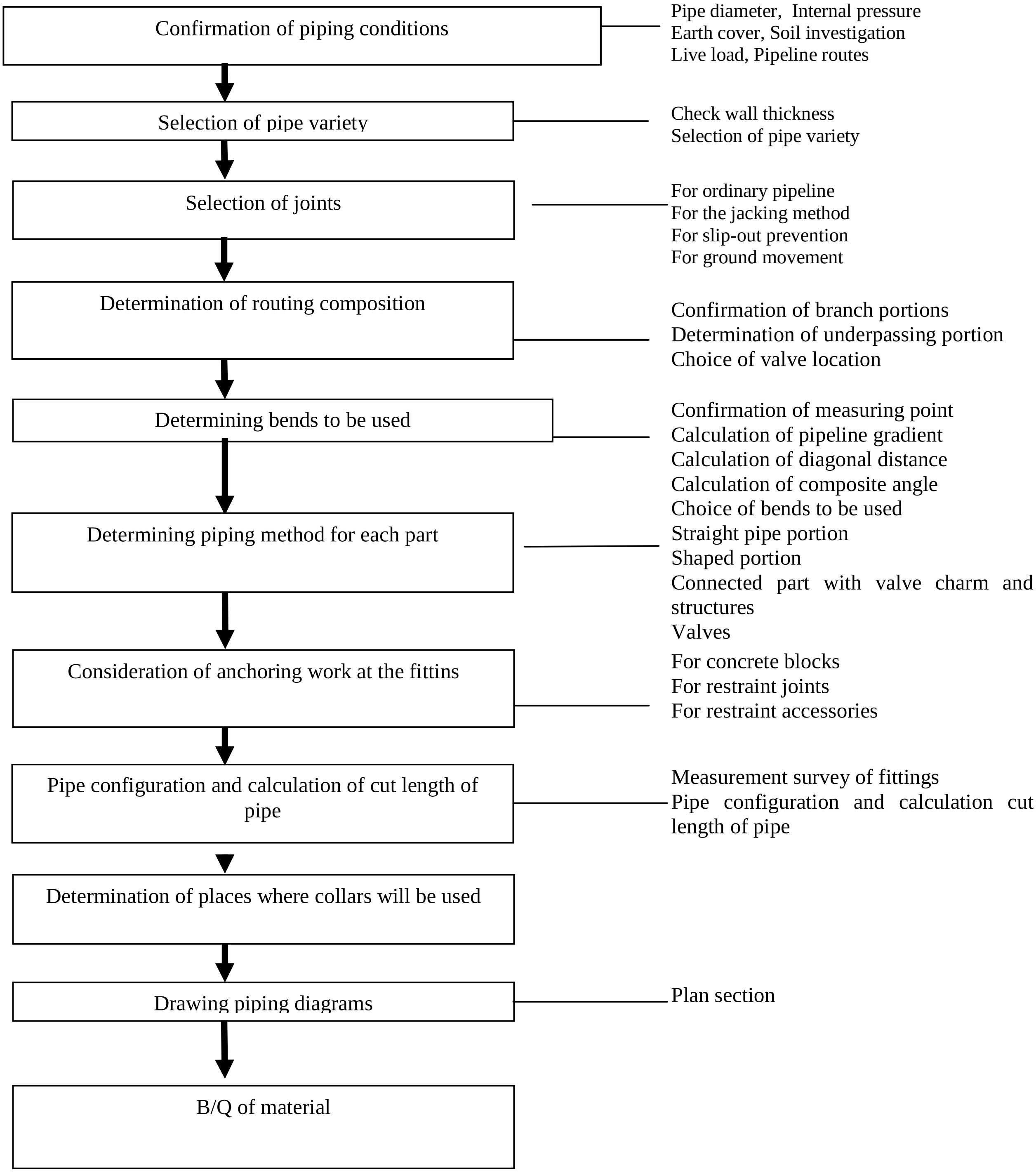
As shown in Appendix E

14.5 PIPELINE DRAWINGS

14.5.1 Plan and Profile Drawings of Pipeline

- Plan and profile shall be drawn on the same sheet and to the same horizontal scale. The plan shall be placed above the profile.
- The horizontal scale shall be 1:2000 and the vertical scale 1:200 except when more detailed drawings are required.
- Chainage of pipelines to start at the inlet and run in the same direction as the flow.
- The profile shall show ground levels of all surveyed points and invert levels at points where the gradient changes.
- All levels to be given to two decimals of metre and the chainage to the metre only.
- The profiles should also show static head, the hydraulic gradient in percent, pipe material size and class, design flow and ground material.
- The plans shall show enough features of the terrain to make it possible to find the surveyed line at the time of the construction. At critical points it may become necessary to shown the distance in metre between the center line of the pipeline and well defined objects. North direction line should be shown on all plan drawing.
- A general Layout Plan to suitable scale covering the whole supply system shall be provided. The plan should show pipe dimensions and classes, reservoir capacities and section valves. References should be made to the detail pipeline drawings
- An overview profile drawing to suitable scale covering the main pipeline routes. The drawing should show the gradients, the static pressures and the pipe size, material class.

14.5.2 Flow chart for drawing procedure of piping diagrams



14.6 BUIDLING AND STRUCTURAL DRAWINGS

14.6.1 General

- Reinforced concrete structures shall be presented on separate drawings for the shuttering, the reinforcement and the installation of pipes and mechanical equipment.
- Shuttering and reinforcement drawings should be to scale 1:20 or 1:50. Details should be to scales as required.

14.6.2 Shuttering

- All measurements should be shown on this drawing, and must not be shown more than once to minimize the risk for mistakes at later revisions.
- Foundation details like hardcore depth, thickness of blinding and existing ground levels must be indicated.
- All construction joints and box outs should be shown

14.6.3 Reinforcement

- Regarding symbols and bar scheduling see section 15.4.4 and 15.5.1
- All data required to identify dimensions, shape and position of steel bars shall appear.
- Reference should be given to separate bar bending schedules.

14.6.4 Installations

- All fitting should be given position number and must be clearly marked with dimension, material and measurements.

14.7 STANDARD DRAWINGS

14.7.1 Definitions

The Ministry has set up the following conditions which have to be fulfilled before a drawing can be classified as a standard drawing.

- The design shall be possible to use for several different projects.
- The design shall be of high professional standard
- The design shall comply with recommendations by Kenya Bureau of Standards or with the standards used by the Ministry. SI – units shall be used exclusively.
- The design shall have been discussed with parties involved in the construction and the operation of the design element.

- The design shall be sufficiently detailed to allow construction work to be carried out without additional design work.

The Design Standardization section shall do the marking and registration of standard drawing.

14.7.2 Available Standard Drawings.

The Ministry issues a separate booklet with all approved standard drawings reduced to format A3.

14.8 TYPE DRAWINGS

14.8.1 Definitions

The same as for Standard Drawings except that condition No.5 is omitted.

14.8.2 Available Type Drawings

A separate booklet with all approved type drawings in format A3 is issued by the Ministry.

15. COST ESTIMATES

15.1 GENERAL

15.1.1 Purpose

The cost estimate is used as an instrument for:

- Economic planning
- Selection of technical solutions
- Control of construction costs and consultancy fees.

15.1.2 Methodology

The method for cost estimations will vary with the design stages. The later in the design process the more detailed will the cost estimate be. Further see section 16.2. The basis for the cost estimates must always be actual tender rates whenever available.

15.1.3 Price Changes

The cost estimates must be based on the price level at a particular time, which shall be clearly stated for all estimates.

When necessary the estimated cost can be updated to present-day cost by applying escalation factors. These factors are determined by Central Bureau of Statistics Ministry of Finance and Planning. The monthly civil engineering cost indices are published quarterly in Kenya Statistical Digest and an extract is shown in table 15.1

Table 15.1 CIVIL ENGINEERING COST INDEX

Date	Index	Price Increase %	Date	Index	Price Increase %
Dec. 1972	100.0	-	Dec. 1982	362.6	19
Dec. 1973	119.0	19	Mar. 1983	363.9	
Dec. 1974	157.4	32	Jun. 1983	375.9	
Dec. 1975	175.9	12	Sep. 1983	378.4	
Dec. 1976	190.3	8	Dec. 1983	379.5	5
Dec. 1977	202.9	7	Mar 1984	379.5	
Dec. 1978	216.8	7	Jun. 1984	379.5	
Dec. 1979	245.6	13	Sep. 1984	379.5	
Dec. 1980	282.7	15	Dec. 1984	460.0	21
Dec. 1981	305.6	8			

Example:

The cost escalation between December 1979 and December 1981 was

$$\frac{(305.6 - 245.6) \times 100}{245.6} = 24.4\%$$

- The Civil Engineering Cost Index may not always represent the changes of the cost of water supply well as the pipeline costs represent such a large portion of the total cost
- The price of pipes, in particular UPVC-pipes, should therefore be studied separately by obtaining rates from the latest tenders.
- The building cost index may also be of use for water supply. This index series as well as that of civil engineering comprises detailed information on the cost escalation of common building materials and labour.

15.2 CONSTRUCTION COSTS

15.2.1 Pre-Preliminary Design Stage.

Only very rough estimates can be made based on population figures, the area of supply or the water demand.

The unit costs given in Appendix F may be used.

15.2.2 Preliminary Design Stage

The construction cost of the project shall be estimated and presented as an itemized schedule of units, unit rates and cost of each component of the project. The estimate shall be detailed as much as possible with regard to available data. The following summary will be suitable for many projects but should be modified to suit the individual scheme.

1. Intake works
2. Raw-water main
3. Treatment works
4. Pumping stations
5. Storage tanks
6. Distribution lines
7. Water points
8. Access roads

   Subtotal

9. Preliminaries

   Subtotal

10. Contingencies

   Total

- The preliminaries shall include cost for site camp, insurance, temporary works, setting-out, material testing, superintendence, transport, etc. This amount shall be assumed to be 15% of the sum of 1-8 above.

- The contingencies should cover items which cannot be foreseen in the preliminary design stage but which are likely to become necessary in the final design. The amount shall be assumed to be10% of the sum of 10% of the sum of 1-9 above.

- Unit costs of pipes, pipelines, treatment works, storage tanks and a few other items are given in Appendix F. The prices were valid in late 1983 and should be updated to present-day prices as stated in section 15.1.3.

  The unit costs should be used as indicative only and may have to be modified for the particular scheme. Reasons for modifications may be e.g. difficult ground condition (swamp, rock), simplified treatment, remote locations etc.

  It should be noted that the unit-cost data cover far from all items that may be part of a water supply. Remaining costs should be obtained from manufacturers, actual tenders etc. as required.

  The unit costs given in the Appendix may only be used for rough estimates at the preliminary design stage.

15.2.3 Final Design Stage.

- The cost estimate shall be based on the draft bills of quantities and detailed unit rates obtained from actual tenders, manufacturers quotations etc. and updated to present-day prices valid at the time for the submission of the FD-report.
- The preliminaries shall be assumed to be 15% of the itemized subtotal and there shall be a provision for contingencies of 10% [limit to zero by Treasury April 8]

15.2.4 Tender Stage

- The estimate shall be the Engineer's Estimate and be based on the definite bills of quantities.
- The unit rates shall be obtained as for the final design stage but they should be updated to represent the price level at the expected time of construction.

15.2.5 Supervision Stage

- An estimate of the total construction cost should be made based on the contract price and the expected price escalations during the construction period.

- When the construction work is completed and all payments settled should the actual construction cost be calculated and compared with the estimates made in the different stages. The difference should be analysed and used as feedback.

15.3 MAINTENANCE COSTS

15.3.1 Annual Maintenance Costs

The annual maintenance costs should be calculated as a percentage of the construction (installation) cost as shown in table 15.2, which also shows the economic lifetime for various assets.

Table 15.2: Annual Maintenance Costs

Asset part	Economic lifetime years	Annual maintenance cost in %
Dams	40	0.5
Intake works, including boreholes: Mass concrete structures, such as intakes, underground pits, culverts, etc	40	1
Earthworks generally	40	1
Boreholes and wells	20	1
Pumps:		
Hydrams and hydrostats	15	5
Other pumps	10	5
Power:		
Diesel engines	10	5
Engine and pump sets petrol paraffin	5	5
Electric motors, cables and switch gears	10	5
Piping:		
All types	30	1

Treatment works:		
Treatment works in masonry or reinforced concrete	30	1
Reservoirs:		
Storage tanks in masonry or reinforced concrete	30	1
Storage tanks, sectional steel incl. towers	20	2
Storage tanks, corrugated galvanized steel (C.G.S.) on timber stands	10	2
Building:		
Building C.G.S. on timber	20	1
Building, masonry	30	1
Miscellaneous and items:		
Communal water points (CWP)	10	5
Water kiosks, latrines, licenced retailer points etc.	20	2
Gantries, steelwork etc	20	2
Permanent Tools and plant not mentioned elsewhere	10	2
Water meters	10	5
Chemical Apparatus:		
Chemical dosing gear	10	5
Instruments and testing apparatus	5	5
Roads, fences etc.		
Roads of access, general	30	1
Fences, G.S. wire or mesh on timber	10	1
Fences, G.S. wire or mesh on concrete posts	20	1

15.4 OPERATION COSTS

15.4.1 Manpower Costs

- The number of staff of different categories and salary scales can be found in Appendix F for different types and sizes of water supply.
- The 2004 civil service salaries for different job groups are given in Appendix F.

15.4.2 Energy Costs

- The price of diesel and electricity should be obtained from the O&M Branch and the E.A. Power & Lighting Company respectively.
- Note that the price of diesel varies from area to area.

- Note that E..A.P&L has different methods for charging for power depending on the consumption. The charges comprise three parts, namely:
  a fixed charge per reading period
  a charge per unit of electricity supplied
  a charge per KVA of **maximum** demand during the reading period

- The electricity rates are usually given per KVA. For cost estimate purpose it may be assumed that IKW = 0.80 KVA

15.4.3 Chemical Costs

- For the purpose of cost estimates the following dosage rates may be assumed unless jar tests or other experience indicate different rates:

Alum	0.05 kg/m^3 – water
Soda ash	0.02 kg/m^3 – water

Tropical
Chloride
of lime 0.002 kg/m^3 - water

Unit rates at the price level of late 2004 are given in Appendix F.

15.5 MISCELLANEOUS COSTS

15.5.1 Land Acquisition

Should be estimated after consultation with the Government valuer.

15.5.2 Electricity Power Transmission Line

Connection to the existing grid system should be estimated in conjunction with EAP&L.

15.5.3 Design Costs

Only external design cost should normally be considered. The approximate costs for different stages are shown in table 14.3.

Table 14.3: Design Costs

Cost of the Works	Design cost as a percentage of the cost of works for each design stage				
Million shillings	PD	FD	TD	Survey 1) 2)	Supervision in-chief
Up to 0.5	5	5	2		5
0.5 to 1.0	4	4	1		4
1.0 to 2.0	3	3	1		3
2.0 to 5.00	2.5	2.5	1		2.5
5.0 to 10.0	2	2	0.5		2
10.0 to 20.0	1.5	1.5	0.5		1.3
20.0 to 40.0	1.5	1.5	0.5		1.5
Over 40.0	1.2	1.2	0.4		1.2

1. The land survey cost should be estimated with a unit rate per Km pipeline. See Appendix F.

2. Costs for special investigations e.g. groundwater investigation

15.5.4 Supervision Costs

- The cost of supervision in chief is related to the cost of works as shown in table 16.3.
- The cost of the Resident Engineer should be assumed to be the maximum allowable salary in accordance with the regulations of the Ministry.

15.5.5 Establishment Costs

Establishment charges which are a contribution to the expenses of the Ministry in District, Province and in Nairobi may be assumed to be 35% of the direct costs calculated as described in Section 16.3 and 16.4

15.5.6 Depreciation Costs

The depreciation should be assumed to be on a straight line basis and be based on the economic lifetime shown in table 16.2 for various assets.

15.5.7 Capital Costs

Real loan charges i.e. interest and amortization should not normally be considered in the economic analysis of a particular project.

However the annual capital cost should be estimated assuming a nominal interest rate. A rate of 8% is reasonable for 1983 but the rate has to be determined from time to time. The annuities for different lifetime and discount rates are given in Appendix F.

15.6 FOREIGN AND LOCAL COMPONENTS

15.6.1 Pipelines

The direct foreign currency component of the pipeline cost inclusive material, sales tax, duty, transport, handling, storing, trenching, laying, testing, ancillary cost and preliminaries can be assumed to be:

- 30% for locally manufactured UPVC pipelines
- 40% for imported DI and CI pipelines
- 30% for locally manufactured steel pipelines

15.6.2 Mechanical and Electrical Equipment

The direct foreign currency component of installed or erected mechanical equipment inclusive material, duty, sales tax, transport, handling, storing, mounting and preliminaries can be assumed to be: 50%.

15.7 REVENUE

15.7.1 Present-day Tariff

- The revenue should be estimated using the current structure of the tariff as issued in Kenya Gazette.

- Note that the price will depend on location, whether the water is metered or not, type of use (domestic or industrial) and whether the water is sold to a retailer or direct to a consumer.

- It should be assumed that 75% of the produced water is sold to and paid for by the consumers, 25% being leakage and wastage, and uncollected bills.

15.8 SELECTION CRITERIA AND PRESENTATION OF COSTS

15.8.1 Present Value Analysis

- The economic comparison of alternative technical solutions should normally be made by discounting to present value all investment, operation and maintenance costs.

- The present values of future payments for discount rates 1 to 10% are shown in Appendix F. Generally, the sensitivity of the economic comparison to different discount rates should be tested by making calculations for different discount rates e.g. 6%, 8%, 10%.

15.8.2 Presentation of Costs

The following costs should be computed and presented in all preliminary design and final design reports.

- Investment costs for all implementation phases.
- Investment costs per capita calculated for the initial, future and ultimate population.
- Investment costs per km^2 of supply area and per m^3 water of daily ultimate output.
- Operation and maintenance costs for the initial, future and ultimate year.
- The production costs per m^3 water for initial, future and ultimate year considering only direct operation and maintenance costs.
- The production costs per m^3 water for initial, future and ultimate year considering direct operation and maintenance costs, establishment costs and annual capital costs.
- Miscellaneous costs e.g. land acquisition, connection to the electricity grid, design costs, supervision costs and loan cost when relevant.

15.9 Cost of establishing Monitoring and Evaluation System

15.9.1 General

M & E system should be estimated at 2-3 % of the total project cost. The Budget allocation should be 50 % for Data collection and 50 % for Data analysis.

16 STANDARDS

16.1 GENERAL

16.1.1 Preferred Standards

- Locally manufactured items shall be to Kenya Bureau of Standards (KS) Specification. Where a KS Specification is not published then the items should meet the requirements of the International Standards Organisation (ISO). Where neither a Kenya Standard nor a ISO Specification are published then the locally manufactured item should be in accordance with the relevant British Standard Specification (BS) or other National Standards.

- Imported items should meet the requirements of the International Standards Organization (ISO). Where an ISO Specification is not published then the item should be in accordance with the requirements of the National Standards of the Country of origin (i.e. BS for British Manufacture, DIN for West German Manufacture etc) with the proviso that

  i. the Standards Specification lays down requirements not less than those required by the British Standards Institution

  ii the Standards Specification exists in official English translation

16.1.2 New Standards

The list of KS, BS and ISO standards are expanded continuously and old standards are revised. It is therefore important that design engineers keep up to date with the development.

16.1.3 Standards for Material used in Existing plants

For existing plants where other standards than KS were used originally, the same standards may be kept for the completion and extension work.

16.2 STANDARDS RELATING TO WATER SUPPLY

16.2.1 Standards valid in 2005

KS, BS and ISO Standards, which can be expected to have some application in water supplies, can be found in Appendix C.

16.3 UNITS

16.3.1 SI – Units

Only SI units may be used on drawings and in documents prepared by or for the Ministry. The conversion took place in 1974. (WD/2/1/147 26 September 1974).

A short description of the SI system follows below:

1. There are seven basic units of which the following are most common.

For Length	Metre	-m
Mass	Kilogram	-kg
Time	Second	-s

2. Derived and supplementary units to be used are:-

Area	Square metre	-m^2
Volume	Cubic metre	-m^3
Density		-kg/m^3
Velocity	Metre per second	-m/s
Force	Newton	-N
Pressure stress	Pascal	-Pa
Power	Watt	-W
Temperature	Degree Celcius	-^0C
Angle	Degree	-^{0}

3. Prefixes will be used as an alternative method for writing multiples and submultiples e.g.

One Million	10^6	-Mega - M
One Thousand	10^3	-Kilo – K
One Thousanth	10^{-3}	-Milli - m

4. The following table illustrates the commonly used units, their recommended multiples and sub-multiples together with conversion factors between the SI system and the Imperial system.

Table No. 16.1

TABLE OF UNIT SYMBOLS, MULTIPLES & SUB MULTIPLES

QUANTITY	UNIT SYMBOL	MULTIPLES AND SUB-MULTIPLES	CONVERSION FACTORS
Length Mass Time	m kg s	Km, mm Mg. G. mg Hour (h) day (d)	1m = 3.281 lt = 39.370 inches 1k = 2,205 Ib = 35.274 oz 1h = 3,600 s
Temperature Angle Area	^{0}C a m^2	Minute (‘) Second (“) Km^2, mm^2	^{0}C = 0.556 (^{0}F – 32) (10 = 60^0) Note: 360^0 circle $1m^2 = 10.764\ lt^2$ 1 acre = $4{,}046.86m^2$
Volume Density Force	M^3 Kg/m^3 N	Mm^3, dm^3, 1 G/ml KN, MN	$1m^3 = 35.315\ lt^3$ – 219.969 galls U.K. 1/ = $idm^3 = 0.001m^3$ 1 $kg/m^3 = 0.062428\ ib/lt^3$
Pressure stress Velocity Power	 Pa m/s w	MPa, kPa, MN/m^2, N/mm^2 km/h MW, kW	1Pa = $1N/m^2$ = 10.197 x 10-6 kp/cm^2 = 10^{-5} bar = 0.145 x 10^{-3} ibl/m^2 (psi) 1 m/s = 3.281 lt/s = 2.237 miles/h 1N = 1Nm/s = 1.341 x 10^{-3} hp (U.K.) =3.412 Btu/h

Note that NO plural “s” shall be added to any symbol e.g. write 10km NOT 10kms.

5. Further conversion factors from UK-units and vice versa are given in Table 11.2

16.4 FORMATS

16.4.1 General

The following formats may be used for reports and drawings.

Code	Dimension
A1	594 x 840mm
A2	420 x 594 mm
A3	297 x 420 mm
A4	210 x 297 mm

16.4.2 Preferred Formats

See under chapters “Drawings” and “Design Reports”.

16.5 PIPES

16.5.1 UPVC Pipes

Table No. 16.2 shows the dimensions of UPVC pressure pipes of the Metric series and pipe sizes preferred by the Ministry.

Table No. 16.2: Dimensions of UPVC pressure pipes

NOMINAL OUTSIDE DIAMETER mm	OUTSIDE DIAMETER mm		WALL THICKNESS mm							
			CLASS A P = 0.60 MPa		CLASS B P = 0.90 MPa		CLASS C P = 1.2 MPa		CLASS D P = 1.5 MPa	
	min	max	min	max	min	max	min	max	min	max
* 20	20.0	20.3	-	-	-	-	1.40	1.80	1.40	1.80
25	25.0	25.3	-	-	-	-	1.40	1.80	1.60	2.00
* 32	32.0	32.0	-	-	1.40	1.80	1.70	2.10	2.10	2.60
40	40.0	40.3	-	-	1.60	2.00	2.10	2.60	2.60	3.10
50	50.0	50.3	1.40	1.80	2.00	2.40	2.60	3.10	3.20	3.80
* 63	63.0	63.3	1.70	2.10	2.50	3.00	3.30	3.90	4.10	4.80
75	75.0	75.3	2.00	2.40	3.00	3.50	3.90	4.50	4.80	5.50
* 90	90.0	90.3	2.40	2.90	3.60	4.20	4.70	5.40	5.80	6.60
* 110	110.0	110.4	3.00	3.50	4.40	5.10	5.80	6.40	7.10	8.00
125	125.0	125.4	3.40	4.00	5.00	5.70	6.50	7.40	8.00	9.00
140	140.0	140.4	3.80	4.40	5.50	6.30	7.30	8.30	9.00	10.30
* 160	160.0	160.5	4.30	5.00	6.30	7.20	8.30	9.40	10.30	11.60
200	200.0	200.6	4.80	5.50	7.10	8.00	9.40	10.60	11.60	13.00
* 225	225.0	225.7	5.40	6.20	8.00	9.00	10.50	11.80	13.00	14.50
250	250.0	250.8	6.00	6.80	8.90	10.00	11.70	13.10	14.40	16.00
* 280	280.0	280.9	6.70	7.60	9.90	11.10	13.10	14.70	16.20	18.10
* 315	315.0	316.0	7.60	8.60	11.20	12.60	14.70	16.40	18.20	20.20
355	355.0	356.1	8.50	9.60	12.60	14.10	16.60	18.50	20.50	22.80
400	400.0	401.2	9.60	10.80	14.20	15.90	18.70	20.80	23.10	25.70
450	450.0	451.4	10.80	12.10	16.00	17.80	21.00	23.30	26.00	28.80
500	500.0	501.7	12.00	13.40	17.70	19.70	23.40	26.00	28.90	32.00
560	560.0	561.9	13.40	15.00	19.90	22.10	26.20	29.00	32.30	35.80
630	630.0	632.0	15.10	16.90	22.30	24.80	29.40	32.60	36.40	40.30

- Preferred pipe sizes

16.5.2 Designation of UPVC Pipes

The UPVC pipe shall be designated by its nominal outside diameter and the lettered pipe class in accordance with Kenya Standards as below:

UPVC 63	A
UPVC 90	B
UPVC 110	C
UPVC 315	D

The title drawing (or the first profile drawing) of each project shall have the following explanatory note:-

UPVC PIPES TO KS 06 - 149		
Class	Maximum MP2	Working Pressure Metre Water Head
A	0.6	60
B	0.9	90
C	1.2	120
D	1.5	150

16.5.3 Steel Pipes. Seamless or Welded

Table No. 16.3: Seamless or welded Steel pipes

ISO 559 - 1977		BS 534 : 1981		
ND mm	Outside diameter mm	Outside diameter mm	Wall thickness mm	Test pressure Mpa
40	48.3 Test pressure Minimum 5.0 Mpa.	-	-	-
50	60.3	60.3	2.9	7.0
65	76.1	76.1	3.2	7.0
80	88.9	88.9	3.2	7.0
100	114.3	114.3	3.6	7.0
125	139.7	139.7	3.6	7.0
150	168.3	168.3	3.6	6.0
200	219.1	219.1	4.0	5.1
250	273.0	273.0	4.0	4.1
300	323.9	323.9	4.0	3.5
350	355.6	355.6	4.5	3.6
400	406.4	406.4	4.5	3.1
450	457	457	5.0	3.0
500	508	508	5.0	2.7
etc				
To 1000 fore class A,B,C and To 2200 for class U		Up to 2220 mm		Down to 1.8

16.5.4 Locally Manufactured Steel Pipes

The locally manufactured steel pipes are not exactly to BS or ISO. The data for these pipes are shown in the table below.

Table No. 16.4: Locally manufactured Steel pipes

Diameter		Effective	Wall	Weight	Works Test	Working
Nominal (mm)	Outside (mm)	Length (m)	Thickness (mm)	Coated & lined kg/m	Pressure Mpa	Pressure MPa
100	114.3	6	3.2	11.4	7.0	4.7
150	168.3	6	4.0	20.1	7.0	4.7
200	219.1	6	4.0	26.2	6.4	4.3
250	273.0	8/10	4.0	32.8	5.2	3.5
300	323.9	8/10	4.0	39.0	4.4	2.9
350	355.6	8/10	5.0	54.1	5.0	3.3.
400	406.4	10/12	5.0	61.9	4.3	2.9
450	457.2	10/12	5.0	69.8	3.9	2.6
500	508.0	10/12	5.0	77.6	3.5	2.3
550	558.8	10/12	6.4	104.3	4.0	2.7
600	609.6	10/12	6.4	113.9	3.7	2.5
650	660.4	10/12	6.4	128.5	3.4	2.3
700	711.2	10/12	6.4	138.4	3.2	2.1
750	762.0	10/12	6.4	148.5	3.0	2.0
800	812.8	10/12	7.1	172.2	3.1	2.1
850	863.6	10/12	7.1	183.1	2.9	1.9
900	914.4	10/12	7.1	193.9	2.7	1.8
950	965.2	10/12	7.1	212.1	2.6	1.7
1000	1016.0	10/12	7.1	223.3	2.5	1.7

16.5.5 Polyethylene Plastic (PE) Pipes for Water Supply (low and high density)

The locally manufactured Polyethylene Plastic Pipes are to KS 06-478 Part I and II. The data for these pipes are shown in the tables below:

Table 1: Dimensions of Low Density Polyethylene Pipes

Nominal outside diameter (mm)	Outside Diameter (mm)		Wall Thickness (mm)									
			Class 1 P=0.25Mpa		Class 2 P=0.4 Mpa		Class 3 P=0.6Mpa		Class 4 P=0.8 Mpa		Class 5 P=1.0Mpa	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
12	12	12.3										
16	16	16.3							2.2	2.7	2.8	3.3
20	20	20.3					2.1	2.6	2.8	3.3	3.4	3.9
25	25	25.3					2.7	3.2	3.5	4.0	4.2	4.8
32	32	32.4			2.4	2.8	3.4	4.0	4.4	5.1	5.3	6.1
40	40	40.4			3.0	3.5	4.3	5.0	5.5	6.3	6.7	7.6
50	50	50.5	2.4	2.9	3.7	4.3	5.3	6.2	6.9	7.9	8.3	9.5
63	63	63.6	3.0	3.5	4.7	5.4	6.8	7.7	8.1	9.8	10.5	11.2
75	75	75.7	3.6	4.2	5.6	6.4	8.0	9.1				
90	90	90.8	4.3	5.0	6.7	7.6	9.6	10.9				
110	110	110.1	5.2	6.0	8.2	9.2	11.8	13.2				
125	125	126.2	6.0	6.8	9.3	10.5						
140	140	141.3	6.7	7.6	10.4	11.7						

P = Pressure

Note: Wall thickness of pipes are based on working stress of 2.5Mpa at 20^0C.

Table 2: Dimensions of High Density Polyethylene Pipes

Nominal outside diameter (mm)	Outside Diameter (mm)		Wall Thickness (mm)									
			Class 1 P=0.25Mpa		Class 2 P=0.4 Mpa		Class 3 P=0.6Mpa		Class 4 P=0.8 Mpa		Class 5 P=1.0Mpa	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
20	20.0	20.2							1.4	1.7	1.8	2.1
25	25	25.2					1.1	1.5	1.9	2.2	2.3	2.7
32	32	32.3			1.2	1.6	1.8	2.2	2.4	2.8	2.9	3.4
40	40	40.3			1.5	1.9	2.3	2.7	3.0	3.5	3.6	4.2
50	50	50.5			1.9	2.3	2.8	3.4	3.7	4.3	4.6	5.3
63	63	63.6	1.5	1.9	2.4	2.9	3.6	4.2	4.7	5.4	5.7	6.5
75	75	75.1	1.8	2.2	2.9	3.4	4.3	4.9	5.6	6.4	6.8	7.7
90	90	90.8	2.2	2.6	3.5	4.0	5.1	5.8	6.7	7.6	8.1	9.1
110	110	110.9	2.7	3.2	4.2	4.9	6.2	7.1	8.1	9.1	10.1	11.3
125	125	126.1	3.0	3.6	4.8	5.5	7.1	8.1	9.3	10.4	11.4	12.1
140	140	141.1	3.4	4.0	5.4	6.2	7.9	9.0	10.4	11.6	12.7	14.2
160	160	162.2	3.9	4.6	6.2	7.3	9.1	10.3	11.9	13.3	14.6	16.3
180	180	182.3	4.4	5.1	6.9	7.9	10.2	11.6	13.3	14.8	16.4	18.2
200	200	202.3	4.9	5.6	7.7	8.7	11.3	12.8	14.8	16.5	18.2	20.2

P = Pressure

Note: Wall thickness of pipes are based on working stress of 5Mpa at 20^{0}C.

17 REPORT WRITING

17.1 GENERAL

17.1.1 Purpose

Design reports have two main functions, namely

(i) to record and present all particulars of a project in a clear and concise way for future reference and for information to planners or other parties interested in the project.

(ii) to present basic data assumptions and conclusions regarding the project to enable superiors and other interested parties involved in the project to supervise and approve the design.

17.1.2 Report Standard

The form and contents of the reports will vary considerably depending on the complexibility and size of the water supply projects.

However, reports shall always be compiled to satisfy the two requirements as described above.

Large and complex projects should be reported on in two stages namely preliminary Design and Final Design of which details are given later in this chapter.

For small and uncomplicated schemes one Design Report is often adequate.

17.2 CONTENTS

17.2.1 Preliminary Design Reports (PDR)

The following list of contents will generally be applicable for medium and large size projects. However the contents of the report should always be modified to suit the requirements of the particular project.

1. Executive Summary

1.1 The project summary sheet and the O & M expenditure summary sheet as shown in Appendix D. A scheme boundary map approximately to scale 1:200,000 should be put immediately after the Executive summary.

1.2 A written summary of approximately two pages giving geographic, demographic technical and economic information aboutthe recommended scheme together with background information on any existing supply and on main alternatives which have been studied and the reasons for discarding these.

2. Logical Framework
It is a very useful aid to the Project planning, monitoring and evaluation.
The log frame is a planning and management tool that: -
- Defines Project Inputs, Outputs, Purposes and higher sector/programme goals in measurable or objectively veritable terms.
- Indicates the caustic linkage between Inputs, Outputs, Purposes and Goals.
- Specifies assumptions.
- Establishes Indicators.

3. List of Contents

4. Introduction
4.1 Background
4.2 Previous investigations and reports
4.3 Scope of this report

5. Description of Area
5.1 Location
5.2 Climate
5.3 Topography and Geology

6. Socio-economic Infrastructure
6.1 Administration
6.2 Education
6.3 Health Facilities
6.4 Transport
6.5 Commerce and Industry
6.6 Agriculture

7. Existing Water Supply
7.1 Location, Source and ownership
7.2 Consumers
7.3 Reliability and Constraints
7.4 Technical and Economic assessment of the supply

8. Consumer projections
8.1 Design Period
8.2 Human population

8.3 Livestock population
8.4 Institutions
8.5 Commerce and Industry
8.6 Irrigation and others

9. Water Demand
9.1 Human demand
9.2 Livestock demand
9.3 Institutional demand
9.4 Industrial demand
9.5 Irrigation and other demand
9.6 Total demand

10. Socio-economic Study
10.1 Economy and income situation
10.2 Willingness and ability to pay for water supply
10.3 Conclusions

11. Hydrology
11.1 Groundwater sources
11.2 Surfacewater source
11.3 Other possible sources
11.4 Recommended source(s).

12. Alternatives proposed
12.1 Surface sources
12.2 Groundwater sources

13. Proposed system
13.1 Source
13.1.1 Locations
13.1.2 Safe Yield
13.1.3 Flood Flows
13.1.4 Sizing
13.2 Treatment
13.2.1 Water Quality
13.2.2 Recommended Treatment Process
13.2.3 Location of Treatment Work

(N.B. Detailed drawings must not be submitted in the PD Stage).

13.3 Pumping stations
13.3.1 Raw water pumping stations
13.3.2 Clear water pumping station
13.3.3 Rising Mains

(N.B. Detailed drawings must not be submitted in the PD Stage)

13.4 Distribution System
13.4.1 Alternative Distribution Systems

13.4.2 Recommended Distribution Systems
13.4.3 Phasing

14. Cost Estimate
14.1 Construction costs
14.2 Capital costs
14.3 Operation and maintenance costs
14.4 Revenue
14.5 Conclusion regarding economy

15. Institutional Models
15.1 Available Models and Shortcomings
15.2 Proposed institutional set-up

16. Environmental Impact Assessment

17.2.2 Final Design Reports (FDR)

The following list of contents will generally be applicable for projects for which preliminary design reports have previously been prepared. However the contents of the report should always be modified to suit the requirements of the particular projects.

1. Executive Summary

1.1 The project summary sheet and the O & M expenditure summary sheet as shown in Appendix D. A scheme boundary map approximately to scale 1:200000 should be put immediate after the executive summary.

1.2 A written summary of approximately one page giving geographic, demographic technical, and economic information about the designed supply.

2. Logical Framework
It is a very useful aid to the Project planning, monitoring and evaluation.
The log frame is a planning and management tool that: -

- Defines Project Inputs, Outputs, Purposes and higher sector/programme goals in measurable or objectively veritable terms.
- Indicates the caustic linkage between Inputs, Outputs, Purposes and Goals.
- Specifies assumptions.
- Establishes Indicators.

3. List of Contents

4. Introduction
4.1 Background
4.2 Previous investigations and reports
4.3 Scope of this report.

5. Summary of Design Data from the Preliminary Design Report
5.1 Description of Area (Inclusive map of supply area)
5.2 Social infrastructure
5.3 Existing water supply
5.4 Consumer projections
5.5 Water demand
5.6 Hydrology
5.7 Selected project alternative
5.8 Studied but discarded project alternatives

6. Intake

Sub-sections stating all design criteria and technical details

7. Treatment
As above

8. Pumping Stations
As above

9. Distribution system
As above

10. Cost Estimates
10.1 Construction costs
10.2 Capital costs
10.3 Operations and maintenance costs
10.4 Revenue
10.5 Conclusion regarding the economy

11. Appendices

Detailed calculations to be submitted under separate cover. The calculations should be presented in a way that makes them easy to follow. However they need not be typed.

11.1 Hydraulic calculations
11.2 Structural calculations
11.3 Economic calculations

12. Environmental Impact Assessment

17.2.3 Monitoring and Evaluation

Monitoring and Evaluation of the project should be prepared continually throughout the life of the project. The technical information and the language should be on a level that can easily be understood by the average operator.

The content of the monitoring and evaluation should include:

i Brief description of the water supply

ii Details of equipment used in the supply and information about the suppliers or manufacturers.

iii Operational status and outputs for all components of the scheme e.g. intake, pumping stations, treatment works, storage tanks, distribution system etc. as applicable.

iv Remedial measures undertaken for smooth and targeted outputs. Also daily, weekly, monthly and yearly operation and maintenance routines should be checked to ensure consistency with design requirements.

Table 17.5: Monitoring and Evaluation questionnaire

(a)	Inputs	-	What are the Resources of a project? Their source? Their cost?
(b)	Outputs	-	What specific products or services is the project expected to produce from its inputs? How extensive are the physical outputs? What services are provided?
(c)	Effects	-	what is the outcome of the outputs? What is the use of services? Facilities?
(d)	Impacts	-	What is the outcome of project effects? Have long range objectives been met? What has been the change in the living conditions of the beneficiaries?
(e)	Processes	-	Are procedures followed as outlined? Do the processes facilitate progress of the project
(f)	Schedules	-	Where schedules maintained according to plants? Were there delays? For what reasons?

18 REFERENCES

18.1 GENERAL

The content of the Water Supply Services Manual for water supply services in Kenya is to a large extent based on practical experience and long-established procedures as communicated by a great number of engineers or other experts inside as well as outside the Ministry of Water and Irrigation.

Where such experience has been missing or when supplementary information is found necessary then general textbooks, special studies, handbooks, manuals, etc. in particular those related to water supply services in Kenya or other developing countries have been consulted. The following references have been particularly useful but many other sources of information have been used e.g. design reports on a large number of water projects. Those of the reference, which contain particularly practical and detailed information and/or type drawing, are marked*.

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PRACTICE MANUAL

FOR

WATER SUPPLY SERVICES

IN

KENYA

PART B

IRRIGATION

LIST OF CONTENTS

1. INTRODUCTION

1.1 GENERAL

1.1.1 Definition

Irrigation is generally defined as the application of water to the soil for the purpose of supplying the moisture essential for plant growth. Irrigated agriculture makes food and fiber supplies less dependant on fluctuations in climate.

Irrigation is one of the oldest known agricultural techniques, but improvements in irrigation methods and practices are still being made. The future will require even more improvements, better management and a more efficient use of the water, as competition for limited water supplies continues to increase.

1.1.2 Land Characteristics

The first step in planning an irrigation project is to establish the capability of the land to provide crops, which provides returns on the investment in irrigation works. The land must be arable and irrigable. To be arable means that the land will give sufficient yield in agricultural development. To be irrigable means the land will be suitable for farming.

The land should exhibit the following characteristics to be arable and irrigable: -

- High water holding capacity
- Readily penetrable by water
- Infiltration rate should be very low to avoid excessive loss by percolation below the root zone
- Deep enough to allow root development
- Free of black alkali (Sodium saturated condition) and free of Salts that are not susceptible to removal by leaching.
- Adequate in plant Nutrients
- Land slopes should be such that excessive erosion will not occur and water losses through surface runoff is minimized

1.2 SOIL – WATER – PLANT RELATION

1.2.1 General

The design and operation of efficient irrigation systems require knowledge of the factors and processes controlling movement and storage of water in soil.

1.2.2 Soil Water

Water applied during irrigation enters the soil, and the plants in turn extract water from the soil for their growth. Soil water is of interest largely because of its influence on plant growth and crop production. A growing plant must be able to balance the atmospheric demand for water with the amount it can extract from the soil. The soil water supply is alternately depleted through evapotranspiration and replenished by irrigation, precipitation or groundwater. The soil-water-plant system is a continuous dynamic system where water moves through the soil to the plant root surfaces, into the root, through the plant and into the atmosphere along a continuously decreasing potential energy.

1.2.3 Soil Water "constants"

(a) Field capacity
Field capacity refers to the water content in a field soil after the drainage rate has become small and it is an estimate of the amount of water that may be temporarily stored in the soil profile for plant use.

(b) Wilting point
The permanent wilting point or percentage is the soil water content below which plants growing in that soil remain wilted even when transpiration is nearly eliminated.

(c) Available water
The amount of water released by a soil between field capacity and permanent wilting is called the available water.

As a general indication this is for:
Heavy textured soil : 200mm/m
Medium textured soil : 140mm/m
Coarse textured soil : 60mm/m

The term implies that the available water can be used by plant, but this is misleading. If the soil water content approaches the wilting range, especially during periods of high demands for water yield or quality of most crops will be greatly decreased.

(d) Readily available water
Is that portion of the available water that is depleted before yield reduction occurs. This fraction (p) mainly depends on soil type, the crop and the maximum evapotranspiration ETm, see tables 1 and 2.

Table 1 – Crop Groups according to soil water depletion

Group	Crops
1	Onion, pepper, potato
2	Banana, cabbage, grape, pea, tomato
3	Alfalfa, bean, citrus, groundnut, pineapple, sunflower
4	Cotton, maize, sorghum, soybean, sugarcane, tobacco

1.2.4 Plant Root Zone

When to irrigate and how much water to apply is affected considerably by where and when water is removed from the soil by the roots of the plants. Shallow rooted crops will require more frequent irrigation than deep-rooted crops, since water can be stored in a bigger root zone. Other conditions being equal, roots of plants in a wet soil will extract more water than roots of the same plant growing in a dryer soil. This means, the more water there is available, the more water the plants will use.

As the moisture content of the soil near the surface decreases, more moisture is extracted from lower depths. Since fewer roots exist in the lower portion of the soil profile, more energy must be expended in extracting the moisture required, and frequently and finally insufficient moisture is available to prevent wilting. The distribution of active roots in a normal soil is also approximately triangular in shape, the greatest concentration being near the ground surface. Hence, especially for younger plants, it is essential to keep the upper part of the root zone wet.

Table 2 – Soil Water Depletion Fraction (p) for Crop Group and Maximum Evapotranspiration (ETm)

Crop Group	Etm mm/day								
1	2	3	4	5	6	7	8	9	10
2	0.50	0.425	0.35	0.30	0.25	0.225	0.20	0.20	0.175
3	0.675	0.575	0.475	0.40	0.35	0.325	0.275	0.25	0.225
4	0.80	0.70	0.60	0.50	0.45	0.425	0.375	0.35	0.30
	0.875	0.80	0.70	0.60	0.55	0.50	0.45	0.425	0.40

Table 3: Average depth of root zone in deep well drained soil

Crop	Root zone (m)
Lucerne	1.5 – 3
Beans	0.9 – 1.20
Cabbages	0.60
Carrots	0.60 – 0.90
Cauliflower	0.60
Celery	0.60
Citrus	1.20 - 2
Cucumber	0.3 – 0.60
Cotton	1.2 – 1.80
Grain	1.20
Pasture/grass	0.6 – 1.20
Lettuce	0.15 – 0.3
Onions	0.30
Orchards	1.8 – 2.40

Maize	0.6 – 0.90
Melons	1.50
Peas	0.9 – 1.20
Potatoes	0.9 – 1.20
Sweet potatoes	1.2 – 1.80
Pumpkin	1.80
Sorghum	0.6 – 1.20
Spinach	0.60
Squash	0.90
Tomatoes	1.80 - 3
Tobacco	0.6 – 1.20
Strawberries	0.90 – 1.2
Wheat	0.75 – 1.1
Bananas	0.60

2. CROP WATER REQUIREMENT ET_{CROP}

A crop water requirement is defined as the depth of the water needed by the crop through the period of evapotranspiration.

Evaporation: Transfer of water from the liquid to the vapour state.

Transpiration: Process by which plants remove moisture from the soil and release it to the air as vapour.

Evapotranspiration: The loss of water through the combined effort of evaporation and transpiration. It should be noted that more than half of the precipitation, which reaches the land surfaces, is returned to the atmosphere by the combined processes of evapotranspiration.

$$ET_{CROP} = ET_o \times K_c$$

ET_o = reference evapotranspiration
K_c = crop coefficient

2.1 REFERENCE EVAPOTRANSPIRATION ET_O

Several methods can be used to determine $\mathbf{ET_o}$, like

- Blaney - criddle
- Penman
- Pan evaporation

depending on available climatic data and accuracy required. ET_o is normally expressed in mm/day or mm/period (generally 1 month or 10 days).

ET_o will, however, vary from year to year and a frequency distribution analysis of ET_o is recommended; the selected ET_o value for planning is thus not based on average conditions, but on a likely range of conditions and on an assessment of tolerable risks of not meeting crop water demands. Advised is the 80% dry value (highest value 4 years out of 5).

If average ET_o data are used, a connection factor for peak ET_{crop} requirement is normally done; this generally ranges from 1.1 to 1.2.

For Kenya, ET_o can be found by multiplying the evaporation from a free water surface (E_o) by an adjustment factor, this one being:

- For highlands (above 1,100m) = 0.75
- For hot and dry low areas (below 1,100m) = 0.80

Values for E_o (open water surface evaporation) can be found in "Studies of Potential Evaporation in Kenya" by T. Woodhead, and are represented in table 4 (average year) and table 5 (dry year) for several stations in Kenya.

For computation ET_o using an evaporation pan, the data from the pan must be multiplied by a pan factor (K_{pan}). This factor depends on ground-cover, relative humidity, windspeed, radiation, etc…., and may range from1.05 to 0.35. For Kenya little data is available on this factor, often a value of 0.70 is used.

2.2 CROP FACTOR K_C

Factor affecting the value of K_c are mainly crop characteristics, crop planting data, crop development, length of growing season and climate considerations. Some recommended values are given in table 6.

Table 4: Evaporation from open water surface (average data) in mm

	Station	Altitude m	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
1	Ahero	1200	205	195	212	179	178	169	167	175	194	200	180	182	2236
2	Ainabkoi	2600	165	162	174	126	126	98	110	85	142	150	116	136	1590
3	Archers Post	865	210	210	230	208	215	210	215	230	240	230	182	185	2565
4	Bachuma	400	200	185	198	182	160	158	156	162	177	190	182	189	2139
5	Baricho	70	195	195	215	185	165	165	165	180	185	195	190	195	2230
6	Busia	1180	182	170	184	170	170	158	152	164	183	186	164	173	2056
7	Chebloch	1200	185	176	191	169	171	156	151	164	174	179	164	170	2050
8	Eldoret	2100	182	177	195	160	148	126	118	123	148	170	168	168	1883
9	Equator	2760	179	177	192	151	140	117	104	111	139	161	153	164	1788
10	Garissa	130	201	191	216	203	207	183	188	199	206	219	182	179	2374
11	Gede	30	189	165	191	178	155	137	148	155	176	192	181	185	2052
12	Habaswein	200	246	257	277	248	275	273	272	282	291	286	205	208	3120
13	Hola	90	198	202	221	191	191	168	169	182	191	198	190	192	2293
14	Isiolo	1100	209	208	230	206	216	209	215	231	241	228	181	187	2561
15	Kabondori	1140	180	165	164	146	125	98	120	119	163	157	129	138	1704
16	Kapenguria	2130	145	153	157	131	131	124	101	117	133	131	123	142	1588
17	Kapsabet	2000	177	176	198	162	152	136	138	148	166	176	171	169	1969
18	Kaputir	700	205	200	200	175	180	165	165	175	195	200	185	190	2235
19	Katumani	1600	181	165	166	136	145	126	116	125	153	171	136	170	1790
20	Kedong	1900	176	161	176	147	129	117	111	124	147	171	150	152	1761
21	Kericho	2070	160	152	166	125	130	125	121	120	124	125	121	141	1610
22	Kiambu	1730	192	178	180	138	129	98	109	117	158	166	151	165	1781
23	Kibos	1170	203	197	217	191	188	174	174	187	202	217	192	198	2340
24	Kimakia	2500	150	149	160	132	116	105	89	99	122	143	131	132	1528
25	Kipkabus	2400	178	183	199	152	149	116	124	128	156	177	152	165	1879
26	Kisumu	1140	187	182	195	164	157	143	144	156	165	182	167	176	2018
27	Kitale	1900	180	170	192	167	151	139	131	147	161	169	155	163	1925
28	Kitui	1180	189	191	200	169	168	152	149	162	183	203	163	167	2096
29	Koru	1600	182	174	180	152	148	144	140	145	163	163	158	170	1919
30	Lamu	9	219	199	220	182	173	162	166	188	193	214	206	205	2327
31	Lamuria	1850	132	133	144	136	156	140	146	138	165	147	115	115	1667
32	Lodwar	500	227	210	232	204	235	221	221	226	239	255	220	224	2714
33	Likichokio	1050	200	200	200	175	200	175	175	175	200	210	190	197	2297
34	Likitaung	700	255	255	270	221	232	235	234	242	261	257	238	239	2939
35	Machakos	1650	190	174	182	151	140	129	128	140	169	180	158	166	1907
36	Magadi	613	230	227	246	201	194	185	196	204	223	238	218	223	2585
37	Makindu	1000	175	179	182	160	151	139	139	153	179	191	154	149	1951
38	Malindi	20	210	197	215	186	171	156	156	175	191	202	195	205	2259
39	Mandera	330	233	234	257	210	213	222	223	234	238	205	193	215	2677
40	Maralal	1950	161	159	173	151	151	132	130	132	151	157	139	150	1786
41	Marigat	1070	205	195	212	187	190	173	167	182	193	199	182	189	2274
42	Marsabit	1360	176	168	175	138	155	153	154	162	173	168	134	147	1903
43	Masara	1200	193	184	191	163	171	157	165	179	196	200	172	185	2156
44	Meru	1565	155	155	170	140	150	130	130	150	155	165	135	130	1765
45	Molo	2500	149	147	159	133	127	110	108	110	127	140	123	137	1570
46	Mombasa	60	211	204	221	180	152	148	144	162	181	198	200	204	2205
47	Moyale	1110	220	207	218	160	150	147	144	161	175	165	164	184	2095
48	Muguga	2100	173	171	186	141	116	107	96	109	143	171	149	152	1714
49	Mwea Tebere	1160	197	192	200	173	166	140	123	148	176	196	183	188	2082
50	Mwingi	1050	185	185	190	170	167	143	137	164	180	190	163	163	2037
51	Nairobi Kab.	1737	173	176	183	146	125	113	108	116	140	158	141	159	1738
52	Nairobi Sth	1675	195	189	192	157	144	122	119	132	166	184	169	179	1948
53	Naivasha	1900	167	160	169	134	137	123	126	133	153	160	139	153	1754
54	Nakuru	1890	137	156	163	133	139	132	138	141	145	142	121	146	1693
55	Nanyuki	1950	156	155	158	128	129	125	125	138	150	146	118	135	1663
56	Narok	1900	149	148	156	127	122	113	112	122	143	157	142	147	1638
57	Ngao	15	205	193	220	190	178	165	165	180	191	205	190	200	2282
58	Nyeri	1800	182	171	179	153	138	118	94	120	148	164	133	145	1745
59	Ol Joro Orok	2380	129	131	152	117	122	109	94	101	117	122	110	108	1412
60	Oloitokitok	1850	160	123	116	124	117	107	91	104	128	170	150	148	1538
61	P. Victoria	1200	180	170	184	170	150	145	150	150	175	180	164	173	1991
62	Ruiru	1610	160	151	171	125	115	104	105	107	136	181	150	116	1621
63	Rumuruti	1860	181	177	196	171	168	149	150	158	178	186	167	178	2059
64	Sigor	1050	145	155	170	130	145	135	110	120	125	125	135	165	1660
65	Sth Kinangop	2600	116	113	129	110	99	88	81	86	100	119	105	105	1251
66	Subukia	2100	140	152	165	132	125	119	116	127	137	142	129	137	1621
67	Taveta	770	175	175	175	150	140	135	135	145	165	185	175	175	1930
68	Thika	1460	193	193	195	156	145	124	113	114	153	177	155	167	1885
69	Voi	560	183	187	198	176	166	158	156	162	174	189	182	175	2106
70	Wajir	240	233	225	238	205	205	199	201	206	213	207	187	208	2527
71	Wayu	160	203	190	209	190	190	167	173	187	191	193	182	198	2273
72	Yatta	1220	197	192	200	173	166	140	123	148	176	196	183	188	2082

* Note: Data from "Studies of Potential Evaporation in Kenya", T. Woodhead

Table 5: Evaporation from open water surface (dry data, 1 in 5) in mm

	Station	Altitude	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
		m													
1	Ahero	1200	238	218	235	202	201	188	197	198	223	227	212	216	2555
2	Ainabkoi	2600	186	178	189	139	139	107	126	94	156	166	133	156	1769
3	Archers Post	865	223	226	246	225	234	226	240	250	265	251	204	208	2798
4	Bachuma	400	223	201	214	199	175	171	175	177	197	209	205	215	2361
5	Baricho	70	218	211	232	201	181	178	186	197	205	214	214	221	2458
6	Busia	1180	209	190	200	187	188	172	176	181	206	208	187	199	2303
7	Chebloch	1200	202	187	202	180	184	166	165	176	188	194	181	188	2213
8	Eldoret	2100	205	193	212	176	164	138	134	136	166	189	192	194	2099
9	Equator	2760	202	193	208	167	155	128	119	122	156	179	175	189	1993
10	Garissa	130	224	207	233	222	226	198	211	218	228	241	205	203	2616
11	Gede	30	211	179	206	194	170	148	166	169	195	211	204	210	2263
12	Habaswein	200	275	279	298	271	301	295	306	308	322	315	231	236	3437
13	Hola	90	221	219	238	208	209	181	190	199	212	219	214	218	2528
14	Isiolo	1100	232	224	246	223	235	225	240	251	266	249	203	210	2804
15	Kabondori	1140	205	181	180	162	139	107	138	132	184	176	149	161	1914
16	Kapenguria	2130	167	170	173	147	147	137	117	132	152	147	144	167	1800
17	Kapsabet	2000	200	192	215	178	168	148	157	164	186	195	196	195	2194
18	Kaputir	700	224	212	211	187	193	175	181	187	211	216	204	210	2411
19	Katumani	1600	204	181	180	150	161	138	132	138	172	190	155	196	1997
20	Kedong	1900	204	180	195	166	145	130	130	140	168	194	176	180	2008
21	Kericho	2070	186	169	183	141	146	139	142	136	143	142	143	167	1837
22	Kiambu	1730	216	195	196	153	142	107	125	129	177	185	173	190	1988
23	Kibos	1170	232	217	238	213	210	192	201	209	229	244	222	232	2639
24	Kimakia	2500	171	165	176	147	129	115	102	111	138	161	152	156	1723
25	Kipkabus	2400	201	200	216	168	165	127	141	141	175	196	174	190	2094
26	Kisumu	1140	213	201	214	183	175	157	167	174	187	205	193	205	2274
27	Kitale	1900	207	189	211	187	169	154	152	165	184	191	181	192	2182
28	Kitui	1180	212	208	216	185	184	165	168	178	204	224	185	190	2319
29	Koru	1600	209	193	199	170	166	159	163	163	186	184	184	200	2176
30	Lamu	9	245	215	237	198	189	175	186	206	214	235	232	232	2564
31	Lamuria	1850	150	146	157	151	173	153	167	154	186	164	144	134	1879
32	Lodwar	500	253	227	250	223	257	239	247	247	265	281	248	254	2991
33	Likichokio	1050	223	217	215	191	219	189	196	191	222	230	214	224	2531
34	Likitaung	700	284	277	291	241	253	254	263	265	289	283	268	271	3239
35	Machakos	1650	214	190	198	167	155	141	146	155	190	200	181	191	2128
36	Magadi	613	260	248	267	221	214	202	223	226	250	265	250	257	2883
37	Makindu	1000	197	195	197	176	166	151	157	166	200	211	175	170	2161
38	Malindi	20	234	213	231	203	187	168	175	191	212	222	220	232	2488
39	Mandera	330	260	253	277	229	232	240	250	256	264	225	218	244	2948
40	Maralal	1950	178	171	185	163	164	142	145	144	167	172	155	169	1955
41	Marigat	1070	224	207	224	200	204	184	183	195	209	215	201	209	2455
42	Marsabit	1360	196	182	188	150	170	165	173	177	192	185	151	166	2095
43	Masara	1200	222	205	210	182	192	173	191	210	223	226	200	218	2452
44	Meru	1565	176	171	186	155	166	142	150	171	175	184	156	151	1983
45	Molo	2500	172	164	176	149	143	122	127	124	146	159	145	162	1789
46	Mombasa	60	235	220	238	196	164	160	162	177	200	217	226	231	2426
47	Moyale	1110	246	224	235	175	164	159	161	176	194	181	185	209	2309
48	Muguga	2100	203	193	208	160	132	120	114	125	166	197	179	182	1979
49	Mwea Tebere	1160	223	210	217	191	183	152	140	164	198	218	209	217	2322
50	Mwingi	1050	208	202	205	186	183	155	155	184	201	210	185	185	2259
51	Nairobi Kab.	1737	195	192	199	161	137	123	122	128	156	175	160	182	1930
52	Nairobi Sth	1675	220	207	208	173	160	133	136	145	186	205	193	206	2172
53	Naivasha	1900	191	177	185	150	153	135	145	149	174	179	162	179	1979
54	Nakuru	1890	156	172	`	149	155	145	160	157	164	159	140	170	1727
55	Nanyuki	1950	177	171	173	142	143	137	144	154	169	163	136	157	1866
56	Narok	1900	172	164	172	142	137	125	130	137	163	177	165	173	1857
57	Ngao	15	219	206	237	207	194	178	186	197	212	225	214	228	2503
58	Nyeri	1800	207	188	195	169	154	130	108	133	167	183	153	168	1955
59	Ol Joro Orok	2380	148	145	167	130	136	120	109	113	133	137	128	127	1593
60	Oloitokitok	1850	180	135	126	136	129	117	103	115	144	190	173	171	1719
61	P. Victoria	1200	203	186	200	187	166	160	172	166	196	201	187	199	2223
62	Ruiru	1610	182	167	187	138	127	114	121	119	153	202	173	135	1818
63	Rumuruti	1860	200	191	210	186	183	161	167	171	196	203	186	201	2255
64	Sigor	1050	167	172	187	146	163	149	127	135	143	140	158	194	1881
65	Sth Kinangop	2600	130	124	141	121	109	96	62	94	112	132	121	121	1363
66	Subukia	2100	160	168	181	147	140	131	134	142	155	159	150	160	1827
67	Taveta	770	195	190	189	164	153	146	151	158	183	204	197	198	2128
68	Thika	1460	219	212	213	173	161	136	129	126	172	198	179	193	2111
69	Voi	560	204	203	214	192	182	171	175	177	193	208	205	198	2322
70	Wajir	240	260	244	257	224	224	215	225	225	236	228	211	236	2785
71	Wayu	160	226	206	225	207	208	180	194	204	212	212	205	225	2504
72	Yatta	1220	223	210	217	191	183	152	140	164	198	218	209	217	2322

* Note: Data from "Studies of Potential Evaporation in Kenya", T. Woodhead

Table 6: Crop Coefficients (K_c

CROP	CROP DEVELOPMENT STAGES					TOTAL GROWING PERIOD
	Initial	Crop development	Mid-season	Late season	At harvest	
Alfalfa	0.3-0.4				1.05-1.2	0.85-1.05
Banana						
Tropical	0.4 – 0.5	0.7-0.85	1.0-1.1	1.0-1.0	0.75-0.05	0.7-0.8
Sub tropical	0.5-0.65	0.8-0.9	1.0-1.2	1.0-1.15	1.0-1.15	0.86-0.95
Bean						
Green	0.3-0.04	0.65-0.75	0.95-1.05	0.9-0.95	0.85-0.95	0.85-0.9
Dry	0.3-0.4	0.7-0.8	1.05-1.2	0.65-0.75	0.25-0.3	0.7-0.8
Cabbage	0.4-0.5	0.7-0.8	0.95-1.1	0.9-1.0	0.8-0.95	0.7-0.8
Cotton	0.4-0.5	0.7-0.8	1.05-1.25	0.8-0.9	0.65-0.7	0.8-0.9
Grape	0.35-0.55	0.6-0.8	0.7-0.9	0.6-0.8	0.55-0.7	0.55-0.75
Groundnut	0.4-0.5	0.7-0.8	0.95-1.1	0.75-0.85	0.55-.06	0.75-0.8
Maize						
Sweet	0.3-0.5	0.7-0.9	1.05-1.2	1.0-1.15	0.95-1.1	0.9-0.95
Grain	0.3-0.5	0.7-0.95	1.05-1.2	0.9-0.95	0.55-0.6	0.75-0.9
Onion						
Dry	0.4-0.6	0.7-0.9	0.05-1.1	0.85-0.0	0.75-0.85	0.8-0.9
Green	0.4-0.6	0.6-0.75	0.95-1.05	0.95-1.05	0.95-1.05	0.65-0.8
Pea-fresh	0.4-0.5	0.7-0.85	1.05-1.2	1.0-1.15	0.95-1.1	0.8-0.95
Pepper-fresh	0.3-0.4	0.6-0.75	0.95-1.1	0.85-1.0	0.8-0.0	0.7-0.8
Potato	0.4-0.5	0.7-0.6	1.05-1.2	0.05-0.95	0.7-0.75	0.75-0.9
Rice	1.1-1.15	1.1-1.15	1.1-1.3	0.95-1.05	0.95-1.05	1.05-1.2
Safflower	0.3-0.4	0.7-0.8	1.05-1.2	0.05-0.7	0.2-0.25	0.65-0.7
Sorghum	0.3-0.4	0.7-0.75	1.0-1.15	0.75-0.8	0.5-0.55	0.75-0.85
Soyabean	0.3-0.4	0.7-0.0	1.0-1.15	0.7-0.8	0.4-0.5	0.75-0.9
Sugar beet	0.4-0.5	0.75-0.85	1.05-1.2	0.9-1.0	0.6-0.7	0.8-0.9
Sugar cane	0.4-0.5	0.7-1.0	1.0-1.3	0.75-0.8	0.5-0.6	0.85-1.05
Sun flower	0.3-0.4	0.7-0.8	1.05-1.2	0.7-0.8	0.35-0.45	0.75-0.85
Tobacco	0.3-0.4	0.7-0.0	1.0-1.2	0.9-1.0	0.75-0.85	0.65-0.95
Tomato	0.4-0.5	0.7-0.8	1.05-1.25	0.8-0.95	0.0-0.65	0.75-0.9
Water melon	0.4-0.5	0.7-0.8	0.95-1.05	0.8-0.9	0.65-0.75	0.75-0.85
Wheat	0.3-0.4	0.7-0.8	1.05-1.2	0.65-0.75	0.2-0.25	0.8-0.9
Alfafa	0.3-0.4				1.05-1.2	0.65-1.05
Citrus						
Clean weeding						0.65-0.75
No weed control						0.85-0.9
Olive						0.4-0.6

First figure: Under high humidity (RH min > 70%) and low (U<5m/sec)
Second figure: Under low humidity (RH min < 20%) and strong wind (U>5m/sec)

2.3 OASIS EFFECT, ADVECTION

In arid semi-arid regions, irrigation fields surrounded by extensive dry fallow areas are subject to advection.

^{ET}crop predicted using data prior to irrigation development is usually too high. A correction factor for advection is applied, ranging from 1.1 (1 hectare area) to 0.85 (100 hectare area); but it may vary with crop type.

3. IRRIGATION WATER REQUIREMENT

3.1 NET IRRIGATION REQUIREMENT (NIR)

NIR = $ET_{crop} - P_e - G_e - W_b$ (mm/period)
P_e = effective rainfall (mm)
G_e = groundwater contribution (mm)
W_b = stored soil water contribution (mm)

3.1.1 Rainfall

Dependable rainfall

Crop water needs can be fully or partly met by rainfall. Rainfall will vary from year to year, and rather than using average rainfall data, a dependable level of rainfall should be selected, since a loss in crop yields during a dry year may significantly affect the project's economic viability.

A probability of 80% (dry year) is advised, i.e. the depth of rainfall that can be expected 4 years out of 5. See Appendix 2.

Effective rainfall P_e

Not all rainfall that falls is effective (for the crops), as part of it may be lost by run-off, deep percolation or evaporation, hence the term effective rainfall.

Table 7 gives the relationship between average monthly effective rainfall and mean monthly rainfall for different values of ET_{crop}. A storage factor is applied when effective storage in the root in the root zone differs from 75mm.

Table 7: Average Monthly Effective Rainfall as related to Average Monthly ETcrop and Mean Monthly Rainfall

Monthly rainfall	Mean mm	12,5	25	37,5	50	62,5	75	87,5	100	112,5	125	137,5	150	162,5	175	187,5	200
		Average monthly effective rainfall in mm*															
Average	25	8	16	24													
monthly	50	8	17	25	32	39	46										
ET crop	100	9	18	27	34	41	48	56	62	69							
mm	125	9	19	28	35	43	52	59	66	73	80	87	94	100			
	150	10	20	30	37	46	54	62	70	76	85	92	98	107	116	120	
	175	10	21	31	39	49	57	66	74	81	89	97	104	112	119	127	133
	200	11	23	32	42	52	61	69	78	86	95	103	111	118	126	134	141
	225	11	24	33	44	54	64	73	82	91	100	109	117	125	134	142	150
	250	12	25	35	47	57	68	78	87	96	106	115	124	132	141	150	159
		13	25	38	50	61	72	84	92	102	112	121	132	140	150	158	167

* Where net depth of water that can be stored in the soil at time of irrigation is greater or smaller than 75mm, the correction factor to be used is :

Effective storage	20	25	37.5	50	62.5	75	100	125	150	175	200
storage factor	.73	.77	.86	.93	.97	1.00	1.02	1.04	1.06	1.07	1.08

EXAMPLE:
Given:
Monthly mean rainfall = 100mm; ET_{crop} = 150mm; effective storage = 175mm

Calculation:
Correction factor for effective storage = 1.07
Effective rainfall 1.07 x 74 = 79mm

3.1.2 Groundwater G_e and Soilwater Storage Contribution W_b

Groundwater storage contribution often is said to be equal to zero. The contribution from the soil water can easily be found if depth of root zone and soil characteristics is known: W_b often includes a pre-planting irrigation.

3.2 GROSS IRRIGATION REQUIREMENTS IR

$$IR = \frac{1}{E_p} \frac{(NIR)}{1-LR}$$

E_p = project efficiency
LR = leaching requirement

3.2.1 Efficiency

E_a = application efficiency (ratio water available to crop versus water at field inlet)
E_c = conveyance efficiency (ratio water inlet field versus water at project headwork.
E_p = project efficiency; $E_p = E_a \times E_c$

Field application efficiency ranges from
0.55 – 0.75 surface irrigation
0.60 – 0.85 sprinkler irrigation
0.75 - 0.90 drip irrigation

3.2.2 Leaching Requirement LR

Is the minimum amount of irrigation water supplied that must be drained through the root zone to control salinity. It is expressed as a fraction of the irrigation water, and must be supplied in excess of it.

$$LR = \left(\frac{EC_w}{5\,EC_e - EC_w} \right)$$

where

EC_w – electrical conductivity of irrigation water (mm hos/cm)
EC_e - electrical conductivity of the soil saturation extract for a given crop appropriate to tolerable degree of yield reduction – see table 8

Often leaching is done after the irrigation season. Leaching can be accomplished by excess precipitation.

Table 8: Electrical conductivity of irrigation water, EC_c

CROP TOLERANCE - values given are EC_e (*)

Yield decrement to be expected for certain crops due to salinity of irrigation water (Surface irrigation methods)

FIELD CROPS					
Crop	0%	10%	25%	50%	Maximum
Cotton	7.7	9.6	13	17	27
Wheat	6.0	7.4	9.5	13	20
Soyabean	5.0	5.5	6.2	7.5	10
Groundnut	3.2	3.5	4.1	4.9	6.5
Rice (paddy)	3.0	3.8	5.1	7.2	11.5
Maize	1.7	2.5	3.8	5.9	10
Beans	1.0	1.5	2.3	3.6	6.5
FRUIT CROPS					
Orange/lemon					
Grapefruit	1.7	2.3	3.3	4.8	8
Apple & pear	1.7	2.3	3.3	4.8	8
Peach	1.7	2.2	2.9	4.1	6.5
Avocado	1.3	1.8	2.5	3.7	6
Strawberry	1.0	1.3	1.8	2.5	4
VEGETABLE CROPS					
Broccoli	2.8	3.9	5.5	8.2	13.5
Tomato	2.5	3.5	5	7.6	12.5
Spinach	2.0	3.3	5.3	8.6	15
Cabbage	1.8	2.8	4.4	7	12
Potato	1.7	2.5	3.8	5.9	10
Pepper	1.5	2.2	3.3	5.1	8.5
Lettuce	1.3	2.1	3.2	5.2	9
Onion	1.2	1.8	2.8	4.3	7.5
Carrots	1.0	1.7	2.8	4.6	8
Sweet potato	1.5	2.4	3.8	6	10.5
FORAGE CROPS					
Alfalfa - Lucern	2	3.4	5.4	8.8	15.5
Bermuda grass	6.9	8.5	10.8	14.7	22.5
Sudan grass	2.8	5.1	8.6	14.4	26

(*) EC_e means electrical conductivity of the saturation extract in milli-mohs/cm at 25^0C
EC_w means electrical conductivity of the irrigation water in milli-mohs/cm at 25^0C
$EC_e \approx 1.5 EC_w$

3.3 Irrigation Supply Requirement

$V = 10 \times A \times IR$

Where

V = system requirement (m^3/period)

A = area in hectares

IR = gross irrigation requirement (mm/period)

Often, for small schemes, the system capacity has to be determined over a short period, hence

$$V = 10 \times A \times IR \times \frac{24}{h} \times \frac{7}{v}$$

where

h = hours of operation per day

v = working days per week

(if 7 is replaced with 30/31, v = working days per month)

Normally for smaller schemes (<100 hectares) this requirement is multiplied by a flexibility factor, being 1.1 to 1.25.

4. FIELD IRRIGATION SCHEDULES

4.1 IRRIGATION INTERVAL (FREQUENCY) I

The interval (in days) between successive irrigation

$$i = \left(\frac{p \times AW \times D}{ET_{crop}} \right) \text{ days}$$

p = fraction of available water, without affecting evapotranspiration and/or crop growth (table 2).

AW = available soil water (mm/m)

D = rooting depth (m)

ET_{crop} = see 2.1

4.2 DEPTH OF IRRIGATION D

The actual amount of water given to the soil during a single irrigation, or the depth of irrigation application (d) is equal to the readily available water (p x AW) over the root zone (D). An application efficiency factor (E_a) is always added to account for uneven application.

$$d = (p \times AW) \cdot D/E_a \quad \text{(mm)}$$

4.3 FIELD IRRIGATION SUPPLY

Field supply is primarily determined by the field irrigation schedules (depth and interval) and by the method the water is distributed to and applied over the fields. The method of irrigation (surface, sprinkler, drip) is in turn determined by such factors as type of crop, soil, need for land grading, water use efficiency, erosion hazards, salinity, costs and others. Field irrigation supply at the time of irrigation for a given soil, crop and level of evaporative demand is:

$$q.t = \frac{10}{E_a} (p.\ AW).D.A \quad (m^3)$$

Where

q	=	stream size in 1/sec
t	=	supply duration in seconds
E_a	=	application efficiency, fraction
AW	=	total available soil water, mm/m
p	=	fraction of total available soil water permitting unrestricted Evapotranspiration and/or crop growth,
D	=	rooting depth, m
A	=	acreage, ha

To obtain a first estimate of **q** and **t**, by converting the depth of water to be applied into stream size and supply duration, table 9 can be used. The table does not take into account the irrigation application rate, irrigation methods and practices, and the stream size that can be handled by the irrigator, and as such may give unrealistic estimates. Estimates of **q** and **t** must be evaluated on the basis of the different irrigation methods and practices.

Table 9: Average intake rates of Water in mm/hr for different soils and corresponding stream size 1/sec/ha

Soil Texture	Intake Rate mm/hr Average Range	Stream size q 1/sec/ha
Sand	50 (25 to 250)	140
Sandy loam	25 (15 to 75)	70
Loam	12.5 (8 to 20)	35
Clay loam	8 (2.5 to 15)	22
Silty clay	2.5 (0.03 to 5)	7
Clay	5 (0/1 to 15)	14

An irrigator can handle a flow between 15 to 40 1/sec

5. SURFACE IRRIGATION METHODS

5.1 GENERAL

Several methods are practised, depending on crop type, land and soil characteristics, availability of water, aptitude of the farmer and other factors.

5.2 FURROW IRRIGATION

The flow of water into a furrow should be large enough to reach the end, but small enough not to cause erosion, flooding or excessive tail losses. The size of the flow must be adjusted to the infiltration rate of the soil, land slope, furrow length, erosion hazard and depth to be applied.

The general practise is to allow as large as permissible initial flow Qi into the furrow, and when the water reaches the end, to cut-back this flow Qcb until the desired depth of water is applied.

The two following formula’s can be used as a guide line, as well as table 10. Field experiments will however determine the optimum values

$$Qi = 63/s$$

$$Qcb = 4s^{-1/4} L^{1/2} K^{1/2}$$

Where

Qi	=	initial flow (l/sec)
s	=	slope in (m/m)
Qcb	=	cut-back flow (1/sec)
L	=	length of furrow (m)
k	=	Final infiltration rate (m/sec)

Depending on available water, and skill of irrigator, several furrows are irrigated at the same time.

5.3 BASIN IRRIGATION

Best suited to soil with moderate to low infiltration rates (<50 mm/hr), and gentle land slopes.

Design is based on rapid application of water over the entire basin. Some advised basin areas are given in table 11.

5.4 BORDERSTRIP IRRIGATION

Is a method where a field is divided into a number of level strips between ridges. Water is applied from one end and, in flowing downslope, progressively covers the entire strip with a thin sheet of water. Length and width of the strip depend on soil and land characteristics, suggested values are given in table 12. The flow applied to the strip depends on several factors, mainly slope infiltration – figure 1 can be used for a first appraisal.

Field experiments will determine optimum dimensions.

5.5 DRAINAGE

A drainage system is an essential part of each irrigation scheme. This is particularly true for surface irrigation methods, especially with basin irrigation.

5.6 CALCULATIONS AND SELECTIONS

5.6.1 Calculation Procedure

(a) Determine climatically suitable crops. Collect available, climate data; evaluate climatic conditions in relation to crop requirements; select crops that are most suitable for the given climate and soil, see table 13, 14 and 15.

(b) Determine cropping patterns. Determine most likely length of growing periods of the selected crops in relation to the total growing season and the time required for other farming operations.

(c) Select optimum cropping pattern.

Example:

Given:	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec
T mean ^{0}C	14	15	17.5	21	25.5	27.5	28.5	28.5	26	24	20	25.6

Climatically suitable selected crops

Maize	24 – 28^{0}C 120 days May - August
Cotton	25 – 30^{0}C 180 days April – October/November
Sorghum	25 – 30^{0}C 120 days May - August
Groundnut	26 - 30^{0}C 120 days May - October
Wheat	18 - 30^{0}C 150 days November - April
Bean	15 – 22^{0}C 120 days November - April
Onion	15 – 200C 120 days November - April

Possible cropping patterns:-

Wheat/maize/bean/maize or sorghum
Wheat/maize/bean/bean/groundnut
Wheat/cotton/bean/maize or sorghum

Table 10: Furrow Irrigation – Suggested Values of Infiltration Rates

		Maximum length and maximum discharge of the furrows (m) (l/min)																	
		Low infiltration rate I < 0.75cm/h						Moderate infiltration rate 2.5 > I > 0.75cm/h					High infiltration rate 6 > I > 2.5cm/h				Sandy soil I > 6cm/h		
Slope %	Discharge L/min	DEPTH OF APPLICATION WATER FOR EVERY IRRIGATION (cm)																	
		18	15	13	10	8	5	15	13	10	8	5	13	10	8	5	10	8	5
0,25	150	590	550	500	450	390	320	425	380	350	300	250	230	210	180	150	100	85	70
0.50	75	410	280	350	300	270	220	290	270	240	200	170	160	140	120	100	70	55	45
0.75	50	320	300	270	240	210	173	230	210	190	160	130	130	110	100	85	55	45	40
1.00	38	270	260	240	200	180	140	200	175	160	140	110	110	85	70	55	45	40	30
1.50	26	220	200	180	170	140	110	160	140	130	110	90	85	70	60	45	40	30	
2.00	20	190	175	160	140	120	100	130	120	110	90	75	75	60	55	40	30		
2.50	15	170	150	140	130	110	90	120	110	100	85	70	70	55	45	40			
2.50	13	150	140	130	110	100	85	110	100	90	75	60	60	45	40	30			
3.00	9	130	120	110	100	85	70	90	85	75	60	55	55	40	30				
4.00	7	115	105	100	85	75	60	85	75	70	55	45	45	40	30				
5.00	6	100	90	85	75	70	55	75	70	60	50	40	40	40	30				
6.00																			

Table 11: Suggested Basin Areas (hectares) for Basin irrigation

Flow 1/sec	Soil type: Sand	Sandy loam	Clay loam	Clay
5	0.003	0.008	0.02	0.03
10	0.005	0.015	0.03	0.05
20	0.01	0.03	0.06	0.1
30	0.02	0.06	0.12	0.2
60	0.04	0.12	0.24	0.4
90	0.06	0.18	0.36	0.6
120	0.08	0.24	0.48	0.8
150	0.10	0.30	0.60	1.0
210	0.14	0.42	0.84	1.4
270	0.18	0.54	1.08	1.8
300	0.20	0.60	1.20	2.0

Table 12: Suggested Standards for the design of Border-strip for shallow-rooted crops

Metric units					
Soil Profile	Percent of slope	Unit flow per metre of strip width	Average depth of water applied	Border-strip Width	Border-strip Length
	Metres per 100 metres	Litres per second	Milli-metres	Metres	
CLAY LOAM	0.15-0.6	6-8	50-100	5-18	90-180
0.6 metre deep over permeable	0.6-1.5	4-6	50-100	5-6	99-180
Subsoil	1.5-4.0	2-4	50-100	5-6	90
CLAY	0.15-0.6	3-4	100-150	5-18	180-300
0.6 metre deep over permeable	0.6-1.5	2-3	100-150	5-6	180-300
Subsoil	1.4-4.0	1-2	100-150	5-6	180
LOAM	1.0-4.0	1-4	25-75	5-6	90-300
0.15 to 0.45 metre deep over hardpan					

Suggested standards for the design of border-strip for deep-rooted crops

Soil Profile	Percent of slope	Unit flow per metre of strip width	Average depth of water applied	Border-strip width	Border-strip length
	Metres per 100 metres	Litres Per second	Milli-metres	Metres	
SANDY Infiltration rate of 2.5 + cm per hour	0.2-0.4	10-15	100	12-30	60-90
	0.4-0.6	8-10	100	9-12	60-90
	0.6-1.0	5-8	100	6-9	75
LOAMY SAND Infiltration rate of 1.8 to 2.5cm per hour	0.2-0.4	7-10	125	12-30	75-150
	0.4-0.6	5-8	125	9-12	75-150
	0.6-1.0	3-6	125	6-9	75
SANDY LOAM Infiltration rate of 1.2 to 1.8cm per hour	0.2-0.4	5-7	150	12-30	90-250
	0.4-0.6	4-6	160	6-12	90-180
	0.6-1.0	2-1	160	6	90
CLAY LOAM Infiltration rate of 0.6 to 0.8cm per hour	0.2-0.4	3-4	175	12-30	180-300
	0.4-0.6	2-3	175	6-12	90-180
	0.6-1.0	1-2	175	6	90
CLAY Infiltration rate of 0.25 to 0.6cm per hour	0.2-0.3	2-4	200	12-30	350

Figure 1: Borderstrip Irrigation

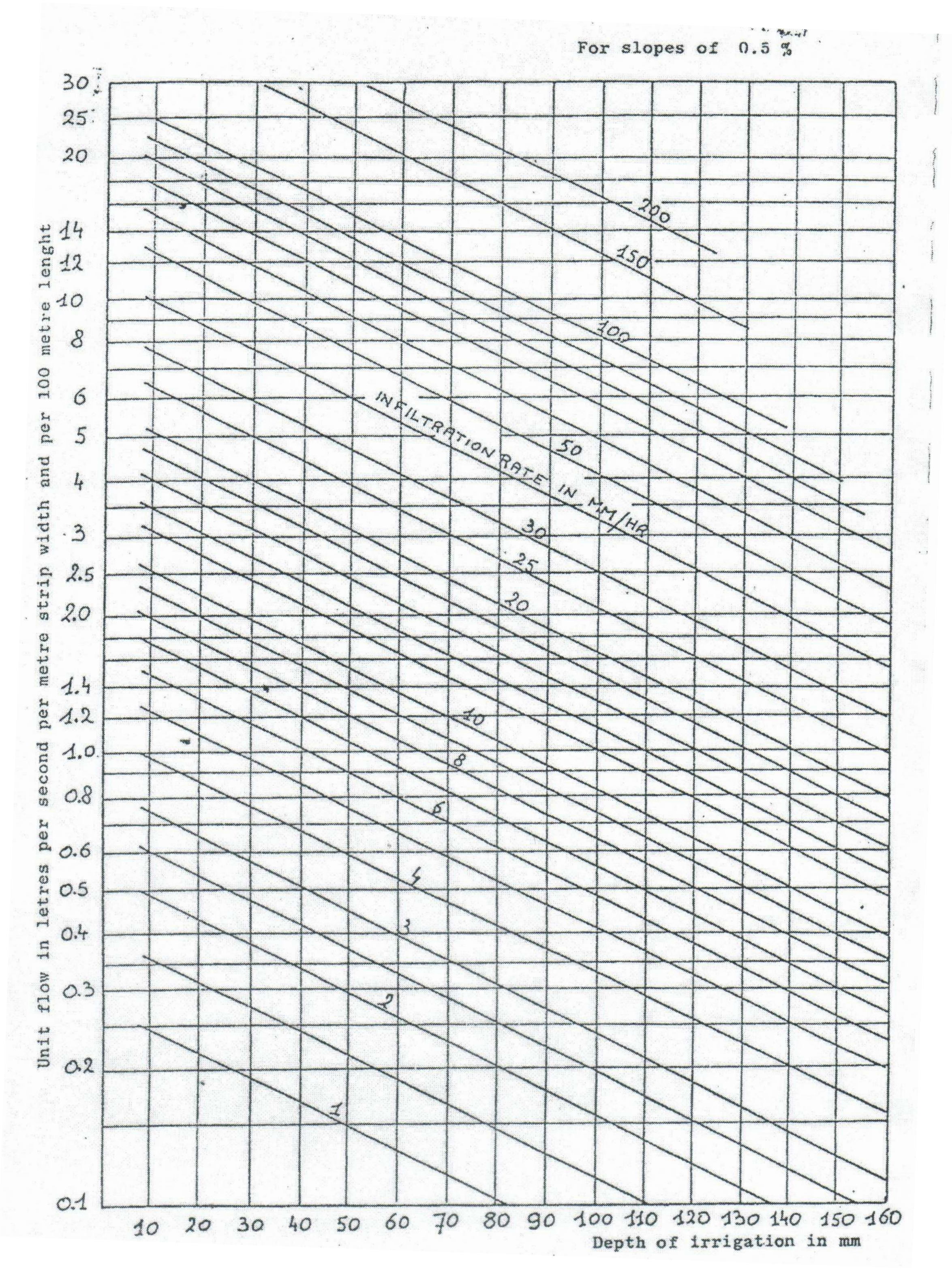


Table 13: Climatic, Soil and Water requirements for Crops.

Crop	Total growing period (days)	Temperature required for growth optimum ^{0}C	Specific climate constraints/ requirements	Soil requirements
Alfalfa	100-365	24-26	Sensitive to frost; cutting interval related to temperature; requires low RH in warm climates	Deep, medium-textured, well drained, pH=6.5-7.8
Banana	300-365	25-30	Sensitive to frost; temperatures < 8^{0}C for longer periods causes serious damage wind < 4m/sec	Deep, well-drained loam without stagnant water pH=5-7
Bean	Fresh: 60-50 Dry: 90-120	15-20	Sensitive to frost; excessive rain and hot weather	Deep, friable soil, well drained and aerated pH = 5-8
Cabbage	100-150	15-20	Short periods of frost not harmful; RH = 60-90%	Well drained; pH=6-6.5
Citrus	240-365	23-30	Sensitive to frost, strong wind high humidity, cool winter or short dry period preferred	Deep, well aerated light to medium textured soils free from stagnant water pH=5-8
Cotton	150-180	20-30	Sensitive to frost; strong or cold winds; temperature required for boll development; dry ripening period required.	Deep, medium to heavy textured soils. PH=5.5-8
Grape	180-270	20-25	Resistant to frost during dormancy but sensitive during growth; long, warm to hot, dry summer and cool winter required	Well-drained, light soils are preferred.
Groundnut	90-140	22-28	Sensitive to frost, for germination temperature > 20^{0}C	Well-drained, friable medium textured soil with loose top soil; pH=5.5-7.0
Maize	100-140	24-30	Sensitive to frost; for germination temperature > 10^{0}C; cool temperature causes problem for ripening	Deep, well-drained aerated soils with deep water table and without water logging pH=5-7.0
Olive	210-300	20-25	Sensitive to frost; low winter temperature required for flower bud initiation.	Deep, well-drained soils free from waterlogging.
Onion	100-140 (30-35) in nursery	15-20	Tolerant to frost; low temperature required for flower initiation; no extreme temperature or excessive rain.	Medium-textured soil, pH=6.0-7.0
Tomato	90-140 (25-35) in nursery	18-25	Sensitive to frost, high RH, strong wind; optimum night temperature – 10 to 2^{0}0C.	Light loam, well-drained without waterlogging pH=5-7.6
Pea: Fresh Dry	 05-100 85-120	15-18	Slight frost tolerance when young	Well drained and aerated soils; pH=5.5-6.5
Pepper	120-150	18-23	Sensitive to frost	Light medium textured soils pH=5.5-7.0

Pineapple	365	22-26	Sensitive to frost; requires high RH; quality affected by temperature	Sand loam, with low lime content pH=5-5.5
Rice	90-150	32-30	Sensitive to frost; cool temperature causes hand sterility; small difference in day and night temperature is preferred.	Heavy soils preferred for percolation losses, high tolerance to 02 deficit; pH=5.5-6.0
Safflower Spring Autumn	 120-160 200-230	Early growth 15-20 Later growth 20-30	Tolerant to frost, cool temperature Required for good establishment And early growth.	Fairly deep, well-drained Soils, preferably medium Textured; pH=6-8.0
Sorghum	100-140	24-30	Sensitive to frost; for germination temperature >10^0C; cool temperature causes head sterility.	Light to medium/heavy soils relatively tolerant to periodic water logging, pH=0-8.0
Soybean	100-130	20-25	Sensitive to frost; for some varying temperature >24^0C required for flowering.	Wide range of soil except sandy, well-drained; pH=6-8.0
Sugar beet	160-200	18-22	Tolerant to light frost toward harvest mean daily temperature <10^0C for high sugar yield.	Medium to slightly heavy textured soils, friable and well drained; pH=6-7.0
Sugar cane	270-365	22-30	Sensitive to frost; during ripening cool, dry, sunny weather is required	Deep well aerated with ground water deeper than 1.5-2m but relatively tolerant to periodic high water tables and 0_2 deficit; pH=5-8.5.
Sun flower	90-130	18-25	Sensitive to frost	Fairly deep soils; pH=6-7.5
Tobacco	90-120 (40-60)in nursery	20-30	Sensitive to frost	Quality of leaf depends on soil texture; pH=5-6.5
Tomato	90-140 (25-35) in nursery	10-25	Sensitive to frost, high RH, strong wind, optimum night temperature 10 to 20^0C	Light loam, well-drained without water logging pH=5-7.0
Water melon	80-110	22-30	Sensitive to frost	Sandy loam is preferred; pH=5.8-7.2
Wheat	Spring: 100-130 winter: 180-250	15-20	Spring wheat; sensitive to frost winter wheat: resistant to frost during dormancy, sensitive during post-dormancy period; requires a cold period for flowering during early growth, for both, dry period required for ripening.	Medium-texture is preferred; relatively tolerant to high water table; pH=6-8.0

Table 14: Crop characteristics.

CROP	Water Requirements mm/growing period	Sensitivity to water supply (1)	Sensitivity to Salinity	Fertilizer requirements N: P: K Kg/Ha/growing period (2)
Alfalfa	800-1600	0.7-1.1	Mod. Sensitivity	0-40:55-65:75-100
Banana	1200-2200	1.2-1.35	Sensitive	200-400:40-60:240-480
Bean	300-500	1.15	Sensitive	20-40:40-60:50-120
Cabbage	380-500	0.95	Mod. Sensitivity	100-150:50-65:100-130
Citrus	900-1200	0.8-1.1	Sensitive	100-200:35-45:50-160
Cotton	700-1300	0.85	Tolerant	100-180:20-60:50-80
Grape	500-1200	0.85	Mod. Sensitivity	100-160:40-60:160-230
Groundnut	500-700	0.7	Mod. Sensitivity	10-20:15-40:25-40
Maize	500-800	1.25	Mod. Sensitivity	100-200:50-80:60-100
Onion	350-550	1.1	Sensitive	60-100:25-45:110-220
Pea	350-500	1.15	Sensitive	20-40:40-60:80-160
Pepper	600-900	1.1	Mod. Sensitivity	100-170:25-50:50-100
Pineapple	700-1000	Low		230-300:45-65:110-220
Potato	500-700	1.1	Mod.	80-120:50-80: 60-100
Rice	350-700	High	Mod. Sensitivity	100-150:20-40:80-120
Sorghum	450-650	0.9	Mod. Tolerant	100-180:20-45:35-80
Soyabean	450-700	0.85	Mod. Tolerant	10-20:15-30:25-60
Sugar Cane	1500-2500	1.2	Mod. Tolerant	100-200:20-90:125-160
Sun flower	600-1000	0.95	Mod. Tolerant	50-100:20-45:60-125
Tobacco	400-600	0.9	Sensitive	40-80:30-90:50-110
Tomato	400-600	1.05	Mod. Sensitivity	100-150:65-110:160-240
Water melon	400-600	1.1	Mod. Sensitivity	80-100:25-60:35-80
Wheat	450-650	1.15	Mod. Tolerant	100-150:35-45:25-50

(1) < 0.85 = Low
o.85 – 1.15 = Medium
> 1.15 = High

(2) 1Kg P = 2.4 kg K_2O_5
1Kg K = 1.2 kg K_2O

Table 15 – Good yields of high – producing varieties adapted to climate conditions of the available growing season under adequate water supply and high level of agricultural inputs under irrigation farming conditions (ton/ha).

<table>
<tr><th colspan="2" rowspan="3">CROP</th><th colspan="6">Climatic Regions</th></tr>
<tr><th colspan="2">Tropics [1]</th><th colspan="2">Subtropics [2]</th><th colspan="2">Temperate [3]</th></tr>
<tr><th>< 20°C [4]</th><th>>20°C</th><th>< 20°C</th><th>>20°C</th><th>< 20°C</th><th>>20°C</th></tr>
<tr><td>Alfalfa</td><td>hay</td><td colspan="2">15</td><td colspan="2">25</td><td colspan="2">10</td></tr>
<tr><td>Banana</td><td>Fruit</td><td></td><td>40-60</td><td></td><td>30-40</td><td></td><td></td></tr>
<tr><td>Bean: fresh</td><td>Pod</td><td>6-8</td><td></td><td>6-8</td><td></td><td>6-8</td><td></td></tr>
<tr><td>dry</td><td>grain</td><td>1.5-2.5</td><td></td><td>1.5-2.5</td><td></td><td>1.5-2.5</td><td></td></tr>
<tr><td>Cabbage</td><td>head</td><td>40-60</td><td></td><td>40-60</td><td></td><td>40-60</td><td></td></tr>
<tr><td>Citrus:</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>grape</td><td>fruit</td><td></td><td>35-50</td><td></td><td>40-60</td><td></td><td></td></tr>
<tr><td>lemon</td><td>fruit</td><td></td><td>25-30</td><td></td><td>30-45</td><td></td><td></td></tr>
<tr><td>orange</td><td>fruit</td><td></td><td>20-35</td><td></td><td>25-40</td><td></td><td></td></tr>
<tr><td>Cotton</td><td>seed cotton</td><td></td><td>3-4</td><td></td><td>3-4.5</td><td></td><td></td></tr>
<tr><td>Grape</td><td>fruit</td><td colspan="2">5-10</td><td colspan="2">15-30</td><td colspan="2">15-25</td></tr>
<tr><td>Groundnut</td><td>nut</td><td></td><td>3-4</td><td></td><td>3.5-4.5</td><td></td><td>1.5-2</td></tr>
<tr><td>Maize</td><td>grain</td><td>7-9</td><td>6-8</td><td>9-10</td><td>7-9</td><td></td><td>4-6</td></tr>
<tr><td>Olive</td><td>fruit</td><td colspan="2"></td><td colspan="2">7-10</td><td colspan="2"></td></tr>
<tr><td>Onion</td><td>bulb</td><td>35-45</td><td></td><td>35-45</td><td></td><td>35-45</td><td></td></tr>
<tr><td>Pea : fresh</td><td>pod</td><td>2-3</td><td></td><td>2-3</td><td></td><td>2-3</td><td></td></tr>
<tr><td>dry</td><td>grain</td><td>0.6-0.8</td><td></td><td>0.6-0.8</td><td></td><td>0.6-0.8</td><td></td></tr>
<tr><td>Fresh pepper</td><td>fruit</td><td colspan="2">15-20</td><td colspan="2">15-25</td><td colspan="2">15-20</td></tr>
<tr><td>Pineapple</td><td>fruit</td><td></td><td>75-90</td><td></td><td>65-75</td><td></td><td></td></tr>
<tr><td>Potato</td><td>tuber</td><td>15-20</td><td></td><td>25-35</td><td></td><td>30-40</td><td></td></tr>
<tr><td>Rice</td><td>paddy</td><td></td><td>6-8</td><td></td><td>5-7</td><td></td><td>4-6</td></tr>
<tr><td>Safflower</td><td>seed</td><td colspan="2"></td><td colspan="2">2-4</td><td colspan="2"></td></tr>
<tr><td>Sorghum</td><td>grain</td><td>3-4</td><td>3.5-5</td><td>3-4</td><td>3.5-5</td><td></td><td>2-3</td></tr>
<tr><td>Soybean</td><td>grain</td><td colspan="2">2.5-3.5</td><td colspan="2">2.5-3.5</td><td colspan="2"></td></tr>
<tr><td>Sugar beet</td><td>beet</td><td colspan="2"></td><td colspan="2">40-60</td><td colspan="2">35-55</td></tr>
<tr><td>Sugarcane</td><td>cane</td><td></td><td>110-150</td><td></td><td>100-140</td><td></td><td></td></tr>
<tr><td>Sunflower</td><td>seed</td><td colspan="2">2.5-3.5</td><td>2.5-3.5</td><td></td><td colspan="2">2-2.5</td></tr>
<tr><td>Tobacco</td><td>leaf</td><td></td><td>2-2.5</td><td></td><td>2-2.5</td><td></td><td>1.5-2</td></tr>
<tr><td>Tomato</td><td>fruit</td><td colspan="2">45-65</td><td colspan="2">55-76</td><td colspan="2">45-65</td></tr>
<tr><td>Watermelon</td><td>fruit</td><td></td><td>25-35</td><td></td><td>25-35</td><td></td><td></td></tr>
<tr><td>Wheat</td><td>grain</td><td>4-6</td><td></td><td>4-6</td><td></td><td>4-6</td><td></td></tr>
</table>

[1] Semi-arid and arid areas only
[2] Summer and winter rainfall areas
[3] Oceanic and continental areas
[4] Mean temperature

	F	M	A	M	J	J	A	S
E_o (table 4) (mm)	185	198	182	160	158	156	162	177
$ET_{crop} = E_o \times 0.80$ (mm)	-	158	146	128	126	125	130	-
K_c	-	0.35	0.6	1.0	1.12	1.15	0.6	-
Advection correction	-	-	-	0.9	0.9	0.9	-	-
Peak month correction	-	-	-	-	1.1	1.1	-	-
ET_{cotton} (mm/month)	0	55	88	115	140	142	78	0
P_e (mm/month)	0	3	26	16	0	0	0	7
G_e (mm)	-	-	-	-	-	-	-	
W_b (mm)		(50 pre-irrigation			-	-	-	
NIR (mm/month)	50*	2	62	99	140	142	78	0

* pre-irrigation

(d) Calculate reference evapotranspiration (ET_o) on a monthly basis, select appropriate crop coefficient (K_c) (Table 6), calculate crop water requirements $ET_{crop} = K_c ET_o$ in mm/period.
Eventually adjust for advection and peak month ET_o.

(e) Determine effective rainfall (P_e) and ground water contribution (G_e) to crop water requirements in mm/period, and actual depth of available soil water over the depth at the start of the growing period (W_b) in mm, calculate net irrigation requirement using the following formula:

$$NIR = ET_{crop} - (P_e + G_e + W_b) \text{ mm/period}$$

Example
Semi-arid, hot and moderate windly climate. Climate data collected prior to irrigation development, cotton, grown early March, harvested end August. Project size 100 ha; cropping intensity is 100%, surrounding is dry fallow land.

(f) Find gross irrigation requirements; evaluate quality of irrigation water and drainage conditions, determine salinity level (table 8) and determine leaching requirement.

Determine irrigation efficiency, considering technical and managerial control, delivery and application methods.

(g) Summarize calculation to find irrigation supply requirements.

Example (Bachuma)
Cotton (as earlier), leaching required to 90% yield, EC of irrigation water = 6 mmhos/cm, leaching is 85% effective, irrigation by furrows, unlined field canal, lined distribution canals. Flexibility factor = 1.05
From table 8, max. salinity for 90% yield = 9.6 mmhos/cm

Ep	=	0.65 (E_a) x 0.75 (E_c)		
LR (net)	=	6/(5 x 9.6 – 6)	=	0.13
LR (gross)	=	0.13/0.85	=	0.153
IR	=	I/E_p (NIR)/(1-LR)	=1/0.488 x NIR/(1-0.153)	
	=	NIR/0.41		
V	=	10 x 100 x NIR		

	F	M	A	M	J	J	A	S
NIR mm	50	2	62	99	140	142	78	-
IR mm	122	5	151	241	341	346	190	-
Flex. Factor	-	-	-	-	-	1.05	-	-
V(‘000m^3)	122	5	151	241	341	363	190	-

If irrigation is done 7 days a week, 12 hours per day, required low at project inlet is:

$$V_{inlet} = \frac{363,000}{31} = 710 \text{ m}^3/\text{day}$$

$$= \frac{11,710}{12} = 976\text{m}^3/\text{hr} = 271 \text{ 1/sec}$$

(h) Determine depth irrigation, irrigation interval, number of furrows/basins/border trips irrigated at the same time, taking into account the available water.

(i) Lay-out of supply system is done, main canals, distribution canals and field canals are designed, taking into account the available water.

(j) Schedule of operation is prepared.

(k) Project costs and economical analysis is carried out.

5.6.2 Remarks

Similar calculations can be made for different crops grown simultaneously in the project area (see example). To arrive at an irrigation supply in terms of flow rate (m^3/sec) and flow duration (hrs), the irrigation interval (i) and the depth of irrigation (d) are weighted for different crops according to their respective field acreages, blocks of fields, acreages served by laterals and main canals, taking into account irrigation efficiency and leaching requirements.

However, to simplify the operation of the supply system, in many irrigation projects fixed irrigation intervals and/or fixed depth or irrigation application are used.

Since crop water requirements change over the growing period, the use of fixed intervals and/or fixed depth for irrigation causes either over or under – irrigation during the different parts of the growing period. This in turn leads to inefficient use of water or causes reduction in crop yield.

EXAMPLE:
Given:
Semi-arid, hot and moderate windy climate. Project size 150 ha; cropping intensity 200%. Cropping pattern: maize (90 ha) from May through September followed by berseem (90 ha) from October through February; cotton (60 ha) from March through August followed by wheat (60 ha) from November through March.

Calculation:

```
                                 Maize 90 ha
        berseem                                                  90 ha
       +++++++                                                 +++++++++++
                                 Cotton 60 ha
                  xxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
         Wheat 60 ha
        ------------------                                        ---------
```

	J	F	M	A	M	J	J	A	S	O	N	D	T
ETmaize					85	140	275	225	115				840
ETberseem	80	95								65	90	70	400
ETcotton			45	110	225	285	280	120					1065
ETwheat	80	100	95								40	60	375
ETcrop	80	100	55	45	140	200	280	185	70	40	70	70	1325

mm/month weighted for acreage, or 90/150. ETmaize + 90/150. ETberseem + 60/150. ETcotton + 60/150. ETwheat, rounded of to nearest 5mm.

6. DRIP IRRIGATION

With drip system, irrigation water is supplied to individual trees, groups of plants or plant rows by emitters placed on laterals delivering a flow (qe) of 2 to 10 1/hr each. The stream size is determined by the number and type of emitters, soil type, crop and allowable water depletion. In a well-operated system a nearly constant low soil water tension can be maintained in the root zone. For a selected level of soil water depletion and knowing ET_{crop} and soil infiltration rate, the frequency and duration of application can be determined. Information on flow rates is given in Tables 16,17 and 18.

EXAMPLE:

Given:

Tomatoes; acreage (A) is 40 ha, ETpeak is 7mm/day; soil intake rate is 5mm/hr; total available soil water (AW) is 140mm/m soil depth, at time of irrigation (1-p)Sa is 90 mm/m soil depth; rooting depth (D) is 1m. Row spacing (1_1) is 1.2m with emitter spacing (1_2) of 0.6m; emission uniformity (Eu) is 0.95; application losses including evaporation 0.90; emitter flow (qe) selected is 2 1/hr.

Calculation:

Fraction of surface area wetted (P) using Table 18 is
w/(1_1 x 1_2) = 0.4/(1.2 x 0.6) = 0.55
Depth of application (d) = (p.AW) D.P/ (Eu. Ea) = 32 mm
Irrigation interval (1) is (p.AW) D.P/ETcrop = 4 days
Flow duration (t) is d x 1_1 x 1_2/qe = 11.5 hours
Operation unit (N) is (I x 24)/t = 8
Stream size required assuming continuous operation of
the system 2.8A/N x qe/(1_1 x 1_2) = <u>39 1/sec</u>

Table 16: Flow rates per Drip Emitter (qe) in l/hr, Continuous flow for different E_{crops} and number of Emitters per ha.

ET_{crop}	Emitters per ha							
mm/day	250	500	750	1000	1500	2000	2500	5000
1.25	2.08	1.04	0.69	0.52	0.35	0.26	0.21	0.10
2.50	4.16	2.08	1.38	1.04	0.69	0.52	0.42	0.21
3.75	6.25	3.12	2.08	1.56	1.04	0.78	0.62	0.31
5.00	8.33	4.16	2.77	2.08	1.39	1.04	0.83	0.42
6.25	10.41	5.12	3.47	2.60	1.74	1.30	1.04	0.52
7.50	12.50	6.25	4.17	3.13	2.08	1.56	1.25	0.63

Table 17: Flow rates per Tree, Continuous flow for different E_{crops} and Tree spacing, l/hr.

Tree Spacing	$ET_{Orchard}$ mm/day		
	5	6.25	7.5
6 x 6	7.5	9.5	11
9 x 9	17	21	25
12 x 12	30	37	45
15 x 15	47	59	70
18 x 18	67	84	101

Table 18: Surface Area Wetted (w) in m^2 for Different Emitter Flow and Soil infiltration rate.

Emitter Flow L/hr	Soil Infiltration Rate mm/hr		
	2.5	5	7.5
2	0.8	0.4	0.25
4	1.6	0.8	0.50
6	2.4	1.2	0.75
8	3.2	1.6	1.00

Table 19: Sprinkler characteristics.

Nozzle mm	Pressure kg/cm^2	Wetted Diameter m	Discharge m^3 / hr	Spacing m	Area Irrigated m^2	Precipitation mm/hr
4.5	2.0	13.5	1.1	12 x 18	215	5.0
	2.5	14.0	1.2	12 x 18	215	5.5
	3.0	14.5	1.3	18 x 18	325	4.1
5.0	2.0	13.5	1.3	12 x 18	215	6.2
	2.5	14.5	1.5	18 x 18	325	4.6
	3.0	15.0	1.6	18 x 18	325	5.0
6.0	2.0	14.5	1.9	18 x 18	325	6.0
	2.5	16.3	2.2	18 x 24	430	5.0
	3.0	16.5	2.8	18 x 24	430	5.5
4.5/4.8	2.0	14.0	2.3	12 x 18	215	10.8
	2.5	14.8	2.6	18 x 18	325	8.0
	3.0	15.5	2.8	18 x 18	325	8.8
5.0/5.5	2.5	16.0	3.3	18 x 18	325	10.2
	3.0	16.3	3.6	18 x 24	430	8.4
	3.5	16.6	3.9	18 x 24	430	9.1
5.0/7.5	3.0	19.0	5.3	24 x 24	575	9.3
	3.5	19.3	5.8	24 x 24	575	10.7
	4.0	20.0	6.2	24 x 24	575	10.7
6.0/7.5	3.0	17.7	6.1	18 x 24	430	14.0
	3.5	18.5	6.6	24 x 24	575	11.3
	4.0	19.0	7.0	24 x 24	575	12.2

7. SPRINKLER IRRIGATION

7.1 GENERAL

In this method the water is applied as a spray above ground, somewhat resembling rainfall.

Most conventional systems, depending on the size of the farm and the lay-out, consists of:

- Pumping plant
- Mainline
- Sub mains or distribution lines
- Lateral line(s) with
- Sprinklers on
- Riser pipe (supported on stabilizers)

Portable systems are those where everything can be shifted; permanent (or stationary) systems have everything permanently installed. Very common are semi-portable systems where pumping plant and eventually the main line is permanent, the rest is not.

Most laterals in portable and semi-portable systems are of quick-coupling aluminium pipes; PVC is often used as buried main/submain. In permanent systems, steel pipes are sometimes used. For smaller diameter laterals, PE pipes are less costly.

7.2 SPRINKLER OVERLAP AND SPACING

In order to achieve an uniform application, adjacent sprinklers must overlap. This overlapping mainly depends on type of sprinkler and occurring wind conditions.

A common practise is a square or rectangular arrangement. Triangular arrangement mainly is used in permanent systems.

Overlap is (when winds are light or absent) some 60-65% of wetted diameter for distance between laterals, and some 40% of wetted diameter for sprinklers along the lateral.

Under high wind velocity spacing is reduced by about 10% for a wind velocity of 3-6 km/hr, some 35% with wind velocities 6-10 km/hr, some 65% for higher velocities.

In practice, most medium pressure sprinkler operate effectively at spacing of about 12 x 12 to 18 x 18m.

7.3 SPRINKLER CHARACTERISTICS

7.3.1 Discharge

Can be computed with

$$q = C.A. \sqrt{2g\ h.} \quad (m^3/sec)$$

C	=	nozzle coefficient (coefficient (0.9 – 0.98)
A	=	cross-sectional area of nozzle (m^2)
g	=	acceleration due to gravity (m/sec^2)
h	=	pressure at nozzle (m)

7.3.2 Wetted Diameter

Indicates the range that the sprinkler will wet a circle. Can be approximated by using the empirical formula:

R	=	1.35 d.h.	(m)
h	=	head in m	
d	=	diameter of nozzle in mm	

7.3.3 Sprinkler Selection

The manufacturer normally provides sprinkler-operating characteristics; an example is given in table 19 and figure 2. The only fixed criteria for sprinkler selection is based on preventing run-off and/or water impounding occurring. Table 20 can be used if no direct local data is available.

7.4 APPLICATION

7.4.1 Application Rate

Application rate (also termed precipitation density/equivalent or accumulation rate) are governed by discharge and spacing.

$$I = \left(\frac{q}{S_m \times S_1} \right) \times 1000$$

I	=	application rate in mm/hr
q	=	discharge in m^3/hr
S_m	=	spacing between laterals in m
S_1	=	spacing between sprinklers on lateral in m.

7.4.2 Duration of Irrigation t

Is the time a given lateral is to stay at a point to apply a given quantity.

$$t = \frac{I_R}{I}$$

t = duration in hours
I_R = gross irrigation requirement in mm
I = application rate in mm/hr

7.4.3 Irrigation Efficiency

Application efficiency E_a general ranges from 0.85 to 0.60 (under high temperatures and high wind velocities). Water conveyance efficiency E_c is high, since water is conveyed in pipelines.

7.4.4 Frequency of Irrigation

Time interval between two successive applications,

$$i = \frac{t \times I}{ET_{crop}} \times E_p$$

Example
Maize, silt loam soil, rooting depth of 0.80 m, maximum ET_{crop} = 6.1 mm/day E_p = 0.70, land slope is 2.5%, 12 working hours a day, no leaching required.
Sprinkler selected (max. application rate = 13mm/hr cfr table 20) from figure 2.

- Spacing 12 x 12m
- Pressure 2.1 kg/cm^2
- Nozzle size 3/16 inch
- Gives application rate of: 8.13 mm/hr
  Capacity of: 1.25 m^3/hr
  Wetted diameter of: 25.6m

Depth of irrigation = AW x D x P
= 140 mm/m x 0.80 x 0.55 = 61.6 ≈ 62mm

Frequency $= \frac{62}{6.1} = 10$ days

Gross water requirement
$= \frac{62}{0.7} = 89$mm

Duration of application
$= \frac{89}{8.13} = 10.9 = 11$ hours

Numbers of daily applications

$$= \frac{\text{working hours}}{\text{duration}} = \frac{12}{11} = 1$$

Table 20: Recommended Maximum allowable Sprinkling Rates (mm/hr)*

Describe of Soil and profile condition	Maximum Allowable Sprinkling Rate, mm/hr							
	0-5 %		5-8 %		8-12 %		Over 12 %	
	With Cover	Bare	With Cover	Bare	With Cover	Bare	With Cover	Bare
Sandy Soil Homogenous profile to depth of 1.8 m.	50	50	50	38	38	25	25	13
Sandy soil over heavier soil	45	38	32	25	25	18	18	10
Light sandy-loamy soil. Homogeneous profile to 1.8 m.	45	25	32	20	25	15	18	10
Sandy-loam over heavier soil.	32	18	25	13	18	10	13	8
Silty-loam soil. Homogeneous profile to 1.8m.	25	13	20	10	15	8	10	5
Silty-loam soil over heavier soil.	15	8	13	7	10	4	8	3
Clay soil, Silty clay-loam soil.	5	4	4	3	3	2	3	2

* Taken from the handbook of Engineering practices for Region 7, U.S.D.A, S.C.S. Part one – Section 6: Planning Sprinkler Irrigation Systems, p. VI-11 (1-6)

Table 21: Coefficient F for Plastic and Aluminum Laterals

n	**Plastic lateral, r = 1.760**			**Aluminum lateral, r = 1.852**		
	(a) F_1	**(b)** F_2	**(c)** F_3	**(a)** F_1	**(b)** F_2	**(c)** F_3
5	0.469	0.337	0.410	0.457	0.321	0.396
10	0.415	0.350	0.384	0.402	0.336	0.371
12	0.406	0.352	0.381	0.393	0.338	0.367
15	0.398	0.355	0.377	0.385	0.341	0.363
20	0.389	0.357	0.373	0.376	0.343	0.360
25	0.384	0.358	0.371	0.371	0.345	0.358
30	0.381	0.359	0.370	0.368	0.346	0.357
40	0.376	0.360	0.368	0.363	0.347	0.355
50	0.374	0.361	0.367	0.361	0.348	0.354
100	0.369	0.362	0.366	0.356	0.349	0.352
200	0.366	0.363	0.365	0.353	0.350	0.352

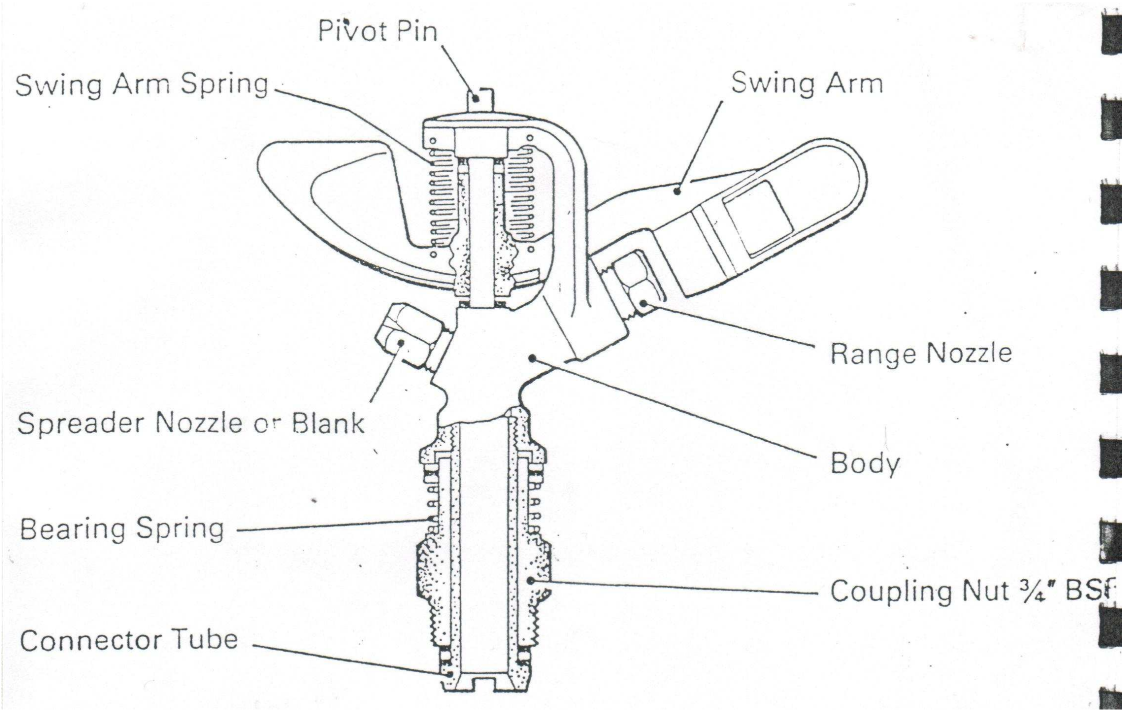
(a) F_1 is to be used when the distance from the lateral inlet to the first outlet is s_1 metres.
(b) F_2 is to be used when the first outlet is near the lateral inlet.
(c) F3 is to be used when the distance from the lateral inlet to the first outlet is s1 / 2 metres.

Table 22: Table on Performance for Single nozzles

Nozzle size	Pressure at nozzle		Discharge		Diameter				SPRINKLER SPACING											
									30' X 30' 9m x 9m		30' x 40' 9m x 12m		30' x 50' 9m x 15m		40' x 40' 12m x 12m		30' x 60' 9m x 18m		40' x 60' 12m x 18m	
			imp		Std		L/A		RAINFALL EQUIVALENTS PER HOUR - STANDARD MODEL											
inch	psi	kg/cm^2	gmp	m^3/hr	ft	m	ft	m	inch	mm	inch	mm	inch	mm	inch	mm	inch	mm	inch	mm
$^{6}/_{32}$	30	2.1	325	89	80	24,4	64	19,5	.41	10,41	.31	7.87	.24	6.10	.23	5.84	_	_	_	_
	40	2.8	3.83	1.04	83	25,3	69	21.0	69	12.45	.37	9.40	.28	7.11	.27	6.66	.24	6.10	_	_
	50	3.5	4.33	1.18	85	25.9	73	22.3	.55	13.97	.41	10.41	.32	8.13	.30	7.62	.27	6.86	.21	5.33
$^{11}/_{81}$	30	2.1	3.81	1.04	83	25.3	65	19.8	.49	12.45	.37	9.40	.28	7.11	.27	6.86	.24	6.10	_	_
	40	2.8	4.50	1.23	86	26.2	69	21.0	.58	14.73	.43	10.92	.34	8.64	.31	7.87	.28	7.11	.22	5.59
	50	3.5	5.05	1.38	88	26.8	74	22.5	.65	16.51	.49	12.45	.38	9.65	.35	8.89	.32	8.13	.24	6.10
	60	4.2	5.41	1.47	90	27.4	79	24.1	.70	17.78	.52	13.21	.40	10.16	.36	9.05	.34	8.64	.26	6.60
$^{3}/_{18}$	30	2.1	4.58	1.25	84	25.6	68	20.7	.58	14.73	.44	11.18	.35	8.89	.32	8.13	.28	7.11	.22	5.59
	40	2.8	5.33	1.45	87	26.5	73	22.3	.68	17.27	.51	12.95	.40	10.16	.37	9.40	.33	8.38	.26	6.00
	50	3.5	6.00	1.64	69	27.1	77	23.5	.77	19.56	.58	14.73	.45	11.43	.42	10.67	.38	9.65	.29	7.37
	60	4.2	6.45	1.76	91	27.7	81	24.7	.83	21.80	.62	15.75	.49	12.45	.46	11.68	.41	10.41	.31	7.87
$^{13}/_{64}$	30	2.1	5.33	1.45	85	25.9	-	-	.68	17.27	.51	12.95	.41	10.41	.38	9.65	.34	8.64	.26	6.60
	40	2.8	6.16	1.68	91	27.7	-	-	.79	20.07	.59	14.99	.48	12.19	.45	11.43	.40	10.16	.30	7.62
	50	3.5	6.91	1.88	94	28.7	-	-	.89	22.61	.66	16.76	.53	13.46	.50	12.70	.44	11.18	.33	8.38
	60	4.2	7.66	2.09	97	29.6	-	-	.98	24.89	.74	18.80	.59	14.99	.55	13.97	.49	12.45	.37	9.40
$^{7}/_{28}$	30	2.1	6.08	1.66	89	27,1	-	-	.78	19.81	.58	14.73	.47	11.94	.44	11.18	.39	9.91	.29	7.37
	40	2.8	7.00	1.91	94	28,7	-	-	.90	22.86	.67	17.02	.54	13.72	.51	12.95	.45	11.43	.34	8.64
	50	3.5	7.91	2.16	97	29,6	-	-	1.02	25.91	.76	19.30	.61	15.49	.57	14.48	.51	12.95	.38	9.65
	60	4.2	8.83	2.41	100	30,5	-	-	1.13	28.70	.85	21.59	.68	17.27	.64	16.26	.57	14.48	.43	10.92

Diameters quoted are maximum obtainable under still air conditions. For normal operating conditions in the field the effective diameters will be approximately 10% less.

Fig 2: Typical Sprinkler diagram



7.5 Lateral hydraulics

7.5.1 Friction losses

The flow in a lateral decreases along its length. The procedure for computing head loss in a lateral is as follows:

(i) select a sprinkler from catalog, capacity q, spacing S_1 and pressure h is thus known

(ii) numbers of sprinklers along the lateral, n, is determined

(iii) discharge at inlet of lateral Q_u is determined

(iv) a lateral diameter, D_1, is selected

(v) head loss (from nomograph) = $7^m/100^m$
lateral length = 10 x 12m = 120m
head loss = $\frac{7 \times 120}{100}$ = 8.4m

(vi) coefficient F = 0.402 (Table 21)
head loss in lateral = 8.4 x 0.402 = 3.4m

7.5.2 Design

Since in a lateral friction losses occur, pressure (also to the sprinkler) will reduce. It is common practice that discharges from sprinklers should not vary by more than 10% of the designed discharge.

Hence, the criteria governing the size (and length) of a lateral in terms of pressure head or known as the “20% rule”. This states that the pressure head variation along a lateral (comprising both friction loss and elevation difference) must be limited to 20% of the pressure head of the design sprinkler.

Rules:
- (a) friction loss in lateral < 20% of design head if higher, select a larger diameter D or reduce length of lateral.
- (b) velocity < 2m/sec
- (c) length for potable laterals should be limited (max. 250m, 400m under specific conditions)

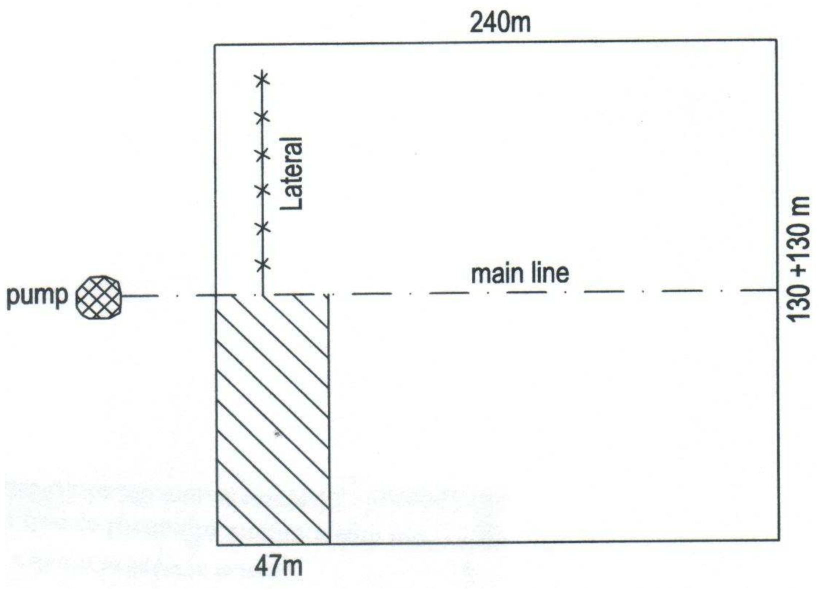
For portable systems, laterals are generally of the same diameter throughout their length.

7.6 NUMBER OF LATERALS OPERATING SIMULTANEOUSLY

The maximum numbers of laterals is governed by

- Available water
- Total investment and economics.

The minimum number of laterals depends on layout and crop water requirements. For example, for the field (see par. 7.3.4 and 7.4.1) being 240 by 260m, the layout is as sketched below.



Number of lateral settings (places where a lateral is supposed to be connected) is

$$Ls = \frac{240}{12} + \frac{(240 - 47)}{12} = 20 + 16.1 = 36$$

It would be possible (if available water permits) to connect 36 laterals (with 10 sprinklers each) to the pump, and irrigate the whole area in 1 day. With a frequency of 10 days (see 7.3.4), this would mean that for the other nine days, we do not use the equipment!

On the other hand, since we require 1 to apply the irrigation requirement (11 hours), it would take us 36 days to irrigate the whole field which is impossible since we have to irrigate every 10 days.
Somewhere between 1 and 36 lies the optimum number of laterals.

The minimum number of laterals is

$$L_{min} = \frac{L_S}{i} \times \frac{1}{m} \quad \text{(rounded up)}$$

Where

m	=	number of daily applications (shifts of lateral per day)
L_S	=	number of lateral settings
I	=	interval in days

In the example

$$L_{min} = \frac{36}{10} = 3.6 = 4 \text{ laterals}$$

Maximum number of laterals L_{max} is limited by available water.

If $L_{max} < L_{min}$ then the area under irrigation should be reduced, or hours of operation extended.

In the example, if we irrigated for 22 hours/day, we could put each lateral on 2 position (m = 2), hence minimum number of laterals required would be:

$$\frac{36}{10} \times \frac{1}{2} = 1.8 = 2 \text{ laterals.}$$

7.7 RISER PIPES

In order to achieve the characteristics indicated by the manufacturers, a riser pipe (generally 1″ diameter) must be installed between lateral and sprinkler.

Recommended minimum values are given in table 22, although often they are dictated by crop type.

Table 22: Riser pipe heights

Flow in 1/sec	Height in cm
0.75	15
0.75 – 1.50	20
1.50 – 3.0	30
3– 8	45
8	90

7.8 MAINLINE DESIGN

The function of the main lines and submains is to convey the required quantity of water at the desired pressure to all lateral lines under maximum pressure conditions. The selection should be based on economic considerations. Mainline frictional lose of about 3 meters for small systems and 12 meters for large systems may be allowed. The friction losses in main lines of portable aluminium pipe, semi-rigid plastic irrigation, PVC pipes, asbestos cement pipes are readily available in the form of tables, formula's monographs. It is basically an hydraulic problem, with the design based on the condition that indicates the maximum hydraulic loss (function of lateral positions in the field).

Main lines, and distribution lines are often for larger schemes or different diameter.

7.9 PUMPING PLANT

In selecting a suitable pump, it is necessary to determine

- the capacity
- total dynamic head

The capacity can easily be found. It is the number of sprinklers times that discharge. Any other requirement (domestic, livestock etc.) has to be added.

The total dynamic head against which the pump is to work, is composed of the following:

(i) Operating pressure at head of lateral, consisting of
- Sprinkler operating pressure
- Riser pipe height
- 75% of friction losses in the lateral
- difference in elevation in lateral
- local losses due to bends, valves etc…….

(between 10 – 20% of friction losses)

(ii) friction and local losses in mainline and eventually distribution lines.

(iii) difference in elevation in mainline and distribution lines

(iv) local losses at pump outlet (valve, elbow, meter, ….)

(v) difference in elevation between mainline and water source

(vi) suction line losses.

Power of pump then can be calculated:

$$HP = \frac{h \times Q}{75 \times E} \quad \text{(horse power)}$$

h = total dynamic head (m)
Q = discharge (l/sec)
E = pump efficiency (fraction)

7.10 DETERMINATION OF SEQUENCE OF OPERATION OF LATERALS ALONG MAINS/SUBMAINS

With the number of laterals operation simultaneously, their sequence of operation is determined.

Two examples are given.

Fig 3: Two laterals coupled to a single off-take

Lateral 1 moves on upper part of field, lateral 2 moves on lower part of field. After they finish in position 8, they move back to position 1. Mainline is design for 2 x flow in a lateral ($2Q_u$)

Fig 4: Two laterals, each covering half the field

Lateral 1 moves from position 1 to 4, then on the lower part from 4 to 1.
Lateral 2 moves from positions 5 to 8, then on the lower part from 8 to 5.

Mainline is designed

- for 2 Q_u for first stretch (up to position 4)
- for Q_u for the last stretch (from position 4 to 8)

It is also possible to have the two laterals working side by side, e.g. in position 1 and 2, 3 and 4 etc. This "block" system is easier to manage with several laterals. But the mainline should then be designed for $2Q_u$.

7.11 CONSIDERATIONS - REMARKS

7.11.1 Smaller Systems

For smaller systems, pressure at sprinkler should range between 2 to 4 kg/cm^2. Although higher pressures allow greater spacing, this should be avoided as there is a risk of decreasing uniformity of application; and at the same time higher pressure will be required.

7.11.2 Labour Requirements

Labour requirements for shifting laterals (and eventually distribution and main lines) have to be taken into account when comparing working hours per day with actual hours of irrigation. For a portable system this is between 1.0 to 2.5 hours/irrigation/hectare depending on skill and layout.

7.11.3 Duration

NEVER design a scheme to operate 24 hours per day.
Advised maximum 20 hrs/day.

7.11.4 Laterals

Laterals are generally positioned at right angles to the prevailing wind direction.

7.11.5 Limited Water Supply

When water supply is limited consideration on crop selection and acreage irrigated should be based on crop yields as affected by the extent to which crop water requirements are met by the available water supply over the growing season. Considering the effect of limited seasonal supply on crop yields and production can make a first evaluation. Where crops under consideration are sensitive to water supply deficits, scheduling of the supply is based on minimizing water deficits on the most sensitive growth periods. During periods of unpredictable water shortages, within season adjustments of water scheduling must be made in relation to the difference in the yield response to water deficits on crops and their individual growth periods. This applies both to controlled and uncontrolled water supply at the head works.

Figure 3: Submain with two hand-moved laterals coupled to a single take-off

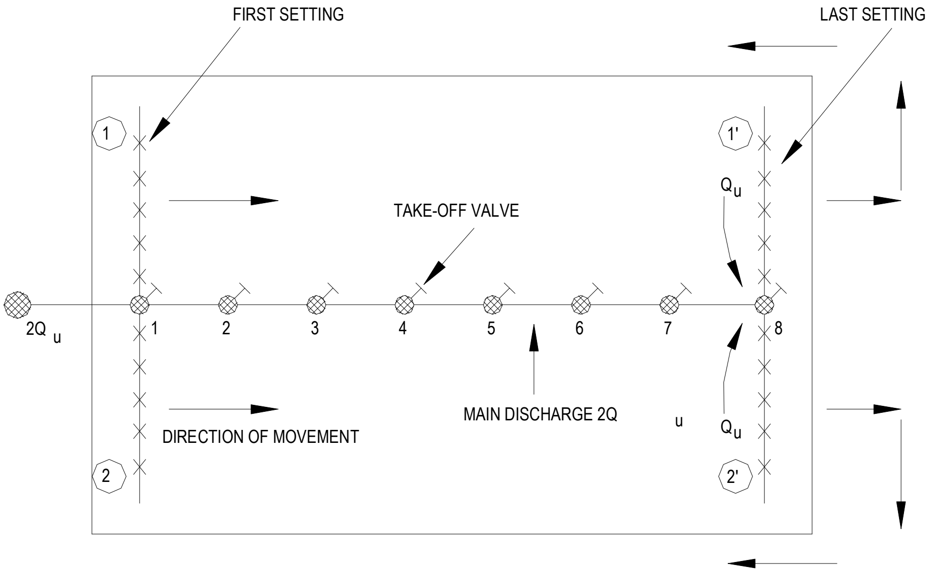
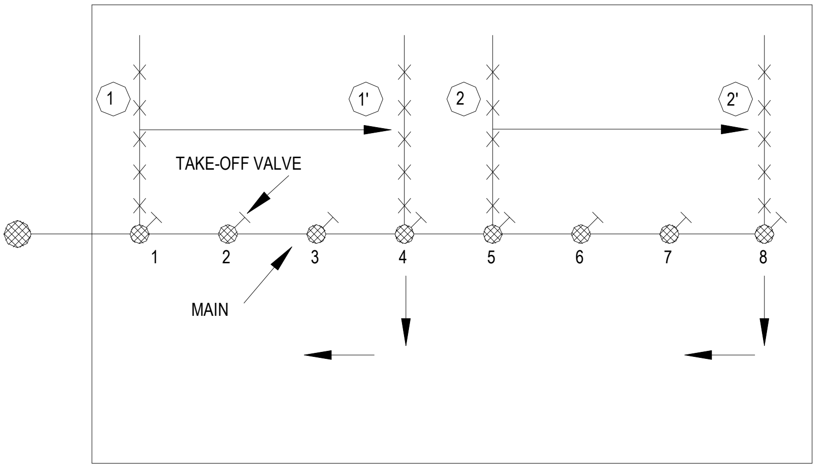
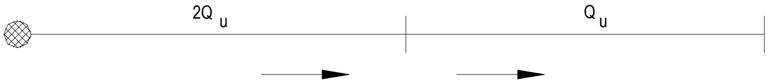


Figure 3: Submain with two hand-moved laterals each irrigating half of the field





8 WATER QUALITY FOR IRRIGATION WATER

8.1. GENERAL

Good quality water has the potential to allow maximum economical returns. With poor quality water, soil and cropping problems have to be expected to develop, which will reduce yields unless special management practices are adopted.

The suitability of water, from quality point of view, is determined by its potential to cause problems and its relation to the special management practices needed.

By selecting crops and adequate management one may find that water considered "unsuitable" under the prior concept of quality may really be "usable" under certain conditions. Poor quality water is often better than no water.

Following is some few guidelines on classification of irrigation water.

8.2 SALINITY CLASSIFICATION

Salts desolved in irrigation water will remain in the soil when the water is used by the plants. High salt content in soils will reduce the availability of soil water to the plants.

Water for irrigation is divided into 4 classes depending on soluble salt content as follows:-

C1 -Low salinity - less than 250 micromohs/cm (at 25^0C)
This can safely be used for irrigation of most corps on most soils, with a very little hazard of salinity.

C2 - Medium salinity – 250 to 750 micromohs/cm
This can be used if moderate amount of leaching occurs. Plants with moderate salt tolerance can easily be grown in most cases without special practices for salinity control.

C3 - High salinity – 750 to 2250 micromohs/cm
This water cannot be used on soils with restricted drainage, even with adequate drainage, special management for salinity may be required and plants with good tolerance must be selected.

C4 - Very high salinity – over 2250 micromohs/cm
This type of water is unsuitable for irrigation, except under very special conditions.

8.3 SODIUM HAZARD CLASSIFICATION

High amount of sodium in irrigation water can cause a severe soil permeability problem. The greater the quantity of sodium present the greater the risk of soil structural and pH problems developing.
The most commonly used method for evaluating the sodium potential is sodium absorption rate (SAR):

$$SAR = \frac{Na^{+}}{\sqrt{\left(\frac{Ca^{++} + Mg^{++}}{2}\right)}}$$

Where: Na, Ca and Mg represent the concentration in milliequivalents per liter.

The SAR determines the exchangeable sodium (ESP) of a soil. A high ESP defines a soil which has a high degree of saturation with exchangeable sodium – or an alkali soil. Four classes are generally recognized or sodium hazard:

S1- Low sodium water – SAR is 0 to 10
Can be used on almost all soils with little danger of the development of harmful levels of ESP.

S2 - High sodium water – SAR is 10 to 18
May present an appreciable Na – hazard in fine textured soils, especially under low-leaching conditions. This water may be safely used on course textured soils or organic soils with good permeability.

S3 - High sodium water – SAR is 18 to 26
Will produce harmful levels of exchangeable sodium in most soils and will require special soil and water management, e.g. good drainage, high leaching and organic matter addition.

S4 - Very high sodium water – SAR is over 26
Is generally unsatisfactory for irrigation purposes, except at low and perhaps medium salinity with special management. With reference to salinity and sodicity, 16 classes are recognised – see figure 5.

8.4 SODIUM TOXICITY

Most tree crops are sensitive to low concentrations of sodium. Most annual crops are affected by higher concentrations. As the crop takes up the water, it also takes up the sodium, which will accumulate in the leaves as the water is lost by transpiration.

Damage (toxicity) will occur when this concentration exceeds the tolerance of the crops. Sodium toxicity is often modified if Calcium (Ca) is present.

8.5 BORON

Boron is one of the essential elements for plant growth but it is needed in relatively small amounts. If excessive, it becomes toxic.

Few surface streams have boron problems; it is more prevalent in wells and springs.

The relative sensitivity of crops for boron varies widely. Lemon is affected when the water contains more than 1 ppm, while Lucerne gives maximum yields by a ppm of 1 to 2.

8.6 CHLORIDE TOXICITY

Chloride is not absorbed by the soil, but moves readily with the soil water. It is taken up by the plants through the roots and accumulates in the leaves.

Most tree crops are sensitive to low concentrations, most annual crops are not that sensitive.

8.7 NITRATE AND AMMONIUM NITROGEN

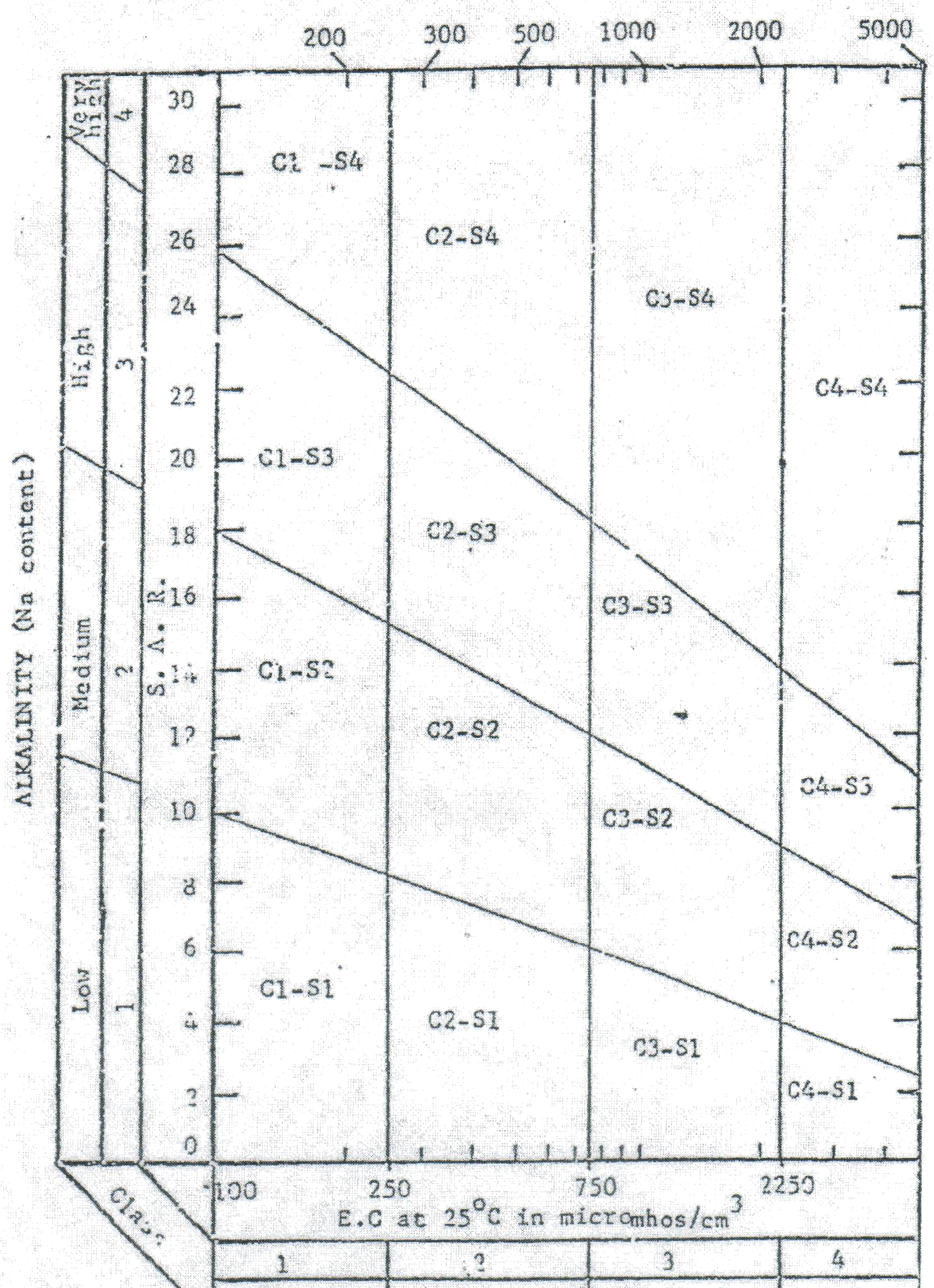
These two forms of nitrogen are nutrients. But excess nitrogen in the irrigation water will cause the same problem as excess fertilizer will cause. These include delaying of maturity, reduction in yield, lodging due to excessive vegetative growth, growth of algae and aquatic plants in ponds and lakes.

8.8 pH

The pH is a measure of the acidity or alkalinity of the water. It is of interest as an indicator, but seldom of real value by itself. The main use of the pH is a quick evaluation of the possibility that the water may be abnormal.

A pH 1 to 7 being acid, 7 to 14 being alkaline, whereas a pH of 7 is neutral. The normal range of irrigation water is from 6.4 to 8.4; within this range crops have done well. Irrigation water having values outside this range may still be satisfactory but other problems of nutrition or toxicity become suspect.

Figure 5: Sodium Hazard - Classification of the US Salinity Laboratory



9. APPENDICES AND EXAMPLE ON IRRIGATION

9.1 PART E OF WATER SUPPLY MANUAL: APPENDIX

Included in part E: Appendix E of this Manual is two tables and two maps giving some general data on irrigation water requirements in Kenya.

The tables give information on irrigation requirements for average and dry months, computed using a crop coefficient of 1.0, no leaching and an application efficiency of 0.60. Water conveyance efficiency is NOT included.

Map 1 gives average annual water requirement in thousands cubic meters. Map 2 indicates design abstraction flow (flow needed at inlet of field) in litres per second per hectares; with an application efficiency of 0.60, no leaching, irrigating 10 hours a day.

This general data can change with crop type, soil type, method of irrigation, etc. and conveyance efficiency should be added.

9.2 EXAMPLE ON COMPUTATION OF RAINFALL PROBABILITY

<u>Computation of rainfall probability</u>

An example is given in the following two pages, based on yearly data. Using the same method, monthly data can be determined.

EXAMPLE:
A simple method is by grouping the rainfall data; a rough indication of rainfall probability is obtained by the number of times the yearly amount falls within a group divided by the number of years of record.

Year	1956	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71
Mm/month	75	85	50	65	45	30	20	65	35	80	45	25	60	75	40	55

Highest value is 85 and lowest 20mm. Using a 10mm grouping:

0 - 9	0	50 - 59	2x
10 – 19	0	60 – 69	3x
20 – 29	2x	70 – 79	2x
30 – 39	2x	80 – 89	2x
40 - 49	3x	90 - 99	0
			16

Rainfall will equal or exceed 40mm for 3 out of 4 years (or 12/16).

An improved estimate can be obtained computing and plotting rainfall probabilities.

The steps involved are:

- Tabulate rainfall totals for given period (line 2)
- Arrange data in descending magnitude and give rank number m (lines 3 and 4)

- Tabulate plotting position (Fa) using 100m/(N + 1). N is total data number, m is rank number with m = 1 for the highest value (line 5)
- Prepare vertical scale and plot rainfall according to Fa position on log-normal probability paper (Fig.A.1)

Line																	
1	Year	1956	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71
2	Mm/given month	75	80	75	75	65	65	60	55	50	45	45	40	35	30	25	20
3	Sequence	85	80	75	75	65	65	60	55	50	45	45	40	35	30	25	20
4	Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
5	Plotting position (Fa)	6	12	18	24	29	35	41	47	53	59	65	71	76	82	88	94

From Fig. 6: dependable rainfall 3 out of 4 years, or 75% probability, for given month is 36mm; 4 out of 5 years, or 80% probability, 32mm etc.

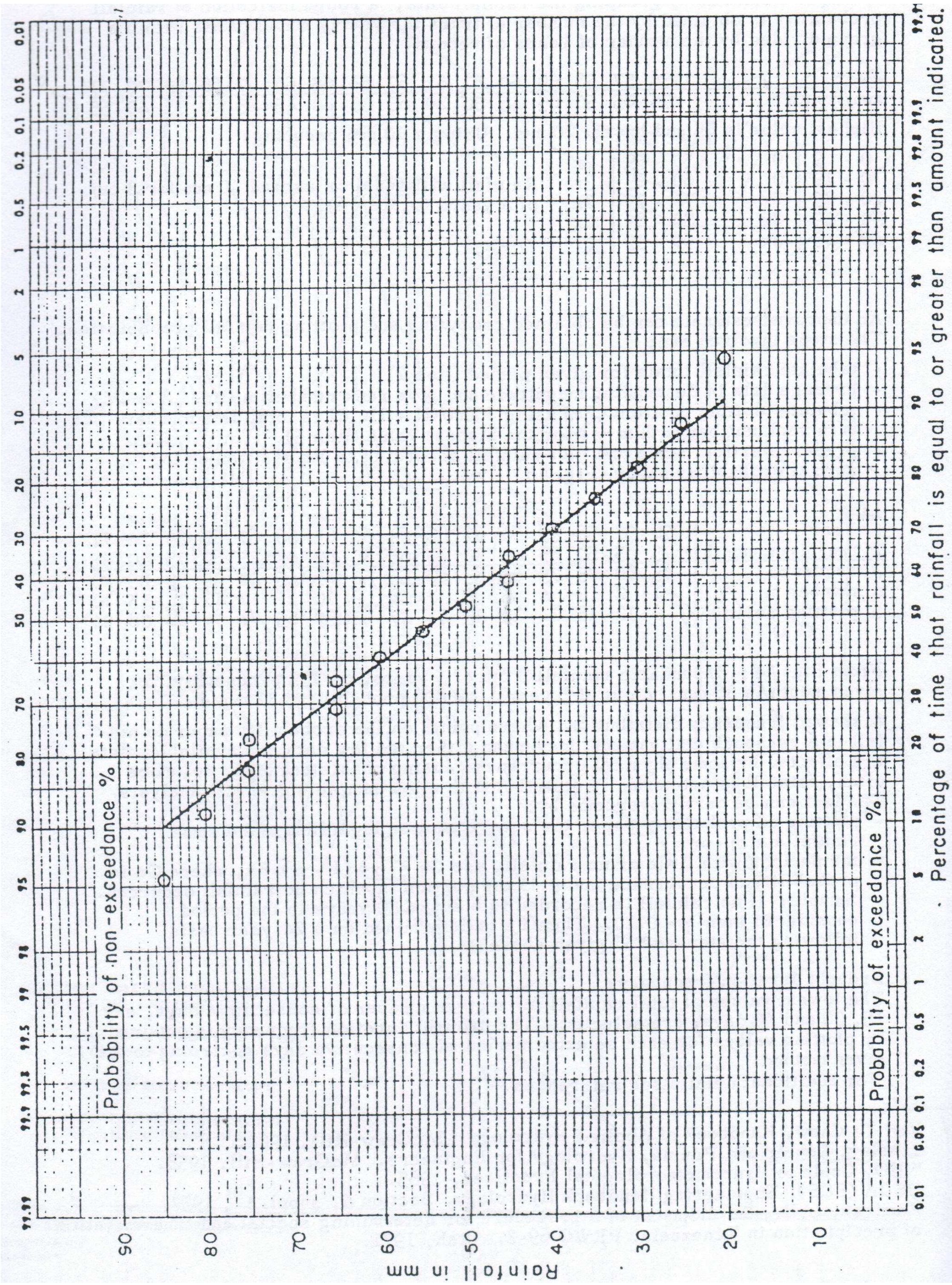
A skewed frequency distribution, where points on the probability paper do not fall in a reasonable alignment, may mean too few data are available, data are affected by some physical occurrence causing consistent bias, or, more often, rainfall is not normally distributed to allow simple statistical analysis. The last can be partly overcome by:

- Plotting on probability paper the square root or logarithm of the same rainfall data; or
- For periods with little or no rainfall, use Ga = p + (1-p). Fa, where Ga is probability of occurrence and p is the portion in which no rainfall occurred. Sample: if no rainfall is recorded in 6 out of 30 years in the period considered then p = 0.20. Then Fa is determined on a 24-year basis following the step method give above.

Drought duration frequency:
The lowest values of total rainfall for a given number of consecutive days, say 15, 30 and 40 days, are selected. The drought duration frequency is obtained by plotting values for each selected period of consecutive days according to the given method. For additional details see references. 1/

1/ Ven Te Chow, Handbook of Applied Hydrology. McGraw-Hill, 1964.
Linsley, Kohler and Paulus, Hydrology for Engineers. McGraw-Hill, 1958.
WHO, Guide to Hydrometeorological Practices, 1965.
USDA (SCS), Engineering Handbook Hydrology, Section 4, suppl.A., 1957.
Ramirez, L.E.< Development of a procedure for determining special and time variation of precipitation
in Venezuela, PRWG 69-3. Utah, 1971.

Figure 6: Rainfall probability calculation



PRACTICE MANUAL

FOR

WATER SUPPLY SERVICES

IN

KENYA

PART C

SMALL DAMS

LIST OF CONTENTS

1. INTRODUCTION

1.1 GENERAL

The purpose of this manual is to serve those concerned with water reservoirs within the Water Sector as a guideline, but also to field officers when sent out in the field for investigations as a kind of check list when siting new possible reservoirs or to make recommendations towards rehabilitation of existing dams.

The manual is also meant for Water Managers in the field when recommending new dam sites for exploitation as certainly in the remote areas of Kenya the more qualified pre-investigations by those officers will save decision-makers considerable time as impossible or bad sites will have already been eliminated when the officers visit the area

It should be recognized that the methodology in this manual for hydrology and soil-mechanics is very brief and that in case of doubt or larger schemes or very high-required reliability reference should be made to international handbooks.

1.2 RESTRICTIONS

The methods for design, described in this manual will, for safety and economical reasons, have to be restricted to dams and reservoirs of certain dimensions and construction methods.

For dams beyond those limitations internationally recognized hand books will have to be utilized, such as:

- Design of small dams, U.S.B.R.
- Engineering of dams, Hinds, Greger & Justin
- Technique des barrages, French Government

The limits set for the dimensions are the following:

(a) Dam height, for earth dams, up to 10 metres with a maximum of 15 metres. For higher walls knowledge of the specific soil characteristics for stability calculations become essential.

(b) Reservoir capacity unto 200,000 m^3 with maximum of 500,000 m^3. Larger reservoirs require more sophisticated water use and loss studies for economical reasons. At the same time damage through dam failure as a result of insufficient stability calculations, will become unacceptable. The upper limit to be chosen, also depends on the extend of the downstream interests to be safeguarded.

(c) Homogeneous earth dams with the possibility of a central core are considered only. Zoned dams require better knowledge of the materials to be used.

(e) Concrete and other dams are not considered in this manual, although all pre-studies and reservoir characteristics are the same as for earth dams.

(d) Concrete and other dams are not considered in this manual, although all pre-studies and reservoir characteristics are the same as for earth dams.

For each decision concerning the limits of construction (a) and (b) should be appraised together, avoiding that both limits will be in their maxima.

1.3 DEFINING PROJECT

Previous to the design a proper investigation into the general requirements of the area under consideration should be carried out.

The objectives and needs should be defined and written down. Contradictory purposes should be marked and a solution or a compromise will have to be found. The possibility of alternative sources to dam construction should be considered, mainly on economical grounds but also for health, esthetical, environmental and property reasons.

The construction of a dam will generally be a multi-purpose project whereunder the next items will have to be considered.

(a) Domestic Supply: Usually Reservoir water is polluted and treatment required. In the case that this is the only purpose natural sources or groundwater should be investigated. The latter two usually may supply water of such a quality that treatment costs will be avoided.

(b) Stock Supply: Water of lesser quality than for the previous purpose is permissible. If however in conjunction with domestic supply, straight watering from the reservoir will not be possible and draw-off facilities will have to be provided.

(c) Irrigation: This is not interfering with the above requirements. It is however in the case of small earth dams usually an expensive source of water for irrigation purposes and requires policy and economical consideration.

(d) Conservation: If the sole purpose of the dam construction is water conservation per se, other means should be sought as this is an expensive and not very efficient way. In combination with other purposes it will be useful.

(e) Flood control and stream flow regulation: Both practicable for smaller dams, but will require reservoirs in ratio to the streams catchments.

(f) Wildlife: As for stock supply, but usually not combinable with domestic supply because of the shy nature of the consumers.

(g) Fish Breeding: This is combinable with most other purposes, but requires perennial over-sized reservoirs as storage depletion will not be permissible.

(h) Industrial Use: Very well possible on small scale.

(i) Hydro-power: This will be hardly practicable, because of the limited storage, irregular draw-off not conceding with electricity requirements (day versus night) and low heads available.

The following negative considerations should be looked into, as they could lead to changing of the plans or even abandoning of the project.

(j) Sedimentation: The sand and silt load of a stream can be such that dam construction will be economically unfeasible as the reservoir's capacity will decrease too fast. Either a concrete dam with scour possibilities or a complete change of plans or source should be considered. If however dam construction will still prove the only possibility, drastic action on soil conservation should be undertaken in the catchment's area.

(k) Site suitability: Are sites available where the ratios between storage and earth movement are favourable?

The following ratios should be considered.

Earth movement (m^3)		Water storage (m^3)	
1	:	2 – 3	poor site
1	:	5	fair site
1	:	10	decent site
1	:	20	good site

Evaporation might be prohibitive, no shallow reservoirs in high evaporation areas, wells or boreholes should be considered. For low water requirements closed steel or concrete reservoirs could be considered.

(l) Availability of other sources: A pumping scheme form a nearby perennial river or other source should be considered and an economical comparison made.

(m) Construction water: Is enough water within decent proximity available for compaction of the embankment? This could schedule the construction period to the times of the year when waterholes and other rivers are still yielding sufficient water.

After having established the objectives of the project a first, rough design and cost estimate should be prepared in order to prove the feasibility of the project, in connection to this the cost estimate should, if possible, be compared with alternative supply systems and sources. The unit cost per capita and per m^3 of water should also be calculated.

1.4 WATER REQUIREMENTS

After having defined the project, the net water requirements at the head of the distribution system will have to be calculated. See Part A "Water Supply" chapter 2.

1.5 STORAGE RELIABILITY

Once the net water requirements are established the period for which the storage should cater will have to be fixed. The storage should last for the length of a dry season. The length of an average dry season will not do, as per definition half of the dry season will not do, as per definition half of the dry season will not do, as per definition half of the seasons are longer than the average dry season and would therefore render the storage insufficient for 50% of the time. Consequently longer periods will have to be accounted for. In the arid areas one of the usual two rainy seasons could very well miss out or render so little run-off that replenishment of the reservoir will not be achieved, consequently storage for a nine to eleven month period will have to be catered for.

The reliability for which the dam shall be designed will depend on the use of the water. For domestic water supply see Part A "Water Supply" Chapter 2.

The thus required period for storage computation can be found from rainfall records and stream discharge records, while local information will complete the picture. If no stations are available within the catchment, comparison with other similar catchments should be made.

However, in many of the catchments in the rural areas no precipitation nor discharge data are available and a guess towards the length of the dry period will have to be made. The next catering periods for different rainfall areas are suggested:

Mean annual precipitation mm	Dry period (month)
1200 - 1000	5
1000 - 800	7
800 - 400	9
below - 400	11

The above suggested periods should be checked against actual information.

The net monthly water requirements and the now established length of the catering period give the total net water or storage requirements.

2 DAM LOCATION

2.1 DAM SITE SELECTION

After having established the water requirements, a rough idea about the required storage is available. The gross storage required in ratio to the net storage calculated ranges from a factor 1.2 to 5.0 (i) depending on the reservoirs being rather large and deep in low evaporation areas to small and shallow in high evaporation areas. An estimate for the gross storage requirements or the reservoirs capacity will have to be made in order to select a suitable site, which will fit the estimated storage.

The following general considerations should be taken into account when looking for a suitable site:

2.2.1 Proximity

The site should be as close to the consumers as possible, in order to avoid long and expensive pipelines or long walking distances.

2.2.2 Size of Catchment

The size of the catchment area should be if possible in relation to the required storage. The catchment area should be big enough to ensure replenishment of the reservoir even in moderate dry years. Yet the catchment area should not be too big as otherwise the spillway and freeboard (dam-height) will become too large, hence too expensive in relation to the dam wall. As a first indication towards the minimum size of catchment area; the storage required should be equal to 5% of the mean annual rainfall on the catchment area. The 5% represents the runoff factor, which includes a safety for dry years as the normal runoff factor for an average year is usually around or above 20% for Kenya.

Cadastral, topographical and geological complications can have serious impacts on this consideration.

2.2.3 Gravity Supply

If possible the site should be selected in such a way that gravity supply is possible, avoiding recurrent pumping costs.

2.2.4 Sedimentation

Heavy sediment transport in a river (erosion upstream) can render a river thus the site useless, on the spot investigations are required.

2.2.5 Pollution

Natural and/or man-made pollution can render a site useless. The stream can carry undesired minerals from its source or be polluted by industry or wet season surface effluent from a village. In both man-made cases a site upstream of the pollution source or on another stream should be considered whenever no treatment works are to be included. In case treatment is considered, a cost comparison between more extensive treatment and a cleaner source further away should be made.

2.2.6 Property

Property to be effected on the site selected can put a constraint, as expropriation might be too expensive or impossible leading to a change in plans. Ownerships of reservoir area and dam site should be sorted out thoroughly.

Technical considerations to be looked into for each possible site individually are the following: -

2.2.7 Topographical

A narrow part on a river valley just downstream of a relatively wide part on a stretch of the river which is sloping as little as possible are sites to be considered.

2.2.8 Geological

Will the reservoir be able to hold the water. This is to say that the area should not be sandy nor have extensive sand layers from old river beds (which can be closed off by a core trench) nor on heavy fissured rock, murram or laterite.

Major geological faults can drain a reservoir seriously, although this becomes relative unimportant for smaller reservoirs. Filling of the reservoir causes a freatic water table in the immediate surroundings changing the stability of the soil, which could cause minor landslides, but more important, heavy structures standing on the shore of the reservoir to be could be seriously affected by the new water table.

As for the foundation of the wall the material should be solid enough to carry the weight of the dam, which usually is in Kenya except for swampy areas.

Care should be exercised with regards to piping (heavy seepage carrying soil particles thereby developing channels which eventually lead to dam failure). Sand layers, rock surface (and bad compaction) can lead to piping. The risk of piping with smaller dams because of the low water heads is usually not too serious.

2.2.9 Construction material

Suitable material should be available nearby.

2.2 SPILLWAY POSSIBILITIES

With nearly all earth dam constructions the spillway should be kept away from the earth wall in order to avoid expensive protection structures.

Consequently steep valleys, requiring much excavation for the spillway channel next to the wall should be avoided.

The places to look for are narrow valley stretches with relative steep side's upto the required normal water level, above which at lease one of the valley embankments flattens off considerably to allow shallow cut for the spillway channel.

In case of a depression in the valley embankment leading into a side valley presents itself this could be used as a natural spillway if the bottom level of this depression is roughly at the required storage level.

If possible the spillway should be located on the bedrock, of which the weathered part is normally rippable by the bulldozer downhill the required level. Channels entirely in earth are usually subject to erosion, certainly in areas where the river's base flow will pass for a few months or more, requiring special structures for protection.

Too steep valleys for earth channel excavation require concrete spillways either as a concrete channel over the dam or as a Morning Glory type through the wall. In those cases, certainly for low walls, all concrete dams should be considered.

The procedure to be followed locating a site is the following keeping the above-mentioned items in mind.

Maps should be consulted as for possible stretches of rivers to be utilized. If the contour interval of the maps is less than $^{1}/_{3}$ of the expected dam height a first selection of sites can be made from the maps. The existing 1:50,000 S.K. maps with 20 metres intervals are **not** suitable for site identification as all dam heights will fall within this interval.

Once river stretches have been identified a safari will have to be organized in order to select a site on the spot. Part of the river stretch will have to be walked, as the local population usually cannot be relied upon for identification since they do not recognize the problems involved with site selection.

It is very well possible that two or even more alternative sites are spotted while heavy bush or other problems prevent a final decision towards the most economical site for the relation between water stored and backfill-excavation required. In those cases a global topographical (and geological) survey should be undertaken to enable the selection of final site after all the cost of the survey is less than 5% of the total cost, while a wrong decision can add considerably more to the cost of the dam as it is difficult without much experience to estimate the backfill and excavation within a range of 25 – 100%, the latter being the major cost components of earth dam construction.

3 FLOOD STUDIES

The majority of earth dam failures (around 90%) in the world and also in Kenya are due to under-designed spillways or under estimated floods which cause over-topping of the wall followed by erosion of the downstream slope and finally the breaching of the wall. Consequently obtaining accurate figures for the flood expectation is the most important part of the design from the safety point of view.

As flood studies are a field for specialized people it should be left to them to prepare the proper calculations and estimates. This will be the hydrology section of the Ministry to which requests for flood expectations will have to be directed.

While investigating the dam site on the spot flood information should be gathered, like looking for flood signs in the valley which give water heights hence discharges, the size of the streambed in relation to the water shed the amount of sediments. Both the latter give an idea of the size of flashfloods in relation to the catchment area. The local population can be questioned as to the height of the floods in the past, this information however is not always reliable as it is subject to subjective observation and exaggeration. All this information can be used as a modification towards the calculations.

For many areas in Kenya, especially the Northern Districts, **no** contour maps are available and as a result the hydrology section cannot do the flood studies. However as the dams still will have to be build a decently sized spillway will have to be included, hence flood expectations will have to be prepared. In all those cases, comparative studies will have to be made. Similar catchment areas in regions where flood studies have been carried out should be identified. The similarity should be as good as possible, precipitation wise, evaporation wise, slope wise, vegetation wise and size wise. The results should then be adapted and extrapolated for the reservoir under study. Slight exaggeration through an over dimensioned spillway will be helpful as a higher factor of safety for dams in those areas is actually desirable and their remoteness prevents quick repairs in cases of failures.

In order to help establishing flood figures for above-mentioned areas and to allow flood estimates for dam feasibility studies, a tentative list is given below, derived from many flood studies for individual dams throughout Kenya.

Catchment area in Km^2	Q (Flocod) *) In $m^3/s/Km^2$ of catchment
< 1	15
1 - 5	12 - 10
5 - 25	3 - 6
25 - 100	3 - 2
100 - 1000	1 - 0.4
> 1000	< 0.3

*) Figures quoted are for Q100 or 100 year return period.

The magnitude of the flood to be expected is given m^3/s per Km^2 of catchment area e.g. for a catchment area of 0.5 Km^2 a flood of 0.5 x 15 = 7.5 m^3/s can be expected.

The range given in the flood column depends on the mean annual precipitation and on the size of the catchment, a larger catchment gives less discharge per Km^2 than a smaller catchment.

The return period for the flood to be calculated depends on the size (cost) of the structure, the importance of the system it supplies (reliability factor), the importance of all kind of woks downstream and whether they get destructed when a dam ***** repaired in case of failure. Another point is the reliability of the date on which the flood calculations are based, quite often rainfall stations or discharge stations rather far from the actual site are utilized for computations making their full credibility slightly doubtful, and hence a higher safety factor will be required.

The presently commonly employed flood return period of 25 years (the chance of a certain magnitude of discharge accruing once in every 25 years) is found to be too small. A minimum return period of 100 years is advocated, while for the bigger (over 20m) or more important dams a return period of 625 years (25 x 25) or even 1000 years should be employed.

4 SURVEY

Except for urban areas and possible private exceptions no maps of a suitable scale for dam design are available within Kenya and topographical teams will have to be sending out to enable their preparation.

The existing aerial photos are generally of too large a scale to enable the preparation of the required maps (except for large parts of Machakos). Aerial surveys to this purpose for the small dams under study are usually too expensive to be considered.

If the final site has not yet been selected the pre-survey for the several possible dam sites should include the following for each site.

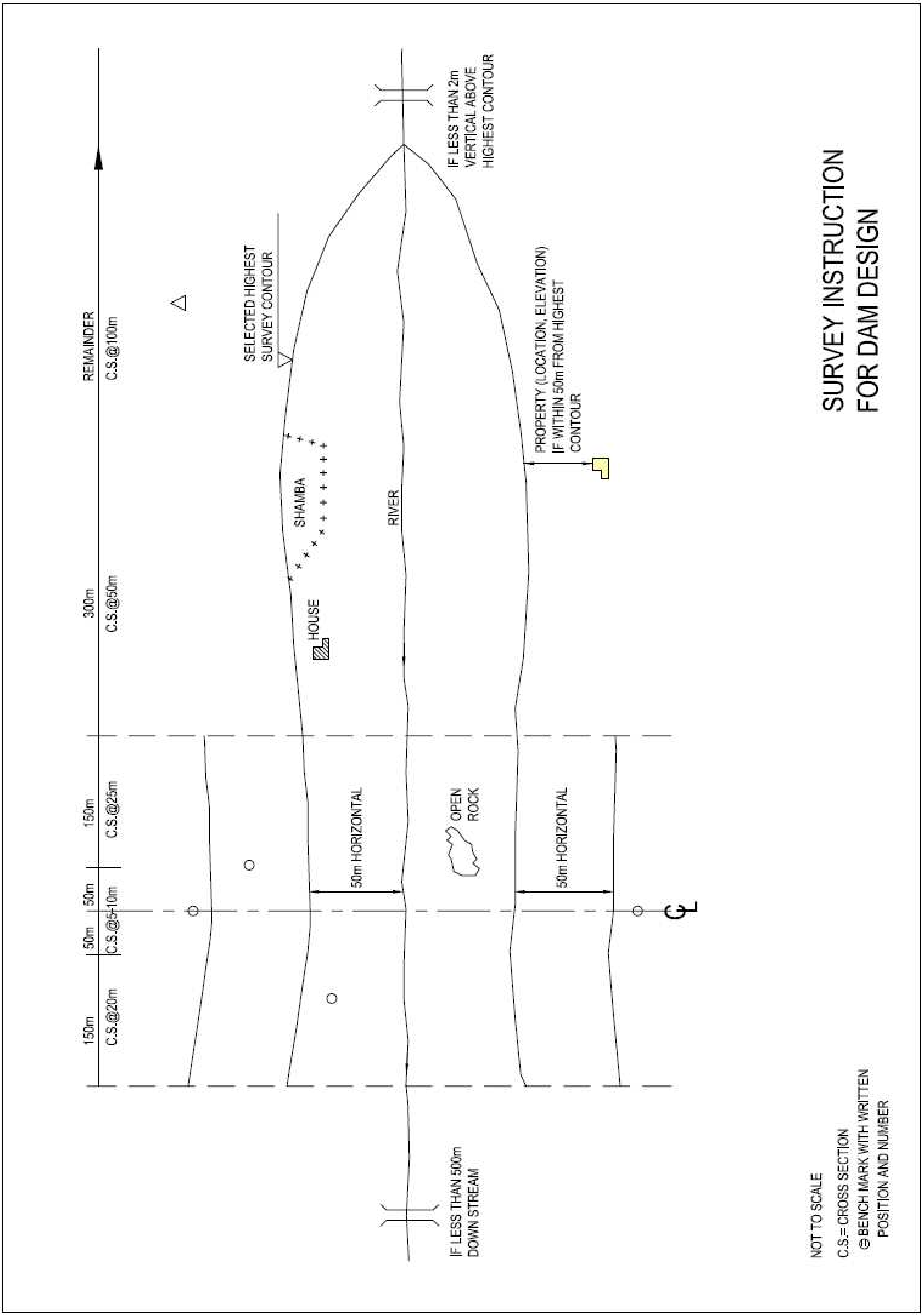
One cross section and valley at the axis of the dam with 1-metre intervals extending well above the expected dam height.

One longitudinal profile of the river with a few cross sections (2-5 depending on the length of the reservoir) up to the expected water level.

The thus acquired information should be sufficient to enable the selection of the most economical site from the topographical point of view.

* *failure occurs and finally on the chances of getting the dam …*

Figure 1: Survey instruction for Dam Selection



For the final site maps with more accuracy are require which should display the following contour intervals and scales:

DESIGN	RESERVOIR	DAM AND SPILLWAY
Scales	1:100 - 1:5000	1:250 - 1:500
Contour intervals	1m - 2m	0.5m - 1.0m

The survey for the final dam site should meet the criteria listed below:

- The planning and/or design engineer should indicate the dam axis and the required level above the river bed (in metres) to which extend the reservoir area should be surveyed to ensure that the required storage capacity will be within the surveyed range.

- The dam area survey should be extended by 50m horizontal above the indicated reservoir level and for 200 metres up and downstream of the dam axis in order to allow proper dam and spillway design.

- The first 50m up and downstream of the dam axis should have a grid or cross-section distance of 10 or 5 metres depending on regular or highly irregular topography.

- The next 150m up and downstream should have a cross section every 25 metres.

- From 200 metres to 500 metres upstream of the dam axis the reservoir area should be covered by cross section every 50 metres.

- Above 500 metres one cross section every 100 metres.

- The contour at the level indicated by the engineer must close upstream.

- Within the area covered, all men made structures and cultivations should be indicated, if possible with their ownership.

- As above but for a horizontal distance of 50 metres outside the reservoir area, elevation should be indicated.

- A river crossing (road, pipeline etc within 500 metres downstream of the dam axis should be indicated.

- As above but within a two metres vertical range upstream of the highest closing contour.

- Additional areas to the above mentioned may require surveying and should be added. This may be the case if very large spillways are expected or depressions in the valley embankment.

- At least four solid concrete Bench Marks (B.M.) should be placed of which two should indicate the dam axis. The Bench Marks should be underground for at least 0.30m to avoid removal or displacement of Bench Marks. Levels should be indicated by writing in the concrete, together with their numbering.

5 SITE INVESTIGATIONS

Site investigations will have to be carried for the following reasons:-

- Location and condition of the bedrock. In Kenya the bedrock is often very close to the surface and can easily give (spillway) construction or seepage problems.

- Location of permeable layers which might cause excessive leakage or even undermine the dam.

- Location of foundation material under the wall and possible other structures.

5.1 BEDROCK

Bedrock at shallow depths should be detected. The weathered parts under the dam's foundation can cause serious leakage and should therefore be removed.

Heavy fractured bedrock can cause serious losses through infiltration and another dam site should be considered, or clay blanket applied to the bottom of the reservoir. The spillway could be badly affected by the bedrock as simple excavation and ripping might not be possible, requiring expensive blasting with the risk of fissuring the rock. On the other hand the presence of the bedrock might enable the construction of the spillway on the bedrock thereby avoiding the erosion problems common to earth channels. In this connection it might be more economical to raise the dam use the bedrock as a natural spillway, in order to avoid expensive blasting.

5.2 PERMEABLE LAYERS

All permeable layers, like gravel and sand (old river-beds) or murram and laterite under the dam or underlying the reservoir should be detected as extensive losses and even dam failure can occur if their presence is not discovered.

As a result a rather extensive investigation of the dam site is required. International handbooks advise on investigation into the substrata as deep as the dam is expected to be high. However, in view of the limited dam height dealt with in this manual and in view of the generally very tight soils in Kenya, a depth of 4.00 metres below the surface being investigated is in most cases considered sufficient, certainly since this depth can be reached by hand, using local labour, while deeper investigations will require local labour, while deeper investigations will require mechanical power, which often might be difficult to get on the site.

Each site should still be considered individually on its own merits and it necessary the general test pit layout given in this manual adapted accordingly.

As for the reservoir area no test pits should be dug, but visual reconnaissance should be made, as to the presence of sand or other out crops. Only in case of doubtful areas a few test pits can be sunk to ensure the quality of the underlying layers. Those test pits should be resealed properly as otherwise extensive sand layers hit might cause serious losses.

5.3 BORROW AREA

As we are dealing with earth dams and earth channel spillways, the first investigation is to see whether the material excavated from the spillway channel is suitable for dam construction, as this will save considerably on construction costs. This is usually not the case, but if the material is considered suitable it will normally not be in sufficient quantities, therefore other borrows areas for embankment material will have to be located.

In the first instance borrow areas should be located within the reservoir area, as this will increase the storage capacity. If this proves to be impossible borrow outside the reservoir area should be located.

A few test pits should be sunk in order to test the suitability of the material and to establish the quantity available, which should at least be double the requirements for the embankment backfill.

5.4 FOUNDATION

The test pits sunk in order to detect sand lenses and layers under the dam to be, will serve also for testing suitability of the walls foundation. In Kenya there are generally no problems with the foundation as the materials in site are consolidated (natural compaction) in such a way that practically any earth wall can be build on it without any problems. Only in swampy areas should care be exercised. Heavy structures, coming with the construction of the dam, like all-concrete spillways or power stations etc. will require additional foundation tests.

5.5 SANDY REGIONS

There are large areas in Kenya of which considerable parts are very sandy making dam construction uneconomical because of heavy losses through leakage and infiltration on top of the already enormous evaporation. In those cases clay deposits should be found in order to give the complete reservoir a clay blanket (clay lining) or the ultimate expensive solution of an artificial lining – butyl lining – could be applied. Those solutions should only be considered when all other solutions (e.g. boreholes-long distance piping etc) are impossible or even more expensive.

5.6 TEST PITS (T.P.)

Test pits should be at least 4.00 metres deep or down to bedrock if struck before. The first two metres should be dug by hand the last two metres by hand auger.

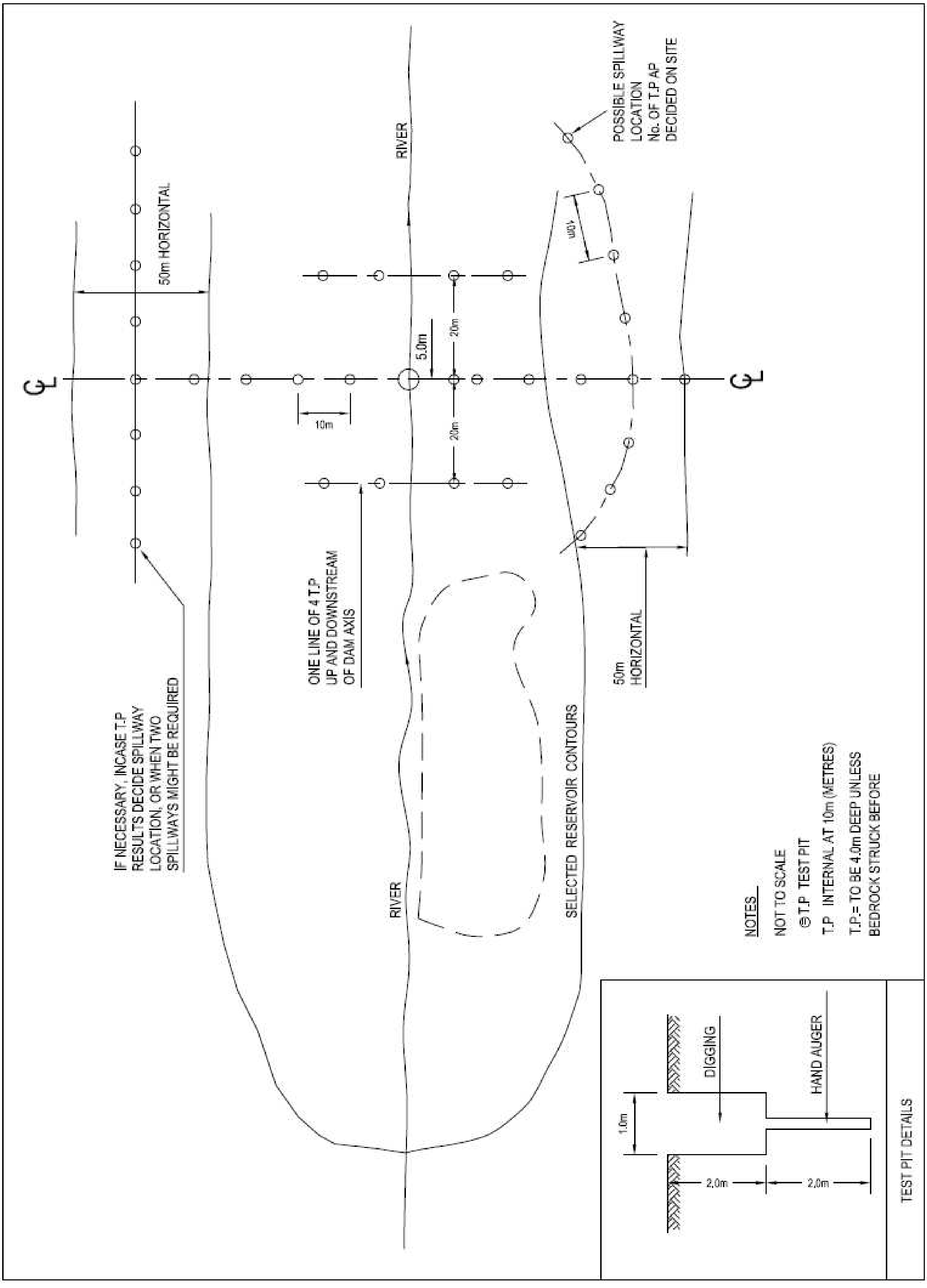
Along the dam axis test pits should be made every 10.00 metres and extended well above the expected dam height or 50.00 metres horizontal either side of the expected reservoir height, in order to cover the spillway as well. Eight more test pits should be sunk in the valley, four each at 20.00 metres either side of the dam axis to detect the extent of permeable layers.

The future centreline(s) of the spillway(s) should be established, as good as possible and T.P. should be dug every 10.00 metres along the centreline in order to establish the position of the bedrock and for possible use of the material for backfill. For T.P. location see sketch below.

While making the test pits, records of the materials found and over which depth should be kept. Descriptions of the materials should be simple and understandable for every layman, e.g. solid rock, weathered rock, and sandy clay etc.

When laboratory test will be made, samples of the different materials found should be taken. Those samples should be marked with the T.P. number and the depth of which it was taken, date when it was taken and should be put in plastic bags.

Figure 2: Test pits for Dam Selection and Design



6 SOIL TESTS

6.1 VISUAL TESTS

With the eye a first classification of the available soils can be made, being unsuitable, heavy, clays can be detected and classified as undesirable, as their swelling, shrinking and extensive cracking makes them highly dangerous construction or foundation material.

Soils with most particles visible with the naked eye should be classified as sandy and generally considered unsuitable for homogeneous dam construction soils with around 50% of the particles visible with the naked eye should be considered good for homogeneous dam construction.

Soils with hardly any particles visible with the naked eye should be treated carefully and preferably laboratory tested, as the clay fraction will probably be too large. Organic material rich soils should be avoided.

6.2 SIMPLE FIELD TESTS

Add roughly 10% (volume) water to a soil sample to see if cohesion is existant, and try the following tests.

Soils which cannot be rolled into a ball are either too sandy or have a highly irregular granular distribution and should be avoided. Alluvial deposits of a fine sand, silt mixture show the last feature.

Soils which can be rolled out to relative thin threads contain too much clay. Consequently suitable soils are those which can at least be rolled into a ball and no further than a thread with a diameter of a cigarette with a length of maximum 15cm, before it starts breaking into pieces.

In case the cohesion appears to be suitable it should be confirmed that organic matters do not cause this. Freshly sampled organic soils have a distinctive odour.

A ball made of the selected moisten material and placed in water should disintegrate slowly. Too fast (clearly visible) or no disintegration at all makes the material unsuitable for construction purposes. For this test the particles larger than 0.42 mm are usually removed from the sample.

The dilatancy or shaking test can be applied to the moisten sample. The pat should be placed on the open hand palm shaken horizontally. If water comes to the surface making it glossy, the sample should be squeezed between the fingers and water and gloss should disappear from the surface.

To rapid a reaction indicates lack of plasticity, which indicate fine sand or silt (not enough small particles for construction) no reaction at all indicates a plastic clay or peat (organic), making construction application doubtful.

The dry strength or breaking test. Remove all the larger particles form the moisten sample and let it sun dry. Test the sample by breaking it and crumbling it between the fingers. A slight strength indicates silt or silty sand or which the latter should be felt between the fingers, making the souls unsuitable for constructions. A high strength indicates large quantities of clay, which could be unsuitable as difficult compaction, and extensive cracking of the wall may occur.

Clay can also be distinguished by rubbing a near dry sample with the nail or a piece of metal; a shiny surface indicates high clay contents.

In case a glass jar or something similar is available the sample should be thoroughly mixed in water. The deposits, which occur, will be in layers of granular distribution and give a fair indication of the composition of the composition of the sample.

6.3 LABORATORY TESTS

Tests are done for two purposes:

- Suitability of foundation and construction material
- During the construction the checking of the placed material, whether it meets the required compaction specifications or not.

If there is a possibility to get testing done by experienced people in a laboratory undisturbed samples should be taken from test pits, right away after excavation and be brought to a (field) laboratory in sealed containers where at least the following test should be curried out:

i) Gradation test: to see if a fairly even distribution of granularity is available

ii) Water contents: of the soil on site in order to compute the additional water requirements for optimum compaction

iii) Atterberg limits: to evaluate volume change potential and shooting strength of soils.

iv) Proctor Compaction: test indicates the greatest dry unit weight obtainable under optimum water contents. This should be controlled during construction. Usually 95% of the optimum compaction is specified as minimum construction requirement.

Above described tests are to ensure good placing and compaction of the soils to be used for construction, in case laboratory equipment is available. Otherwise compaction will have to be done by feeling and between five and ten percent – volume wise – water should be added to ensure decent compaction.
The percentage mentioned is commonly required with the soils in Kenya.
Water should be added at the borrow area, to ensure better distribution in the soil.

Other test on the materials can be carried out to allow stability calculations for the embankment (selection of embankment slopes).

For those test and calculations well equipped laboratories and good knowledge of soil mechanics are required, but are not essential for the design of small dams as only relative minor savings on the backfill of the embankments are to be found in the earlier mentioned handbooks. Guidelines, based on experience, for the selection of embankment slopes for different types of soils are given in section 7.2, Dam Design.

Dispersive soils: Cases are known of dispersive soils in Kenya. This is also known as deflocculating of clays and the result is that those clays do not have resistance to erosion. Very small leaks in the dam wall can lead very rapidly to failure of the dam. The dispersivity should be checked by laboratory tests.

7 DESIGN

7.1 RESERVOIR CAPACITY

The reservoir capacity is the required volume of water stored behind the dam in order to cater for the expected water demand – including for losses – on a year round base.

For the selection of reservoir capacity a surface and a capacity (volume) curve should be prepared from the contour map previously drawn.

The reservoir capacity curve shows the accumulated volume for different (water) heights, or the volume stored below different (water) levels. The capacity curve is calculated from the surface curve. Hereto small enough height intervals (0.50 – 1.00m) are multiplied by the average surface over this height interval which give the volume for the specific height interval which is them accumulative represented in capacity curve.

The net requirements calculated in chapter 2 of part A ‘Water Supply’ will be used on a monthly base for the capacity calculation (weekly or daily figures will give more accurate information, but are not recommended as data to be used for the calculations are not very accurate for most areas).

See also “graphical determination of reservoir capacity” in Part E, Appendix of this manual, Appendix I.1.

The following parameters are considered:

i) Dead storage

ii) Monthly water demand of monthly consumption

iii) Flow of the steam in the dry period.

iv) Evaporation losses.

v) Other losses are supposed to be included under (iv) as evaporation losses are over-estimated since the4 total monthly evaporation losses are over-estimated since the total monthly evaporation loss is considered to take place at the beginning of each month when the reservoir surface is the largest.

vi) Sedimentation. This is only briefly considered in the example given below, however, as it is of great significance in many areas with soil erosion in Kenya some representative figures of the sediment loads are given in the following table:

Erosion	Sediment Load
	$m^3/km^2/year$
Low	500
Moderate	1000
Heavy	1500

All items mentioned are measured or estimated in volume (m^3) except for evaporation which is measured in water depth (height) (m or mm) and consequently the volume going into evaporation is a function of the reservoir surface at the time of consideration.

The graphical method comprises the following steps, going back in time, considering the reservoir empty just before the rains – river discharge – start:

1) Dead storage remains at the very end of the dry season, or the last dry month. The volume reserved for the dead storage is set out horizontally from point zero. Vertical transport of the volume gives the corresponding reservoir depth or water height, indicated by point a.

2) The monthly consumption (M.C.) minus the in-flow during the month in question is set out horizontally from point a. and then transported vertically back to the curve giving point b. which indicates storage requirements for one month or the last month of the dry period, excluding evaporation losses. The vertical transport indicates the loss of water depth for the consumption volume.

3) The monthly evaporation (EV) for the month under consideration is set out vertically from point b. and then transported horizontally back to the curve giving point c. which now indicates the gross storage requirements for one month or the last month of the dry period. The horizontal transport indicates the volume of water lost through evaporation.

4) As for 2 starting from point c. set out M.C. horizontal and transport vertical to point d.

5) As for 3 starting from point d, set out EV vertical and transport horizontal to point e. which now indicates the gross storage required for the last and the forecast month or a two month dry period.

6) As for 4 and 5 giving storage requirements for a three-month dry period.

7) And so on uphill the number o month required to cater for the dry period as calculated under section 1.5 has been covered.

An example for the graphical method is presented in ‘Appendix I.1 of Part E of this manual. In the example the following fictitious values have been used:

Dead Storage	:	7,500 m^3 (EL.3.65)
Monthly consumption	:	10,000 m^3
Monthly flow of the stream in the dry period	:	zero
Monthly evaporation	:	0.20 m

Those values are assumed constant over the dry period. In case the monthly consumption or evaporation is expected to vary over the dry period, the varying values should be incorporated for the different months. This will mainly be important in case large variations are expected due to for example the irrigation season starting and/or ending. Changes in evaporation values are hardly realistic, as selected values are long-term averages and not the actual value for the month in the year under consideration, nor does one know exactly in which month the river discharge will cease and drawing water from storage will start.

Below are two examples for the use of the capacity curve shown using the example in Appendix I.1 of Part E of this manual.

1. 7 months dry period :

Reservoir capacity or gross storage	:	122, 500 m^3
Dead storage	:	7,500 m^3
		115,000 m^3
Net storage require: 7x10,000 m^3	:	70,000 m^3
Total evaporation losses	:	45.000 m^3

Evaporation losses are 65% of the net storage requirements. Total losses are 75% of the net storage requirements. OR: Evaporation losses 36% of reservoir capacity, total losses 42% of reservoir capacity, the latter meaning that nearly the double of the expected water demand has to be stored to safeguard the continuous water supply.

The capacity curve also shows that the hydraulic height of the dam or the water depth in the reservoir will be 9.95m. This is in turn the design level of the spillway crest above riverbed level.

2. 4 months dry period :

Hydraulic height	:	8.65 m
Reservoir capacity or gross storage	:	60,500 m^3
Dead storage	:	7,500 m^3
		53,000 m^3
Net storage require: 4x10,000 m^3	:	40,000 m^3
Total evaporation losses	:	13,000 m^3

Evaporation losses 32.5% of net requirements or 21.5% of reservoir capacity. Total losses respectively 50% and 34%.

Note that evaporation losses are not only larger in volume but also in percentage for longer dry periods as a result of the flattening out of the valley (hence the capacity curve) for higher water levels. This is exposed by the increasing length of the (dotted) line for the horizontal transport of the evaporation back to the curve for larger water depths.

In Appendix 2H mathematically calculated values are given for evaporation losses to be expected as a percentage of the net-storage for various reservoir shapes and consumption patterns. Those values will help during the feasibility studies as indicators for dam siting and project evaluation.

If the catchment area is 3km2 and with a low rate of siltation of 500/m3/km2/year, for a design life of 20 years a volume of 3 x 500 x 20 = 30,000m^3 should be added.

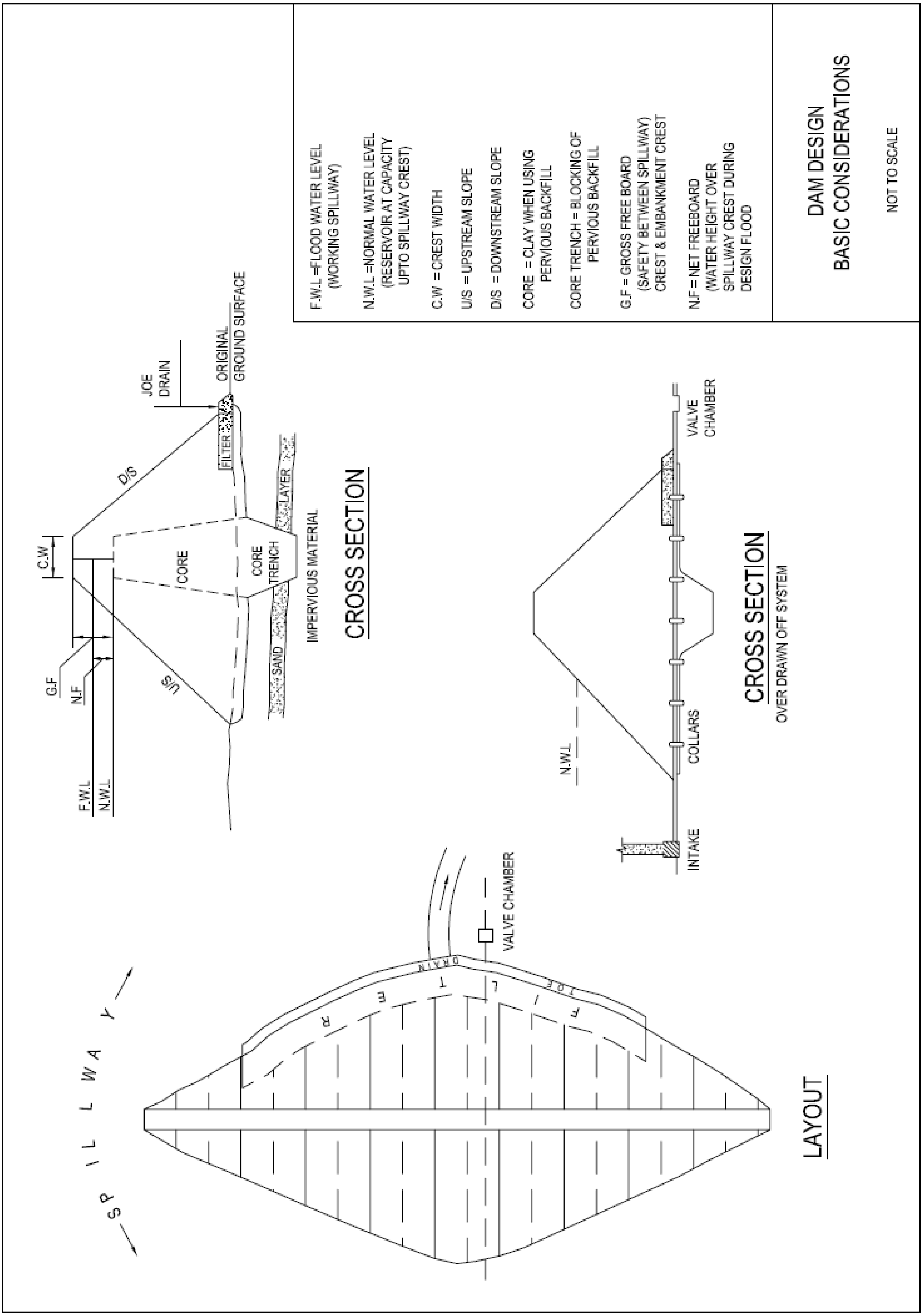
7.2 DAM DESIGN

In those cases that proper soil sample taking is possible and qualified personnel for supervised laboratory tests are available and then only when safety of the structure or the economy or backfill (volume of the dam wall) warrants those lengthy and relative expensive procedures a dam design engineer should be requested to do the design as a good knowledge of soil mechanics is essential for the necessary stability calculations.

In all other cases for designs of dams meeting the restrictions set out in chapter 2 a more simplified design method will have to be followed, as outlined below:

The basic design of the embankment will include but not necessarily be limited to all components identified in figure 3.

Figure 3: Dam Design Basic Considerations



7.2.1 Dam Axis

The dam axis should normally be designed straight unless special topography features dictate a bend or a curved axis. The dam axis should be located in such a way that the minimum amount of backfill will be required for the embankment. A narrowing down of the contour intervals on the topographical map usually indicates this.

7.2.2 Foundation

It should be expected that the whole foundation area of the dam will have to be cleared of all vegetation and the topsoil containing organic materials, as their rotting will create seepage paths and localized settlements. Equally should all the sand (or mud) be cleared from the riverbed. No foundation slopes of over 20% (max 30%) should be allowed.

Steep slopes will create either dangerous sliding planes, through embankment settlement against the naturally compacted original soil (in situ) making seepage lines (planes). In case of riverbed gullies unequal settlement between two embankment sections might occur, also creating seepage planes.

7.2.3 Height of Wall

Establish approximate height of the dam wall, this is the water depth calculated in chapter 11.1 plus the required freeboard (safety) see chapters 11.3 & 4.

It is noted that dams with a height under 5.00 metres should be carefully considered before further design as the freeboard is usually around 1.00 – 1.50 metres, evaporation in arid areas 2.00 – 2.50 metres per year leaving 1.50 metres or less out of the 5.00 metres for actual net storage or consumption, which means relative large quantities of backfill for little effective storage.

Only special cases will economically justify the construction of very low dams.

7.2.4 Fill Material

Decide whether it will be an all homogeneous embankment or whether a core will be required. The latter in case the soil to be used for backfill is rather sandy. Decision to be based on the investigation of the Borrow Area.

7.2.5 Slopes of Wall

Slopes of the dam are dependent on the height as well as the materials used. The following slopes are recommended:

DAM HEIGHT (m)	DAM TYPE	SLOPES	
		Upstream	Downstream
Below 5	Good granular distribution	1:2.5	1:2
5 - 10	Good granular distribution; very clayey	1:2.5 1:2.5	1:2 1:2.5
10-15	Good granular distribution, clay	1:2.5 1:3	1:2.5 1:2.5

Application of a clay core does not change the slopes, however, in cases that the foundation proves to be bad (e.g. clay) flatter slopes should be selected.

7.2.6 Crest Width

The crest width of the dam should be 5.00 metres minimum in case heavy machinery will be used for the construction, as this is a reasonable minimum width for the machines to be manoeuvred comfortably.

A 1.50 metres to 2.00 metres crest width can be employed in cases of labour intensive construction methods.

Dam crest. If the dam crest is not used for traffic, it should be covered with 200 mm gravel to avoid drying out and shrinking.

7.2.7 Core Trench

A core trench is used to cut off bad underlying layers, See chapter 9. In case machinery will be used for the construction the minimum width of the core trench should be about 1.5 times the excavation width of the machine. The depth of the core trench that should have been established by the test pits will have to be confirmed by the Resident Engineer during the excavation, the trench should not exceed a depth of 1/3 – 1/2 of the dam's height.

7.2.8 Filter

A toe filter and drain are used to prevent piping or seepage appearing on the downstream slope.

The toe filter is made from sand. In case graded sand is available it should be employed if not any sand without organic matters to be found in the vicinity can be used.

The thickness of the toe filter can be 1.00 metre, of width 5.00 – 1.00 metres depending on the height of the dam. The extent of the filter should be up to an elevation 4.00 to 5.00 metres under the crest level of the embankment.

The toe drain made from rocks or riprap of variable size (25mm – 250mm diameter) to be placed against the sand filter to prevent the water draining from the filter of carrying too many soil particles (piping)

The toe drain is kept outside the embankment for easier construction purposes, simple lorry dumping is possible and no angled filters are required. It also forms a barrier against cattle wandering up the embankment.

7.2.9 Upstream Slope Protection

In case of a long reservoir (large fetch) when strong wave development can be expected, the upstream slope of the embankment should be protected by rip-rap (rough stone pitching) over the area, which will be affected by the wave action.

7.2.10 Draw Off

A draw off system or compensation outlet should be included, as pumping straight from the reservoir will prove to be awkward with the fluctuating water levels.

Either mentioned system will be crossing in or under the embankment for either gravity supply or connection to a pumping system.

As this crossing pipe is a possible seepage hazard care should be taken with the backfill (well compacted) around the pipe concrete collars should be constructed at set intervals around the pipe to lengthen the seepage path and render it less straight. Those collars are usually made around the pipe joints in order to strengthen those.

7.3 INTAKE

7.3.1 Intake Structure

A simple intake structure, made from a perforated pipe (G.S.), weld mesh or a similar wire netting to prevent leaves or other materials from blocking the perforations in the pipe to be fixed in a concrete block to ensure stability.

7.3.2 Valve Chamber

A gate or sluice valve just downstream the embankment, preferably with a washout possibility to be fixed in a valve chamber (lockable if possible).

7.4 SPILLWAY DESIGN

7.4.1 Spillway Type Selection

For smaller earth dams he common type selected in the earth channel spillway excavated next to the embankment. Only in rare cases where the valley sides are too steep and channel excavation would become too costly (always) in comparison, an all concrete overflow spillway (weir) forming on integrated part of the embankment could be opted for, however as cubic metres of concrete are about 50 times as expensive as cubic metres of backfill (or excavation in case of an earth channel spillway and a concrete section in the middle of an earth embankment gives seepage problems along the concrete surface, careful consideration should be given to this solution.

Another alternative when the valley slides are too steep and then only when the catchment area is very small (under 25 ha), hence the return flood to be expected which might have to be evacuated through the spillway is a concrete channel "floating" on the embankment. This solution should be a last resort solution should be a last resort solution as seepage will follow the under side of the channel, this can be prevented by collars at regular intervals. Secondly settlement of the embankment might cause the channel; collars can prevent this at regular intervals. Secondly settlement of the embankment might cause the channel to crack seriously as a result of which water might escape form the channel and erode/undermine the embankment.

N.B. For very small discharges culverts through the top of the embankment, could be considered.

7.4.2 Spillway Location

The concrete spillway have been basically discussed in the previous chapter. One has to assure however that they discharge on bedrock, otherwise costly stilling basins will have to be provided to prevent erosion and embankment undermining.

The earth channel spillway have to be located in such a way that minimum quantities of excavation are required and at the same time the spillway discharge channel should have slopes flat enough in order to deep water velocities down to an acceptable level preventing erosion of the channel. Maximum acceptable velocities on Kenya's hard soils, also in view of the short discharge periods are thought to be 2.50m/s. Consequently the spillway usually is located on that side of the dam where the valley side is flattest. With large (wide) spillways it might be cheaper-less excavation – to have two spillways at either side of the dam.

Because of erosion considerations the spillway should always be excavated in the original soil (soil in situ) avoiding the relative loose backfill of the embankment or any other backfill for the matter.

Because of the cost of rock blasting extensive rock excavation should be avoided, this might well invalidate one valley side or even the whole design in the latter case an all concrete spillway be considered for smaller reservoirs or other special solutions falling beyond the scope of the manual. On the other hand a rather horizontal rock bed level provides a natural lined spillway channel free of erosion problems. Such a rock strata could very well lead to a change in the reservoir capacity, certainly in cases where the rock level is higher above the river bed than the required water height (reservoir capacity) behind the dam. In those cases it is probably cheaper t increase the height of the wall (reservoir capacity) moderately than to start expensive rock excavation.

The possibility that the spillway evacuates the floodwater into a valley other than that of it is original can be considered in case a depression in the ridge separating the two valleys could be employed as a natural spillway or when this requires only moderate excavation. It should be realized that the evacuation of water from the valley could adversely affect the consumer's down-stream of the construction.

7.4.3 Spillway Design

For the design and calculations only earth channel spillways will be considered in this manual, for all other types of spillway reference to international bankbooks will have to be made.

The spillway normally consists of thee parts:
i) Crest
ii) Inflow section
iii) Outflow section

ad i. The crest of the spillway controls the normal water level in the reservoir, hence the reservoir capacity. The crest is the higher elevated part of the spillway and unless the crest is naturally fixed by bedrock it should be fixed by a solid concrete sill at the designed storage – capacity level, as erosion of the earth channel will lower this level (probably only section) and will consequently reduce the reservoir's capacity. This still which normally designed in line with the dam axis. The angle of the sill and the exact location however remain dependant on local topography.

ad ii. The inflow section leads the flood water to the crest. This part usually slopes moderately – one to two percent – upwards to the sill and it's cross section narrows down towards the sill, both features to prevent a too abrupt transition of water velocities, as the latter is an erosion hazard. The length depends entirely on the topography, although it should be made sure that the water flowing to the intake remains far enough away from the dam wall.

ad iii. The outflow section evacuates the floodwater to a safe distance from the embankment preventing the washing out or the undermining of the wall. This section discharges the excess or floodwater back into the riverbed (or a neighbouring river), either straight away or through a minor tributary. The outflow section is usually the largest erosion risk because of increasing water velocities; hence the design should be such that those velocities are kept down to acceptable levels – below 1.50m/s. This is mainly done through slope selection, while the final spillway outlet should either be back to riverbed level or on solid rock. Whenever this would lead to too long channels a drop-structure (concrete, gabions etc) should be in-corporated. A cost comparison will indicate whether this drop structure is more economical and also where it should be located.

Further erosion preventive measures are the inclusion of more sills in the outlet section, which sills will fix the channel elevations at certain points. The lining of the channel bottom with riprap is an additional possibility.

This riprap should be made of solid rock, at least 0.30m thick and provided with an underlying coarse sand filter.

In regions where relative long base discharges are to be expected the design of the spillway channel should be such that a 2.50 – 10.00 metre wide section away from the dam wall can contain this base flow in order to keep possible erosion as far away from the dam wall as possible.

This section is created by raising the remainder of the bottom by 0.05 – 0.25 metres, leaving the lower section as final control structure for the reservoir levels.

Instead of the bottom elevation difference mentioned above, a culvert could be included, collecting the water just after the crest sill and brining the base flow as quick as possible through the ground to the riverbed, this solution will be applicable for minor base flows only.

7.4.4 Spillway Dimensions

The calculations covering this chapter are explained in Appendix I.3. The return flood calculated under section 3 is the base of the calculations. Before deciding on the final dimensions of the spillway which will have to be done in relation to the embankment height as explained in section 7.5, the decision whether one or two spillways will be employed will have to be made.

For several alternatives for the spillway width an excavation estimate for the following possibilities must be made: -

1. Full spillway capacity (width) on one side.
2. Half spillway capacity (width) on either side.

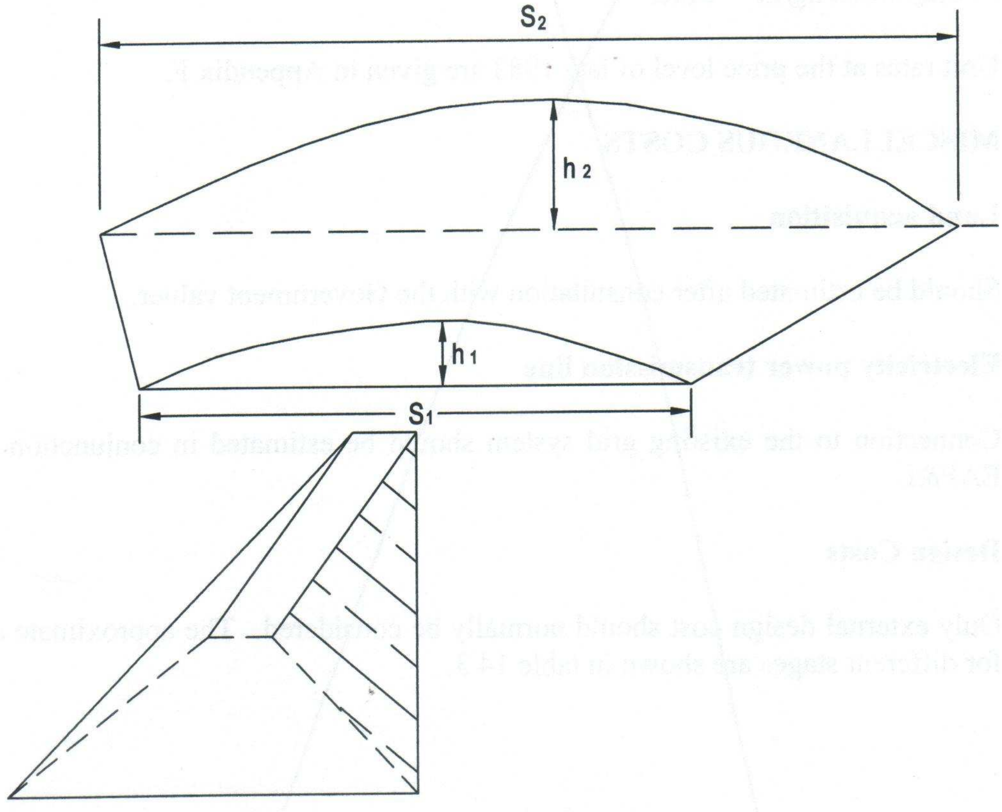
For the excavation estimates the following formula can be employed as explained in the figure in Appendix I.3 of Part E of this manual.

$$V_E = \frac{2}{9} \text{ x b x } \left(h_1 \text{ x } s_1 \text{ x } h_2 \text{ x } S_2 + \sqrt{h_1 x S_1 x h_2 x S_2} \right)$$

This formula does not allow for the slopes in the spillway, a horizontal spillway bottom is assumed. It should also be realized that this formula is too inaccurate for the calculations for the Bill of Quantities, for this purpose cross sections at regular intervals over the final selected spillway design should be employed.

In case property or real estate dictates the side of the spillway the estimates for the several alternatives should be made for one side only.

Figure 4: Spillway Excavation Estimate



PARABOLA SURFACE $= \frac{2}{3}.h.s$

EXCAVATION VOLUME:

$$V_E = \frac{2}{9} . b . \left(h_1 . s_1 + h_2 . s_2 + \sqrt{h_1.s_1.h_2.s_2} \right)$$

IN CASE $S_1 \cong S_2$

$$V_E = \frac{1}{9} . b . (s_1 + s_2) \left(h_1 + h_2 + \sqrt{h_1.h_2} \right)$$

FORMULAS ARE VALID FOR SPILLWAYS

WITH 0% SLOPE

FORLULAS TO BE USED FOR ROUGH ESTIMATES

OR CALCULATION CHECKING PURPOSES ONLY

7.5 DAM AND SPILLWAY DIMENSIONS

The hydraulic height of the dam has been established under section 7.2

The total height of the dam is equal to the hydraulic height plus the gross freeboard, the latter being a function of the width of the spillway. The wider the function of the width of the spillway (more excavation) becomes, the smaller the freeboard require, therefore the lower the dam (less backfill) and vise versa.

The optimal solution is the construction though which the cost of spillway excavation plus the cost of embankment backfill is lowest as the cost of intake and other structures remain the same. Only in highly cultivated areas the cost of expropriation will have to be included.

For cost optimilization the final dam heights should be established for the different spillway widths calculated under the previous sections, 7.2 to 7.4, but alternatives are usually sufficient.

Quantity estimates for spillway excavation and embankment backfill should be prepared for the selected alternatives. For the spillway excavation as explained before for the embankment backfill a minimum of 10 cross sections (at least every 5.00 – 10.00 metres) of each alternative for volume calculation should be used. In case local rates for excavation and compacted backfill are known, they should be employed for the comparative cost calculations.

However the cost are not constant in time (inflation) nor is the cost of backfill equal to the cost of excavation. On the average the cost of excavation is about ¾ of the cost of compacted backfill, which allows a volume comparison instead of a cost comparison (in Kenya), as a result the sum of the total volume of embankment backfill plus three quarters of the volume of spillway excavation for each alternative can be compared.

$$V_{TOT} = V_{EMBANKMENT} + \tfrac{3}{4}\, V_{SPILLWAY\ EXCAVATION}$$

The alternative with the lowest total volume is the cheapest and should be adopted, better however is to put the total volume in a graph against the spillway width, which will show the most economical spillway width.

In case spillway excavation is partly common and partly rock excavation, this separation is included into the calculations. For the final selected dimensions proper volume calculations should be made in case the construction is put up for tender. For the spillway this is to be performed through cross sections at 5.00 – 10.00 metes interval, for the embankment through the plan metering of the horizontal sections at 1.00-metre interval.

7.6 OUTLET WORKS

It is in most cases not recommendable to let either people or livestock near or in the reservoir, because of the pollution risk. Consequently a draw off system will have to be included.

This draw off system is normally an intake at the bottom of the reservoir with a pipeline crossing under the dam at low level. This pipeline is then connected either to the gravity supply or to the pump house in cases when pumping is required as pumping straight from the reservoir will be difficult because of the rather great fluctuations of the reservoir's water level common with small dams.

This pipeline can also be used for compensation flows to safeguard downstream water rights.

The intake should be placed in such a way that not too much sediments or floating materials will get near nor that siltation will cover the intake.

The intake structure can be made of a concrete block supporting a standing perforated pipe, preferably not in plastics as they degrade in the sun during low water levels.

The perforations should be protected form rotten leaves or other debris by a wiring at some distance from the pipe.

The draw off pipe should have a minimum diameter of 100mm in order to decrease the risk of the pipe getting blocked by debris or silt entering the pipe, as this pipe can never be reached any more. As this pipe forms a seepage path collars should surround the pipe at regular intervals to decrease the seepage possibility, as a high degree of seepage might lead to piping.

7.7 SCOUR

Scour is the flushing of sediments from the reservoir by the run off through a fairly sized bottom outlet.

The minimum diameter of a scour outlet should be 500mm. The control structures for such a system through an earth dam are difficult to build, as a downstream control valve (or penstock) will give the risk of a sediment filled pipe, thus useless. The risk, however, can be reduced if the valve is opened every week.

An upstream control can not be reached because of the sediments. Scours are usually adopted for minor concrete dams only.

A possibility of cleaning the reservoir is scooping, this however gives $1m^3$ of water stored for every m^3 of excavation contains voids (porosity) thus water while even a poor dam site gives at least 2 to $3m^3$ of water stored for every m^3 of backfill.

Consequently it will be cheaper to build a new dam downstream (possibly using the sediments for the old dam) and use the old dam as silt trap. This of course only when topography, property, etc. allow to do so. It should also be considered whether a possible change from gravity supply to pumped supply (or increased pumping head) will in the long run not be more expensive than scooping.

8 CONSTRUCTION ASPECTS

The main items to be watched during the construction are the foundation and the compaction of the backfill.

During excavation of the core trench it should be observed whether the test hole results gave the right information, during this stage it should be decided whether excavation should go deeper in case more or deeper pervious layers are found than indicated by the test pits.

Control of the compaction should be exercised as bad compaction can lead to seepage and piping and eventually dam failure. Compaction by sheep foot roller or similar rollers should be done in layers not exceeding 0.15m an easy method of compaction control when standard equipment, is impossible to obtain is to have backfill excavated by hand. In case this excavation is possible only by hoe, a decent degree of compaction has been obtained in all cases where fairly easy excavation by shovel is possible compaction should be considered insufficient.

For the actual construction heavy machinery or labour intensive methods can be used.

Labour intensive methods require very good supervision of the compaction as failures because of insufficient compaction have been observed. It should also be noticed that manual labour can do about 2m2 of fill a day a person which means 10,000 man days for 20,000 m3. This means an army of 100 men for at least 100 days.

Heavy machinery has problems in remote areas with the supply lines (fuel), which should be considered.

Unworkable whether will have to be taken into account. Heavy machinery will be unable to move on muddy grounds. Rainy seasons could carry the risk of an embankment not yet raised to final/level being washed away by the floods. In case the rains are expected before the termination of the construction, the construction could be started at either end of the dam wall leaving a gap in the middle to be filled at a later stage after the rains, it is however preferable to continue the raising of the embankment in one go during the dry season.

Actual construction should be carried out in layers, which can be compacted. So the wall's cross-section will show a pyramid shape.

The pushing of soil by bulldozer will not do as step like formations will show up during construction anywhere over the unfinished embankment surface, which will create long holes which will be difficult to compact when the next load is pushed over. Those incompact long holes are a dangerous seepage and piping risk.

The best way for construction and setting out the construction is by 0.5m or 1m horizontal layer through many wooded pegs which indicate the width and height of the layer to be backfilled. The layer is then filled by spreading the material in 5cm layers, with a compactor (herds of sheep or goats could be employed)
moving around continuously and water spraying for obtaining the required moisture contents.

9 MAINTENANCE ASPECTS.

Maintenance to dams is little and not very costly. It should however be attended to as minor erosion signs on the embankment and in the spillway still can lead to serious erosion and possible dam failure. A responsible person should inspect the site once in a while and at least at the end of every rainy season.

The items to be watched are:

i) Erosion by rain or runoff in the spillway. Embankment erosion should be refilled with compacted material or puddle clay and grassed. Spillway erosion should be refilled with riprap.

ii) Possible population of rats and or coypus should be removed as they can seriously undermine the dam.

iii) No livestock should be crossing the dam as they wear out the embankment. The possible fencing should be repaired.

iv) The outlet works should be checked and the main draw off be flushed in order to remove possible sediments. Gates should be operated once every week.

v) The spillway channel should be kept clear of high vegetation and should be cleared before the wet season. A short grass cover of the channel should be encouraged as this provides a good erosion protection.

PRACTICE MANUAL

FOR

WATER SUPPLY SERVICES

IN

KENYA

PART D

ENVIRONMENTAL IMPACT ASSESSMENT (EIA)

LIST OF CONTENTS

1.0 INTRODUCTION

1.1 BACKGROUND

1.1.1 Basic Concepts

- EIA is the study of the effects of a proposed action on the environment. The "environment" is taken to include all aspects of the natural and human environment.

- EIA identifies both negative and positive impacts on development activities and how they affect the people, their property and the environment. EIA in addition identifies measures to mitigate the negative Impacts, while maximizing on the positive ones. EIA is essentially a preventive process, which seeks to minimize adverse impact on the environment.

- The EIA Studies covers all physical, biological, social, economic and other impacts arising from an action on the environment. The studies vary from action and include studies on weather, flora and fauna, soil erosion, human health, urban irrigation and employment.

- EIA seeks solutions to all feasible alternatives that represents an optimum mix or balance of environmental and social-economic costs and benefits.

- The technical work involved in EIA is the estimation of the changes in environmental quality which may be expected as a result of the proposed action i.e. EIA attempts to weigh environmental effects on a common basis with social-economic costs and benefits in the overall project evaluation.

- The ultimate objective of EIA is to aid judgmental decision making by giving the decision maker a clear picture of the alternatives which are considered, the environmental changes which are predicted, the trade-offs of advantages and disadvantages for each alternative.

1.1.2 Objectives

The objective of Environmental Impact Assessment (EIA) Study is to ensure that the project conforms to regulations of the "National Environmental Management Authority (NEMA)" of Kenya and in general of the prevailing Kenya's environmental rules and regulations.

1.1.3 Legal Framework

- Environmental Management Coordination Act (EMCA) 1999 give authoritative legislature framework within which the environmental protection and management shall take place.

- The Environmental Impact Assessment (EIA) is guided by the Environmental and coordination Act, 1999 through National Environmental Management

Authority (NEMA). The Act became effective on 14th January 2000 after receiving Presidential Assert on 6th January 2000.

- The provision of water supply services falls under Water Supply and Sewerage disposal works EMCA schedule 2 activities and is therefore, expected to comply with the EIA requirement under the same Act and its regulations.

- The preparation of EIA Study and subsequent approval procedures are set out in the Environmental Impact Assessment Study, legal notice 101 of 2003.

1.1.4 EIA Regulations

In accordance with NEMA regulations, a proponent shall submit to the Authority an environmental impact assessment study report incorporating but not limited to the following information.

(i) The proposed location of the project
(ii) The objective of the project
(iii) The technology, procedures and process to be used, in the implementation of the project
(iv) The materials to be used in the construction and implementation of the project.
(v) The products and by-products and waste generated by the project
(vi) The environmental effects of the project including the socio-cultural impacts, effects and direct, indirect, cumulative, irreversible, short-term and long-term effects anticipated.
(vii) A concise description of national environmental legislative and regulatory framework, baseline information and any other information related to the project.
(viii) A description of the potentially affected environment.
(ix) Alternative technologies and processes available and reasons for preferring the chosen technology and processes.
(x) An analysis alternative including projects site, design and technologies and reasons for preferring the proposed site, design and technologies.
(xi) An environmental management plan proposing the measures for eliminating, minimizing or mitigating adverse impacts on the environment, including the cost, time frame and responsibility to implement the measures.
(xii) The provision of an action plan for the prevention and management of foreseeable accidents and hazardous activities in the cause of carrying out activities or major industrial and other development projects.,
(xiii) The measures to prevent health hazards and to ensure security in the working environment for employers and for the management of emergencies.
(xiv) An Identification of gaps in the knowledge and uncertainties which were encountered in compiling the information.
(xv) An economic and social analysis of the project
(xvi) An indication of whether the environment of any other state is likely to be affected and the available alternative and mitigating measures.

(xvii) Any other matters as the authority may require.

1.1.5 Reasons for EIA Study

Reasons for carrying out an EIA will include:

- Assurance of adequate procedures for managing environmental risks and compliance with procedures.
- Improved statutory compliance
- Identification of environmental risks and problem areas, early warning and prevention of potential adverse environmental effects.
- Improved planning through the identification of future and potential capital, operating and maintenance costs, associated with environmental activities.
- Improved preparation for emergency and crisis situation management.
- Improved corporate image and positive public relations
- Enhancement of environmental awareness and responsibility throughout the corporate hierarchy.
- Improved relations with regulatory authorities
- Facilitation of obtaining insurance coverage for environmental impairment liability.

1.2 PROJECTS TO UNDERGO EIA

According to Clause 58(1), (4) of EMCA Act, projects listed in 2nd schedule require to undergo EIA study before being issued with an Environmental Licence. Sections 1.2.1 -1.2.9

1.2.1 General

(a) an activity out of character with its surrounding
(b) any structure of a scale not in keeping with its surrounding
(c) major changes in land use.

1.2.2 Urban Development Including: -

(a) designation of new townships
(b) establishment of industrial estates
(c) establishment or expansion of recreational areas
(d) establishment or expansion of recreational townships in mountain areas, national parks and game reserves
(e) shopping centers and complexes.

1.2.3 Transportation Including: -

(a) all major roads
(b) all roads in scenic, wooded or mountainous areas and wetlands
(c) railway lines
(d) airports and airfields
(e) oil and gas pipelines
(f) water transport

1.2.4 Dams, Rivers and Water Resources Including: -

(a) storage dams, barrages and piers

(b) river diversions and water transfer between catchments
(c) flood control schemes
(d) drilling for he purpose of utilizing ground water resources including geothermal energy.

1.2.5 Aerial Spraying

1.2.6 Mining, Including Quarrying and Open-cast Extraction of: -

(a) precious metals
(b) gemstones
(c) metalliferous ores
(d) coal
(e) phosphates
(f) limestone and dolomite
(g) stone and slate
(h) aggregates, sand and gravel
(i) clay
(j) exploration for the production of petroleum in any form
(k) extracting alluvial gold with use of mercury

1.2.7 Forestry Related Activities Including: -

(a) timber harvesting
(b) clearance of forest areas
(c) reforestation and afforestation

1.2.8 Agriculture Including: -

(a) large-scale agriculture
(b) use of pesticide
(c) introduction of new crops and animals
(d) use of fertilizers
(e) irrigation

1.2.9 Processing and Manufacturing Industries Including: -

(a) mineral processing, reduction of ores and minerals
(b) smelting and refining or ores and minerals
(c) foundries
(d) brick and earthware manufacture
(e) cement works and lime processing
(f) glass works
(g) fertilizer manufacture of processing
(h) explosive plants
(i) oil refineries an petro-chemical works
(j) tanning and dressing of hides and skins
(k) abattoirs and meat-processing plants
(l) chemical works and process plants
(m) brewing and malting
(n) bulk grain processing plants

2.0 EIA REQUIREMENTS

2.1 EIA EXPERTS

2.1.1 Criteria for Listing

Below is the criterion for listing EIA Experts.

	Category expert	Assignment	Requirements
1	Lead Experts	Entitled to Conduct EAI studies independently under their name as professional	1. **Minimum 2nd** degree in an field **and:** Trained in EIA and 2 years of experience in conducting EIA or 2. 5 years research work in any discipline **and** 2 years experience in conduction EIA
3	Associate Experts	Entitled to undertake EIA studies only under the guidance of a Lead Expert.	3. **Phd**. In environmental related studies **and** 2 years research experience or 4. **Phd** in any discipline, an environmental course **and** 2 years research experience. or
			5. **Engineers** with an environmental course **and** 2 years of working experience.
			6. **2nd degree** in any discipline **and** 2 years of working experience or **First degree** in Environmental Sciences, environmental Law, Natural Resources Management, Social Sciences **and** 3 years of field experience or **Diploma** in Environmental Management Studies **and** 4 years of field experience.

However the following should be noted: -

- The criteria for listing of EIA Experts can at a later stage be revised to fit the National capacity requirement.
- The rational of selecting the Lead and Associate Expert is mainly based on the experience in conducting EIA studies. One finishes a minimum of 2 years as Associate Expert. Preference is given on number of EIA studies undertaken rather than the years.

2.1.2 Procedures for Registering with NEMA

An applicant should prepare the following:-

- An application in the subscribed form to the Director General NEMA
- A detailed CV mentioning the academic performance, signed and dated highlighting relevant experience in the field of EIA.
- Three letters of recommendation (references) from different recognized experts of Institutions acting in the field of environmental management and/or EIA.
- A detailed list of studies, publications and consultancies undertaken indicating whether acting in a team, his role and contribution.
- Any other relevant material which may help appreciate the qualification of the applicant.

2.1.3 Fees

The charge is KShs 6,000/= to 20,000/= per year as per attached schedule:-

Fees

		Application fee (KShs.)		Annual fee (KShs.)	
		citizen	Non citizen	citizen	Non citizen
1	Application by individual expert:				
	(a) Lead Expert	3,000	9,000	5,000	15,000
	(b) Associate Expert	2,000	6,000	3,000	9,000
2	Application by a firm of Experts:				
	(a) The firm's fee	5,000	15,000	20,000	60,000
3.	Inspection of records/register ………. 200 per record/register				
4.	Environmental Impact Assessment Licence ….. 0.1% of the total cost of the project				
5.	Surrender, transfer or variation of Environmental Impact Assessment licence …..5,000				

Source: - Kenya Gazette Legal Notice No.101 of June 2003.

2.2 LICENCES FOR EIA

2.2.1 EIA Licence Fees

To obtain EIA licence, KShs. 5,000 to 500,000 is paid as detailed in schedule below.

2.2.2 Inspection Records Fee

Between KShs.200 to 500 is required to inspect records. However if photocopy is to be obtained, and additional KShs.4 per page is paid attached is the schedule.

		Inspection fees (KSh.)	Extraction fee (KSh.)
1.	Environmental Impact Assessment Report	200	4 per page
2.	Environment Audit Report	200	4 per page
3.	Environmental Impact Assessment Study Report	500	4 per page
4.	Register of Experts	500	Not for extraction
5.	Access to Data Banks	500	Not for extraction
6.	Register of projects	500	Not for extraction
7.	Other/public documents	500	4 per page.

2.3 TERMS OF REFERENCE (TOR)

2.3.1 TOR of EIA Study

(a) An environmental Impact Study shall be conducted in accordance with the TOR to be developed by the Applicant in consultation with the authority.

(b) Based on Environmental, Social, Cultural, Economic and Legal considerations, the EIA Study shall:-

- (i) Identify the anticipated Impacts of the project
- (ii) Predict the extent and scale of the impacts
- (iii) Identify mitigation measures including alternatives
- (iv) Identify mitigation measures including alternatives
- (v) Propose a monitoring and evaluation plan

(c) The proponent shall take appropriate measures to seek the views of the persons who may be affected by the project during the study by:

- (i) Publicising the intended projects and its anticipated effects in the local mass media and by public postures.
- (ii) Holding public meetings to consult the local people on their views of the projects
- (iii) Incorporating the views of the local communities in the report of the study.

2.3.2 Environmental Audit

The Audit shall be carried out through questionnaires, site visits and Test Analysis and in the manner shown hereunder:-

(a) Initial Auditing

The Auditor in conduction an Initial Audit shall:-

make a precise description of the project

present the objective, the scope and the criteria of Audit

collect and review all relevant environmental law and regulator frameworks on health and safety, sustainable use of natural resources and or acceptable national and international standards.

Verity the level of compliance by owner with conditions of regulations 31(iii)

Evaluate knowledge and awareness of and responsibility for application of relevant legislation on the laws:

Review existing project documentation related to all infrastructural facilities and designs.

Examine monitoring programmes parameters, procedures in place for control and corrective actions.

Evaluate the relationship with the Authority or other relevant bodies

Inspect all buildings, premises and yards in which manufacturing, testing, transportation within and without the project area s well as storage and disposal of goods is carried out and give a record of all significant environmental risks associated with such activities.

Example and seek views on health and safety issues form both the project employees, the local and other potentially affected communities and

Prepare a proritised list of health and environmental concerns of past and on going activities and provide recommendations on estimated costs, corrective and rehabilitation measures and test implementation

METHODOGOLOGY

3.1 DEFINITIONS RELATED TO EIA

3.1.1 EIA

EIA refers to a critical examination of the effects of a project on the environment before its implementation.

3.1.2 Screening

A process used to determine whether a project requires an Environmental Assessment and what type and level of Assessment would be necessary.

3.1.3 Scoping

A procedure for attempting to ensure that an Environmental Assessment focuses on the key Environmental issues associated with a project on omitting irrelevant material.

3.1.4 Environmental Management Plan (EMP)

An Action Plan or system which addresses the how, when, who, where and what of integrating environmental mitigation and monitoring measures throughout an existing or proposed operation or activity. It encompasses all the elements that are sometimes addressed separately in mitigation, monitoring and Action Plans.

3.1.5 Environmental Audit

A management too comprising a systematic document, periodic and objective evaluation of how well a project, organization or equipment is performing with the aim of helping to safeguard the environment. The Audit should facilitate management control of environmental practices and assess compliance with policy objectives and regulatory requirements.

3.1.6 Strategic Environmental Assessment (SEA)

A similar technique to EIA but normally applied to policies, plans, programmes and groups of projects. SEA provides the potential opportunity to avoid the preparation and implementation of inappropriate plans, programmes and projects and assists in the identification and evaluation of projects.

Alternatives and Identification of cumulative effects, SEA comprises two main types:-
Sectorial SEA (Applied when many new projects fall within one sector) and
Regional SEA (Applied when broad economic development is planned with one region)

3.1.7 Project

A project is defined as a specific set of Human activities in a particular location and time frame and intended to achieve any objective(s)

3.1.8 Environment

The term “Environment is used in its broadest possible sense to embrace not only physical and biological systems, but also socio economic systems and their inter-relationships.

3.2 LEVELS OF ENVIRONMENTAL IMPACT ASSESSMENT

3.2.1 Small Projects

Small scale projects whose potential Adverse Environmental Impacts can easily be identified and for which mitigation measures can readily be prescribed and can be included in the Design and/or Implementation of the project. This type of project will be normally approved on the basis of mitigation measures without the need of a detailed EIA study requiring field investigations.

3.2.2 Projects Suspected

Projects for which there is some level of uncertainty on the nature and level of Impacts. Decisions will be made during the scooping phase whether the project requires a full study or a partial study. The review at this stage also examines various alternatives, so that the decision maker can select options which do not have significant environmental impacts.

3.2.3 Projects with Significant Impacts

Projects which clearly have significant adverse impacts. The study will determine the nature and the extent of these impacts and examine the alternatives and the mitigation measures. Conducting such EIA requires greater public participation.

3.2.4 Strategic Environmental Assessment (SEA)

SEA can be undertaken at a range of levels, from policies to programmes and plans.

3.2.5 Auditing and Monitoring

An environmental Audit differs for EIA which aims to predict Environmental Impacts in that it is multidisciplinary process of objectively reviewing the environmental performance of an operating project or enterprise including its processes, material storage, operating procedures and Environmental Management to identify potential environmental impacts and liabilities.

3.3 APPROACH PRINCIPLES

3.3.1 General

- Focus on the main issues
    - Avoid covering too many topics in too much detail
    - Screen to limit to only the most likely and most serious of the environmental impacts.
    - Work on mitigation measures that are workable and acceptable

- Involve the appropriate persons and groups
    - Resourceful actors for EIA Study
    - Decision makers

- Link information to decision about the project
    - Organize EIA early enough to provide information to improve basic designs, and progress through the several stages of the project planning and implementation.

- Present clear options for mitigation measures and sound environmental management.

The approach to EIA study should include:

Preliminaries activities
Impact identification
Baseline study
Impact evaluation
Mitigation measures
Assessment
Documentation
Decision making
Post auditing.

3.3.2 Preliminary Activities

Identification of the activities and the actors in the project that will include:-
Decision makers
Project team
Description of actions
Review of existing legislation

3.3.3 Impact Identification

- Check list of impacts
- Selection of important impacts based on
  Magnitude
  Extent
  Significance and

Special sensitivity

3.3.4 Baseline Study

Data prior to proposed action – bench mark

3.3.5 Impact Evaluation

Quantification or qualification of impacts

3.3.6 Mitigation Measures

Identification of actions to eliminate or minimize adverse environmental impacts.

3.3.7 Assessment of Alternatives

Combination of environmental losses and gains with the socio-economic costs and benefits for each alternative.

3.3.8 Documentation

- Reference documents that will confirm detailed record of the work done in the EIA.

- Working documents that convey information for immediate action.

3.3.9 Decision Making

Decision making to be based on:-

- Socio-economic and environmental considerations
- Political considerations
- Choice not between "bad" and "good" but between "good" and "better". However, the decision maker must:

Accept one of the project alternatives
Request further study or
Reject ant proposed action altogether.

3.3.10 Post Audits

Conducted to determine the closeness or how accurate are the predations of the EIA.

- Provide information in a form useful to decision makers
  - Terms and formats of conclusions of EIA Must be understandable by decision makers.

REPORT WRITING

4.1 EIA SCOPING

(a) Executive summary
(b) The proposed project
(d) Approach and methodology
(e) Potential alternatives
(f) Study limits
(g) Environmental policy, legislative and planning framework
(h) Existing environmental conditions
(i) Scoping the environmental impacts
(j) Scooping the assessment
(k) Scooping the environmental management aspects
(l) Recommendations
(m) Technical appendices
(n) Administrative appendices – this will be consultants itinerary, etc

4.2 FULL EIA STUDY

(1) Executive summary

(2) Presentation of the project

- project selection
- project location
- project description and associated activities
- alternatives.

(3) Approval and methodology

- General approach
- Assumptions, uncertainties and constraints.

(4) Environmental policy, legislative and Planning Framework

- Environmental policy and legislative framework
- Consultation and public participation
- Institutional capacity
- Relevant ongoing projects

(5) Existing environmental conditions

(6) Assessment of environmental impacts and mitigation measures

- Prediction of potential environmental impacts and benefits
- Enhancements

(7) Environmental Management Implementation

- Environmental monitoring
- Environmental management capacity
- Environmental management

(8) Conclusion and recommendations

(9) Technical appendices

(10) Administrative appendices

5.0 EIA FORMS

Attached are forms in relation with EIA.

FORM 1 (r.6)

Application Reference No:…………………………….

THE ENVIRONMENT MANAGEMENT AND COORDINATION ACT

SUBMISSION OF PROJECT REPORT

PART A: DETAILS OF PROPONENT

A1 Name of proponent (Person or Firm ……………………….…………..……..

A2 PIN No. ……………………………………………………..……..……..

A3 Address ……………………………………………………..……..……..

A4 Name of contact person ……………………………………..………..…….

A5 Telephone No. …………………...…….. A6 Fax No. …………..……..………..

A7 E-mail …………………………………………………………………..………

PART B: DETAILS OF THE PROJECT REPORT

B1 Title of the proposed project……………………..………...……………..…….

…………………………………………….……………………………

B2 Objectives and scope of the project ……………………………………..…..

………………………………………………………………………..………

……………………………………………………...……………..…………

B3 Description of the activities …………………………..…………..…………..

…………………………………………………….……………..………

B4 Location of the proposed project ………………………..……………..………..

PART C: DECLARATION BY THE PROPONENT

I hereby certify that the particulars given above are correct and true to the best of my knowledge.

……………………………… ……………………………………..……………………

Name …………………………………….. Position ……………..………………………

Signature ……………………………..………………

On behalf of…………………………………………….

Date ……………………

(Firm name and seal)

PART D: DETAILS OF ENVIRONMENTAL IMPACT ASSESSMENT EXPERT

Name (individual/firm) ……………………………………………………………………

Certificate of registration No. ……………………………………………………...………

Address ………………………………………………………………………………..

Tel: ………………………. Fax.: ……………………… E-mail: ……………………

PART E: FOR OFFICIAL USE

Approved/not approved …………………………………………………………………….

Comments: …………………………………………………………………………….

…………………………………………………………………………………………….

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…………………………………………………………………………………………….

Officer: ………………. Sign: ………………….. Date:………………………

NB: 1. If the Project Report does not contain sufficient information required under the Environmental (Impact Assessment and Audit) Regulations the applicant may be requested to give further information concerning the project or be notified of any defects in the application and maybe required to provide the additional information.

2. Any person who fraudulently makes a false statement in a project report or alters the project report commits an offence.

Important Notes: Please submit the following:

(a) Three copies of this form

(b) 10 copies of the project report

(c) the prescribed fees to:

Director-General,
The National Environment Management Authority,
Kapiti Road, South C,
P.O. Box 47146,
NAIROBI.

Tel. 254-02-609013/27/79 or 608900 Fax. 254-02-608997

Email …………………………………………

FORM 2 (r.19)

Application Reference No…………………………….

FOR OFFICIAL USE

THE ENVIRONMENT MANAGEMENT CO-ORDINATION ACT

SUBMISSION OF ENVIRONMENTAL IMPACT ASSESSMENT STUDY REPORT

PART A: DETAILS OF PROPONENT

A1. Name of proponent (Person or Firm) ………………………………………..

A2 PIN No. ……………………………………………………………………

A3 Address. ……………………………………………………………………

A4 Name of contact person ……………………………………………………

A5 Telephone No. ………………….. A6 Fax No. ………………………………

A7 E-mail ………………………………………………………………………

PART B: DETAILS OF THE ENVIRONMENTAL IMPACT ASSESSMENT STUDY REPORT.

B1 Title of the propose project ……………………………………………………

……………………………………………………………………………….

B2 Objectives and scope of the project ……………………………………………

……………………………………………………………………………….

B3 Description of the activities ……………………………………………………

……………………………………………………………………………….

B4 Location of the proposed project ………………………………………………

……………………………………………………………………………….

PART C: DECLARATION BY THE PROPONENT

I hereby certify that the particulars given above are correct and true to the best of my knowledge.

Name ………………………………. Designation ………………………………

On behalf of…………………………………………….

Date ……………………

(Firm name and seal)

PART D: DETAILS OF ENVIRONMENTAL IMPACT ASSESSMENT EXPERT

Name (Individual/firm) …………………………………………………………..

Certificate of registration No. ………………………………………………………

Address ……………………………………………………………………………….

Tel: ………………. Fax: ……………………… E-mail ………………………

PART E: OFFICIAL USE

Approved/not approved …………………………………………………………

Comments: ………………………………………………………………………

…………………………………………………………………………………………

…………………………………………………………………………………………

…………………………………………………………………………………………

Officer……………………… Sign …………………… Date ……………

Important Notes: Please submit the following:

(a) Three copies of this form

(b) 10 copies of the project study report

(c) the prescribed fees to:

Director-General,
The National Environment Management Authority,
Kapiti Road, South C,
P.O. Box 47146,
NAIROBI.

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

Email …………………………………………

FORM 3 (r.24)

Application Reference No: ……………………………..

Registration No.: ……………………………………….

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

ENVIRONMENTAL IMPACT ASSESSMENT LICENCE

This is to certify that the Project Report/Environmental Impact Assessment Study Report ……………………………………… received from …………………………………. (name of individual/firm) ………………………………..(address) submitted to the National Environment Management Authority in accordance with the Environmental Impact Assessment & Audit Regulations regarding …………………………………. ………………………………………… (title of project whose objective is to carry on …………………………………………………………………………………….. …………………………………………………………………………………….. …………………………………………………………………………………….. ……………………………………………………………………………………. (briefly describe purpose) located at ………………………………….. (locality and District) has been reviewed and a licence is hereby issued for implementation of the project, subject to attached conditions.

Date this ……………… day …………………… of 20……..….

Signature ……………………………………………………….

(seal)

Director-General

The National Environmental Management Authority.

Conditions of Licence:

1. This licence is valid for a period of ………………….. (time within which the project should commence) from the date hereof.
2. The Director General shall be notified of any transfer/variation/surrender of this licence.

FORM 4

(r.14 (2))

Application Reference No. ……………………..

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

APPLICATION FOR REGISTRATION AS AN ENVIRONMENTAL IMPACT ASSESMENT/AUDIT EXPERT

PART A: DETAILS OF APPLICANT

A1 Name of proponent (Individual or Firm) ………………………………….

A2. Nationality ……………………………………………………………..

A3 PIN No. ……………………………………………………………..

A4 Firm (Local/Foreign) ……………………………………………………..

A5 Address ……………………………………………………………..

……………………………………………………………………………..

A6 Telephone No. ……………………….. A7. Fax No. ……………………

A8 E-mail ……………………………………………………………….

A9 Applicants academic/professional qualifications: ……………………….

………………………………………………………………………………

A10 List of professional and their academic/professional qualifications and their nationalities (where applicable)

…………………………………………………………………………………….

…..……………………………………………………………………...…………….

……………………………………………………………………….……………

A11 Experience in Environmental Impact Assessment related activities: ………………

…………………………………………………………………………….……

…………………………………………………………………………….………

…………………………………………………………………………….………

A12 Application for registration of Lead Expert or Associate Experts ………………

…………………………………………………………………………………….

A13 Previous registration No. and date of registration (if applicable)…………..………

……………………………………………………………………………………..

PART B: DECLARATION BY APPLICANT

B1 I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief.

……………..…….. …………………….. …………………

Signature of applicant Full name in Block letters Position

On behalf of ………………….. ……………………………

Firm name and seal Date

PART C: FOR OFFICIAL USE

Approved/Not approved …………………………………………………………………….

…………………………………………………………………………………………….

Comments ……………………………………………………………………………..

……………………………………………………………………………………………

……………………………………………………………………….…………………

……………………………………………………………………..…………………

Official …………………… Sign ……………….. Date ……………………..

Important Notes: Please submit the following

(a) Application Form in duplicate

(b) Curriculum vitae of all applicants: and

(c) the prescribed fee, to:

Director-General,
The National Environment Management Authority (NEMA),
Kipiti Road, South C,
P.O. Box 47146,
NAIROBI, KENYA

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

E-mail …………………………….

FORM 5 (r.14(4))

Application Reference No: …………………..
Registration No: …………………………….

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF REGISTRATION AS AN ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT EXPERT

This is to certify M/s ………………………………………………………… of ……………………………………………………………………………………………… ……………………………….. (Address) has been registered as an Environmental Impact Assessment Expert in accordance with the provisions of the Environment Management and Coordination Act and is authorized to practice in the capacity of a Lead Expert/Associate Expert/Firm of Experts (Type) ………………………………………… ……………………………………………………………………………………………..

Date this ………………… day ………………….. of 20……………..

Signature …………………………………………..
(seal)

Director – General.
The National Environmental Management Authority

FORM 6 (r.15(1))

Application Reference No. …………………….

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

APPLICATION FOR LICENCE TO PRACTICE AS AN ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT EXPERT

PART A: DETAILS OF APPLICANT

A1 Name of proponent (Individual of Firm)…………………………………….

A2 Nationality …………………………………………………………………….

A3 PIN No. ……………………………………………………………………….

A4 Firm (Local/Foreign) ………………………………………………………….

A5 Business registration No. (where applicable) ……………………………..

Date ………………………………..

A6 Address ……………………………………………………………………..………

A7 Telephone No. ………………………… A7 Fax No. …………………

A9 E-mail……………………………………………………………………………….

A10 Applicants academic/professional qualifications: ……………………..…………

……………………………………………………………………………...………..

…………………………………………………………………………………………

A11 List of professionals and their academic/professional qualifications and their nationalities (where applicable) ……………………………………….……………

……………………………………………………………………………………..

……………………………………………………………………………………..

……………………………………………………………………………………..

……………………………………………………………………………………..

A12 Experience in Environmental Impact Assessment related activities………………

…………………………………………………………………………………………

PART B: DECLARATION BY APPLICANT

B1 I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief.

……………………….	……………………….	……………………..
Signature of applicant	***Full Name in Block letters***	***Position***

On behalf of …………………..…… …………………………..

Firm Name and seal *Date*

PART C FOR OFFICIAL USE

Approved/Not approved …………………………………………………………….

Comments ……………………………………………………………………………

……………………………………………………………………………………..

……………………………………………………………………………………..

…………………………………………………………………………………….

Official …………………. Sign ………………… Date …………………

Important Notes

Please submit the following:

(a) Application Form in duplicate

(b) Curriculum vitae of all applicants; and

(c) The prescribed fee, to

Director-General,

The National Environment Management Authority (NEMA),

Kapiti Road, South C,

P.O. Box 47146,

NAIROBI, KENYA.

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

E-mail …………………………….

FORM 7 (r.15(2))

Application Reference No.:…………………………
Licence No.: ………………………………………..

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINAITON ACT

ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT PRACTICING LICENCE

M/s ……………………………………………………….(Individual or firm) of Address ……………………………………………….……………………………….

…………………………………………………………………………………………

……………………………………………………..……………………………………

…………………………………………………………………… is licensed to practice in the capacity of a (Lead Expert/Associate Expert/Firm of Experts) ……………………..

……………………………………………………… in accordance with the provisions of the Environmental Management and Coordination Act.

Date this …………………….. day ……………………. of 20 …………………

Signature ………………………………………………………………………

(Seal)

Director-General,

The National Environmental Management Authority.

Conditions of Licence:

1. This licence expires on 31st December, 20…………….

FORM 8 (r.21)

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINAITON ACT

NOTICE TO THE PUBLIC TO SUBMIT COMMENTS ON AN ENVIRONMENTAL IMPACT ASSESSMENT STUDY REPORT.

Pursuant to Regulation 21 of the Environmental (Impact Assessment and Audit) Regulations, the National Environmental Impact Assessment Study Report for the implementation of the proposed project……………………………………….……………

…………………………………………………………………………….………………

……………………………………………………………………………………………….

…………………………………………………………(brief description of project)

at …………………………………………………………………………………………….

…………………………………………..(locality) of ………………………………… District. The said project anticipate the following impact ………………………………… ……………………………………………………………………………………………

……………………………………….…………………………………………………

(describe anticipated impacts and proposed mitigation measures).

The full report of the proposed project may be inspected during working hours at:

(a) The NEMA Headquarters.

(b) …………………………..

(c) …………………………..

NEMA invites members of the public to submit oral or written comments within ……..… days of the date of publication of this notice to assist the Authority in the approval process of the project to:

(a) Director – General, NEMA,

(b) …………………………….

(d) …………………………….

Dated this ……………………day…………………..of 20……….

Signature ………………………………………………

(seal)

Director-General,

The National Environmental Management Authority.

FORM 9 (r.25)

Application reference No. ……………

Licence No. …………………………..

FOR OFFICIAL USE

THE ENVIRONMENT MANAGEMENT AND CO-ORDINATION ACT

APPLICATION FOR VARIATION OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

PART A: PREVIOUS APPLICATION

No previous application for variation of an environmental impact assessment licence.*

The Environmental Impact assessment licence was previously amended.*

PART B: DETAILS OF APPLICANT

B1 Name (Individual or firm): ……………………………………………………………

B2 Business Registration No: ………………………………………………… ……

B3 Address: ……………………………………………………………………………

B4 Name of contact person: ………………………………………………………………

B5 Position of contact person: ……………………………………………………………

B6 Address of contact person: ……………………………………………………………

Telephone No.: ……………………… Fax: …………………. No: ……………

E-mail: ……………………………………………………………………………….

PART C: DETAILS OF CURRENT ENVIRONMENTAL IMPACT ASSESSEMENT LICENCE.

C1 Name of the current Envir+onmental Impact Assessment licence holder ………….

……………………………………………………………………………………..

……………………………………………………………………………………..

C2 Application No. o the current Environmental Impact Assessment licence ………..

………………………………………………………………………………………

C3 Date of issue of the current Environmental Impact Assessment licence …………..

………………………………………………………………………………………

PART D PROPOSED VARIATION STO THE CONDITIONS IN CURRENT ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

D1 Conditions in the current Environmental Impact Assessment license…………….

…………………………………………………………………………………

……………………………………………………………………………………...

…………………………………………………………………………………

D2 Proposed variation(s)…………………………………………………………..

…………………………………………………………………………………

…………………………………………………………………………………

…………………………………………………………………………………

D3: Reason for variation(s) ……………………………………………………………

…………………………………………………………………………………

…………………………………………………………………………………

…………………………………………………………………………………

D4 Describe the environmental changes arising from the proposed variation(s) ……....

…………………………………………………………………………………

…………………………………………………………………………………

…………………………………………………………………………………

D5 Describe how the environment and the community might be affected by the proposed variation(s)……………………………………………………………….….

…………………………………………………………………………………

…………………………………………………………………………………

D6 Describe how and to what extent the environmental performance requirements set out in the EIA report previously approved or project profile previously approved or project profile previously submitted for this project may be affected …………………………………………………………………………………

…………………………………………………………………………………

…………………………………………………………………………………

D7 Describe any additional measures proposed to eliminate, reduce or control any adverse environmental impact arising from the proposed variation(s) and to meet the requirements in the Technical Memorandum on Environmental Impact Assessment Process……………………………………………………….………...

..……………………………………………………………………….……….

…………………………………………………………………………………

…………………………………………………………………………………

PART E: DECLARATION BY APPLICANT.

I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief. I understand the environmental impact assessment licence may be suspended, varied or cancelled if any information given above is false, misleading or incomplete.

…………………….. …………………….. ……………………………

Name *Position* *Signature*

on behalf of ……………………………….. …………………………………….

Company name and seal *Date*

PART F: OFFICIAL USE

Approved/Not approved ………………………………………………………..

………………………………………………………………………………..

Comments …………………………………………………………………….

…………………………………………………………………………………

…………………………………………………………………………………

…………………………………………………………………………………

Officer …………….. Signature………………… Date ………………..

Important Notes

Please submit:-

(a) 3 copies of this completed Form: and

(b) The prescribed fee to:

Director-General,

The National Environment Management Authority,

Kapiti Road, South C,

P.O. Box 47146,

NAIROBI.

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

E-mail …………………………….

*Delete where applicable

FORM 10 (r.25)

Application Reference No…………………….
Certificate No…………………………………

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF VARIATION OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

This is to certify that the Environmental Impact Assessment Licence No. ……………… …………………….. Issued on …………………………….(date) to …………………. …………………………………………..(name of individual/firm) of …………… ……………………………………………………………………(address) regarding ……………………………………………………………………………. …………………….(title of project) whose objective is to …………………………….. ……………………………………………………………………………………… ……………………………………………………(briefly describe purpose) located at ……………………………………………………(locality and District) has been varied to ………………………………………………………………………… ………………………………………………………………………………………… ………………………………………………………………………………………… …………………………(nature of variation) with effect from …………………….(date of variation) in accordance with the provisions of the Act.

Dated this ………………………… day …………………..of 20……………..

Signature ……………………………

(Seal)

Director-General

The National Environmental Management Authority.

FORM 11 (r.26)

Application reference No……………………

Licence No:…………………………………

FOR OFFICIAL USE

ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

NOTIFICATION OF TRANSFER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

PART A: DETAILS OF CURRENT LICENCE

A1 Name of the current Environmental Impact Assessment licence holder………… ………………………………………………………………………………………

A2 PIN No. ……………………………………………………………………….

A3 Address ……………………………… A4 Tel: …………………………………

A5 Fax No. ……………………………….A6. E-mail ……………………………..

A7 Application No. of current Environmental Impact Assessment license ………. ……………………………………………………………………………………..

A8 Date of issue of current Environmental Impact Assessment license …………. …………………………………………………………………………………….

PART B DETAILS OF THE TRANSFEREE

B1 Name (Individual/firm)…………………………………………………………

B2 PIN No. ……………………………………………………………………….

B3 Address …………………..………………… A4 Tel: …………………………

B5 Fax No. …………………………. B6. E-mail …………………………………

B7 Name of contact person…………………………………………………………

B8 Capacity of transferee to run the project (financial, technological, manpower) ….. ……………………………………………………………………………………… ……………………………………………………………………………………… ………………………………………………………………………………………

PART C: REASONS(S) FOR TRANSFER OF LICENCE

………………………………………………………………………………………

……………………………………………………………………………………….

……………………………………………………………………………………….

PART D: DECLARATION BY RANFEROR AND TRANSFEREE

It is hereby notified that ……………………………………. of …………………… on this day of ……………………….. transferred EIA licence No………………….to ……..

…………………………………………..of ……………………………………………
who will assume his responsibility for all liability under this project.

Transferor	Transferee
Name …………………………..	Name …………………………..
Address …………………………	Address ………………………..
Signed …………………………..	Signed …………………………
Date ………………………………..	Date ……………………………

PART E: FOR OFFICIAL USE

Approved/Not approved…………………………………………………………………

………………………………………………………………………………………

Comments ……………………………………………………………………….

…………………………………………………………………………………………

Officer………………… Signature ………………… Date…………………………

Important Notes:

1. Where an Environmental Impact Assessment licence is transferred, the person to whom it is transferred and the person transferring it shall jointly notify the Director-General, of the transfer.
2. The person holding an environmental impact assessment licence assumes responsibility for the transfer of the licence only in respect of the project to which this licence was issued.
3. Any transfer of an environmental impact assessment licence, shall take effect on the date the Director General is notified.
4. This Form must be submitted in quadruplets with
5. Prescribed fees, to:

Director General,

The National Environment Management Authority,

Kapiti Road, South C,

P.O. Box 47146,

NAIROBI.

Tel. 254-02-609013/27/79 Fax. 254-02-608997

E-mail ……………………………….

FORM 12 (r.26)

Application Reference No: …………

Certificate No. ………………………

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF TRANSFER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE

This is to certify that the Environmental Impact Assessment Licence No………………..

Issued on …………………………………… (date) to ………………………………….

………………………………………. (name of previous holder) of …………………….

…………………..(address) regarding …………………………………………………… (title of project) ………………………………………………………………………..

whose objective is to ……………………………………………………………….

……………………………………………………………………………………….

……………………………………………………………………………………….

(briefly describe purpose) located at ………………………………………..(locality and District) has been transferred to ……………………………………………….(name of new holder) of …………………………………………………..(address) with effect from ………………………………………………………………..(date of transfer) in accordance with the provision of the Act.

Dated this ……………………day ……………………of 20……………

Signature …………………………………….

(seal)

Director – General,

The National Environmental Management Authority.

Important notes

1. The transferee as well as the transferor of a license under this regulation shall be liable for all liabilities and the observance of all obligations imposed by the transfer in respect of the license transferred.
2. The transferor shall not be responsible for any future liabilities or any obligations so imposed with regard to the license form the date the transfer is approved.

FORM 13 (r.27)

Application reference No: …………………..

Licence No: …………………………………

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

NOTIFICATION OF SURRENDER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENSE.

PART A: PROPONENT DETAILS

A1 Name: (Individual or firm) …………………………………………….

A2 PIN No. ………………………………………………………………….

A3 Address ………………………………………………………………….

A4 Name of contact person ………………………………………………

A5 Position of contact person ……………………………………………

A6 Address …………………… Tel …………………. Fax No…………

E-mail ……………………………….

PART B: DETAILS OF THE CURRENT ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

B1 Environmental Impact Licence No…………………………………..

B2 Title of project under the current Environmental Impact licence

…………………………………………………………………………………

…………………………………………………………………………………

…………………………………………………………………………………

B3 Please state the following details of the Environmental Impact Assessment licence to be surrendered.

(a) Scope/scale of project(s)……………………………………………………

………………………………………………………………………………….

………………………………………………………………………………….

(b) Conditions on the EIA licence ……………………………………...……….

………………………………………………………………………………….

………………………………………………………………………………

………………………………………………………………………………

…………………………………………………………………………………….
…………………………………………………………………………………….

PART C: REASON(S) FOR SURRENDER

…………………………………………………………………………………………….
…………………………………………………………………………………………….
…………………………………………………………………………………………….
…………………………………………………………………………………………….

PART D: DECLARATION BY PROPONENT

I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief.

…………………………. ………………………………… ……………………..
Name of applicant *Full name in block letters* *Position*

on behalf of ………………………………. ………………………….
Company name and seal *Date*

PART E: FOR OFFICIAL USE

Approved/Not approved ………………………………………………………………….
………………………………………………………………………………………………
Comments ……………………………………………………………………………….
………………………………………………………………………………………………
………………………………………………………………………………………………
………………………………………………………………………………………………
Officer ………………….. Signature ……………………. Date…………………

Important Notes:

Intent to surrender an environmental impact assessment license should be communicated to the Authority at least six months before the date of surrender.

FORM 14 (r.27)

Application Reference No.: …………………….

Certificate No.: …………………………………

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF SURRENDER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

This is to certify that the Environmental Impact Assessment License No:……………… Issued on ………………………………..(date) to ……………………………………… (name of individual/firm) of ………………………….………..(address) regarding …………………………………………………………………………………(title of project) whose objective is to …………………………………………………(briefly describe purpose) located at ……………………………………………………(locality and District) has been duly surrendered with effect from ………………………………… (date) to the National Environment Management Authority in accordance with the provisions of the Act.

Dated this ……………………… day ………………………. of 20………..

Signature ………………………………………..

(seal)

Director-General

The National Environmental Management Authority.

Important Note:

A surrender shall be without prejudice to any liabilities or obligations which have accrued on the holder of the license prior to the date of surrender.

FORM 15 (r.29)

Form No. ……………………………..

Reference No. …………………………

FOR OFFICIAL USE

ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

NATIONAL ENVIRONMENT MANAGEMENT AUTHORITY
APPLICATION FOR ACCESS TO INFORMATION

PART A: DETAILS OF APPLICANT

A1. Name: ……………………………………………………………….

Address: …………………………………………………………..

…………………………………………………………………….

…………………………………………………………………….

…………………………………………………………………….

Telephone: …………………………. Fax …………………………

E-mail ……………………………………………………………

Profession ………………………………………………………….

Date ………………………………………………………………….

A2 NAME OF EMPLOYER (if applicable)……………………………………………

Address: ……………………………………………………………………………

……………………………………………………………………………………

Telephone: …………………………… Fax: ……………………………………

E-mail ………………………………………………………………………….

Designation ……………………………………………………………………….

PART B: INFORMATION DETAILS

B1. TYPE OF INFORMATION REQUIRED (tick as appropriate)

- ☐ Project Report
- ☐ Environmental Impact Assessment Study Report
- ☐ Environmental Audit Report
- ☐ Strategic Environmental Assessment Report
- ☐ Environmental Monitoring Report
- ☐ Record of Decision (ROD) for Environmental Impact Assessment Approvals
- ☐ Licenses for Project Reports
- ☐ Licenses for Environmental Impact Assessment
- ☐ Environmental Impact Assessment Experts (Individual)
- ☐ Environmental Impact Assessment Experts (Firms)

B2 DOCUMENT

Title of the document ……………………………………………………………………

Author ……………………………………………………………………………………..

Year ………………………………………………………………………………………..

B3 HOW THE INFORMATION IS EXTRACTED?

- ☐ Reading
- ☐ Environmental Impact Assessment Experts (Individual)

B4 PURPOSE FOR REQUIRING THE INFORMATION

- ☐ Educational
- ☐ Research
- ☐ Affected party
- ☐ Interested party

Important note:

A prescribed fee of KShs.200 will be charged for access to information per record/register.

FORM 16A (r.47)

REGISTER OF ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT EXPERTS: (FIRMS)

Name of Firm	Registration No.	Date of Registration	Contact Address	Area of specialization	Experience & Category (eg. Lead, Associate)	Signature of Filing Officer.

FORM 16B (r.47)

REGISTER OF ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT EXPERTS: (INDIVIDUALS)

Name of Person	Registration No.	Date of Registration	Contact Address	Area of specialization	Experience	Signature of Filing Officer.

FORM 16C (r.47)

REGISTER OF ENVIRONMENTAL IMPACT ASSESSMENT PROJECT REPORTS SUBMITTED TO NEMA)

Name of Proponent	Title of Project	Location of Project	Sector	Date Received	Status of Project	Signature of Filing Officer.	Decision of NEMA

FORM 16D (r.47)

REGISTER OF ENVIRONMENTAL IMPACT ASSESSMENT STUDY REPORTS SUBMITTED TO NEMA

Name of Proponent	Title of Project	Location of Project	Sector	Date Received	Status of Project	Signature of Filing Officer.	Remarks

FORM 16E (r.47)

REGISTER OF STRATEGIC ENVIRONMENTAL ASSESSMENT REPORTS SUBMITTED TO NEMA)

Name of Proponent	Title of Project	Location of Project	Sector	Date Received	Status of Project	Signature of Filing Officer.	Remarks

FORM 16F (r.47)

REGISTER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCES

Project Title	Name of Proponent	Location of Project	Value of Project (KShs)	Licence No.	Date of Issue	Conditions attached to Licence (if any)	Date filed	Status of Licence	Name of Filing Officer	Date and Signature of filing Officer	Remarks

Note: Details of Status of Licence
(a) New
(b) Transferred
(c) Surrendered

FORM 16G (r.47)

REGISTER OF ENVIRONMENTAL IMPACT ASSESSMENT AUDIT REPORTS

Title of project	Name of Proponent	Location of Project	Sector	Date received	Status of project	Signature of Filing Officer	Date	Remarks

FORM 16H (r.47)

REGISTER OF ENVIRONMENTAL IMPACT ASSESSMENT MONITORING REPORTS

Title of project	Name of Proponent	Location of Project	Sector	Date received	Status of project	Signature of Filing Officer	Date	Remarks

FORM 16I (r.47)

REGISTER OF THE PROPRIETARY INFORMATION

Title of Firm	Name of Proponent	Licence Register	Contact Address	Type of Proprietary Information	Status of project	Signature of Filing Officer	Remarks

PRACTICE MANUAL

FOR

WATER SUPPLY SERVICES

IN

KENYA

PART E

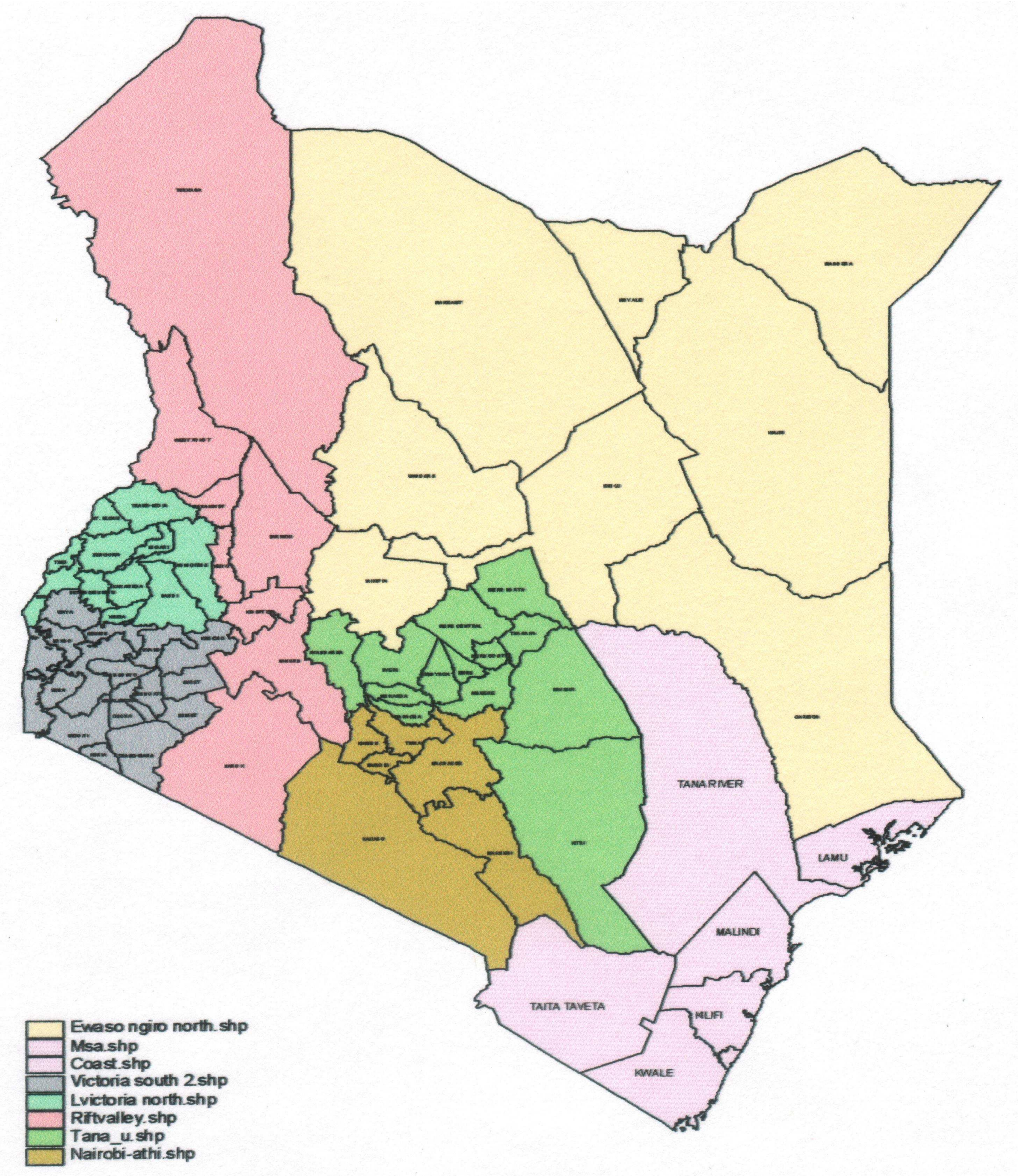
APPENDIX

LIST OF CONTENTS

PART A: WATER SUPPLY

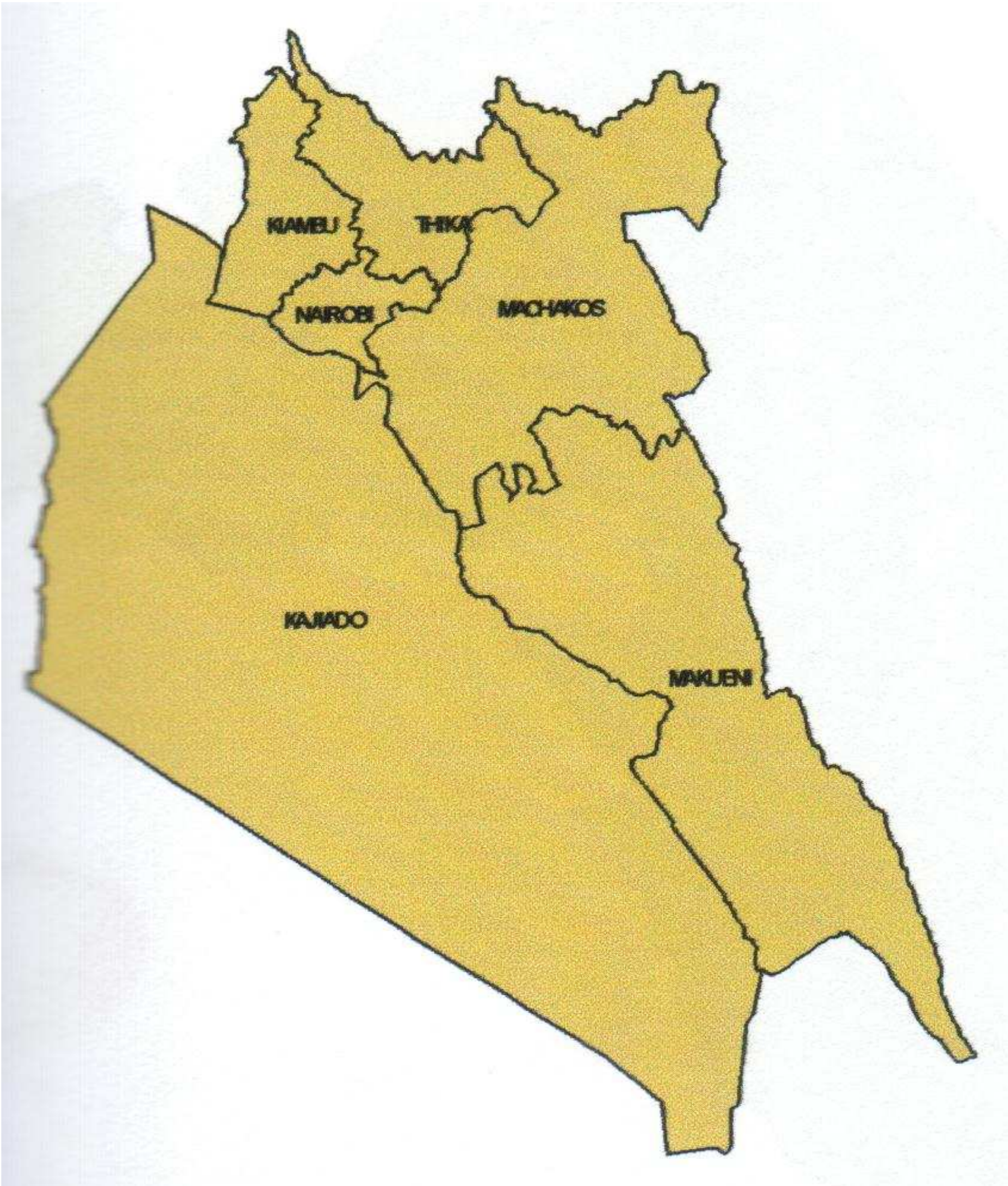
APPENDIX A:

DISTRICT BOUNDARIES & WATER SERVICE BOARDS



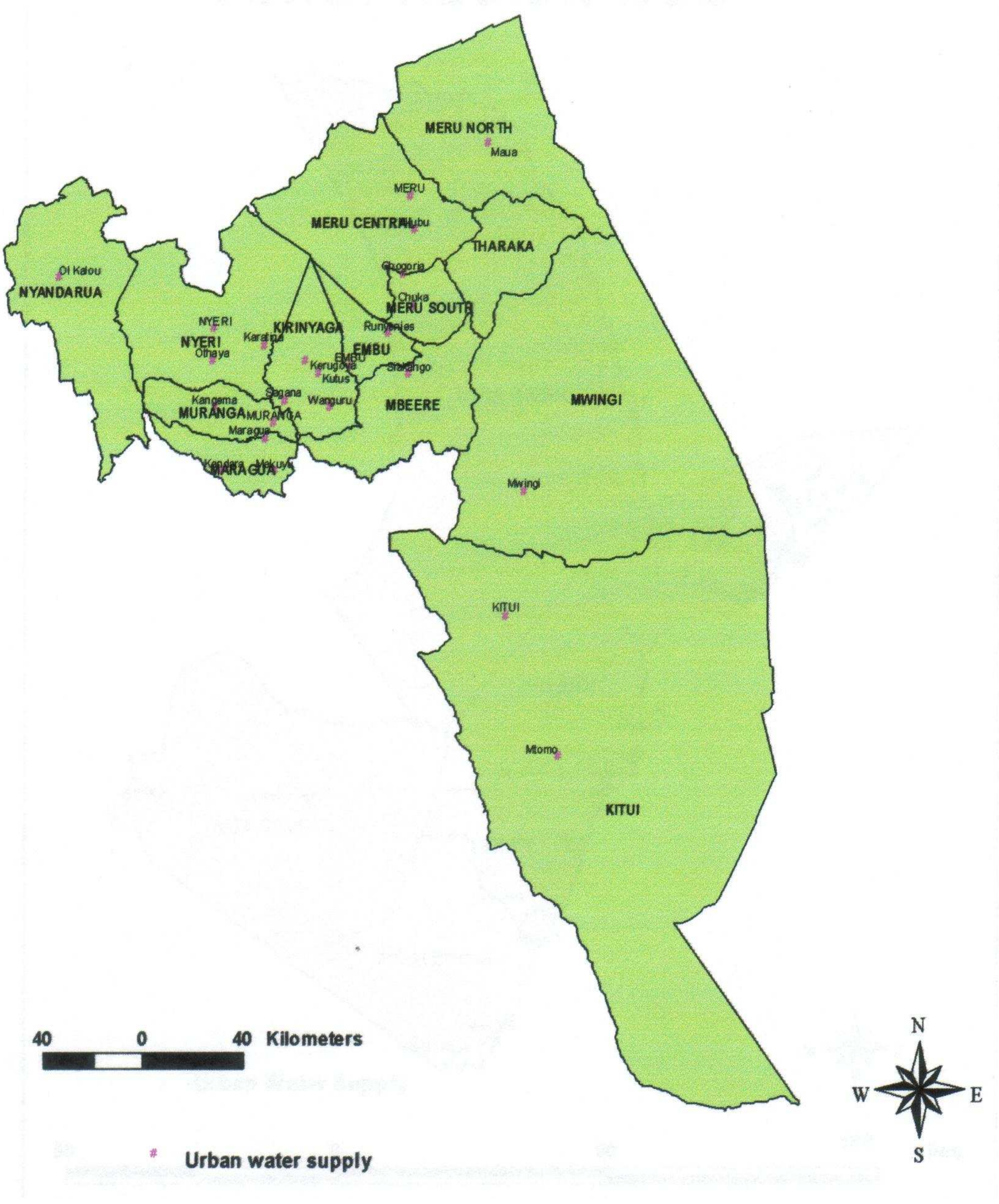
A.1.1:

ATHI WATER BOARD



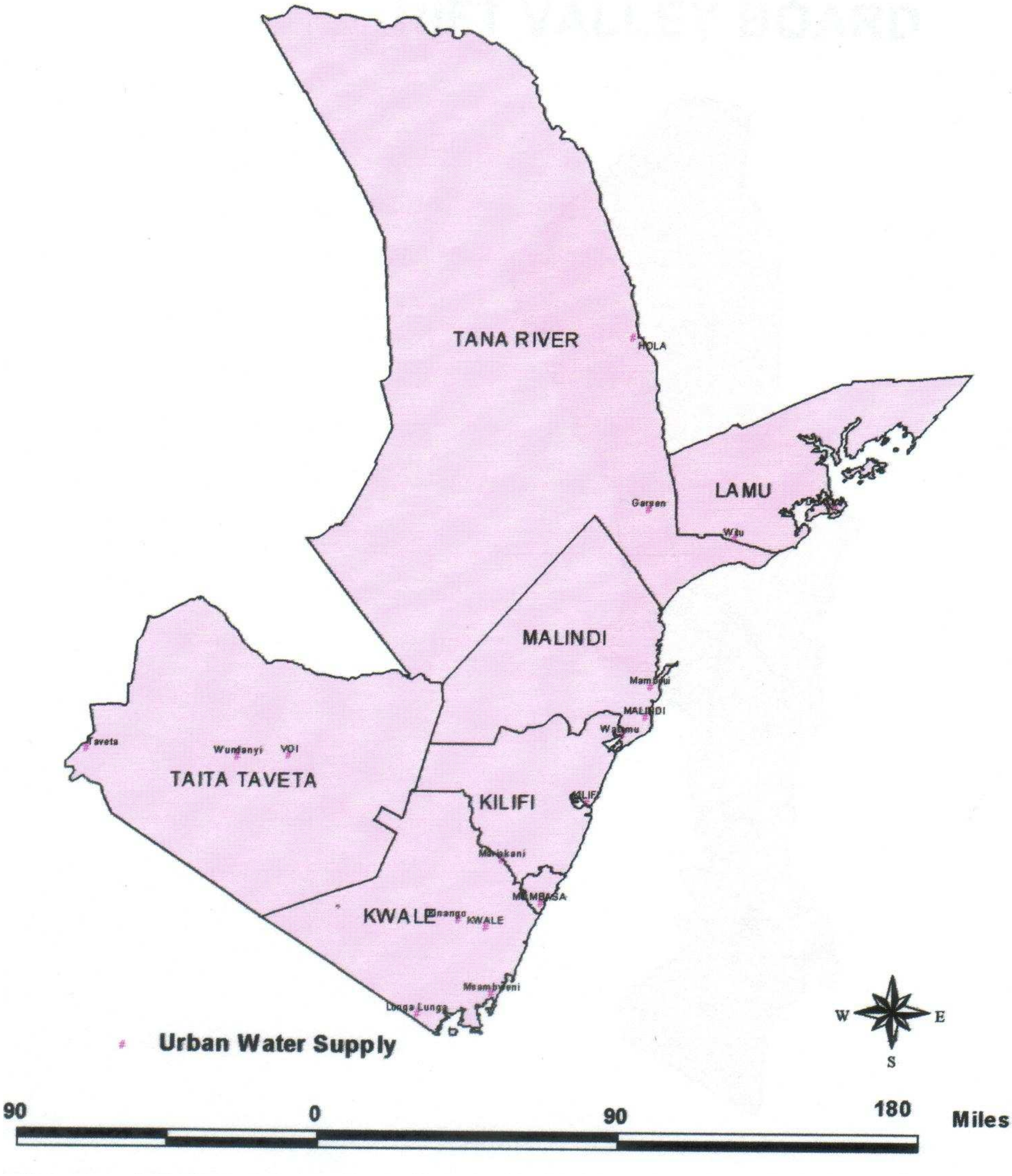
A.1.2:

CENTRAL REGION WSB



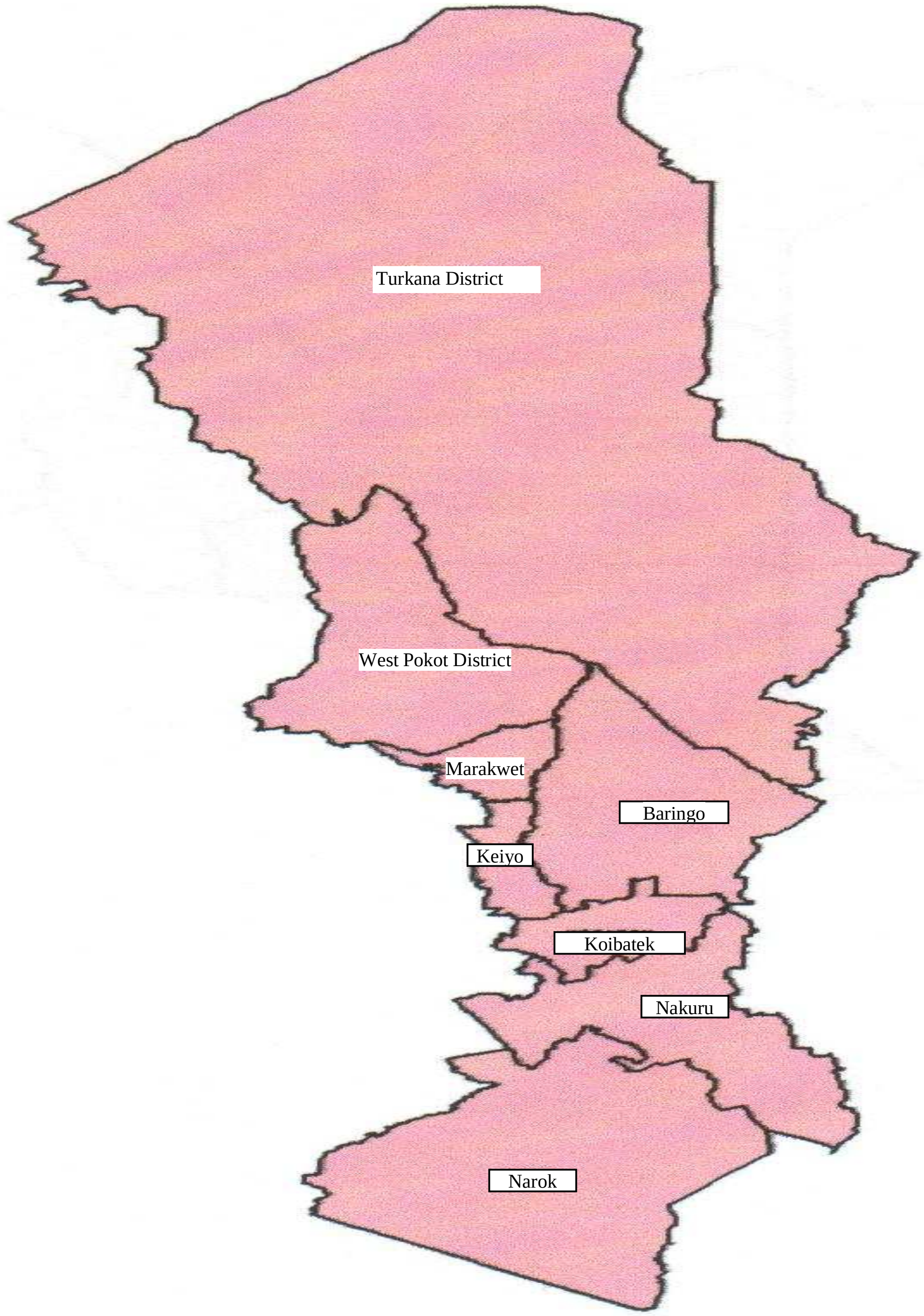
A.1.3:

COAST REGION WSB



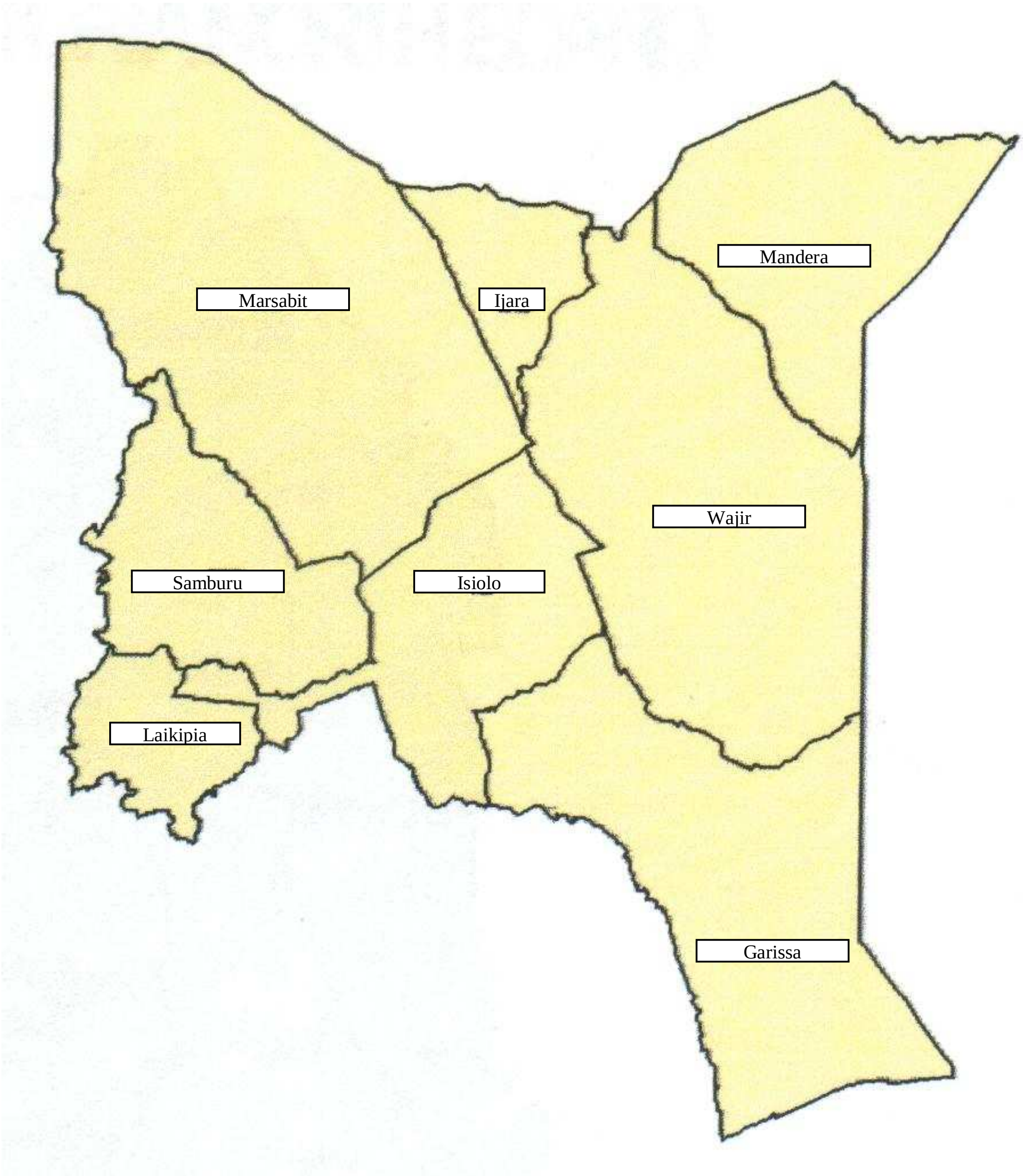
A.1.4:

RIFT VALLEY BOARD



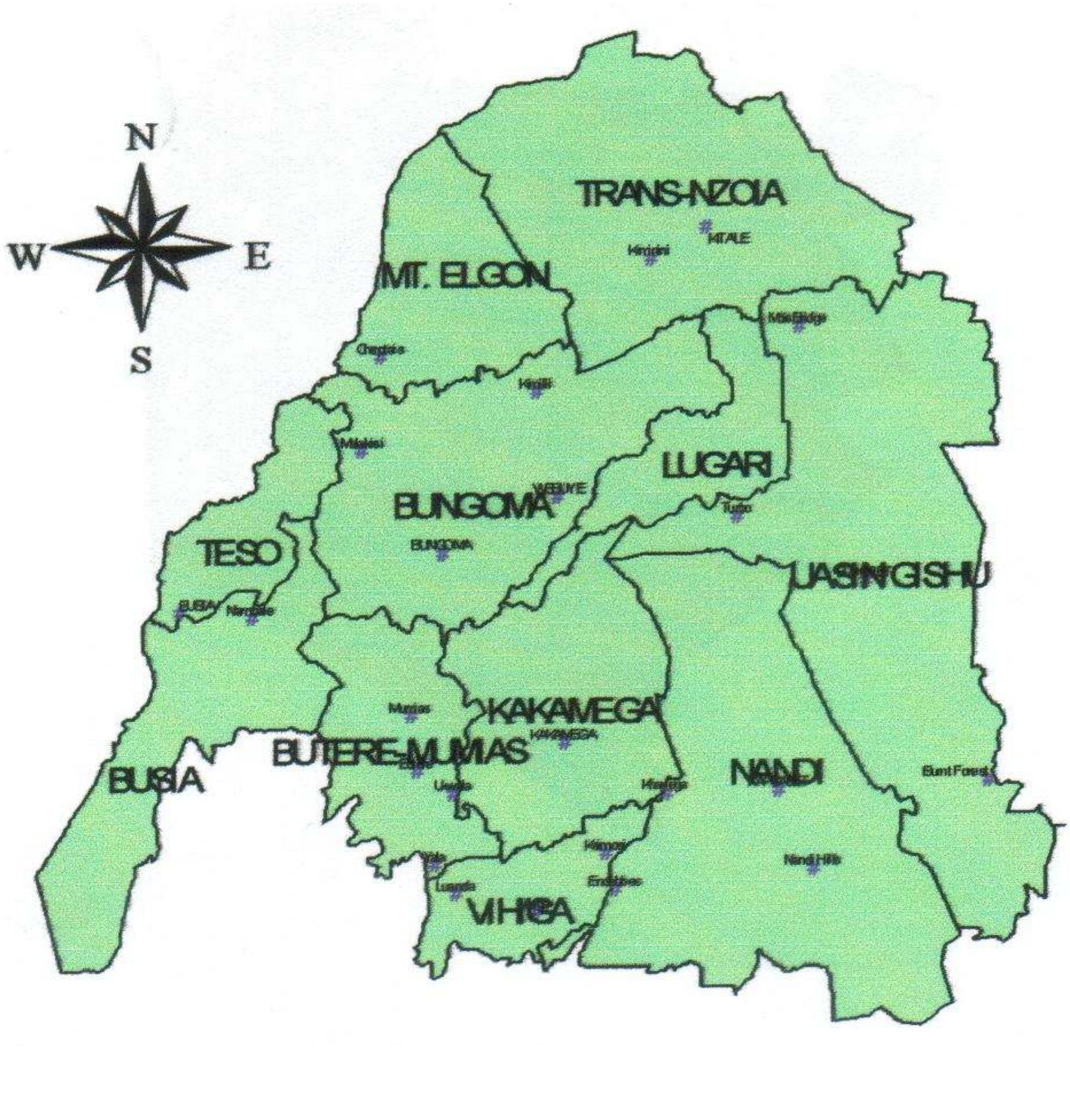
A.1.5:

NORTHERN BOARD



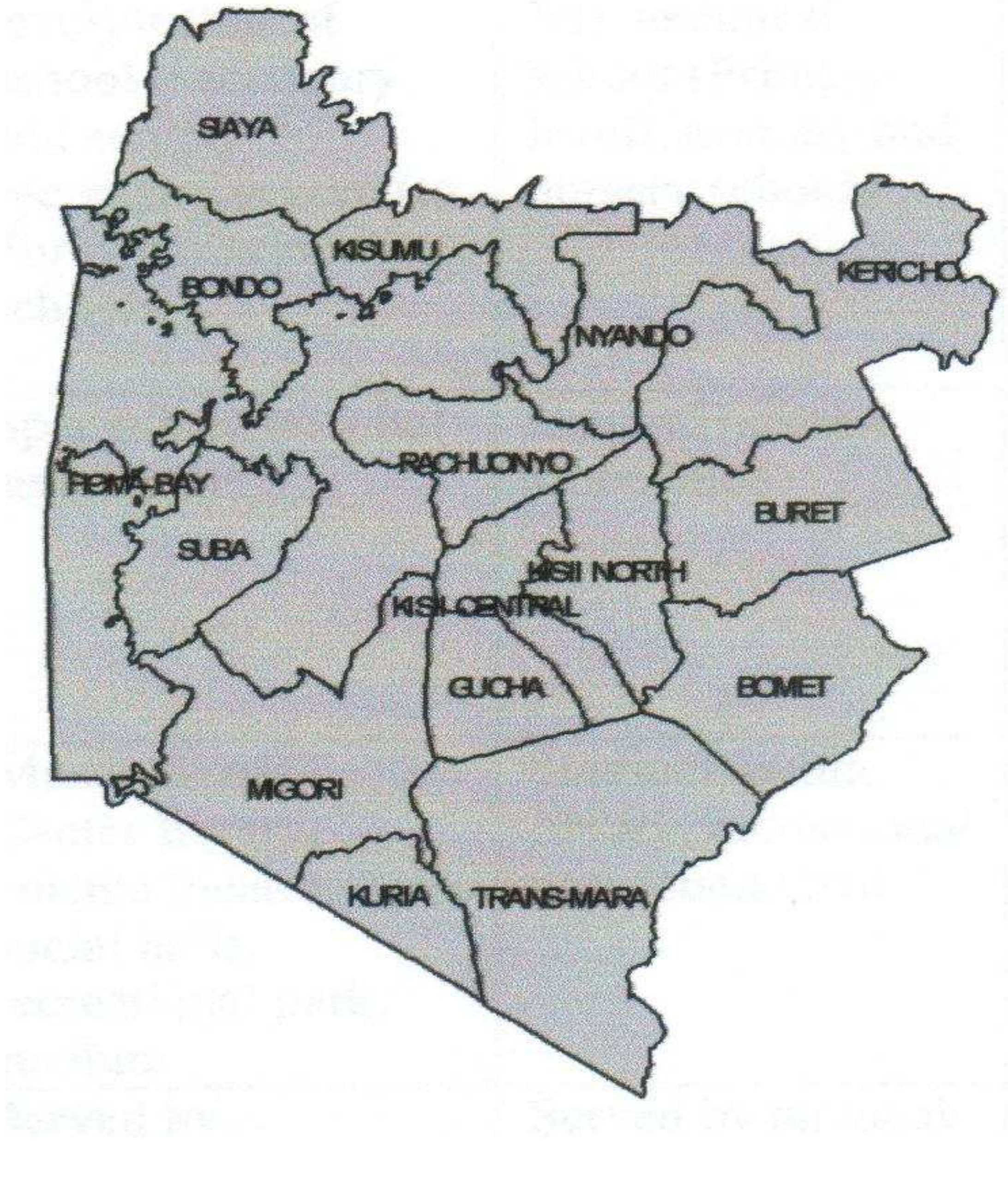
A.1.6:

LAKE VICTORIA NORTH BOARD



A.1.7:

LAKE VICTORIA SOUTH BOARD



A.2: LONG TERM GUIDELINES FOR LOCATION OF INFRASTRUCTURAL FACILITIES IN THE VARIOUS LEVELS OF CENTRES

(SOURCE: PPD'S CHART FOR URBAN AND RURAL ORGANISATION).

LEVEL/ FACILITY	PRINCIPAL TOWNS	URBAN CENTRES	RURAL CENTRES	MARKET CENTRES	LOCAL CENTRES
Population served	> 1,000,000	> 100,000	> 50,000	> 15,000	> 5,000
Resident population	> 100,000	> 5,000	> 2,000	Negligible	Negligible
Civic services	Sewage disposal system, water supply, electricity, fire station.	Sewage disposal system, grid water supply, electricity	Sewage disposal system, grid water supply electricity	public water supply	Public water supply
Education services	Teacher training college, (Primary level) technical school (secondary and senior secondary school (to Form VI) primary schools.	Senior secondary school (To form VI), technical school (Primary level), primary and nursery schools.	Secondary school (at least to form IV). Technical school (primary level) primary and nursery schools. Village polytechnic.	Secondary school, primary and nursery schools	Full primary school (2-3 streams), nursery school
Health services	Specialized hospital ambulance	General hospital ambulance	Health center + maternity unit	Health sub centers including family planning service	dispensary
Recreation and other social services	Museum/ ART. Center library, cinema theatre, social halls, recreational park, stadium	Stadium, public library, recreational park, social hall	Mobile library service sports field, social hall, mobile cinema show ground	Public meeting place, bars	Public meeting place, bars
Communi-cation	Served by international/national trunk road, head post office, telephone facilities automatic exchange) regional bus service airfield.	Served by national/ primary road departmental post office, telephone facilities, regional bus service, air strip	Served by primary/ secondary road, departmental post office, radio telephone facilities, local bus service air strip	Served by secondary/ minor road radio call or telephone, sub post office, airstrip (in remote areas only) local bus	Served by secondary/ minor roads radio call telephone, sub post office, local bus service air strip (in remote areas only)
Commercial	Residential hotels and restaurants, banks and insurance companies, wholesale facilities – warehousing, shopping center with spec. market retail facilities	Local hotel and restaurant, branch bank, wholesale and retail facilities, large market rural industrial development centres	Local hotel and restaurant mobile bank service, general shops and medium size market.	General dukas and small market teashops.	General dukas and small market tea shops

A.3: SCHEDULE OF PRINCIPAL TOWNS AND SERVICE CENTERS

CENTRAL PROVINCE

Principal towns	Admin. District	Urban centres	Rural centres	Market centres		Local centres		
Nairobi (2,143,2 54)	Nairobi							
Thika (106,707)	Thika	Ruiru (53,000) Juja (6,009)						
	Kiambu	Limuru (68,326) Kiambu (60,605) Kikuyu (156,131) Githunguri (5,370)	Gatunguya/ Mangu Uplands Kalimoni Gatundu Karuri	Kilima- Mbogo Kabati Kartu Kiganjo Ngenda (Kiamwani) Kamwangi Ngorongo Mataara Tigoni Muguga Ngecha Kijabe Wangige (Mughumu)	Lusigeti Kamangu Cianda Kagwe Githiga Gighioro Ngewa Kairi Tinganga Marigo Kimende	Mitubiri Kirinyange Munyu Wamwangi Kimunyu Kamunyaka Gachika Gatitu Nyamanger a Kanyoni Biberioni Makutano Marboukia Kahuho	Kiranga Thogoto Uthiru Rironi Mwimuto Lower Kabete gikambura Kamangu Ndumberi Ruaka Kiambaa Miguta Kamburu Kamuchege	Gathangari Ikinu Kibichoi Kiratina Gacoiri Katwa Riuki Kambaa Kihara Magina Gacharage Gatunguru
	Kirinyaga	Kagio (260) Kerugoya ( ) Sagana (8,439) Wanguru (8,181) Baricho (261) Kutus ( ) Kianyaga (2,541) Kimunye (1,606)	Kianyaga Kutus Baricho	Mururi Kiamutugu Ithareni Mukarara Kimbimbi	Mwea/ Karima Kagumo Makutano Gatithi	Murinduki/ Tongonyes Githure Karucho Karumandi Kiangueny Kimunye Gathigiriri Nyangati Kiriko	Nguka Mutithi Kiamwanja Gitumgi Kangei Kiamichiri Kiangai Kiamaina Mutitu Kiburu	Kibirigwi Rukanga Kiandai Kiamuthabi (kanyekini) Riakiania Rwambiti Kadongu Ndindiruku Muthigiini
	Muranga	Muranga (58,007)	Kaburo (Muriranjes) Kirwara (Gatanga) Kigumo Kiriaini Kangema	Kahuhia (Karuri) Gitugi Kanyenyaini Njumbu (Nyakianga) Gacharageini Githumu Gaichanjiru (Kangunduin i) Kaharati Gacharage Kateti Githiga Gatara	Gatura Mugomoini Kangare Kinyona Muthulu Ndunyu chege Kaweru (Mugeka) Gaturi) Kiria Kiunyu (Kihumbui ni) Kigumo Ithanga Mariira	Gakubwe Gakuyu Gakoe Githunguri Kiamaia Kiathe Kiangage Muchungu- cha Kirate Githembe Kipanga Gaitwa Giatutu Muruka Githunguri Makutano Mariaini Kaburumo (Muthithi)	Gaikoigo Maragua Ridge Muri Kano Nyagainga Tuso Kiruri Chichi Ichagaki Kamahuha Rwathis Kahuti Kariua Kagumoini Ngutu Gitugu Murarandia (1) Koimbi	Wajengi (Gituto) Mukarara Chumini Kigwa Makangu Ndakaini Gatakaini Makombok i Gacharage Gakui Machocho Kamokabi Githagara Giloe Gathera Kaganda Mitubiri
	Maragua	Kandara (37,311) Makuyu (28,878) Kabati (1,696) Saba saba (1,600)						
	Nyandaru a	Ol Joro Orok (2,223) Nyahururu (16,000) Njabini (4,251) Ol Kalou (49,865) Maili Inya (3,580)	N. Kinangop Kipipiri (Miharati)	Ngano Ndaragwa Kaheho Rurii Ngorika	Ndungu Njeri Murungaru Magumu Milangine Gete	Igwamiti Kangui Dundori Sabugo Githioro (Mawingo) Koinange	Nyairoko (Passenga) Pondo Wanjohi Weru (Muruaki) Gathanji	Kimathi (Malewa) Kiriko (Turasha) Gathandia (Tumaini) Mukeo

		Engineer (3,292) Mugumo (2,949) Ndunyu Njeru (2,446) Miharati (1,479) Ndaragwa (1,101) Wanjohi (1,035)				(Karati) Heni (Karati)	Kambaa (Pesi) Gathara (Tulafa)	Gichaka Munaka (Githioro) Migaa Gichungo Karangatha
Nyeri (101,238)	Nyeri	Karatina (126,337) Othaya (9,966) Mweiga (2,707) Naro moru (2,861) Endarasha (2,862) Mukurweini (1,758)	Wamagana Kigannjo (Kirichu)	Ihururu Kinunga Gatitu Kibiruini Ruthagati Giakanja Ihithe Kiandu	Tumutumu Giakaibai Dichichi Mahiga Gathinga Mahuti Gakindu Waraze	Ndathi Muthuaini Muruguru Gichira Gachika Kiamariga Gatunganga Ngorano Gachatha Gatumbiri Gathuthi Kigwandi	Kangaita Kiawarigi Gikororo Ndimaini Kianjogu Ngandu Kaikuyu Gatondo Ihuririo Karema Dirithia Kagicha Chinga Kagere	Kahati Kaharo Thageini Tambaya Ichamara Gumba Gilondi Kanunga Gatarakwa Hombe Ambogi Embaringo Muyogo

COAST PROVINCE

	Kilifi	Kilifi (75,527) Mariakani (68,509) Mtwapa (18,397) Kaloleni (4,431) Majengo (4,427) Bamba (1,369)	Kakoneni Mazeras	Gongoni Mambrui Gede Watamu Vitangeni Ganze Takaungu Vipingo Rabai Kinagoni	Fundisha (Kibaoni) Hadu Ngomani Marikubuni Marata Garashi Baricho Chakama Dugamra (Bulu) Jilore Kakuyuni Maziwani (Ganda) Msabana	Roka Mtondio Sikoke Dida Jaribuni Junja Jeuri Ribe Jimba Kiduluni Kwa Damu Golani Kibaoni Mdangoni Gongoni Mbuyuni (Tsagwa)
	Kwale	Kwale (15,890) Kinango (1,707) Lunga lunga (967) Vanga (2,532) Ukunda (Diani) (43,916)	Msambweni	Tiwi Ngombeni Ramisi Kikoneni Ndavaya Samburu	Tsunza Gandini Matuga Waa Kigato Munuka (Mwabungu) Gazi Shiraoni Majoreni Mwangoye	Mwareni Gulanzi Shambini (Vigurungani) Matumbi Kinangoni Mackinnon Road Maji ya chumvi Silaloni Makamini
	Lamu	Lamu (11,831 Matondoni (1,722) Mokowe (1,387) Mpeketoni (867)	Witu	Hindi Manda bay Kizingitini	Kiunga Mkunumbi Siyu Majengo	Faza Pate Bargoni Mangai Mararani
Mombasa (665,018)	Mombasa					
	Taita	Voi (33,077) Wundanyi (6,930) Taveta (55,580) Mwatate (5,613)		Bura Station Maktau Nganga Msau Ghazi Rukanga Sugala Kimongo	Dembwa (Wusi) Kungu (Wesu, Ngaronyi) Bura (Mission) Mtugua Mwanda Warughu Mgambonyi Kiwa Kigombo, Ndemi	Kajire Bungula Ghala Ziwani Mazera (Nghonji) Kilometa Saba Kiwaiwa Eldoro Kijabe Timbila

	Tana River	Hola (10,543) Garsen (4,977)		Kipini Bura Masalani Matambala Mororo	Masabubu Saka Karokora Sankuri Wanja	Tarasa/Ngao Nanighi Mnazini Maxora
	Malindi	Malindi (118,428) Mamburui (1,658)				

EASTERN PROVINCE

Principal towns	Admin. District	Urban centres	Rural centres	Market centres		Local centres		
Embu (52,446)	Embu	Runyenjes (58,223)	Manata (Kairuri) Ishiara Kiritia	Karingari (Nembure) Kianjokoma Kavoe Ena (Kangethia)	Kathanjure Karurumo Kanyambora Gachoka Karaba	Kibugu Kiamuringa Musonoka Mbebori Kanja Mukuuri Kyani Mulu	Gikuuri Kathunjuri Kigumo Kangunu (Gichicho) Ugweri Keria	Kiambera Mavuria Ngandura Gathomu Riakanau Makutano Kanguru
	Isiolo	Isiolo (32,684) Merti (3,446) Garba Tula (1,991) Kinna (4,122)	Modo Gashe Garba Tula	Maai		Saricho Ol Doinyo Nyiro	Kinna Bulossa	Kula mawe
	Kitui	Kitui (106,873) Mutomo (1,674) Tulia (631) Mwingi (2,000)	Ndoa (Mutito) Kyoso Kisasi Matinyani Ikutha Migwani	Mutune Kalulani Mulango Nuu Mui Zambe Kanziko Voo Matha	Kabati Katsa Tselkuru Walta Ngomeni Miambani Chuluni Endau	Kathibo Syongila Kakumuti Musebe Yatta Tiva Kavisuni Nguni Ikanga Mbitini	Enziu Mwitika Inyui Kyatuna Kyoani Muthue Mosa Wikiilya Ithookwe	Nseluni Thitani Tharaka Kimangau Muvokoni Usueni Kamugwangu Mathunyami Kwa-vonza
	Machakos	Machakos (143,274) Kangundo/Tala (179,449) Mavoko (Athi River) (27,168)	Masii Kithimani (Yatta) Uaani/Tawa Mitaboni Kileme (Nunguni) Mbooni (Kikima)	Kaani Kimulwa Mitituni Muthethaai Wamunyu Kivaani Matuu Mumandu Kalawa	Kimutwa Ekarakara Kalangi Kangondi Mbiuni Matheeni Kathiani Kawathei Kaviani Nzaini Konza Mutula Upete	Kibandhini Mutondoni Kiatineni Kamuthangu Ngelani Kivumbu Lema Kinyata Ithaeni Kaloleni Vyale Nguluni Kianzabe Mwaani (Mukaa)	Kanzulu Mutondoni Kiitwii Ikombe Nzukini Myanani Makutano Kwandoo Kitandi Kivani Kyangata Iluni (Katuaa) Kali Utangwa Ngolenii Kaiani Kusyomuomo Makaveti Miu Kithioko Kabaa Mikuyuni Daajani Kaloleni	Kyamua Kitwil Kailli Kilili Thabu Maiara Okia Kavumbo Mutembuku Kihungo Ndauni Mwanyani Kithumain Wathini Muyambua Kathama Ithetani Kambuu Mbuani Koma Rock Miondoni Kithayoni Muisuni
	Mwingi	Mwingi (66,212) Migwani (462)						
	Makueni	Wote (58,195) Makindu (6,226) Sultan Hamud (4,452) Kibwezi (3,824) Emali (3,649) Kikina (1,871) Machinery (1,581)	Mwata/ Siadhani	Kikoko Kila Nziu Katulani Mbumbuni Matiliku Kasikeu Maiyani	Kathonzweni Mavindini Kikumini Ngwata	Kiongwani Masongaleni Syumila Katangini Masumba Kiu Kako		

		Mbumbuni (1,258) Talw (693) Mtito Andei (53,745)		Makindu Kilala Kiboko/ Ngonyo Sabuk		Ilani Enzai		
	Marsabit	Marsabit (11,982) Sololo (2,541) North Horr (2,401) Kargi (1,064)	Moyale	Laisamis		Loiengalani Ileret Maikona	Kalacha	Illaot Logo Logo
Meru (65,700)	Meru Central	Nkubu (7,725) Chogoria (2,000) Mitunguu (3,724) Timau (1,564) Chuka (2,000) Maua (2,000)	Igoji Kanekine Marima Muthamba Mariamanta Miathene Kangeta Late	Thimagiri Timau Kiirua Githengo Kibirichia Mujwa (Imenti) Gaitu Mariene Miguriri Mikumbuna Kibugw Kiriani	Mukuuni Mitheru Kaanwa Chakanyinga Chiokarige Gathunga Kianjai Muthara Mikinduri Kiguchwa Gatumune Muluati	Kienduru Kathurina Ngenyi Nthimbiri Mpuri Katheri Kaongo Kithirune Thege Giaki Ruiri Kiangua Kionyo Keiemana Kathara Nyandene Keringa Itugururu Ikuu Mwiria Keria	Mariani Kiereni Chera Weru Momboni Karimba Makatuni Kanjuki Iriga Kanjora Tunyai Kalangachini Kanuti Karama Ruitu Mbaranga Anchenge Kiengu Milu Tatu Atheru Ruujine Theera Kaongo Kamau	
	Meru North (Nyambene)	Maua (15,475) Lare (2,141)						
	Meru South	Chuka (7,271)						
	Mbeere	Siakago (3,230)						

NORTH EASTERN PROVINCE

	Garissa	Garissa (69,203) Dadaab (4,580) Liboi (2,540) Mundo Gacha (4,148) Anuna (1,971		Mulugho	Balambala Bura Masalani	A;omkigir Banana Wardeglo Hara	Sankuri Korokora Masabubu Saka	
	Ijara	Ijara (1,265)						
	Mandera	Mandera (43,916)		Asahaba Malka Mari	Arabia Takaba	Asabito Kalaliyo El Roba Banisa Harere Gololibia Ramo Dimtu	Simmbirr Faduma Gadeiff Wangai Dahan Bur Manisa Nur Mansa Nur Mansa Omar Jilo Figho Dandu Koba Arba Fakat	
	Wajir	Wakor (44,272) Buta (2,808) Eldas (2,575) Buna (1,709)	Habasweom	Grillu Bura	Tarbao	Gurar Kulaley Ajao Banana	Elben Knorot Harar Korondile Wajir Bot	

		El Wak (11,128) Rhamu (7,437)			Batalu Luhalay
	Moyale	Moyale 924,874)			

NYANZA PROVINCE

	Kisii	Keroka (40,328) Manga (2,000) Kisii (65,235) Suneka (31,739) Masimba (21,219) Ogembo (2,000)	Nyambunwa Kabirigo Kaumbu Nyamacha Kenyanya Nyamarambe	Myamaia Gesima Mogonga Gesusu Igara Nyanturago Magomgo Magwagwa Ikonge Nyangusu Tina Riosiri Kiamorkama Marani Ramasha Magencha Etago Mosocho Birongo Chepnyalil Nyamira	Monianku Miruka (Riochanda) Obwari Tombe Riana Ikoba Mukomani Itumbe Matonto Kegogi Maroo Pala Matongo Karota Nyarambe Iseche Rioma	Nyakoa Matongo Kamagambo Kiogoro Makairo Kenyenya Musobeti Rigoma Nyamasibi Nyacheki Magema Marabu Nyamesege Kerongo Kegeti Chengombei Matutu Mogumo
	North Kisii (Nyamira)	Nyamira (100,083) Nyasiongo (32,305)				
	Migori	Migori (95,446) Awendo (90,153) Rongo (79,817) Sorr (4,339)				
	Rachuonyo	Oyugis (65,894) Kendu Bay (22,048)				
	Homa Bay	Homa Bay (56,297) Ndhiwa (1,498)				
	Suba	Mbita (46,223)				
	Nyando	Muhoroni (35,145) Ahero (30,327) Chemelil (6,864) Katito (4,042) Koru (992) Kabongo Oiyier (201)				
	Bondo	Bondo (29,202) Usenge (5,771)				
	Suba	Sindo (1,637) Wadiaga (156)				
	Kuria	Kehancha (151,465)				
Kisumu (322,734)	Kisumu	Ahero (2,000) Maseno (2,900)	Miwani Chemelil	Chiga Kibigori	Kibos Rabula	Nyabondo Makindu

			Sondu Muhoroni Kambawa Kiboswa	Rabuor Awach Kusa Paponditi Awasi Kisian Pawakucha Awach Oronglo Daraja Mbili		)Ombeyi) God Abuoro Songhor Masogo Ambaka Nyamasaria Karowa Onjiko (Otha) Kiboko Onyungo Nyamarimba Sigeti Bodi Nyanganda Dajaja Mbili	(Kowawa) Koru Tamu Kipasi Kaloka Magwar Reru Akaco Bodi Kondik Wath Orego Ulalo Nyag Bondo
	Siaya	Ukwala (1,768) Yala (40,125) Siaya (41,162) Ugunja (29,513)	Bondo Asembo Rangala Nyangweso Ngiya	Boro Nderi Usengi Wagusu Aram Madiani Saga Sigomere Sindindi Akala Ndori		Randago Kadenge Urenga Mwer Nyalgunga Nyapiedho (Maranda) Wangarot Anguongi Usigu Nyamonye God Kapolo (Nyagoma) Nyilima Lwak Ragengani Manywanda Ndigwa	Uwai Siluo Bondo Jera Umina Nzoia Mbosia Madhiero Malanga Muhanda Sianga Ramula Kudho Nyagongo Sirembe Kambare Nyang'oma Ajigo
	S. Nyanza		Mbita Kihancha Karungu Macaler Mine Sara	Omoya Lwanda Ogongo Homa Lime Kadel Doho Kosaie Kabondo Ober Samba Orinda Miriu Gucha Ranon Rangwe Rodi Kpany	Isibania Taranganya Mariwa Rapogi Uriri Mugoro Mohoru Magunga Aora Chuodho Ratanga Kagaga Mawego	Lieta (Kabunde) Ngegu Nyangweso Ndiru Awach Tende Kamasengra Sena Ukula Kwoyo Obando North Huri Sidede Kandiege Kanam Omboga Wagwe Otaro Nyahera Rioma Kwoyo Oyombe Nyabisawa Masaba NyangoreKie rege Kurliange Nyairtiro Intimaru Ndonyo Kagega Oyoni Osogo Gogo Mikei	Gendia Ramula Karota Mambolea Pala Ringa Nyamburu Gangre (Sino) Oring Rakwaro Nyambija Rakwaro Odienya Opaya Dede Asumbi Nyawi;a Oboke Imbo Ongeng Magina Kakrao God Jope Okenge Bondo Nyironge God Kwer Othoch Rakuom Wath Onger Lwanda Migwar Nyandhiwa Kiabuya Agolo Muok Otati Osani Mirogi Rapedhi

RIFT VALLEY PROVINCE

Principal towns	Admin. District	Urban centres	Rural centres	Market centres	Local centres		
	Baringo	Kabarnet (16,931) Marigat (4,738) Kampi ya Samaki (728) Tangulbei (586) Karbartonjo (384)		Kituro Tenges Kinyang Emening	Kabluk Kapturwo Saretunin Pemwai Kamungei Kipcherere Bariolimo Poi Bartabwa Timborwo Mogorwo	Kamimoi Sinonin Sabatia Saos Esageri Torongo Sirwa Makutano Radad Loiminnage Loruk Katwa	Tangulbet Churo Kipedo Mugurin Kisanana Olkokwa Maji ya moto Loboi Sagut Noiwel
	Keiyo	Tambach/Item (22,710) Kimwerer (965) Kanwosor (621) Cheserek (288)	Chepkoiro Chebiemit Kapsowar	Kaptarakwa Tot Chebororwa Kapcherop	Cheptongei Chebara Chesegon Chesongoch Koitilial Kaptalamwa kipsoen	Kamorin Bugar Seroit Biretwo Kessup Kapieren Nyeru	Turesia Kipsaos Tumeiyo Musgut Chebloch Chesoi
	Kajiado	Kajiado (9,165) Ngong (20,701) Kitengela (9,327) Namanga (6,205) Loitokitok (6,150) Rombo (1,885)	Magadi	Il Bisil Mashuru Kiserian	Nonkoopirr Elangata waus Isinya Oliosetti Ngata taak Salengai Lengesim Rongai Kimana Lassit	Oltepis Mosiro Ol Kilamattan Bulbul Waso Kadong Malasia Ongata Rongai	
	Kericho	Kericho (93,213) Londiani (34,187) Kipkelion (26,786)	Sosiot Rorat Longisa Lumbwa Kaboson	Kaitui Isondo Kapkatet Kapkoros Chemosit Ndanai Tenwek Chebuno Sigor Kedowa Fort Ternan	Ainamoi Kimutot Kapkugerwet Chpsir Kapsuser Kipchimchim Kapsoit Kiptera Kapsarok Kipsitet Cheborge	Mogogosiek Kabenet Loiwa Ainabkoi Kabianga Sotik North Kaitet Kabkuress Keseagetiet Ng'oina	Chepelelwa Gorgor Gelegele Siongiroi Moyet Tarakwa Merigi Kapkimoiwa Mulot Sorget
	Laikipia	Nyanyuki (49,330) Doldol (695)	Rumuruti	Ngarua	Lumuria Umanda Kimanjo Nyahururu Murara	Sosian South Marmanet Mithigia Noprth Marmanet	
Nakuru (231,262)	Nakuru	Njoro (16,510) Molo (96,156) Naivasha (158,678) Elburgon (23,173) Gilgil (20,362) Dundori (6,056) Mau Narok (3,344) Salgaa (875) Subukia (1,905) Rongai (1,499) Olenguruone (1,362) Keringet (1,007)	Mau Narok Elburgon Gilgil	Bahari Solai Subukia Rongai Olenguorone Longonot Kijabe	Mabruk Elementaita Banita Nyamamithi Kabasi Kandutura Kampi ya Moto Siape Kamwaura	Mgwatamorp Kibunja Turi Mausummit Molo South Ikumbi Kerisoi	Ambuskat Morendat North Karati Maraigushu Kongoni Kariandus Karunga Eburru Mount Margaret (Mai Mahiu)
	Nandi	Kapsabet (48,729) Nosoriot (1,507) Kabiyet (1,015)	Kabiyet Lessos Kaptumo	Cheptarit Kaiboi Kipkarren Kaigat Chipterwai Kilibwoni Mugondoi	Chepsonol Chepkumia Baraton Chepterit Kapsisywa Kipsigek Kosiral	Sangato Gurgung Ndalat Kablemit Kipyesi Chemnowet Arwos	Kamngoriam Ndutia Kemeloi Chekunan Maraba

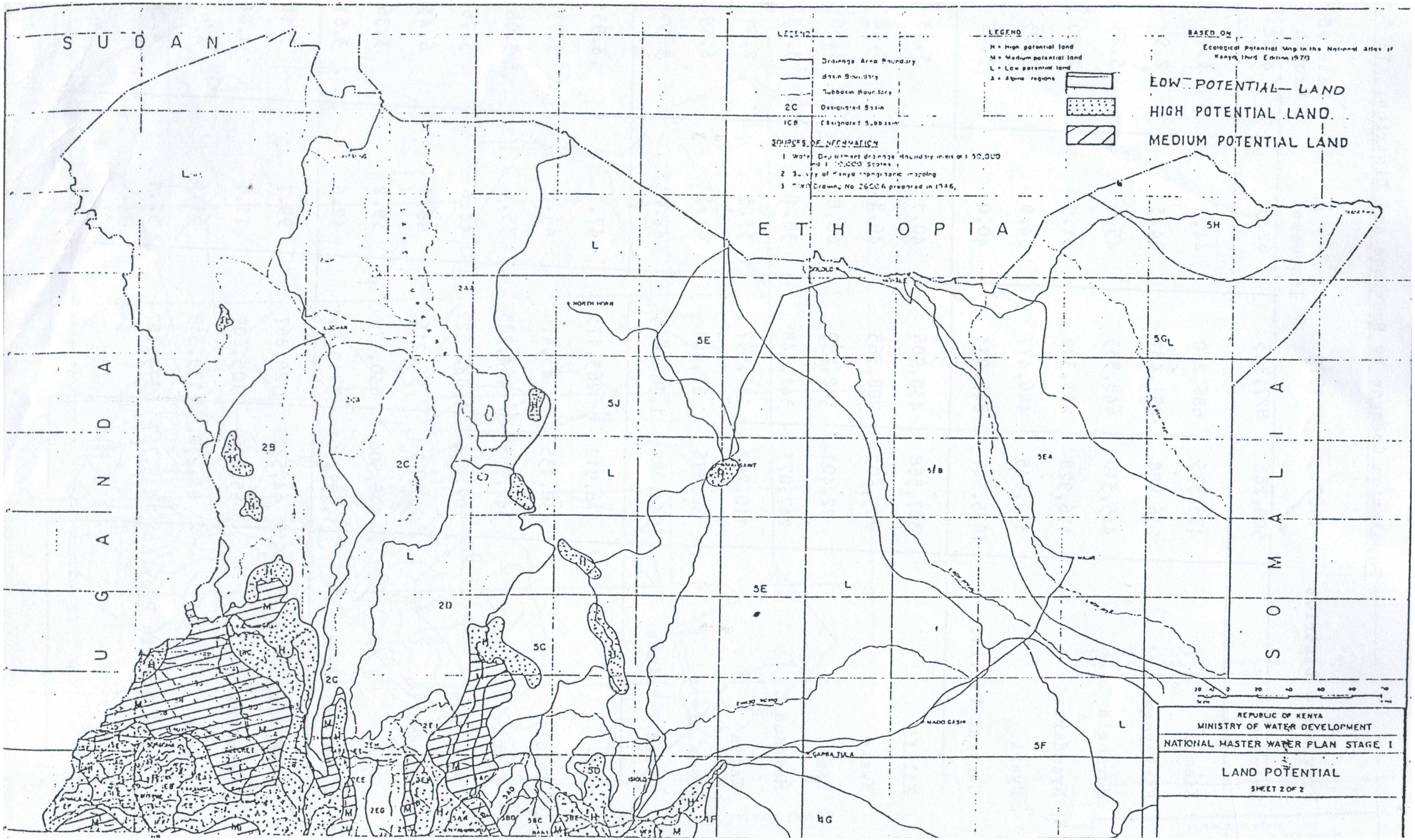
				Chemmase Kobujoi Serem	Mutwol Birbirlet Lelmokwo Kabisaga Lolkeringet	Chepkunyuk	
	Narok	Narok (41,162) Nairegie (1,970) Laisamis (1,938) Olukurto (869) Suguta (1,369)		Enabelbel Narosura Mulot Lolgorien	Siyapa Siyabel Olchoro Sakutiek Lidamat Olulunga Lemak Altong	Mosiro Rotich Enangipeti Olokurto Morijo Naikarra Maguarra	Entasekera Emaril Shartuka Enosaan Angata Baragoi Keekorok
	Samburu	Maralal (24,502) Kisima (580) Baragoi (4,345) Archers post (3,966) Wamba (3,950)		South Horr	Opiroi Losuk Barsaloi Kisima	Kirimon Marii Illaut Nguronit	Tum Lodongokwe Serolevi
Kitale (86,282)	Trans Nzoia		Endebess Kiminini	Kwanza Saboti Cherangani Suwerwa	Suam Kimothon Kimothon Chepchoina Kiptagat Sikhendu	Kapretwa Kisawai Teldet Kipsoen Kibuswa	Makutano Nzoia Kapsara Sirende
	Turkana	Lodwar (34,904) Lokitaung (1,617) Katilu (406) Turkwel (392) Kapendo (46) Lokichogio (13,728) Kakupia (9,107)	Kalokol Lokitaung Lokori	Kakuma Lorekumu Kaputir	Eliye Kerio Kataboi Oropoi	Loiya Khaekongole Lokichar Lowerangak Nachakwi	Todenyang Kating Lomelo Lochakula
Eldoret (197,449)	Uasin Gishu	Timboroa (3,038) Burnt Forest (30,166) Turbo (2,701) Olessos (796) Soy (523) Kipkaren (145) Moi's Bridge (21,844) Matunda (11,214)	Kipkabus Turbo Moiben Moi's Bridge	Soy Kaptagat Ainabkoi West	Leseru Kapsaret Elgeyo Border Plateau Panon	Kiboloss Chepsaita Ngenyilel Osorongai Tatsagoi town Ziwa	Matunda Ainabkoi East Nabkoi Cengalo Kipkurere Kaptabei
	West Pokot	Kapenguria/ Makutano (48,730) Cheparenia (18,470)	Kacheliba (Kongelai) Sigor	Kaibibich Kanyarkwat Kanyao Nyapong Ortum	Morkwijit Kisaunet Serewa Kerenget Chepkono Kapsait Kapsangar	Kanyerus Koidich Nakuyen Kasei Kowriong Chepnyai Ploiyo Chepkobei	Kapchemogen Chesegon Lomut Lolongot Tankal Sebit Wakor
	Bomet	Bomet (84,320) Sotik (8,042) Litein (2,637)					
	Nandi	Nandi Hills (77,574)					
	Koibatek	Eldama Ravine (32,192) Maji mazuri (4,635) Mogotio (3,816) Emining (449)					
	Trans Mara	Kilgoris (2,478) Nkararo (1,296) Lolgorian (1,119)					
	Buret	Chebitat (1,142)					

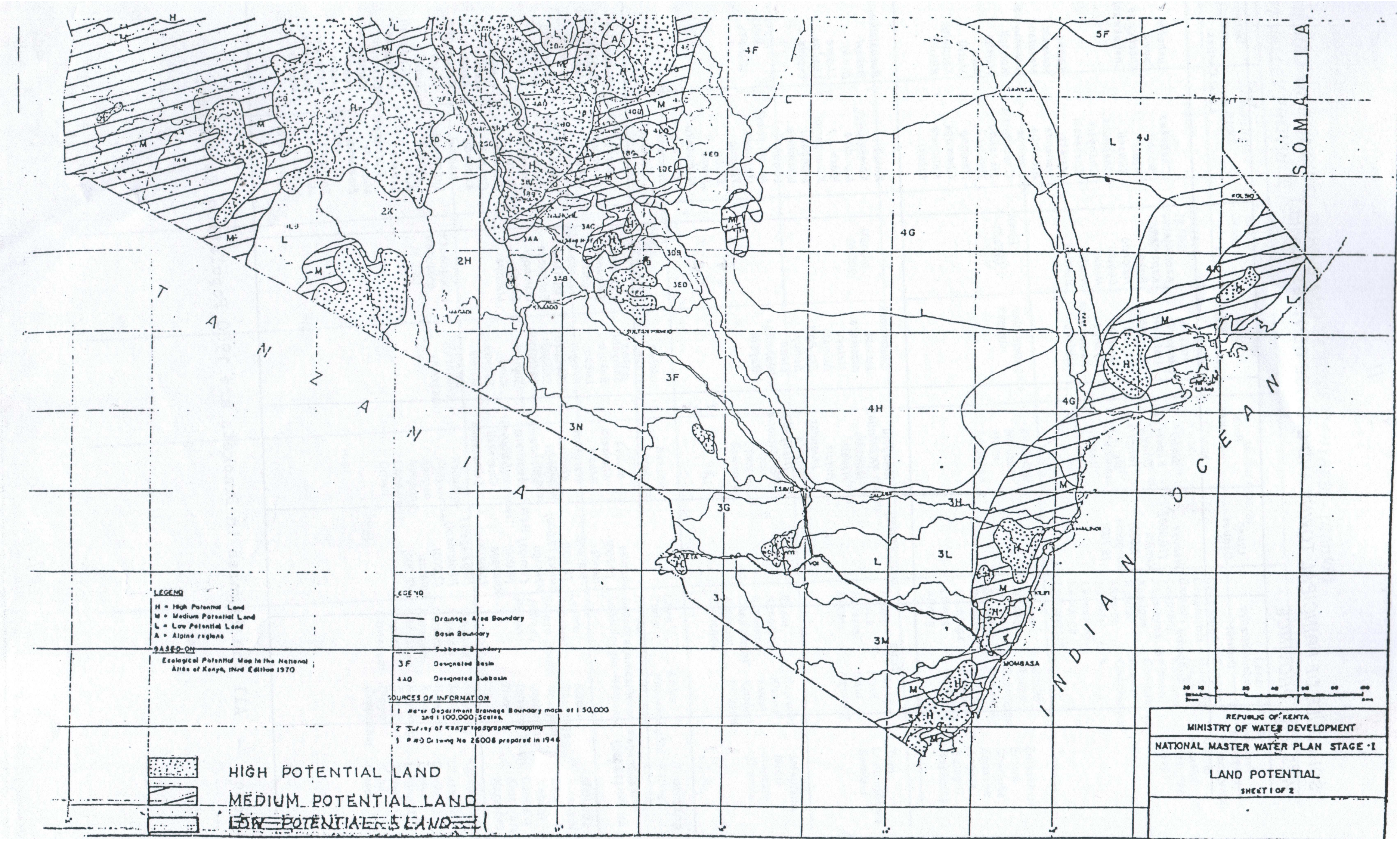
WESTERN PROVINCE

	Bungoma	Webuye (70,137) Kimilili (71,461) Bungoma (73,048) Sirisia (23,520) Malakasii (12,102) Chwele (3,018) Misikhu (1,495)	Kapsakwany Tongareni	Mayanja Makatero Sangalo Kabula Nyanga Kamakoiwa	Kaptema Kapkateny Bokoli Nalondo Lugulu Naitiri Ndalu Bumula	Chebukwa Kibabil Nzoia Kimaeti Chepkube Changara Maleka Nalianda Sibembe Butonge Chesamisi Sikhendu Ndivisi Namotio Chemogu Kaboywa Sikusi	Lukhone Kuywa Khachonge Chebukwabi Sitikho Mbakalo Makutano Makukuywa Kongoli Majaha Milo Muchi Kandunyi Kibuke
	Busia	Busian (44,211) Port Victoria (23,009) Bumala (19,791)	Nambale Hakati Butula Muandas Fungula (Nyangina)	Chelelemuk Mataba Amukura Matayo Sio Port Buhuyi Likoli Murumba Buyolu		Kolait Kolanya Mundika Igara Kwangamor Mungatsi Luamwa Lugulu Alupe Mabunge Tingolo Siribo Jairos Namuduru Machakus Chamasiri	Bukhalatire Bakayi Bukiri Lugara Luanda Lupida
Kakamega (74,115)	Kakamga	Malava (24,248)	Khwisero Navakholo Kiboswa Shianda Ebusiralsi (Esibuye) Khayege (Mukumu) (Sigalagala) (Ikolomani	Shinyalu Bukura Lubao Likmgili Koyonzo Shikulu (Musoli) Soy Matere	Kakunga (Mwanza) Indoise Magada (Muganda) Eshibinga Eregi Malinya	Mahanga Ekambuli Ilungu Jeprok Kinu Banja Senenda Kiritu Chamakanga Cheptik Kagubdu (Ishiru) Litambitsa (Igunu) Sabatia (Buchenya) Manyula Ematundu (Wambulishi) Shiatsala Musanda Virembe (Isechero)	Bungasiwa Samaki Munami Matungu Makunga Eshisiru Matana Muregu Lukumo Samitsi Kambiri Chimangeti Butali (Chebwai) Chimuche Luandeti Nambirima Mabusi Likhuyani Mauwama Mbaraka.
	Mt. Elgon	Kapsokwony (5,687) Cheptais (3,675)				Chesakaki	
	Vihiga	Vihiga (109,508) Luanda (66,289)	Kaimmosi Hamisi Maseno	Sabati Serem Gambogi			
	Lugari	Lumakanda (11,386) Lugari (2,769)		Soy			
	Teso	Malaba (45,637)	Amagoro				
	Butere/ Mumias	Butere (9,771) Mumias					

Note: - All the figures in Brackets are 1999 population estimates

A.4: LAND POTENTIAL




REPUBLIC OF KENYA
MINISTRY OF WATER DEVELOPMENT
NATIONAL MASTER WATER PLAN STAGE ·I
LAND POTENTIAL
SHEET 1 OF 2
LEGEND
H = High Potential Land
M = Medium Potential Land
L = Low Potential Land
A = Alpine regions
BASED ON
Ecological Potential Map in the National Atlas of Kenya, third Edition 1970
LEGEND
Drainage Area Boundary
Basin Boundary
Subbasin Boundary
3F Designated Basin
4A0 Designated Subbasin
SOURCES OF INFORMATION
1 Water Department Drainage Boundary maps at 1:50,000 and 1:100,000 Scales.
2 Survey of Kenya topographic mapping
HIGH POTENTIAL LAND
MEDIUM POTENTIAL LAND
TANZANIA
INDIAN OCEAN
SOMALIA
MOMBASA
TSAVO
3F
3N
3G
3J
3M
3L
3H
4H
4G
4J
4F
5F
2H
2K
3AA
3AC
3ED

A.5: POPULATION GROWTH RATES IN THE DISTRICTS 1979 TO 1999

Province / District	1979 (000's)	1989 (000's)	1999 (000's)	Rate of growth % per annum 1979 to 1989	Rate of growth % per annum 1989 to 1999
Nairobi	828	1,325	2,137	6.00	6.13
Kiambu**	686	914	742	3.32	5.20
Thika***			647		5.20
Kirinyaga	291	392	455	3.47	1.61
Murang'a** +	648	858	351	3.24	2.01
Maragua*** +			389		2.01
Nyandarua**	233	345	466	4.81	3.51
Olkalao***					3.51
Nyeri	486	607	655	2.49	0.79
Central Province	2,344	3,116	3,705	3.29	1.89
Kilifi +	431	592	555	3.74	4.14
Malindi			282		4.14
Kwale	288	383	497	3.30	2.98
Lamu	42	57	73	3.57	2.81
Mombasa	341	462	653	3.55	4.13
Taita	148	207	248	3.99	1.98
Tana River	92	128	183	3.91	4.30
Coast Province	1,342	1,829	2,491	3.63	3.62
Embu +	263	370	277	4.07	0.72
Isiolo	43	70	101	6.28	4.43
Kitui** +	464	653	517	4.07	0.72
Mwingi*** +			303		0.72

Province / District	**1979 (000's)**	**1989 (000's)**	**1999 (000's)**	**Rate of growth % per annum 1979 to 1989**	**Rate of growth % per annum 1989 to 1999**
Machakos** +	1,023	1,402	915	3.70	2.00
Makueni*** +			767		2.00
Marsabit** +	96	129	122	3.44	3.64
Moyale*** +			54		3.64
Meru	830	1,145		3.80	
Mbeere*** +			173		3.86
Meru Central*** +			500		3.86
Meru North*** +			608		3.86
Tharaka*** +			101		3.86
Nithi*** +			205		3.86
Eastern Province	2,719	3,769	4,643	3.86	2.32
Garissa	129	125	390	-0.31	21.20
Mandera	106	124	250	1.70	10.16
Wajir	139	123	321	-1.15	16.10
North Easten Province	374	372	961	-0.05	15.83
Kisii*	870	1,137		3.07	
Kisii Central*** +			487		2.76
Gucha*** +			464		2.76
Kisii North*** +			500		2.76
Kisumu** +	482	664	500	3.78	2.00
Nyando*** +			297		2.00
Siaya* +	475	639	480	3.45	1.25
Bondo*** +			239		1.25

Province / District	**1979 (000's)**	**1989 (000's)**	**1999 (000's)**	**Rate of growth % per annum 1979 to 1989**	**Rate of growth % per annum 1989 to 1999**
South Nyanza*	818	1,067		3.04	
Migori*** +			517		3.35
Kuria*** +			152		3.35
Homa Bay*** +			291		3.35
Rachuonyo*** +			308		3.35
Suba*** +			156		3.35
Nyanza Province	2,645	3,507	4,391	3.26	2.52
Baringo** +	204	348	265	7.06	1.58
Koibatek*** +			138		1.58
Elgeyo Marakwet*	149	216		4.50	
Keiyo*** +			142		3.10
Marakwet*** +			141		3.10
Kajiado +	149	259	405	7.38	1.08
Kericho** +	633	901	472	4.23	4.93
Bomet*** +			384		4.93
Trans Mara*** +			172		4.93
Buret*** +			317		4.93
Laikipia****	135	219	323	6.22	4.75
Nakuru	523	849	1,197	6.23	4.10
Narok +	210	434	363	10.67	1.08
Nandi	299	398	582	3.31	4.62
Samburu	77	109	142	4.16	3.03
Trans-Nzoia	260	394	575	5.15	4.59

Province / District	1979 (000's)	1989 (000's)	1999 (000's)	Rate of growth % per annum 1979 to 1989	Rate of growth % per annum 1989 to 1999
Turkana	143	184	447	2.87	14.29
Uasin Gichu	301	446	617	4.82	3.83
West Pokot	159	225	309	4.15	3.73
Rift Valley Province	3,242	4,982	6,991	5.37	4.03
Bungoma** +	504	679	878	3.47	4.92
Mt. Elgon*** +			135		4.92
Busia** +	298	402	371	3.49	3.48
Teso*** +			171		3.48
Kakamega** +	1,031	1,464	605	4.20	2.29
Butere/Mumias*** +			478		2.29
Lugari*** +			217		2.29
Vihiga*** +			499		2.29
Western Province	1,833	2,545	3,354	3.88	3.18
Total Kenya	**15,327**	**21,445**	**28,673**	**3.99**	**3.37**

NOTES:

* Districts subdivided after 1989 and name ceased to exist

** Districts subdivided after 1989 but old name retained

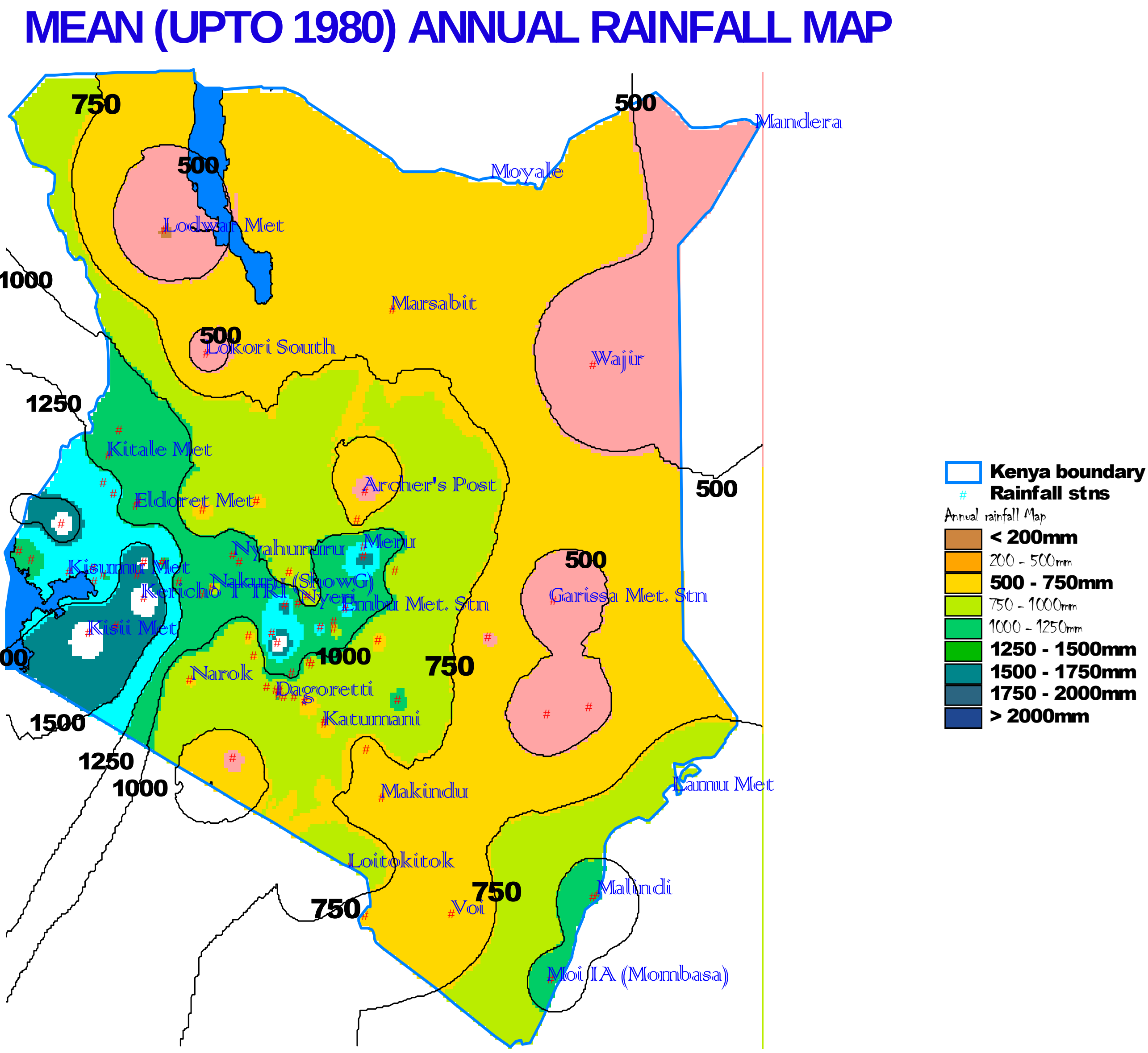
*** Districts created after 1989

*** Part of District shifted elsewhere

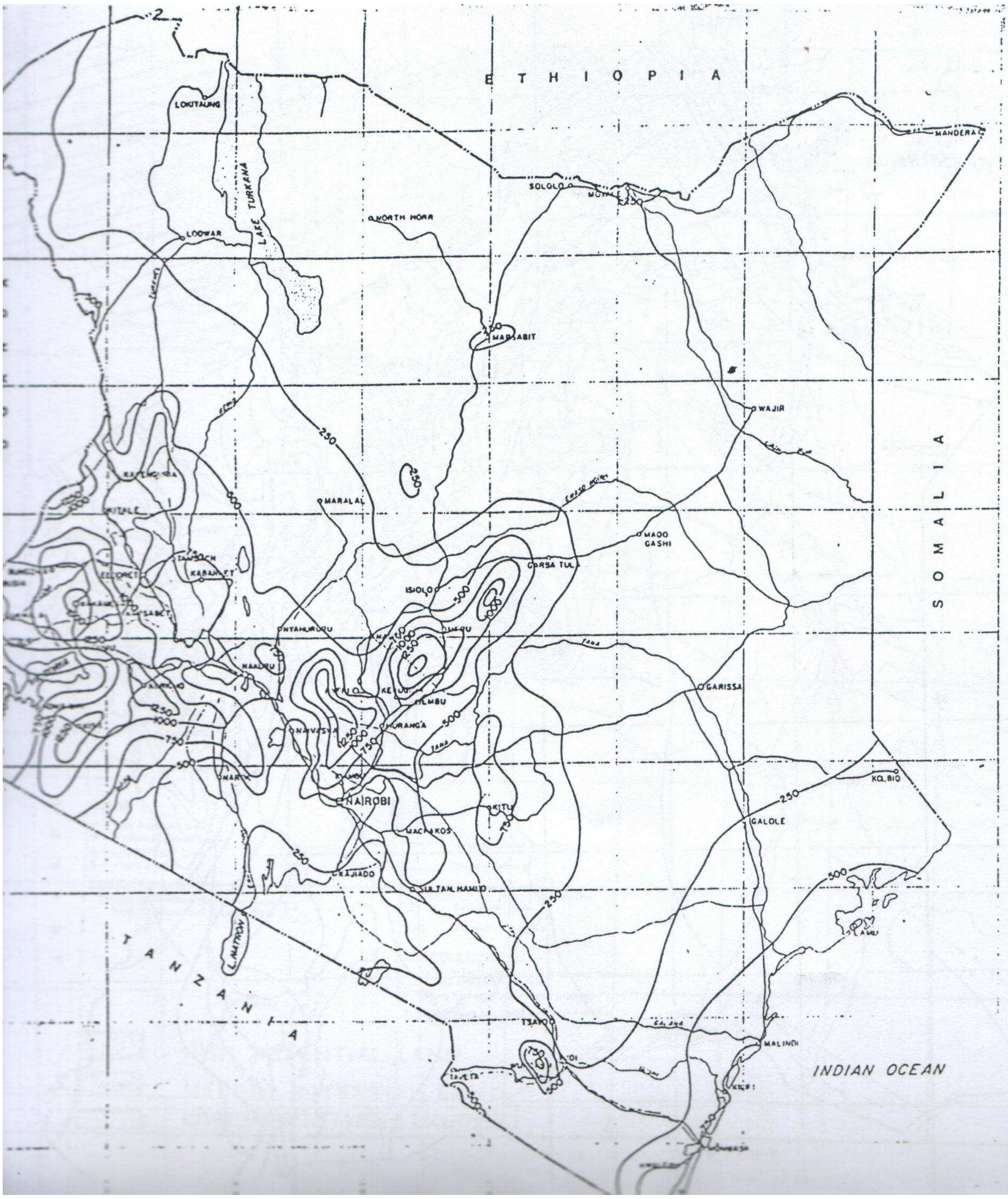
\+ Growth rates for 1989-1999 based on figures prior to sub-divisions of 1989.

APPENDIX B:

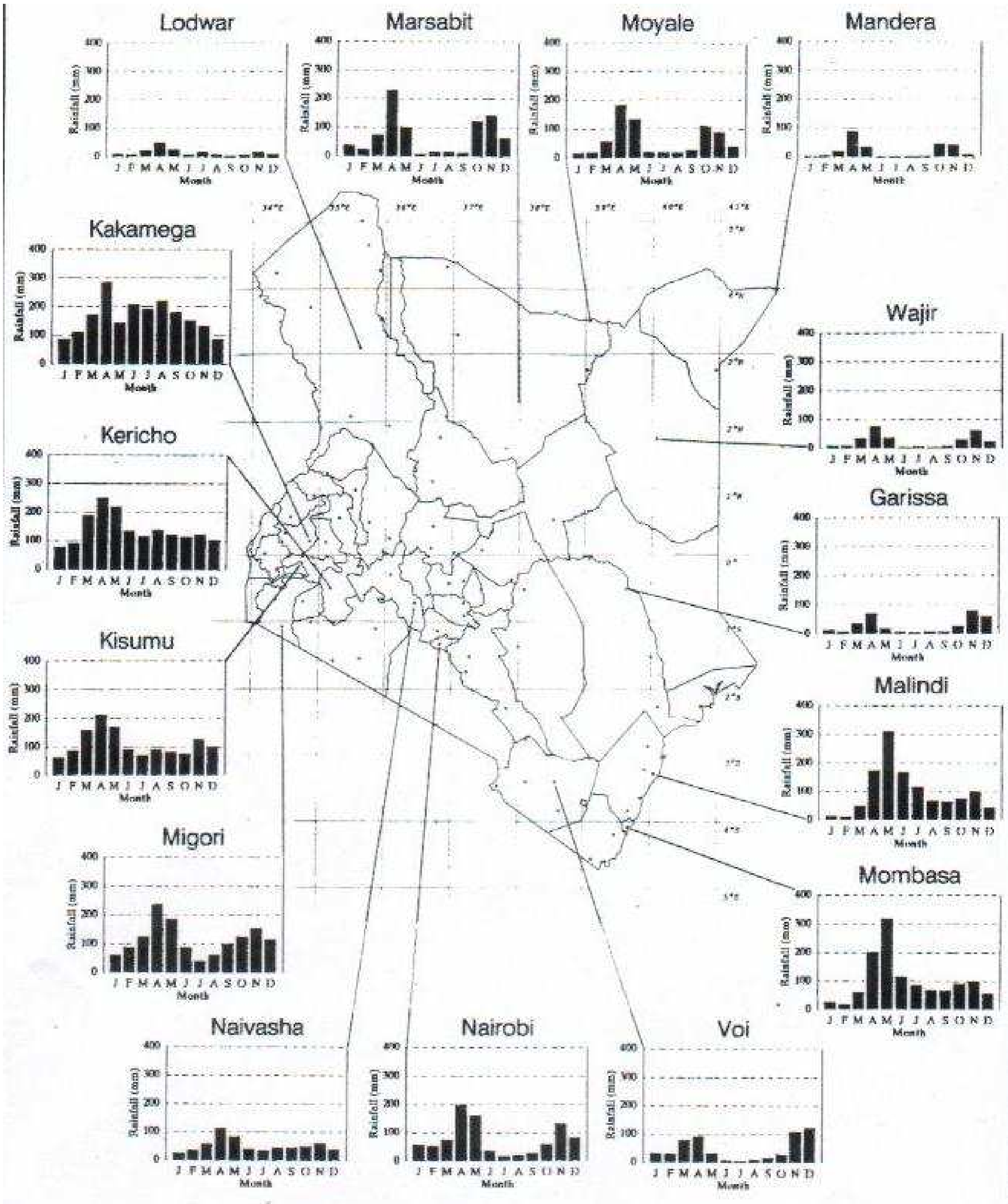
B.1:



B.2: 90% PROBABILITYMAPOF ANNUAL RAINFALL IN KENYA



B.3: MONTH-TO-MONTH VARIATIONS OF RAINFALL IN KENYA



APPENDIX C:

C.1: TOOLS AND LABORATORY EQUIPMENT

<u>ITEMS WITH NO RELEVANCE TO THE PARTICULAR SCHEME SHOULD BE DELETED</u>

ITEM	DESCRIPTION OF ITEMS			QUANTITY
	Tools			
1.	Pipewrench	48” for pipesize	25-125 mm	2 No.
2.	Pipewrench	36” for pipesize	8-80 mm	2 No.
3.	Pipewrench	24” for pipesize	8-65mm	2 No.
4.	Pipewrench	18” for pipesize	8-50 mm	2 No.
5.	Pipewrench	14” for pipesize	8-40 mm	4 No.
6.	Pipewrench	12” for pipesize	8-40 mm	4 No.
7.	Pipewrench	10” for pipesize		4 No.
8.	Chain Pipewrench	48” for pipesize	40-200 mm	2 No.
9.	Chain Pipewrench	36” for pipesize	50-150 mm	2 No.
10.	Adjustable screw spanner	18” for pipesize		2 No.
11.	Adjustable screw spanner	12” for pipesize		2 No.
12.	Adjustable screw spanner	8” for pipesize		2 No.
13.	Adjustable screw spanner	4” for pipesize		2 No.
14.	Open spanner ½”–2” (14 pos per set)			2 sets
15.	L and Key set			1 set
16.	Plier Combination			1 No.
17.	Poly grip			1 No.
18.	Stock & Dies	15-50 mm		1 set
19.	Stock & Dies	65-100 mm		1 set
20.	Stock & Dies	100-150 mm		1 set
21.	Potable pipe vice	8-50 mm		1 set
22	Pipecutter			1 No.
23	Torches			1 No.
24	Brease hand drill *Up to ¾ holes)			1 No.
25	Cold chisel 200 mm			2 No.
26.	Punches			1 set
27	Crow bar			1 No.
28	Hack saw frame 12”			2 No.
29	Blades to –do- (plet of 100)			1 No.
30	Screw driver 9 x 200			2 No.
31	Screw driver 6 x 100			2 No.
32	Screw driver 4 x 75			2 No.
33	Screw driver 6 x 100 star type			2 No.
34	Medium coarse file 6” flat tyre			1 No.

ITEM	DESCRIPTION OF ITEMS	QUANTITY
35	Medium coarse file 8” flat tyre	1 No.
36	Medium coarse file 10” flat tyre	1 No.
37	Medium coarse file 6” round tyre	1 No.
38	Medium coarse file 8” round tyre	1 No.
39	Medium coarse file 10” round tyre	1 No.
40	Hammer engine ½ kg	1 No.
41	Hammer sledge 5 kgs	1 No.
42	Knife	1 No.
43	Axe	1 No.
44	Tool box 18”	1 No.
45	Hurrican lamp	1 No.
46	Pressure lamp	1 No.
47	Oil can 0.5 litre + 5 litres of oil	2 No.
48	Grease gun standard with 10 nipples and 10 refill catridges	1 No.
49	Bucket 10.0 litres	2 No.
50	Jerry can 10.0 litres	2 No.
51	Empty steel drum 200 litres	3 No.
52	Wall clock (battery)	1 No.
53	Stop watch	1 No.
54	Ladder light alloy 5m	1 No.
55	Tape measure 50 m	1 No.
56	Tape measure 3m	1 No.
57	First aid kit (10 persons) + box	1 No.
58	Bow saw + 2 extra blades 36”	1 No.
59	Wheel barrow heavy duty	1 No.
60	Shovel standard type	5 No.
61	Pick with handle	5 No.
62	Garden rake with handle	2 No.
63	Panga	2 No.
64	Grass slasher	2 No.
65	Jembe with handle	4 No.
66	Forkjembe with handle	4 No.
67	Karai	4 No.
68	Broom	2 No.
69	Paint brush 50 mm	2 No.
70	Miscellaneious paint	
71	Diesel, 400 litres if necessary	
	Office Equipment	
72	1.5 x 0.9 mm office table with 6 No. drawers (3 lockable	1 No.
73	Office chairs	2 No.
	Laboratory Apparatus	
74	Beaker 600 cc plain (Borosillicate hard glass)	6 No.
75	Beaker 400 cc (Borosillicate hard glass)	6 No.

ITEM	DESCRIPTION OF ITEMS	QUANTITY
76	Burette 50 cc	4 No.
77	Winchester quart bottle (400 ml) for calcium stability test	4 No.
78	Graduated glass measuring cylinder 25-250 cc	4 No.
79	Conical flask 250 cc	6 No.
80	Plain glass funnel (for Whatman No.1 filter)	3 No.
81	Whatman's No.1 filter paper 185 mm (box of 100)	10 No.
82	Speciman jar cylindrical ground with glass stopper 500 cc capacity for alum and soda ash solutions (jar tests)	4 No.
83	Pipette graduated 0.100 to 1.0 cc	6 No.
84	25 cc pipette with one mark to deliver precise quantity	4 No.
85	Glass stirring rods min. 20 cm long	4 No.
86	Wooden double tripod stand for burette	2 No.
87	Comparator an dcolourmetric determination block type 2000 lovibond complete with indicator disc in leatherette	1 No.
88	Test tubes 10cc (for residual chlorine test)	12 No.
89 a	Disc Cresol Red 7.2 to 8.8 (for pH test)	1 No.
b	Disc Bromo Thymol Blue 6.0 to 7.6 (for pH test)	1 No.
c	Thymol Blue 8.0 to 9.0 (for pH test)	1 No.
d	Disc Chlorine 0.1 to 1.0 ppm (resid. cholor.test	1 No.
e	Disc Chlorine 0.2 to 2.0 ppm (resid. chlor. Test)	1 No.
90	D.P.D. Tablet (box of 100 No.1 for free residual chlorine	1 No.
91	D.P.D. Tablet (box of 1000 No.4 for total residual chlorine)	1 No.
92	Dropping bottles 60 cc	4 No.
93	Sulphuric acid N/50 in 0.25 litre bottle	10 No.
94	Counter scale Avery C/with weights measuring up to 15 kb	1 No.
95	Photographic scale complete with weights form 1 to 20g	1 No.
96	Turbidity rod	1 No.
97	Cresol red indicator solution 0.25 litre bottle	2 No.
98	Bromo Thymol blue solution in 0.25 litre bottle	2 No.
99	Thymol blue solution in 0.25 litre bottle	2 No.
100	Thymol red solution in 0.25 litre bottle	2 No.
101	Phenol phethelein solution in 0.25 litre bottle	2 No.
102	Distilled waer 5 litre container	1 No.
103	Fire extinguisher (CO_2) 5kg	1 No.

C.2: KENYA STANDARDS
(SOURCE: CATALOGUE OF KENIA SRANDARDS, 2005)

Building Industry

KS 02-17 : 1976	Specification for sawn timber
KS 02-18 : 1978	Specification for steels for building and construction.
KS 02-19 : 1976	Nomenclature for commercial Kenya timbers
KS 02-20 : 1976	Definitions and terminology for cements
KS 02-21 : 1976	Specificaitons for Portland cement
KS 02-22 : 1976	Specifications for hot rolled mild steel bars for reinforcement of concrete.
KS 02-93 : 1983	Glossary of terms used in timber
KS 02-95 : 1983	Specification for natural aggregates for concrete
KS 02-97 : 1982	Specification and methods for test for building limes, quick lime and hydrated lime.
KS 02-98 : 1982	Glossary fo terms used in lime and lime products
KS 02-99 : 1982	Glossary of terms relating to bituminous materials
KS 02-100 : 1982	Specification for asbestos-cement corrugated sheets for roofing and cladding
KS 02-101 : 1983	Methods for detainling reinforced concrete
KS 02-102 : 1983	Architectural and building drawings vocabulary
KS 01-103 1083	Specification for the use fo cold formed steel structural members
KS 02-104 : 1983	Specification for cold rolled steel sections.
KS 02-105 : 1982	Specification for hard drawn steel wire for reinforcement of concrete.
KS 02-106 : 1983	Standard bending dimensions and scheduling of steel bars for reinforcement of concrete
KS 02-108 : 1983	Glossary of termsfor concrete and reinforced concrete
KS 02-110 : 1983	Symbols for plumbing, water supply, heating, ventilating and ducting

Chemical process

KS 03-85 : 1979	Specifications for aluminium sulphate
KS 03-155 : 1978	Specifications for sodium carborate – Technical grade
KS 03-221 : 1981	Specification for lime for chemical industry

Electrical Engineering

KS 04-178 : 1980	Glossary of terms related to cables, conductors and accessories for electricity supply
KS 04-179 : 1978	Specifications for rigid PVC conduits and fittings for electrical wiring.
KS 04-182 : 1982	Specification for contactors
KS 04-183 : 1982	Specification for low-voltage fuses
KS 04.187 : 1980	Specificaiton for conductors and insulated cables

Mechanical Engineering

KS 06-02 : 1975	Part 1. Specification for galvanized plain steel sheets
1976	Part 2. Specification for galv. Corrugated steel sheets
KS 06-08 : 1976	SI units and recommendations for the use of their multiples and certain other units
KS 06-72 : 1978	Specification for nails
KS 06-149 : 1079	Specification for unplasticized uPVS pipes for cold water services.
1981	Part 2. Dimensions, nominal pressure (metric series)
1981	Part 3. Dimensions, nominal outside diameter and wall thickness (metric series)
KS 06-217 : 1981	Specification for unplasticized polyvinyl chloride (uPVC) pipes and fittings for buried drains and sewer pipes
KS 06-219 : 1981	Specification for hexagonal square bolts, screws, nuts, and lock nuts, metric and inch threads.
KS 06-259 : 1981	Specification for steel pipes for water and gas suitable for screwing
KS 06-320 : 1982	Specificaiton forPVC water hoses
KS 06-321 : 1981	Specification for low pressure rubber water hoses
KS 06-323 : 1982	Specification for ISO metric threads
KS 06-324 : 1982	Specification for high pressure rubber water hoses
KS 06-355 : 1982	Hot dip galv. Coatings on iron and steel articles
KS 06-356 : 1982	Specification for mild steel products
KS 06-430 : 1982	Specification for eddy current detection of defects.

C.3: ISO STANDARDS

Reference	Ed	Page	Title
TC 5 Ferrous metal pipes amd metallic fittings.			
			UDC 621 643 21 24 – 4
ISO 7/1-1982	2	4	Pipe threads where pressure-tight joints are made on the threads. Part 1 : Designation dimensions and tolerances.
ISO 7/2/1982		9	Pipe threads where pressure-tight joints are made on the threads – part 2 : Verification by means of limit gauges.
ISO 13-1978		33	Grey iron pipes, special castings and grey iron parts for pressure main lines.
ISO 49-1983		30	Malleable cast iron fittings thread to ISO 7/1
ISO 50-1977	2	1	Metal pipes – Steel sockets screwed according to ISO 7
ISO 65-1981	2	3	Carbon steel tubes suitable for screwing in accordance with ISO 7/1
ISO 221-1976		1	Steel tubes – wall thicknesses
ISO 228/1-1982	2	4	Pipe threads where pressure-tight joints are made on the threads – Part 1 : Designation, dimensions and tolerances.
ISO 228/2-1980		13	Pipe threads where pressure-tight joints are not made on the threads – Part 2: Verificaiton by means of limit gauges.
ISO 559-1977		8	Welded or seamless steel tubes for water, sewage and gas.
ISO 657/14-1982	2	9	Hot-rolled steel sections – Part 14: Hot-finished structural hollow sections – Dimensions and sectional properties.
ISO 1127-1980	2	3	Stainless steel tubes – Dimensions, tolerances and conventional masses per unit length.
ISO 1129-1980	2	4	Steel tubes for boilers, superheaters and heat exchangers – Dimensions, tolerances and conventional masses per unit length.
ISO 1179 - 1991	2	3	Pipe connnecitons, threaded to ISO 228/1, for plain end steel and other metal tubes in industrial applications.
ISO 2016-1981		8	Capillary solder fittings for copper tubes. Assembly dimensions and tests
ISO 2037-1980		2	Metal pipes and fittings – stainless steel tubes for the food industry
ISO 2084-1974		5	Pipelene flanges for general use – Metric series, Mating dimensions.
ISO 2441-1975		5	Pipeline flanges for general use – Shapes and dimensions of pressure-tight surfaces.

ISO 2531-1979	2	36	Ductile iron pipes, fittings and accessories for pressure pipe-lines
		4	Addendum 1-1980
ISO 2546-1973		2	Seamless plain end tubes made from unalloyed steel and without quality requirements.
ISO 2547-1973		2	Welding plain endtubes made form unalloyed steel and without quality requirements.
ISO 2851 - 1973		5	Metal pipes and fittings – Stainless steel bends and tees for the food industry.
ISO 2852-1974		10	Metal pipes and fittings – Stainless steel clamp liners with gaskets for the food industry
ISO 2853-1976		21	Metal pipes and fittings – Stainless steel screwed couplings for the food industry.
ISO 2937-1974		4	Plain end seamless steel tubes for mechanical application.
ISO 2938-1974		4	Hollow steel bars for machining
ISO 3304-1985	2	8	Plain end seamless precision steel tubes – technical conditions for delivery
ISO 3305-1985	2	7	Plain end welded precision steel tubes – Technical conditions for delivery.
ISO 3306-1995	2	7	Plain end as welded and sized precision steel tubes – Technical conditions for delivery.
ISO 3419-1981	2	17	Non-alloy and alloy steel butt-welding fittings
ISO 3545 - 1981		3	Steel tubes and tubular shaped accessories with circulr cross-section – Symbols to be used in specifications.
ISO 4019-1982		11	Cold-finished steel structural hollow sections – Dimensions and sectional properties.
ISO 4054-1980		13	Couples, loose spigots and base-plates for use in working scafforlds made of steel tubes. Requirements and test procedures
ISO 4144-1979		10	Stainless steel fittings bhreaded to ISO 7/1
ISO 4145-1979		12	Non-alloy steel fittings threaded to ISO 7/1
ISO 4179-1985	2	4	Ductile iron pies for pressure an dnon-pressure pipelines – Centrifugal cement mortar lining – General requirements
ISO 4200-1985	3	6	Plain end steel tubes, welded and seamless – General tables of dimensions and masses per unti length.
ISO 5251-1991		25	Stainless steel butt-welding fittings
ISO 5252-1977		3	Steel tubes – Tolerance systems
ISO 5256 - 1985		54	Steel pipes and fittings for uried or submerged pipe lines – External and internal coating by bitumen or coal far derived materials.
ISO 6594-1983		12	Cast iron damage pipes and fittings. Spigot series.
ISO 6600-1980		2	Ductile iron pipes. Cenrigugal cement mortar lining. Composition controls of freshly applied mortar.
ISO 6600-1980		2	Ductile iron pipes. Centrifugal cement mortar lining. Composition controls of freshly spplied mortar.
ISO 6708-1980		1	Pipe components. Definition of nominal size
ISO 6758 198		3	Welded steel tubes for heat exchangers
ISO 6759-1980		3	Seamless steel tues for heat exchangers

ISO 6761-1981		1	Steel tubes. Preparation of ends of tubes and fittings for welding.
ISO 7186-1983		8	Ductile iron pipes and accessories for non pressure pipe-lines
ISO 7268-1983			Pipe components – Definition of nominal ressure
		1	Amendment 1-1984
ISO 7369-1983		6	Pipework – flexible metallic hoses – Vocabulary of general terms Bilingual edition
ISO 7598 - 1982		3	Stainless steel tubes suitable for screwing in accordance with ISO 7/1
		1	Amendment 1-1984
ISO 7657 - 1984		3	Pipework – Stripwound flexible metal hoses – Specificaitons and temperature – related requirements for use.
ISO 7658 - 1984		4	Pipework – Stripwound flexible metal hoses – Testing and verification fo characteristics
ISO 8179 - 1985		2	Ductile iron pipes – External zinc coarting
ISO 8180-1985		2	Ductile iron pipes – Polyethylene sleeving
ISO 8444-1985		2	Pipework – double overlap flexible metal hoses (coper packing, limited tightness, circular section, in protected carbon steel)
ISO 8445-1985		2	Pipework – double overlap flexible metal hoses (asbestos packing, leakproof, circular section, in protected carbon steel)
ISO 8446-1985		2	Pipework – double overlap flexible metal hoses (asbestos packing, leakproof, circular section, in austenitic stainless steel)
TC 10			**Technical drawings** **UDC 744.4**
ISO 128-1982		15	Technical drawings – General principles of presentation.
ISO 129-1985		12	Technical drawings – Dimensions – General principles, definitions, methods of execution and special indications
ISO 406-1982		3	Technical drawings – Linear and angular tolerncing – indications on drawings
ISO 1046-1973		2	Architectural and building drawings – Vocabulary – Bilingual edition
ISO 1101-1983		24	Technical drawings – Geometrical tolerancing – Tolerancing of form, orientation, location and run-out – Generalities, definitions, symbols, indications on drawings.
Extract/Extrait			Technical drawings – Geometrical tolerancing. Toleranced characteristics and symbols – Examples of indication and interpreration. Bilingual edition
ISO 1101/2-1974		7	Technical drawings – Tolerances of form and of position – Part II: Maximum material principle.
ISO 1302-1978	2	13	Technical drawings – Method of indicating surface

		texture on drawings.
ISO 1660-1982	4	Technical drawings – Dimensioning and tolerancing of profiles
ISO 2162-1973	4	Technical drawings – representation of springs
ISO 2203-1973	6	Technical drawings-conventional representation of gears
ISO 2594-1972	2	Building drawings-Projection methods
ISO 2595-1973	3	Building drawings – Dimensioning f production drawings – Representaiton of manufacturing work sizes.
ISO 3040- 1974	11	Technical drawings – Dimensioning and tolerancing cones
ISO 3098/1-1974	6	Technical drawings – Lettering – Part 1 : Currently used characters.
ISO 3098/2-1984	5	Technical drawings – Lettering – Part 2: Greek characters
ISO 3098/4-1984	21	Technical drawings-Lettering – Part 4 : Cyrillic characters
ISO 3511/1-1977	10	Process measurement control functions and instrumentation – Symbolic representation – Part 1: Basic requirements.
ISO 3511/2-1984	8	Process measurement control functions and instrumentation – Symbolic representation – Part 2: Extension of basic requirements.
ISO 3511/3-1984	19	Process measurement control functions and instrumentation – Symbolic representation. Part 3 : Detailed symbols or instrument interconnection diagram.
ISO 3511/4-1985	8	Industrial process measurement control functions and instrumentation symbolic representation. Part 4: Basic symbols for process computer, interface, and shred display control functions.
ISO 3753-1977	13	Vacuum technology – Graphical symbols
ISO 3766 - 1977	5	Building and civil engineering drawings – Symbols for concrete reinforcement.
ISO 3952/1-1981	18	Kinematic diagrams – Graphical symbols – Part 1 – rilingual edition
ISO 3952/2-1981	12	Kinematic diagrams – Graphical symbols – Part 2 – Trilingual edition.
ISO 3952/301979	9	Kinematic diagrams – Graphical symbols – Part 3 – Trilingual edition
ISO 3952/4-1984	6	Kinmatic diagrams – Graphical symbols – Part 4 – Trilingual edition
ISO 4066-1977	7	Building and civil engineering drawings – Bar scheduling.
ISO 4067/1-1984	9	Technical drawings – Installations – Part 1: Graphical symbols for plunbing, heating, ventilation and ducting.
ISO 4067/2-1980	2	Building and civil engineering drawings – Installations 0 Part 2 : Simplified representation of sanitary appliances.
ISO 4065/6 1985	5	Technical drawings – Installations – Part 6: Graphical

		symbols for supply water and drainage systems int eh ground.
ISO 4068-1978	2	Building and civil engineering drawings. Reference lines.
ISO 4069-1977	2	Building and civil engineering drawings – Representaiton of areas on sections and views – General principles.
ISO 4157/1-1980	3	Building drawings – Part 1: Designation of buildings and parts of buildings.
ISO 4157/2-1982	2	Technical drawings – Construction drawings – Designation of buildings and parts of building – Part 2 : Designaiton of rooms and other areas.
ISO 4172-1981	8	Building and civil engineering drawings – Drawings for the assembly of prefabricated structures
ISO 5261-1981	10	Technical drawings for structural metal work
ISO 5455-1979	1	Technicl drawings - Scales
ISO 5457-1980	5	Technical drawings – sizes and layout of drawing sheets
ISO 5459-1981	16	Technical drawings – Geometrical tolerancing – Datums and datum-systems for geometrical tolerances.
ISO/TR 5460-1985	71	Technical drawings – Geometrical tolerancing – Tolerancing of form, orientation, location and run-out-Verification principles and methods Guidelines.
ISO 6410-1981	2	Technical drawing – Conventional representation of threaded parts.
ISO 6411 - 1982	5	Technical drawing s- simplified representation of center holes.
ISO 6414-1982	6	Technical drawings for glassware
ISO 642801982	3	Technical drawings – Requirements for microcopying
ISO 6433-1981	2	Technical drwins – item references
ISO 7083-1983	9	Technical drawings – Symbols for geometrical tolerancing – Proportions and dimensions.
ISO TR 70684-1991	11	Technical drawings – coding and referencing systems for building and civil engineering drawings and associated documents.
ISO 7200-1984	3	Technical drawings – Title blocks
ISO 7518-1983	2	Technical drawings – Construction drawings – Simplified representation of demolition and rebuilding.
ISO 7573-1983	2	Technical drawings – Item lists
ISO 8015-1985	5	Technical drawings – Fundamental tolerancin gprinciple
ISO 8048-1984	5	Technical drawings – construction drawings – representation fo views, sections and cuts
ISO/TR 8545-1984	21	Technical drawings – Installaions – Graphical symbols for automatic control.

TC 12			**Quantities, units, symbols, conversion factors and conversion tables** UDC 389.16 + 53.081
ISO 31/0-1981	2	13	General principles concerning quantities, units and symbols.
ISO 31/1-1978		13	Quantities and units of space and time
		1	Amendment 1-1985
ISO 31/2-1978		6	Quantities and units of periodic and related phenomena
		1	Amendment 1-1985
ISO 31/3-1978		18	Quantities and units of mechanics
		1	Amendment 1-1985
ISO 31/4-1978		14	Quantities and units of heat
		1	Amendment 1-1985
ISO 31/5-1979		23	Quantities and units of electricity and magnetism
		1	Amendment 1-1985
ISO 31/6-1980	2	13	ISO 31 – Quantities and units – Amendment 1
		1	Amendment 1 1985
TC 24			**Sieves, sieving and other sizing methods** **UDC 621,928**
ISO 565-1983	2	2	Test sieves – woven metal wire cloth, perforated plate and electroformed sheet – Nominal sizes of openings.
ISO 2194 - 1972		3	Wire screens and plate screens for industrial purposes – Nominal sizes of apertures
ISO 2395-1972		3	Test sieves and test sieving - vocabulary
ISO 2591-1973		11	Test sieving
ISO 3310/1-1982	2	5	Test sieves – Technical requirements and testing – Part 1 : Test sieves or metal wire cloth.
ISO 3310/2-1982	2	5	Test sieves – Technical requirements and testing – Part 2: Test sieves of metal perforated plate.
ISO 4782-1981		2	Industrial wire screens and woven wire cloth – Diameters of metal wire.
ISO 4783/1-1981		11	Industrial wire screens and woven wire cloth – Guide to the choice of aperture size and wire diameter combinations – Part 1 : Generalities.
ISO 4783/2-1981		7	Industrial wire screens and woven wire cloth – Guide to the choice of aperture size and wire diameter combination – Part 2 : Preferred combinations for woven wire cloth.
ISO 4783/3-1981		6	Industrial wire screens and woven wire cloth – Guide to the choice of aperture size and wire diameter combinations – Part 3 : Preferred combinations for pre-crimped or pressure-welded wire screens.
ISO 7805/1:1984		5	Industrial plate screens – Part 1 : Thickness of 3 mm and above
ISO 7806-1983		5	Industrial plate screens – Codification for designating perforations.

Reference	Ed	Page	Title
TC			**Measurement of fluid flow in closed conduits UDC 532.57**
ISO 2186-1973		34	Fluid flow in conduits – Connections for pressure signal transmissions between primary and secondary elements.
ISO 2975/1-1974		11	Measurement of water flow in closed conduits – tracer methods – Part 1 : General
ISO 2975/2-1975		9	Measurement of water flow in closed conduits – Tracer methods – Part II : Constant rate injection method using non-radioactive tracers.
ISO 2975/3-1976		11	Measurement of water flow in closed conduits – Tracer methods – Part III : Constant rate injection method using radioactive tracers
ISO 2975/6-1977		13	Measurement of water flow in closed conduits – Tracer methods – Part VI : Transit time method using non-radioactive tracers.
ISO 2975/7-1977		9	Measurement of water flow in closed conduits – Tracer methods – Part VII: Transit time method using radioactive tracers.
ISO/TR 3313-1974		24	Measurement of pulsating fluid flow in a pipe by means of orifice plates, nozzles or venturi tubes, in particular in the case of sinusoidal or square wave intermittent periodic-type fluctuations.
ISO 3354-1975		26	Measurement of clean water flow in closed conduits – velocity area method using current meters.
ISO 3966-1977		39	Measurement of fluid flow in closed conduits – velocity area method using pitot static tubes.
ISO 4006-1977		23	Measurement of fluid flow in closed conduits – Vocabulary and symbols – Bilinual edition
ISO 4053/1-1977		10	Measurement of gas flow in conduits – Tracer methods – Part 1: General
ISO 4053/4-1978		9	Measurement of gas flow in conduits – Tracer methods – Part IV : Transit time method using radioactive tracers.
ISO 4064/1-1977		6	Measurement of water flow in closed conduits – Meters for cold potable water – Part I: Specificaiton
ISO 4064/2-1978		2	Measurement of water flow in closed conduits – Meters for cold potable water – Part II: Installaltion requirements.
		2	Addendum 1-1983
ISO 4064/3-1983		16	Measurement of water flow in closed conduits – Meters for cold potable water – Part 3 : Test methods and equipment.
ISO 4185-1980		21	Measurement of liquid flow in closed conduits – weighing method.
ISO 5167 - 1980		65	Measurement of fluid by means of orifice plates, nozzles and venturi tubes inserted in circular cross-

			section conduits running tull
ISO 5168-1978		25	Measurement of fluid flow – Estimation of uncertainty of a flow-rate measurement
ISOTR 6817-1980		15	Measurement of conductive fluid flowrate in closed conduits – method using electromagnetic flowmeters
ISO 7145-1982		11	Determination of flowrate of fluids is closed conduits of circular cross-secion – Method of velocity measurement at one point of the cross-section
ISO 7194-1983		24	Measurement of fluid flow in closed conduits – Velocity area methods of flow measurement in swirling or asymmetric flow conditions in circular ducts by means of current meters or pitot static tubes.
ISO 7858/1-1985		3	Measurement of water flow in closed conduits – meters for cold potable water – Combination meters – Part I : Specifications.
TC 70			**Internal Combustion engines** **UDC 621 – 43**
ISO 1204 - 1972		1	Reciprocating internal combustion engines – Designation of the direction of rotation.
ISO 1205 - 1972		4	Reciprocating internal combustion engines – Designation of the cylinders
ISO 2261-1972		4	Reciprocating internal combustion engines – Hand operated control devices – Standard direction of motion.
ISO 2274-1972		3	Reciprocating internal combustion engines – Definition of right-hand and left-hand single bank engines.
ISO 2314-1973		20	Gas turbines – Acceptance test.
ISO 2710-1978	2	22	Reciprocating internal combustion engines. Vocabulary - Trilingual edition
		4	Addendum 1-1982
ISO 3046/1-1981	2	25	Reciprocating internal combustion engines – Performance – Part 1 : Standard reference conditions and declarations of power, fuel consumption and lubricating oil consumption
ISO 3046/2-1977		6	Reciprocating internal combustion engines – Performance – Part 2 : Test methods
ISO 3046/3-1979		4	Reciprocating internal combustion engines : Performance – Part 3: Test measurements
ISO 3046/4-1978		7	Reciprocating internal combustion engines : Performances – Part 4 : Speed governing
ISO 3046/5-1978		1	Reciprocating internal combustion engines – Performances – Part 5 : Toprsional vibrations
ISO 3046/6-1982	2	2	Reciprocating internal combustion engines – Performance – Part 6 : Overspeed protection
ISO 3249-1975		2	Reciprocating internal combustion engines – Definitions of locations on an engine.
ISO 3977-1978		33	Gas turbines – Procurement
ISO 4548/1-1982		7	Methods of test for full flow lubricating oil filters for

		internal combustion engines – Part 2: Pressure drop flow characteristics
ISO 4548/2-1982	5	Methods of test for full flow lubricating oil filters for internal combustion engines.- Part 2: Errent by-pass component characteristics.
ISO 4548/3-1982	4	Methods of test for full-flow lubricating oil filters for internal combustion engines – Part 3 : Resistance to high pressure drop and to elevated temperature.
ISO 4548/6-1985	2	Methods of test for full-flow lubricating oil filters for internal combustion engines – Part 6 : Static bust pressure test (*see also TC 22*)
ISO 6826-1982	3	Reciprocating internal combustion engines – Fire protection.

TC 71		**Concrete, reinforced concrete and pre-stressed concrete** **UD 691.32**
ISO 1920-1976	2	Concrete tests – Dimensions, tolerances and applicability of test specimens.
ISO 3893-1977	1	Concrete – classification by compressive strength
ISO 4012-1978	4	Concrete – Determination of compressive strength of test specimens
ISO 4013-1978	4	Concrete – Determination of flexural strength of test specimens
ISO 4103-1979	2	Concrete – Classificaiton of consistency
ISO 4108-1990	3	Concrete – Determination of tensile splitting strength of test specimens
ISO 4109-1990	2	Fresh concrete – Determination of the consistency – Slump test
ISO 4110-1979	2	Fresh concrete – Determination of the consistency – Vebe test
ISO 4111-1979	2	Fresh concrete – Determination of consistency – Degree of compactibility (Compaction index)
ISO 4848-1980	9	Concrete – Determination of air content of freshly mixed concrete – Pressure method
ISO 6274-1982	2	Concrete – sieve analysis of aggregates
ISO 6275-1982	2	Concrete, hardened – Determination of density
ISO 6276-1982	1	Concrete, compacted fresh – Determination of density
ISO 6782-1982	2	Aggregates for concrete – Determination of bulk density.
ISO 6783-1982	2	Coarse aggregates for concrete – Determinaiton of particle density and water absorption – Hydrostatic balance method.
ISO 6784-1982	3	Concrete – Determination of static modulus of elasticity in compression.

TC 77		**Products in fibre reinforcement cement** **UDC 691.328.5**
ISO 160-1980	9	Asbestos-cement pressure pipes and joints
ISO 390-1977	12	Asbestos-cement products – Sampling and inspection
ISO 391-1982	8	Building and sanitary pipes in asbestos-cement
ISO/R 392-1964	9	Asbestos-cement pipe fittings for builing and sanitary purposes
ISO 393/1-1983	11	Asbestos cement products – Part 1 : Corrugated sheets and fittings for roofing and cladding
ISO 393/3-1984	13	Asbestos cement products – Part 3: Asymmetrical section corrugated sheets and fittings for roofing and cladding.
ISO/R 394-1964	11	Asymmetrical section corrugated sheets in asbestos – cement for roofing and cladding.
ISO 395-1983	7	Asbestos cement slates
ISO 396/1-1980	8	Products in fibre reinforced cement – Part 1 : Asbestos – cement flat sheets.
ISO 396/2-1980	8	Produtcts in fibre reinforced cement – Part 2: Selica-asbestos-cement flat sheets.
ISO 396/3-1980	8	Products in fibre reinforced cement – Part 3 : Cellulose – asbestos – cement flat sheets.
ISO 880-1981	7	Asbestos-cement siding shingles
ISO 881-1980	10	Asbestos-cement pipes, joints and fittings for sewerage and drainage.
ISO/R 1896-1971	6	Thermal insulating asbestos boards
ISO 2785-1974	49	Guide to the selection of asbestos-cement pipes subject to external loads with or without internal pressure.
ISO 4482-1979	6	Asbestos-cement pipelines – guide for laying
ISO 4483-1979	8	Asbestos-cement pipelines – field pressure testing.
ISO 4486-1985	8	Asbestos-cement ventilation ducts and fittings – Dimensions and characteristics
ISO 4488-1979	10	Asbestos-cement pipes and joints for thrust-boring and pipe jacking
ISO 7336-1984	5	Asbestos-cement pipelines – Guidelines for hydraulic calculation
ISO 7337-1984	9	Asbestos reinforced cement products – Guidelines for one-site work practices.
TC 113		**Measurement of liquid flow in open channels** **UDC 532.57:532.543:627.133**
ISO 555/1-1973	16	Liquid flow measurement in open channels – dilution methods for measurement of steady flow – Part 1: constant rate injection method.
ISO 555/2-1974	16	Liquid flow measurement in open channels – Dilution methods for measurement of steady flow – Part II : Integration (sudden injection) method
ISO 555/3-1982	19	Liquid flow measurement in open channels – Dilution methods for measurement of steady flow – Part 3 :

			Constant rate injection method and integration method using radioactive tracers.
ISO 748-1979	2	23	Liquid flow measurement in open channels – velocity area methods
ISO 772-1978	2	35	Liquid flow measurement in open channels – Vocabulary and symbols – Bilingual edition
ISO 1070-1973		6	Liquid flow measurement in open channels – Slope area method.
ISO 1088-1985	2	21	Liquid flow measurement in open channels – Velocity area methods – Collection a processing of data for determination of errors in measurement.
ISO 1100/1-1981		21	Liquid flow measurement in open channels – Part 1 : Establishment and operation of a gauging station.
ISO 1100/2-1982		33	Liquid flow measurement in open channels – Part 2: Determinaiton of the stage-discharge relation
ISO 1438-1975		36	Liquid flow measurement in open channel using thin plate weirs and Venturi flumes.
ISO 1438/1-1980		27	Water flow measurement in open channels using weirs and Venturi flumes – Part 1 : Thin-plate weirs
ISO 2425-1974		15	Measurement of flow in tidal channels
		4	Amendement 1-1982
ISO 2537-1985	2	4	Liquid flow measurement in open channels – Cup-type and propeller-type current-meters
ISO 3454-1983	2	4	Liquid flow measurement in open channels – Director depth sounding and suspension equipment.
ISO 3455-1976		8	Liquid flow measurement in open channels – calibration of rotating-element current meters in sraight open tanks.
ISO 3716-1977		6	Liquid flow measurement in open channels – functional requirements and characteristics suspended sediment load samples.
ISO 3846-1977		9	Liquid flow measurement in open channels by weirs and flumes – free overfall weirs of finit crest width (rectangular broad0cresed weirs).
ISO 3847-1977		7	Liquid flow measurement in open channels by weirs and flumes – End depth method of estimation of flow in rectangular channels with a free overfall.
ISO 4359-1983		51	Liquid flow measuremnt in open channels, rectancular, rapezoidal and U-shaped flumes
ISO 4360-1984	2	13	Liquid flow measurement in open channel by weirs and flumes – Triangular profile weirs.
ISO 4363-1977		6	Liquid flow measurement in open channels – Methods for measurement of suspended sediment
ISO 4364-1977		13	Liquid flow measurement in open channels – Bed material sampling
ISO 4365-1985		30	Liquid flow in open channels – sediment in streams and canals – Determination of concentration, particles size distribution and relative density.
ISO 4366-1979		6	Echo sounders for water depth measurements
ISO 4369-1979		27	Measuremnt of liquid flow in open channels – Moving boat method.

ISO 4371-1984		11	Measurement of liquid in open channels by weirs and flumes – End depth method for estimation of flow in non-rectangular channels with a free ovrfall (approximate method)
ISO 4373-1979		18	Measurement of liquid flow in open channels – water level measuring devices.
ISO 4374-1982		17	Liquid flow measurement in open channels – round hose horizontal crest weirs.
ISO 4375-1979		8	Measurement of liquid flow in open channels – cableway system for stream gauging.
ISO 4377 1982		23	Liquid flow measurement in open channels – Flat V weirs
ISO 6416-1985		20	Liquid flow measurement in open channels – Measurement of discharge by he ultrasonic (acoustic) method.
ISO 6418-1985		13	Liquid flow measurement in open channels – Ultrasonic (acoustic) velocity metres.
ISO 6419/1-1984		18	Hydrometric data transmission systems Part 1 : General
ISO 6420-1984		9	Liquid flow measurement in open channels – position fixing equipment for hydrometric boats.
ISO/TR 7178-1983		27	Liquid flow measurement in open channels velocity area methods – Investigation of total error.
ISO 8333-1985		16	Liquid flow measurement in open channel by weirs and flumes – V shaped broad cested weirs.
ISO 8368-1985		5	Liquid flow measurement in ope channels – Guidlelins for the selection of flow gauging structures
TC 115			**Pumps** **UDC 621.65/.69**
ISO 2548-1973		34	Centrifugal, mixed flow and axial pumps – Code for acceptance tests – Class C
ISO 2858-1975	2	2	End-suction centrifugal pumps (rating 16 bar). Designation, nominal duty point and dimensions.
ISO 3069-1974		1	End suction centrifugal pumps – Dimensions of cavities for mechanical seals and for soft packing.
ISO 3555-1977		36	Centrifugal, mixed flow and axial pumps – Code for acceptance tests – Class B
ISO 3661-1977		3	End-suction centrifugal pumps – Baseplate and installations dimensions.
TC 138			**Plastics pipes, fittings and valves for the transport of fluids** **UDC 621.643.29_4.678**
ISO 161/1-1978	2	2	Thermoplastics pipes for the transport of fluids – Nominal outside diameters and nominal pressure – Part 1 : metric series.
ISO 161/2-1977		2	Thermoplastic pipes for the transport of fluids –

			Nominal outside diameters and nominal pressures – Part II : Inch series.
ISO 264.1976		5	Unplasticized polyvinyl chloride (PVC) fittings with plain sockets for pipes under pressure – laying lengths – metric series.
		2	Addendum 101982
ISO/R 265-1962		5	Pipes and fittings of plastics materials – Socket fittings with spigot ends for domestic and industrial waste pipes – Basic dimensions : Metric seris
ISO 580-1973		1	Moulded fittings in uplasticized polyvinyl chloride (PVC) for use under pressure – Oven test
ISO 727-1985	4	2	Fittings of unplasticized polyvinyl chloride (PVC-U), chlorinated polyvinyl chloride (PVC-C) or acrylonitrile/butadiene/styrene (ABS) with plain sockets for pipes under pressure – Dimensions of sockets – Metric series.
ISO 1167-1973		4	Plastics pipes for the transport of fluids – Determination of the resistance t internal pressure.
ISO 2035-1974		1	Unplasticized polyvinyl chloride (PVC) moulded fittings for elastic sealing ring type joints for use under pressure – pressure-resistance test.
ISO 2043-1974		1	Unplasticized polyvinyl chloride (PVC) moulded fitting sfor elastic sealing ring type joints for use under pressure – Oven test
ISO 2044-1974		1	Unplasticized polyvinyl chloride (PVC) injection moulded solvent welded socket fittings for use with pressure pipe – Hydraulic internal pressure test
ISO 2045-1973		1	Single sockets for unplasticized polyvinyl chloride (PVC) pressure pipes with elastic sealing ring type joints – Minimum depth of engagement.
ISO 2048-1973		1	Double socket fittings for unplasticized polyvinyl chloride (PVC) pressure pipes with elastic sealing ring type joints – Minimum depths of engagement.
ISO 2505 - 1981	2	3	Unplasticized polyvinyl chloride (PVC) pipes – Longitudinal reversion – Test methods and specification.
ISO 2506-1981	2	3	Polyethylene pipes (PE) – Longitudinal reversion – Test methods and specification
ISO 2507-1982	2	4	Unplasticized polyvinyl chloride (PVC) pipes and fittings – Vicat softening temperature – Test method and specification.
ISO 2536-1974		2	Unplasticized polyvinyl chloride (PVC) pressure pipes and fittings, metric series – Dimension sof flanges.
ISO 2703-1973		3	Buried unplasticized polyvinyl chloride (PVC) pipes for the supply of gaseous fuels 0 Metric series – Specification.
ISO 3114-1977		2	Unplasticized polyvinyl chloride (PVC) pipes for potable water supply – Extractability of lead and tin – Test method.
ISO 3126-1974		2	Plastics pipes – Measurement of dimensions
ISO 3127-1980		8	Unplasticized plyvinyl chloride (PVC) pipes for the

			transport of fluids – Determination and specification of resistance to external blows.
ISO 3212-1975		1	Polypropylene pipes – bust test requirements
ISO 3213-1975		1	Polypropylene pipes – Reduction of permissible stress as a function of time and temperature.
ISO 3458-1976		2	Assembled joints between fittings and polyethylene (PE) pressure pipes – Test of leakproofness under internal pressure.
ISO 3459-1976		2	Polyethylene (PE) pressure pipes – Joints assembled with mechanical fittings – Internal under-pressure test method and requirement
ISO 3460-1975		2	Unplasticized polyvinyl chloride (PVC) pressure pipes. Metric series – Dimensions of adapter for backing flange.
ISO 3472-1975		1	Unplasticized polyvinyl chloride (PVC) pipes – Specificaiton and determination of resistance to acetone.
ISO 3473-1977		2	Unplasticized polyvinyl chloride (PVC) pipes – effect of suphuric acid – Requirement and test method.
ISO 3474-1976		1	Unplasticized polyvinyl chloride (PVC) pipes – specification and measurement o capacity.
ISO 3477-1981	2	1	Polyprophylen (PP) pipes and fittings – Density – Determination and specification.
ISO 3478-1975		2	Polypropylene (PP) pipes – Determination of longitudinal reversion.
ISO 3480-1976		1	Polypropylene (PP) pipes – Maximum permissible longitudinal reversion.
ISO 3501-1976		2	Assembled joints between fittings and polyethylene (PE) pressure pipes – Test of resistance to pull out.
ISO 3503-1976		2	Assembled joints between fittings and polyethylene (PE) pressure pipes – Test of leakproofness under internal pressure when subjected to bending.
ISO 3514-1976		1	Chlorinated polyvinyl chloride (CPVC) pipes and fittings – Specification and determination of density.
ISO 3603-1977		2	Fittings for unplasticized polyvinyl chloride (PVC) pressure pipes with elastic sealing ring type joints – Pressure test fr leakproofness.
ISO 3604-1976		2	Fittings for unplasticized polyvinyl chloride (PVC) pressure pipes with elastic sealing ring type joints – Pressure test for leakprofness under condition of external hydraulic pressure.
ISO 3603-1976		2	Unplasticized polyvinyl chloride (PVC) pipes – Tolerances on outside diameters and wall thicknesses.
ISO 3607-1977		1	Polyethylene (PE) pipes – Tolerances on outside diameters and wall thicknesses.
ISO 3608-1976		2	Chlorinated polyvinyl chloride (CPVC) pipes – Tolerances on outside diameters and wall thicknesses.
ISO 3609-1977		1	Polypropylene (PP) pipes – Tolerances on outside diameters and wall thicknesses.
ISO 3663-1976		2	Polyethylene (PE) pressure pipes and fittings, metric series – Dimensions of flanges

ISO 4056-1978		1	Polyethylene (PE) pipes and fittings – Designation of polyethylene, based on nominal density and melt flow index.
ISO 4059-1978		3	Polyethylene (PE) pipes – Pressure drop in mechnical pipe-jointing systems – method of test and requirements.
ISO 4065-1978		4	Thermoplastic pipes – Universal wall thickness table.
ISO 3459-1976		2	Polyethylene (PE) pressure pipes – joints assembled with mechanical fittings – Internal under pressure test method and requirement.
ISO 3460-1975		2	Unplasticized polyvinyl chloride (PVC) pressure pipes – Metric series – Dimensions of adapter for backing flange.
ISO 3472-1975		1	Unplasticized polyvinyl chloride (PVC) pipes – specification and determination of resistance to acetone.
ISO 3473-1977		2	Unplasticized polyvinycl chloride (PVC) pipes – effect of sulpuric acid – Requirement and test method.
ISO 3474-1976		1	Unplasticized polyvinyl chloride (PVC) pipes – Specification and measurement of capacity.
ISO 3477-1981	2	1	Polypropylene (PP) pipes and fittings – Density – Determinaiton and specification.
ISO 3478-1975		2	Polypropylene (PP) pipes – Determination of longitudinal reversion
ISO 3480-1976		1	Polypropylene (PP) pipes – maximum permissible longitudinal reversion
ISO 3501-1976		2	Assembled joints between fittings and polyethylene (PE) pressure pipes – Test of resistance to pull out.
ISO 3503-1976		2	Assembled joints between fittings and polyethylene (PE) pressure pipes – Test of leakproofness under internal pressure when subjected to bending.
ISO 3514-1976		1	Chlorinated polyvinyl chloride (CPVC) pipes and fittings – Specification and determination of density.
ISO 3603-1977		2	Fittings for unplasticized polyvinyl chloride (PVC) pressure pipes with elastic sealing ring type joints – Pressure test for leakproffness.
ISO 3604-1976		2	Fittings for unplasticized polyvinyl chloride (PVC) pressure pipes with elastic sealing rin type joints – Pressure test for leakproofness under conditions of exernal hydraulic pressure.
ISO 3606-1976		2	Unplasticized polyvinyl chloride (PVC) pipes – Tolerances on outside diameters and wall thicknesses.
ISO 3607-1977		1	Polyethylene (PE) pipes – Tolerances on outside diameters and wall thicknesses
ISO 3608-1976		2	Chlorinated polyvinyl chloride (CPVC) pipes – Tolerances on outside diameters and wall thicknesses.
ISO 3609-1977		1	Polypropylene (PP) pipes – Tolerances on outside diameters and wall thicknesses.
ISO 3663-1976		2	Polyethylene (PE) pressure pipes and fittings, metric series – Dimensions of flanges
ISO 4056-1978		1	Polyethylene (PE) pipes and fittings – Designation of

		polyethylene, based on nominal density and melt flow index.
ISO 4059-1978	3	Polyethylene (PE) pipes – Pressure drop in mechanical pipe-jointing systems – Method of test and requirements.
ISO 4065-1978	4	Thermoplastic pipes – Universal wall thickness table.
ISO 4132-1979	5	Unplasticized polyvinyl chloride (PVC) and metal adaptor fittings for pipes under pressure – Laying lengths and size of threads – Metric series.
ISO 4433-1984	13	Polyolefin pipes – Resistance ot chemical fluids – Immersion test method – System for preliminary classification.
ISO 4434-1977	3	Unplasticized polyvinyl chloride (PVC) Adaptor fittings for pipes under pressure - Laying length and size of threads – Metric series.
ISO 4439-1979	1	Unplasticized polyvinyl chloride (PVC) pipes and fittings – Determination and specification of density
ISO 4440-1980	2	Polyethylene (PE) PIPES AND FITTINGS – determination of melt flow rate.
ISO 4451-1980	4	Polyethylene (PE) pipes and fitting – Determination of reference density of uncoloured and black polyethylenes.
ISO/TR 6285-1980	7	Pipes and fittings of acrylonitrile/butadiene/styrene terpolymer (ABS) chemical resistance with respect to fluids.
ISO 6455-1983	7	Unplasticized polyvinyl chloride (PVC) fittings with elastic sealing ring type joints for pipes under pressure – Dimensions of laying lengths – Metric series.
ISO/TR 7024-1985	37	Above-ground drainage – Recommended practice and techniques for the installation of unplasticized polyvinyl chloride (PVC-U) sanitary pipework for above-ground systems inside buildings.
ISO 7245-1984	3	Pipes and fittings of acrylonitrile – butadiene – styrene (ABS) – General specification fo rmoulding and extrusion materials.
ISO 7246-1984	3	Pipes and fittings of acrylonirtile/styrene/acrylester (ASA) – General specification for moulding and extrusion materials.
ISO 7279-1984	3	Polypropylene (PP) fittings for pipes under pressure – Sockets for fusion and using heated tools – Metric series – Dimensions of sockets.
ISO 7349-1983	2	Thermoplastics valves – connection references.
ISO 7370-1983	2	Glass fibre reinforced thermosetting plastics (GRP) pipes and fittings – Nominal diameters, specified diameters and standard lengths.
ISO 7387/1-1983	10	Adhesive with solvents for assembly of uPVC pipe element – Characterization – Part 1 : Basic test methods.
ISO/TR 7471-1978	12	Polypropylene (PP) pipes nad fittings – Chemical resistance with respect to fluids.
ISO/TR 7472-1979	10	Low density polyethylene (idPE) pipes and fittings –

		chemical resistance with respect to fluids to be conveyed.
ISO/TR 7473-1979	10	Unplasticized polyvinyl chloride pipes and fittings – Chemical resistance with respect to fluids.
ISO/TR 7474-1979	10	High density polyethylene pipes and fittings – Chemical resistance with respect to fluids to be conveyed.
ISO 7508-1985	5	Unplasticised polyvinyl chloride (PVC-U) valves for pipes pressure. Basic dimensions – Meric series.
TC 147		**Water quality** **UD 614-777:543.3**
ISO 5663-1984	4	Water quality – Determination for Kjeldahi nitrogen – Method after mineralization with selenioum.
ISO 5665-1984	3	Water quality – Determination of ammonium – Distilation and titration method.
ISO 5666/1-1983	6	Water quality – Determination of total mercury by flameless atomic absorption spectrometry – Part 1: Method after digestion with permanganate-peroxodisulfate
ISO 5666/2-1983	8	Water quality – Determination of total mercury by flameless atomic absorption spectrometry – Part 2 : Method after pretreatment with ultraviolet radiation.
ISO 5666/3-1984	6	Water quality – Determination of total mercury by flameless atomic absorption specutrometry – Part 3 ; Method after digestion with bromine.
ISO 5667/1-1980	13	Water quality – Sampling – Part 1 : Guidance on the design of sampling programmes.
ISO 5667/2-1982	8	Water quality – Sampling – Part 2 : Guidance on sampling techniques.
ISO 5667/3-1985	13	Water quality – Sampling – Part 3 : Guidance on the preservation and handling of samples.
ISO 5813-1983	5	Water quality – Determination of dissolved oxygen – lodomeric method.
ISO 5814-1984	8	Water quality – Determination of dissolved oxygen – Electrochemical probe method.
ISO 5815-1983	4	Water quality – Determination of biochemical oxygen demand after n days (BODn) – Dilution and seeding method.
ISO 7346/3-1984	10	Water quality – Determination of the acute lethal toxicity of substances to a freshwater fish (Brachydanio rerio Hamilton – Buchanan (Teleostei, Cyprinidae)) – Part 3 : Flow-through method.
ISO 7393/1-1985	8	Water quality – Determination of free chlorine and total chlorine – Part 1 : Titrimetiric method using N,N-diethyl-1,4-phenylenediamine
ISO 7393/2-1985	8	Water quality – Determination of free chlorine and total chlorine – Part 2 : colorimetric method using N, N-diethyl-1, 4-phenylenediamine, for routine control purposes.

ISO 7704-1985	4	Water quality – Evaluation of membrane filters used for microbiological analyses.
ISO 7827-1984	5	Water quality – Evaluation in an aqueous medium of the "Ultimum" aerobic biodegradability of organic compounds – Method by analysis of dissolved organic carbon (DOC).
ISO 7828-1985	6	Water quality – Methods of biological sampling – Guidance on handnet sampling of aquatic benthic micro-invertebrates.
ISO 7875/1-1984	6	Water quality – Determination of surfactants – Part 1 : Determinaiton of anionic surfactants by the methylene blue spectrometric method
ISO 7875/2-1984	6	Water quality – Determination of surfactants – Part 2 : Determinaiton of non-ionic surfactants using dragendorff reagent.
ISO 7887-1995	7	Water quality – Examination and determination of colour.
ISO 7888-1985	6	Water quality – Determination of electrical conductivity.
ISO 7899/1-1984	3	Water quality – Detection and enumeration of faecal streptococci – Part 1 : Method by enrichment in a liquid medium.
ISO 7899/2-1984	4	Water quality – Detection and enumeration of faecal streptococci – Part 2 : Method by membrane filtration
ISO 5961-1985	10	Water quality – Determination of cadmium – Flame atomic absorption spectrometric methods.
ISO 6058-1984	3	Water quality – Determination of calcium content – EDTA titrimetric method
ISO 6059-1984	4	Water quality – Determination of the sum of calcium and magnesium – EDTA titrimetric method.
ISO 6107/1-1980	18	Water quality – Vocabulary – Part 1 Trilingual edition.
ISO 6107/2-1981	33	Water quality – Vocabulary – Part 2 Trilingual edition.
ISO 6107/3-1985	23	Water quality – Vocabulary – Part 3 Trilingual edition.
ISO 6107/4-1984	5	Water quality – Vocabulary – Part 4 Trilingual edition.
ISO 6332-1982	5	Water analysis – Determination of iron – 1,10-phenanthroline photometric method
ISO 6341-1982	9	Water quality – determination of the inhibition of the mobility of Daphnia magna Straus (Cladocera, Crustacea)
ISO 6439-1984	7	Water quality – Determination of phenol indix –4-A minoantipyrine spectrometric methods after distillation.
ISO 6595-1982	5	Water quality – Deermination of total arsenic – Silver diethyldithiccarbamate spectrophotometric method.
ISO 6703/1-1984	11	Waer quality – Determination of cyanide – Part 1: Determination of total cyanide.
ISO 6703/2-1984	11	Water quality – Determination of cyanide – Part 2 : Determinaiton of easily liberatable cyanide.
ISO 6703/3-1984	6	Waer quality – Determination of cyanide – Part 3 : Determination of cyanogens chloride.
ISO 6703/4-1985	6	Water quality – Determination of cynide – Part 4 :

		Determinaiton of cyanide by diffusion at pH6.
ISO 6777-1984	5	Water quality – Determination of nitrite – Molecular absorption spectrometric method.
ISO 6778-1984	21	Water quality – Determination of ammonium – Potentiometric method.
ISO 7027-1984	5	Water quality – Determination of turbidity.
ISO 7150/1-1984	24	Water quality – Determination of ammonium – Part 1 : Manual spectrometric method.
ISO 7346/1-1984	9	Water quality – Determination of the acute lethal toxicity of substances to freshwater fish (Brachydanio rerio, Hamilton-Buchanan (Teleostei, Cyprinidae)) – Part 1 : Static method.
ISO 7346/2-1984	9	Water quality – Determination of the acute lethal toxicity of substances to a freshwater fish (Brachydanio rerio Hamilton-Buchanan (Teleostei, Cyprinidae)) Part 2 : Semi-static method.

C.4: BRITISH STANDARDS

<u>BRITISH STANDARD AND CODE OF PRACTICE (BS AND CP)</u>

<u>Pipes, fittings and accessories.</u>

21	:	1973	Pipe threads
65	:	1981	Clay drain and sewer pipes including surface water and fittings
78	:	1965	Part 2. Cast iron spigot and socket pipes (vertically cast) and spigot and socket fittings.
143	:	1968	Malleable case iron and cast copper alloy pipe fittings for steam, air, water, gas and oil. Screwed BSP taper thread or API line pipe thread
336	:	1980	Fire house couplings and ancillary equipment
416	:	1973	Cast iron spigot and socket soil, waste and ventilating pipes (sand cast and spun) and fittings
437	:	1973	Cast iron spigot and socket drain pipes
460	:	1964	Cast iron rainwater goods
486	:	1981	Asbestos cement pressure pipes
534	:	1981	Steel pipes, fittings and specials for water and sewerage
567	:	1973	Asbestos-cement flue pipes and fittings (ligh quality)
567	:	1973	Asbestos-cement rainwater goods
760	:	1984	Underground fire hydrants and dimensions of surface box openings.
778	:	1966	Steel pipes and joints for hydraulic purposes. Obsolescent
1010	:	1973	Draw-off taps and stop valves for waerservices (screw – down pattern). Part 2
1194	:	1969	Concrete porous pipes for under-drainage
1196	:	1971	Clayware field drain pipes
1211	:	1958	Centrifugally cast (spun) iron pressure pipes for water, gas and sewage.
1212	:		Specificaiton for float operated valves (excluding floats)
		1953	Part 1. Piston type
		1970	Part 2. Diaphragm type (brass body)
		1979	Part 3. Diaphram type of cold water services (plastic body)
1387	:	1967	Steel tubes and tubulars suitable for screwing to BS 21 pipe threads
1710	:	1984	Specification fo identification of pipelines and services
1740	:	1971	Wrought pipe fittings, iron and steel (screwed BSP thread)
1965	:	1963	Butt-welding pipe fittings for pressure purposes
1968	:	1953	Floats for ball valves (copper)
2035	:	1966	Cast iron flanged pipes and fittings (1981)
2051	:	1973	Tube and pipe fittings for engineering purposes. Part 1-2
2456	:	1973	Floats and ball valves (plastics) for cold water
2494	:	1976	Joint rings for pipework and pipelines
2879	:	1980	Draining taps (screwed – down pattern)
3063	:	1965	Dimensions of gaskets for pipe flanges
3251	:	1976	Hydrant indicator plates
3284	:	1967	Polythene pipe (Type 50) for cold water services. Obsolescent
3505	:	1968	Unplastized PVC pipe for cold water supply (1982)

3506	:	1969	Unpalstized PVC pipe for industrial uses
3601	:	1974	Steel pipes and tubes for pressure purposes
3604	:	1978	Steel pipes and tubes for pressure purposes Low and medium alloy steel.
3605	:	1973	Steel pipes and tubes for pressure purposes Austenitic Stainless steel
3656	:	1981	Asbestos-cement pipes and fittings for sewerae and drainage.
3974	:	1974	Pipe supports. Part 1. Pipe hanges, slider and roller type
		1978	Part 2. Pipe clamps, cages, contilevers and attachments
		1980	Part 3. Large bore, high temperature, marine and other applic.
4127	:	1972	Light gauge stainless steel tubes. Part 2 (1981)
4504	:		Flanges and bolting for pipes, valves and fittings
		1969	Part 1. Ferrous
		1974	Part 2. Copper alloy and composite flanges
4576	:	1970	Unplasticized PVC rainwater goods (1982)
4622	:	1970	Grey iron pipe and fittings (1983)
4625	:	1970	Prestressed concrete pressure pipes (including fittings)
4660	:	1973	Unplasticized PVC underground drain pipe and fittings
4771	:	1980	Specification for ductile iron pipes and fittings.
4772	:	1980	Specification for ductile iron pipes and fittings.
4865	:		Dimensions of gaskets for pipe flanges to BS 4504
		1972	Part 1. Dimensions of non-metallic gaskets
		1973	Part 2. Dimensions of metallic spiral wound gaskets
5105	:	1974	Cast iron wedge and double disk gate valves for general purposes.
5152	:	1974	Cast iron globe and globe stop and check valves for general purposes (1983).
5153	:	1974	Cast iron check valves for general purposes (1983)
5154	:	1983	Copper alloy globe, globe stop and check, check and gate valves for general purposes.
5155	:	1984	Specificaiton for butterfly valves
5156	:	1974	Screw down diaphragm valves for general purposes
5157	:	1974	Steel gate (parallel slide) valves for general purposes
5158	:	1974	Cast iron and carbon steel plus valves for eneral purposes
5159	:	1974	Cast iron carbon steel ball valves for general purposes
5160	:	1977	Specification for flanged steel globe valves, globe stop and check valves for general purpose.
5164	:	1974	Double flanged cast iron wedge gate valves for waterworks purposes
5178	:	1975	Prestressed concrete pipes for drainge and sewage
5255	:	1976	Plastic waste pipe and fittings
5292	:	1980	Specificaiton for jointing materials and compounds for installations using waer etc.
5412	:	1976	Specifications for the performance of draw-off taps with metal bodies for water serices part 1-5
5413	:	1976	As above but with plastic bodies
5433	:	1976	Specificaitons for underground stopvalves for water services
5481	:	1977	Specification for unplasticized PVC pipe and fittings for gravity sewers.
5556	:	1978	Specification for general requirements for dimensions and pressure ratings for pipe of termoplastics material
5572	:	1978	Code of practice for sanitary pipework

5834	:		Surface boxes and guards for underground stopvalves for gat and waterworks purposes
		1985	Part 1. Specification for guards, incl. Foundation units
		1983	Part 2. Specification for small surface boxes
5911	:		Precast concrete pipes and fittings for drainage and sewerage
		1981	Part 1. Specification for concrete cylindrical pipes, bends, junctions and manholes, unreinforced and reinforced with steel cages and hoops.
		1982	Part 2. Specification for inspection chambers and gullies
		1982	Part 3. Specification for ogee jointed concrete pipes.
5927	:	1980	Guide for laying asbestos-cement pipe
5949	:	1980	Asbestos-cement pipes and joints for thrust-boring and pipe jacking.
5955	:		Code of practice for plastics pipework
		1980	Part 6. Installaiton of unplasticized PVC pipework for gravity drains and sewers.
		1983	Part 7. Recommendations for methods of thermal fusion jointing.
6087	:	1981	Specificaitons for flexible joints for cast iron drain pipes and fittings (BS 437) and for cast iron soil waste and ventilating pipes and fittings (BS 416)
6437	:	1984	Specificaiton for polyethylene pipes (type 50) in metric diameters for general purposes.
CP 312	:	1973	Plastics pipework. Part 1-3
CP 2010			Pipelines. Part 1-5 1966-1975

Concrete and reinforcement

12	:	1978	Portland cement (ordinary and rapid-hardening)
882	:	1983	Aggregates for natural sources for concrete (including granolithic)
890	:	1972	Building limes
1370	:	1979	Low heat Portland cement
3797	:	1976	Lightweight aggregates for concrete (1982)
4027	:	1980	Specificaiton for sulphate-resisting Portland cement.
4449	:	1978	Specification for hot rolled steel bars for the reinforcement of concrete (1984)
4461	:	1978	Specification for cold worked steel bars for he reinforcement of concrete (1984)
4466	:	1981	Specificaiton for bending dimension and scheduling of reinforcements of concrete.
4482	:	1969	Hard drawn mild steel wire for the reinformcement of concrete.
4483	:	1969	Steel fabric for the reinforcement of concrete
5075	:		Concrete admixtures
		1982	Part 1. Specificaiton for acelrating admixtures, retarding admixtures and water-reducing admixtures
		1982	Part 2. Specification for air-entraining admixtures
		1985	Part 3. Specificaiton for superplasticizing admixtures
5328	:	1981	Methods for specifying concrete ; including ready-mixed concrete
5337	:	1976	Code of practice for structural use of concrete for retaining aqueous liquids.
6073	:		Precast concrete masonry units
		1981	Part 1. Specificaiton for precast concrete masonry units

		1981	Part 2. Method for specifying precast concrete masonry units
6089	:	1981	Guide to the assessment of concrete strength in existing structures.
6588	:	1985	Specification for Portland pulverized-fuel ash cement
8110	:		Structural use of concrete
		1985	Part 1. Code of practice for design and construction
		1985	Part 2. Code of practice for special circumstances
CP 110	:	1972	The structural use fo concrete. Part 2-3
CP 114	:	1969	Structural use of concrete in buildings

Structural steel and other building materials.

4	:	1980	Structural steel sections
			Part 1. Specification for hot-rolled sections.
405	:	1945	Expanded metal (steel) for general purposes
449	:	1973	The use of structural steel in building
648	:	1964	Schedule of weights of building materials
690	:		Asbestos-cement slates and sheets
		1981	Part 2. Specification for asbestos-cement and cellulose asbestos-cement flat sheets.
		1973	Part 3. Corrugated sheets
		1974	Part 4. Slates
		1975	Part 5. Lining sheets and panels
		1976	Part 6. Fittings for use with corrugated sheets.
743	:	1970	Materials for damp proof cources. Metric units
747	:	1977	Roofing felts (bitumen and fuxed pitch)
1199	:	1976	Building sands from natural sources
1281	:	1974	Glazed ceramic tiles and tile fittings for internal walls
1286	:	1974	Clay tiles for flooring
1369	:	1947	Metal lathing (steel) for plastering
1494	:	1951	Fixing accessories for building purposes
		1964	Part 1. Fixing for sheet, roof and wall covering
2094	:		Glossary of terms relating to iron and steel
		1954	Part 2. Steel making
		1954	Part 3. Hot rolled steel products (excl. sheet, strip, tubes)
		1954	Part 4. Steel sheet and strip
		1954	Part 5. Bright steel bar and steel wire
		1954	Part 6. Forgings and drop forgings
		1956	Part 7. Wrought iron
		1956	Part 8. Steel tubes and pipes
		1964	Part 9. Iron and steel founding
3189	:	1973	Phosphate treatment of iron and steel for protection against corrosion.
3416	:	1975	Black bitumen coating solutions for cold application
3921	:	1974	Bricks an dblocks of fired brickearth clay or shale
4848	:		Hot rolled structural steel sections.
		1975	Part 2. Hollow sections
		1972	Part 4. Equal and unequal angles
		1980	Part 5. Bulb flats
4868	:	1972	Profiled aluminium sheet for building
5247	:	1975	Corrugated asbestos cement

5390	:	1976	Code of practice for stone masonry (1984)
5400	:		Steel, concrete and composite bridges. Part 1-10c 1978-82
5493	:	1977	Code of practice for protective coating of iron and steel structures against corrosion
5628	:		Code of practice for use of masonry
		1978	Part 1. Unreinforced masonry (1985)
		1985	Part 2. Structural use of reinforced and prestressed masonry
		1985	Part 3. Materials and components, design an workmanship
CP 102	:	1973	Protection of buildings against water from the ground
CP 143	:		Sheet roof and wall coverings. Part 1, 5, 10, 11, 12, 15, 16, 1958-74

Testing and measuring

410	:	1976	Test sieves
599	:	1966	Methods of tesing pumps
812	:		Testing aggregates
		1975	Part 1. Methods for determination of particle size and shape
		1975	Part 2. Methods for determination of physical properties
		1975	Part 3. Mechanical properties
		1976	Part 4. Methods for determination of chemical properties
		1984	Part 101. Guide to sampling and testing aggregates
		1984	Part 102. Methods for sampling
		1985	Part 103. Method for determination o fparticle size distribution.
		1985	Part 105. Section 105.1. Flakiness index
		1985	Part 106. Method for determination of shell content in coarse aggregate.
		1985	Part 119. Method for determination of acid-soluble material in fine aggregate.
1042	:		Methods of measurement of fluid flow in closed conduits.
		1981	Part 1. Section 1.1 Orifice plates, nozzles and Venturi tubes inserted in circular cross-seciton conduits running full
		1984	Part 1. Section 1.2. Specification for square-edged orifice plates and nozzle (with drain holes, in pipes below 50 mm diameter, as inlet and outlet devices) and other orifice plates and Borda inlets.
		1984	Part 1. Section 1.4. Guide to the use of devices specified in sections 1.1 and 1.2.
		1983	Part 2. Section 2.1. Method of using pitot-static tubes
		1983	Part 2. Section 2.2. Method of measurement of velocity at one point of a conduit of circular cross section.
		1984	Part 2. Seciton 2.3. Methods of flow measurement in swirling or asymmetric flow conditions in circular ducts by means of current-meters or pitot static tubes.
		1965	Part 3. Guide to the effects of departure from themethod in Part 1.
1377	:	1975	Methods of testing soils for civil engineering purposes
1427	:	1962	Routine control methods of testing water used in industry
1647	:	1961	PH scale
		1984	Part 1. Specification for pH scale
		1984	Part 2. Specification for reference value standard and operational standard solutions.
1796	:	1976	Method for test sieving (1985)

1881 : Methods for testing concrete

1970 Part 1. Methods for sampling fresh concrete

1970 Part 5. Methods for testing hardened concrete for other than strength.

1971 Part 6. Analysis of hardened concrete

1983 Part 101. Method of sampling fresh concrete on site

1983 Part 102. Method of determination of slump

1983 Part 103. Method of determination of compacting factor.

1983 Part 104. Method for determination of Vebe time.

1984 Part 105. Method for determination of flow

1983 Part 106. Method for determination of air content of fresh concrete.

1924 : 1975 Methods of test for stabilized soils

2690 : Methods for testing water used in industry

1964 Part 1. Copper and iron (1974)

1965 Part 2. Dissolved oxygen, hydrazine and sulphate

1967 Part 4. Calcium and magnesium

1968 Part 6. Chloride and sulphate

1968 Par 7. Nitrate, nitrate and ammonia (free, saline and albuminoid)

1970 Part 9. Appearance (colour and turbidity), odour, susp. and dissolved solids and electricity conductivity

1971 Part 11. Anionic, cationic and non-ionic detergents and oil

1972 Part 12. Nickel, zink, chromates, total chromium and manganese

1972 Part 13. Dichromate value (chemical oxygen demand), non-volatile organic carbon, tannins and chlorine

1972 Part 14. Arsenic, lead and sulphide

1974 Part 15. Free EDTA, total salts of EDTA, polyacrylate and polymetachrylate

1978 Part 100. Foreworkd, scope and general requirements

1984 Part 101. Dissolved oxygen

1979 Part 10-2. Hydrazine : Spectraphotometric method (4-dimethl-aminobenzadehyde)

1983 Part 104. Silica : reactive, total and suspended

1983 Part 105. Soluble phosphate and organophosphorus compounds

1979 Part 106. Reactive aluminium : spectrophotometric method

1984 Part 109. Alkalinity, acidity, pH value and cargon dioxide

1983 Part 115. Cyclohexalamine : spectrophotometric method

1983 Part 116. Morpholine : spectrophotometric method

1983 Part 117. Long-chain fatty amines; spectrophotometric methods

1981 Part 119. Odour

1981 Part 120. Suspendid solids

1981 Part 121. Dissolved solids

1984 Part 123. Sodium

1981 Part 124. Potassium : flame photometric method

1981 Part 125. Lithium : flame photometric method

3148 : 1980 Tests for water for making concrete.

3680 : Methods of measurement f liquid flows in channels

1983 Part 1. Glossary of terms

Part 2. Dillution methods

1964 Part 2A. Constant rate injection

1983 Part 2C. Method of measurement using radioactive tracers

			Part 3. Stream flow measurement
		1980	Part 3A. Velocity area method
		1983	Part 3B. Guide for estalishment and operation of a gauging station.
		1983	Part 3C. Methods of determination of the stagedischarge relation
		1980	Part 3D. Moving-boat method
			Part 4. Weirs and flumes
		1981	Part 4A. Thin-plate weirs.
		1969	Part 4B. Long-base weirs.
		1981	Part 4C. Flumes
		1981	Part 4D. Compound gauging structures
		1980	Part 4E. Free overfall weirs of finite crest width (rectangular broad-crested weirs)
		1981	Part 4F. Round-nose horozontal crest weirs
		1981	Part 4G. Flat-V weirs
		1970	Part 5. Slope area method of estimation
		1973	Part 6. The measurement of flow in tidal channels
		1971	Part 7. The measurement of liquid level (stage)
			Part 8. Measuring instruments and equipment
		1973	Part 8A. Current meters incorporating a rotating element
		1983	Part 8B. Recommendations for direct depth sounding and suspension equipment
		1980	Part 8C. Calibration of rotating-element current-meters in straight open tanks.
		1980	Part 8D. Cableway system for stream gauging
			Part 9. Water level instruments
		1971	Part 9A. Specification for the installation and performance of pressure actuated liquid levelmeasuring equipment.
		1981	Part 9B. Float operated water level recorders (mechanical and electromechanical)
			Part 10. Sediment transport
		1980	Part 10B. Measurement of suspended sediment
		1980	Part 10C. Bed material sampling
3681	:	1973	Methods for the sampling and testing of lightweight aggregates for concrete. Part 2. Metric units (1983)
3889	:		Non-destructiove testing of pipes and tubes
		1983	Part 1. Method of automatic ultrasonic testing for the detection of imperfections in wrought steel tubes
		1965	Part 2A. Eddy current testing of ferrous pipes and tubes
		1966	Part 2B. Eddy current testing of non-ferrous tube (1982)
4408			Recommendations for non-destructive methods of test for concrete
		1969	Part 1. Elecromagnetic cover measuring device
		1969	Part 2. Strain gauges for concrete investigations
		1970	Part 3. Gamma radiography of concrete
		1971	Part 4. Surface hardness methods
		1974	Part 5. Measurement of the velocity of ultrasonic pulses in concrete
4426	:	1969	Method of test for sodium hypochlorite solution
4624	:	1981	Methods of test for asbestos-cement building products

5080	:	1974	Methods of test for structural fixings in concrete and masonry. Part 1. Tensile loading (1982)
5316	:		Acceptance tests for centrifugal, mixed flow and axial pumps.
		1976	Part 1. Class C tests
		1977	Part 2. Class B tests
5728	:		Measurement of flow of cold potable water in closed conduits
		1979	Part 1. Specifications for meters
		1980	Part 2. Specification for installation conditions for meters
		1984	Part 3. Methods for determining principal characteristics of meters
5835	:	1980	Recommendations for testing of aggregates Part 1. Compatibility test for graded aggregate
5857			Measurement of waer flow
		1980	Part 1. Section 1.1 General
		1980	Part 1. Seciton 1.2 Constant rate injection method using non-radioactive tracers.
		1980	Part 1. Section 1.3. Constant rate injetion method using radioactive tracers
		1980	Part 1. Section 1.4. Transit time method using non-radio-active tracers
		1980	Part 1. Section 1.5. Transit time method using radio-active tracers.
5886	:	1980	Methods for field pressure testing of asbestos-cement pipelines.

Drawing Practice

308	:		Engineering drawing practice
		1984	Part 1. Recommendations for general principles
		1972	Part 2. Dimensioning and tolerancing of size
		1972	Part 3. Geometrical tolerancing
1192	:		Construciton drawing practice
		1984	Part 1. Recommendations for general principles
		1984	Part 4. Recommendations for landscape drawings
1553	:		Specification for graphical symbols for general engineering
		1977	Part 1. Piping systems and plant
		1950	Part 2. Graphical symbols for power generating plant.
		1950	Part 3. Graphical symbols for compressing plant.

Miscellaneous

417	:		Galvanized mild steel cisterns and covers, tanks and cylinders
		1964	Part 1. Imperial units
		1973	Part 2. Metric units
879	:		Water well casing
		1985	Part 1. Specification for steel tubes for casing
916	:	1953	Black bolts, screws and nuts. Obsolescent
1083	:	1965	Precision hexagon bolts, screws and nuts. Obsolescent
1244			Metal sinks for domestic purposes
		1956	Part 1. Imperial units with metric equivalents (1977)
		1982	Part 2. Specification for stainless steel sink tops
1247	:	1975	Manhole step irons (Malleable cast iron)
1563	:	1949	Cast iron sectional tanks (rectangular) 1964

1564	:	1975	Presses steel sectional rectangular tanks (1983)
1565	:		Galvanized mild steel indirect cylinders, annular or saddle-back type.
		1949	Part 1. Imperial units
		1973	Part 2. Metric units
1580	:		Unified screw threads
		1962	Parts 1 & 2. Diameters ¼ in the larger (1985)
		1965	Part 3. Diameter below ¼ in (1985)
1768	:	1963	Unified precision hexagon bolts, screws and nuts (UNC and UNF threads)
1769	:	1951	Unified black hexagon bolts, screws and nuts. (UNC an d UNF threads) Heavy series. Obsolescent.
1782	:	1951	Hose couplings ($1^{1}/_{2}$ in to 8 in nominal sizes) other than fire hose couplings.
1906	:	1952	Hose couplings (air and water) $^{1}/_{8}$ in to 1¼ in nominal sizes.
2633	:	1973	Class 1 arc welding of ferritic steel pipework for carry fluids (1981)
2971	:	1977	Specification for class II arc welding of carbon steel pipework for carrying fluids (1982)
3628	:	1963	Hose clip (worm drive type) for general purpose use. Obsolescent
3674	:	1981	Specification for sodium carbonate (tecknical grade)
3746	:	1964	PVC garden hose (1983)
3978	:	1966	Water for laboratory use
4082	:		External dimensions for vertical in-line centrifugal pumps
		1969	Part 1. "I" type
		1969	Part 2. "U" type
4213	:	1975	Cold water storage cisterns (polyolefin or olefin copolymer) and cistern covers.
4515	:	1984	Specification for process of welding of steel pipelines on land and offshore
4882	:	1973	Bolting for flanges and pressure containing purposes.
4994	:	1973	Vessels and tanks in reinforced plastics.
5000	:		Rotating electrical machines of particular type or for particular applications. Part 2,3,10,11,15,17,25,40,50,60 and 99. (1973-1984)
5119	:	1980	Specifications for general purpose rubber water hoses
5315	:	1976	Hose clamps
5423	:	1980	Specificaiton for portable fir extinguishers
5514	:		Specificaiton for reciprocating internal combustion engines : performance. Part 1 – 6. 1978 - 1984
5930	:	1981	Code of practice for site investigations (formerly CP 2001)
5973	:	1981	Code of practice for access and working scaffolds and special scaffold structures in steel.
5974	:	1982	Code of practice for temporarily installed suspended scaffolds and access equipment.
5975	:	1982	Code of practice for falsework
5977	:		Lintels
		1981	Part 1. Method for assessment of load
		1983	Part 2. Specificaiton for prefabricated lintels
5997	:	1980	Guide to British Standard codes of practice for building services.
6031	:	1981	Code of practice for earthwork

6068	:		Water quality
		1982	Part 1. Section 1.1. Terms relating to types of water and treatment and storage of waste water
		1982	Part 1. Section 1.2. Additional terms relating to types of water, and treatment and storage of water and waste water, and terms used in sampling and analysis of water.
		1983	Part 2. Section 2.1. Determination of total arsenic, silver diethyldithiocarbamate spectrophotometric method.
		1983	Part 2. Section 2.2. Determination fo iron 1.10 penanthrolin photometric method.
		1984	Part 2. Section 2.3. Determination of dissolved oxygen: iodometric method.
		1984	Part 2. Section 2.4. Determination of toal mercury by flameless atomic absorption spectrometry: method after digestion with permanganate-peroxodisulphate.
		1984	Part 2. Seciton 2.5. Determinaiton of total mercury by flameless atomic absorption spectrometry : method after pretreatment with ultraviolet radiation.
		1984	Part 2. Section 2.6. Determination of total mercury by flameless atomic absorption spectrometry : method after digestion with bromine.
		1984	Part 2. Section 2.7. Determination of ammonium : distillation and titration method.
		1984	Part 2. Section 2.8. Determination of calcium content – EDTA titrimetric method.
		1984	Part 2. Seciton 2.9. Determination of the sum of calcium and magnesium : EDTA titrimetric method.
		1984	Part 2. Section 2.10. Determination of ammonium : potentio-metric method.
		1984	Part 2. Section 2.11. Determination of ammonium : manual spectrometric method.
		1984	Part 2. Section 2.12. Determination of phenol index : 4-aminiantipyrine (4-aminophenazone) spectrometric method after distillation.
		1984	Part 2. Section 2.13. Determination of turbidity
		1984	Part 2. Section 2.14. Determinaiton of biochemical oxygen demand after 5 days (BOD): dilution and seeding method
		1984	Part 2. Section 2.16. Determination of nitrite : molecular absorption spectrometric method.
		1983	Part 5. Section 5.1. Determination of the inhibition of the mobility of Daphnia magne Staus (Chadocera, Crustacera)
		1981	Part 6. Section 6.1. Guidance of the design of sampling programmes
		1983	Part 6. Section 6.2. Guidance on sampling techniques
6150	:	1982	Code of practice for painting of buildings
6565	:	1985	Method for dimensioning and designating bolts, screws, studs and nuts.
CP 310	:	1965	Water supply
CP 2010	:	1974	Foundation for machinery

APPENDIX D:

D.1: PROJECT SUMMARY SHEET

Project ………………………………………………………… Alternative No. …..………..
Design Stage ………………………………. Date …………………………..……………
Province …………………………………… District ……………………….……………
Reference Code ……………………………. Land potential ……………..…………………
Source ……………………………………… Safe Yield …………………………..m^3/day
Treatment …………………………………………………………………………………….
Raw water system …………..…% gravity ……………….……% pumping
Distribution system ……………%gravity ………..…………% pumping
Power ………………………………………………………………………………………..

	(year)	Initial (……)	Future (….)	Ultimate (….)
Population served	(No.)	………………..	……………….	………………..
Livestock served	(L.U.)	………………..	……………….	………………..
Livestock served (out of total)	(L.U.)	………………..	……………….	………………..
Population demand	(m^3/d)	………………..	……………….	………………..
Livestock demand	(m^3/d)	………………..	……………….	………………..
Institutional demand	(m^3/d)	………………..	……………….	………………..
Industrial demand	(m^3/d)	………………..	……………….	………………..
Other demand	(m^3/d)	………………..	……………….	………………..
Total demand	(m^3/d)	………………..	……………….	………………..
Individual connections	(%)	………………..	……………….	………………..

Phase	Year of implemen -tation	Population served (Tot.)	Area served Km^2(Tot)	Designed Demand m^3/d(Tot)	Construc-tion Shs	Costs Shs/ Capita	Shs/ Km^2	Shs/ m^3
I	…….	…….	…….	…….	…….	…….	…….	…….
II	…….	…….	…….	…….	…….	…….	…….	…….
III	…….	…….	…….	…….	…….	…….	…….	…….
Total	…….	…….	…….	…….	…….	…….	…….	…….

Annual Costs	Initial Shs	Shs/m^3	Future Shs.	Shs/m^3	Ultimate Shs.	Shs/m^3
Direct O & M costs	………	………	………	………	………	………
Establishment costs	………	………	………	………	………	………
Capital Costs	………	………	………	………	………	………
(i- ………….%)	………	………	………	………	………	………
Total	………	………	………	………	………	………
Annual revenue	………	………	………	………	………	………

Limits of supply, see attached plan (Format A4)

D.2: O & M EXPENDITURE SUMMARY SHEET

1 Project …………………………………………………………………………………
2. District ……………………………… 3. Province …………………………………
4. Treatment ………………………………………………………………………………
5. Raw Water System Gravity …………% Pumping …………..…………………%
6. Distribution System Gravity…………% Pumping……………………………%
7. Power ……………………………………………………………………………………..

8.	Year Item - Operation	Initial (I) Shs.	1 + 5 years Shs	1 + 10 years Shs.	1 + 15 years Shs.	1 + 20 years Shs.
	Chemical					
	Electricity					
	Diesel					
	Lubricating Oil					
	Laboratory Reagents					
	Staff					
	Transport					
9.	Item – Maintenance	**Shs.**	**Shs.**	**Shs.**	**Shs.**	**Shs.**
	Tools					
	Laboratory					
	Spare parts – Mechanical					
	Spare-parts – Electrical					
	Meters					
	Pipelines					
	Paints, Timber etc.					
10	Revenue	**Shs.**	**Shs.**	**Shs.**	**Shs.**	**Shs.**
	Revenue Earning					
11	Operation	**Hrs**	**Hrs**	**Hrs**	**Hrs**	**Hrs**
	Operation per day					

All costs valid for the price level in year 20….

D.3: CHECK LIST FOR PRELIMINARY DESIGN

The aspects to be checked in a Preliminary Design when applicable are:

1. Report coverage – use of correct wordings.

2. Compliance with the scope of engagement.

3. Date of submission

4. Names of District indicating boundaries of supplies

5. Complete summary – compared with the information given in the main test. Standard Project Summary Sheet and O & M Expenditure Summary Sheet.

6. Map showing the supply area

7. Existing Supplies:
(a) Details regarding ownership
(b) Operating agency
(c) Source
(d) Quality
(e) Treatment
(f) Constraints
(g) Original Design
(h) Reliability
(i) Technical and economical assessments of the various components.
(j) Number of connections and CPW's.

8. Traditional sources – their quality and distance, is in use.

9. Climate, Hydrology described in detail

10. Recommended sources-Reliability, quality and economy. Comparison between other alternatives basis for selection.

11. Actual abstractions – existing water permits.

12. Surfaces water sources – flows – Flood and low flows probability.

13. Rainfall comparisons and correlation of flows

14. Rainwater harvesting – rainfall probability, pattern. Roof area, tanks.

15. Geological data – soil profile, soil samples, permeability and seepage, possible siltation calculation and control measures, grading of soils, site investigations.

16. Ground water sources – Boreholes, existing records, dimensions, depth, details of investigation of aquifers, adequacy of test pumping done, capacity of test pump, drawdown level, rest level, recovery period, influence on observation holes, determination of recharge, basis for siting.

17. Sprints and shallow wells, existing and potential, reliability.

18. Dams:
    (a) Selection of site, subsurface conditions
    (b) Storage volume, methods of calculations based on run-off data draught periods, evaporation, seepage losses, silt road and sitation losses.
    (c) Stability – kind of material for core and shoulder cover, provision of free board, correlated wave action, fetch and wind velocities, influence of earthquakes.
    (d) Study of alternative construction methods depending on availability of materials.
    (e) Provision of surplussing arrangements and spillway based on adequate calculating – compensation water – consequences resulting for population both up and down stream – comparison of alternatives for spillways and cutlets – calculations and basis for selection.
    (f) Provision made and control features for passing greater floods.
    (g) Arrangements for scour release – study and water shed management, for control of erosion.

19. Subsurface dams – Details in para 15 and 18 – satisfy conditions therein.

20. Protection of the catchment area.

21. Water quality – Adequate number of samples during various seasons and during maximum – minimum flows, physical, chemical and bacteriological tests.

22. Pollution control – Existing and known future waste water and sewage outlets, other possible sources of pollution, means of control and protection.

23. Population – People and livestock growth rates, projections, industries, institutions affecting demography, immigration and emigration.

24 Sources of information of demography, demand forecasts for population, industries, schools, health facilities and other institutions – studies on local conditions present and future – correlation with W.D. criteria.

25. Socio-economic aspects

26. Economic Analysis
    (a) Economics and Quality of proposed system compared with alternatives – include all relevant items.
    (b) Justification for proposal selected.

(c) Relevancy of rates – construction cost including contingencies and over heads, power, fuel, chemicals, interest rates for present value comparisons, escalation costs, depreciation, staff costs etc.
(d) Maintenance cost – O & M cost rates.
(e) Production costs per m^3 for present, and final stage of design period.
(f) Cost per capita – present and final
(g) Expected revenue – present and final

27. Technical review of proposed system
(a) Selection of intake site – study of siltation and erosion problems, access to site.
(b) Intake structures – levels, variation of capacity with water level at intake.
(c) Dimensions and alignment of raw water mains, comparison and alternatives, running time, capacity surge.
(d) Pumping stations – access, energy supply, economical selection fo site, means of control and operation staff (-houses), foundation, pump installations, stand by, phasing, sump.
(e) Treatment plants – siting including preliminary soil investigations, drainage, justifications for the selected method of treatment, capacity.
(f) Data for dosage of chemicals, coagulants, pH adjustment, prechlorination etc.
(g) Mixing means.
(h) Flocculation – mechanical or other means.
(i) Sedimentation – surface load, detention period, sludge storage
(j) Filters – type of filters, surface load, backwash system, economy.
(k) Balancing tank, pump capacity
(l) Sludge and waste disposal
(m) Access to sites, power supply.
(n) Requirement of staff for proper maintenance, housing of staff.
(o) Possibility of phased implementation and future extensions.

28. Distribution system -
(a) Basis and justification for the proposed lay-out, alignment, cross connections, pressure zones, and economics
(b) Balancing tanks, consumption pattern, peak flows, degree of service
(c) Means of distribution – criteria as per MOWD standards, for location of CWPs, people served and up take rates for various periods and various areas of the project, kiosks, retailers, cattle troughs (-dips), connection to various industries and institutions.
(d) Means and Design Flows for mains, levels of critical sections.
(e) Pipe line material proposed – basis.

29 Optimum scope of the project and phasing, priorities between different areas within the project boundaries and adjacent projects.

30 List of materials, design criteria, material standards

31 Recommendations on further investigations and survey.

32 Maps – Kenya, district, scale, contour intervals, year.

33. Any other relevant aspect concerning the scheme.

D.4: CHECK LIST FOR FINAL DESIGN

The following aspects are to be checked for Final Design when applicable.

1. Whether the agreed additions and alterations to the Preliminary Design have been incorporated.

2. Whether all the other instructions issue have been complied with.

3. Whether any additional investigations were carried out.

4. Whether the summary is sufficiently detailed.

5. Whether the report includes an adequate summary of the Preliminary Design findings.

6. Whether detailed site investigations have been carried out and detailed data acquired and used in the design calculations for the structures involved in the project.

7. Distribution system
   - (a) Basis and calculations for design flow.
   - (b) Pipe materials, pressure classes and characteristics
   - (c) Formula used for frictional flow
   - (d) Allowance for water hammer
   - (e) Location and sizes for air valves, scours, hydrants and sectional valves.
   - (f) Position of Water points – Traditional Community points.
   - (g) Optimised used of balanced zones.
   - (h) Economic calculations of storage volumes for balancing, emergency and breakdown purposes. Comparison with variation n pipe dimensions and pump capacities.
   - (i) Details of thrust and anchor blocks, valve and meter chamber, junctions, air valves and wash outs, markers.
   - (j) Any unnecessary air valves or sectional valves.
   - (k) Minimum and maximum pressure requirements.
   - (l) Utilisation of the existing system.

8. Treatment plant
   - (a) Any additional analysis of water required.

(b) Justification for the proposed method.
(c) Capacity – allowance for water used in the treatment process (see check list Preliminary Design item 26).

9. Control devices for raw water inflow – overflow and inflow quantities within specified limit.

10. Mixing devices – feeder arrangement, dosing equipment, contact period, energy input, chemical mixing equipment, operation aspects.

11. Pretreatment such as pre-chlorination, pH adjustment, aeration, pre-sedimentation.

12. Flocculation
Horizontal flow arrangements – energy inputs, mechanical means, velocities, detention period, number of compartments, sludge outlet.

13. Flocculation
Vertical flow arrangements – velocities, obstruction's flexible bleeding arrangements, sludge bleeding cone bottom outlet.

14. Sedimentation
Load factor, detention time, sludge storage and outlet (scour), inlet arrangements, inlet velocity, outlet troughs – position , velocities, V-notch, length-bottom slope, width-length ratio, sludge treatment.

15. Filtration - (Rapid gravity filters)
Load factor, level control limits for variable headloss filters, media dimensions of supporting bed, filtration media (grain sizes, uniformity coefficient, availability of media) inlet and collection arrangements, bypass for initially filtered water, overflow, free board, airbinder.

16. Backwash system
Capacity rate of period, pressure (-distribution), air timing, means of control, facility or arrangements available for the operator to observe the filters during the backwash operation, outlet and distribution of wash water, capacity in relation to chosen filermedia, calculation of head losses.

17. Filtration – slow sand filters as for item 15.

18. Chlorination
Mixing and dosing equipments, as separate from other functions, corrosion control, corrosion resistant pipes and ventilation.

19. Pipework
Without treatment plant – economic dimensioning, control valves, meters.

20. Storage of treated water, level indicator.

21. Office, store and laboratory
    Ample space, equipment, installation (electrical, plumbing etc) ventilation, waste water disposal, laboratory equipment, furniture.

22. Drainage
    Where necessary under and around structures.

23. Concrete and reinforcement
    Classes, strength requirement, actual load, earth quake factors, cover, lapping, construction joints, movement joints, water bars, amount of reinforcement with regard to static and dynamic loads and cracking.

    (Note special requirements for water retaining structures)

24. Choice of material for cast-in pipes and fittings, flexible joints where necessary, puddle flanges.

25. Proposed protection against corrosion of metal

26. Whether items such as pipes, fittings, valves are to Water Department standards, nominal diameters of pipes proposed.

27. Pumping Station:
    Foundation, plinths (size), vibration, layout of installations, lifting gear, ventilation, cooling and fuel system for engines, control devices, choice of pumps, pump characteristics, rating of engines (influence of altitude etc.) priming, suction head and other hydraulic factors, gate and check valves.

28. Building (General)
    Lay-out, foundation, proposed structural material (economical consideration), roofing materials method of fixing, roof trusses provision against windload earthquake, rainwater disposal, ventilation, treatment of surface dimensions of doors and windows preservation of timber works installations etc.

29. Sewerage systems
    Manholes septic tanks, soak ways.

30. Access roads, parking areas
    (Layout and section drawing) Surface water drainage system and models of disposal. Bends, slopes on roads inside compound – (Provide roads only where needed and footpaths elsewhere)

31. Lighting
    Standards, insulation, wiring, earthing

32. Fences and Gates
Sizes, foundation, materials, painting coats, colours – provision for future extensions.

33. Staff Houses
Adequacy, standards, future extensions.

34 Energy Supply
Supply of electricity – arrangements with EAP&L, availability and distance, tariffs.

35. Dams
(see check list for Preliminary Design items 18) – Larger dams to be checked by independent, water rights and permits.

36. Intakes
(Surface water) – Layout, screens, scour, safety of foundations of protective measures, flood protection, permits
.

37. Intakes.
(Bore Holes) – design of screen, foot valve, casing, lighting gear for submerged pumps, criteria as discusses under item 27 above, permits.

38. Cost Estimates
Construction cost based on actual bill of quantities, Basis of rates – other details as discussed in item 26 © to (g) in check list for Preliminary Design.

39. Drawings.
    (a) Whether all the drawings are fully dimensioned to be useful for constructional purpose.
    (b) Do the drawings contain all the required and relevant information to fully cover all problems likely to occur on site during construction.

40. Draft Operation and Maintenance Manual (See separate list of requirements)

41. Any other relevant aspects.

D.5: CHECK LIST FOR TENDER DOCUMENTS.

The Tender Documents shall comprise of the following items:

1. Conditions of tender and instructions to tenderers

2. Form of tender with appendix and schedule

3. Form of tender surety

4. Form of performance surety undertaking

5. Form of performance bond

6. Form of site certificate

7. Form of security bond (for advance payment, if any)

8. Form of Agreement

9. Conditions of contract for works of Civil Engineering Construction (FIDIC)

10. Conditions of contract Part II – Conditions of particular application and amendments to Part I General Conditions MOWD.

11. Conditions of contract Part III – Conditions of particular application and amendments to Part II General Conditions.

12. Standard Specification.

13. Special Specification

14. Bills of Quantities incl. Preamble, Schedule of Day Works and Schedule of Basic prices.

15. Drawings

    The "Conditions of tender and instructions to tenders" shall include the following headlines:-

    1 Invitation
    2 Documents
    3 Tender
    4 Tender surety
    5 Performance bond
    6 Variation of tender
    7 Incomplete tender
    8 Duration of tender
    9 Rejection of tender

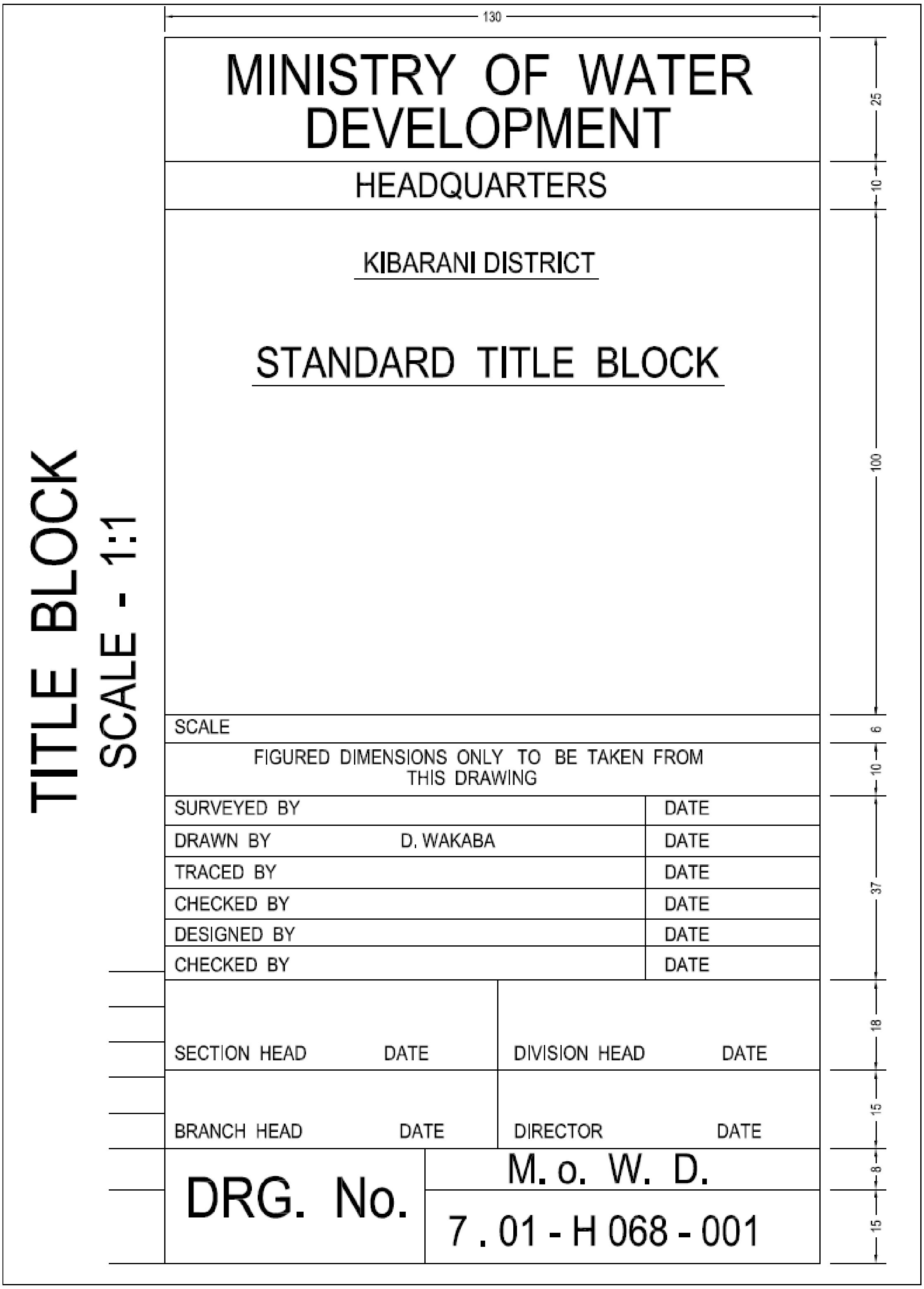
10 Award of contract
11 Service of notice
12 Documents confidential
13 Explanations of documents
14 Completion of tender
15 Errors in tender
16 Site visit
17 Submission of tender
18 Expense of tendering
19 Training levy
20 Price variation
21 Preliminary design
22 Alternative designs
23 Quantities approximate
24 Any additional information

Schedules shall include:-

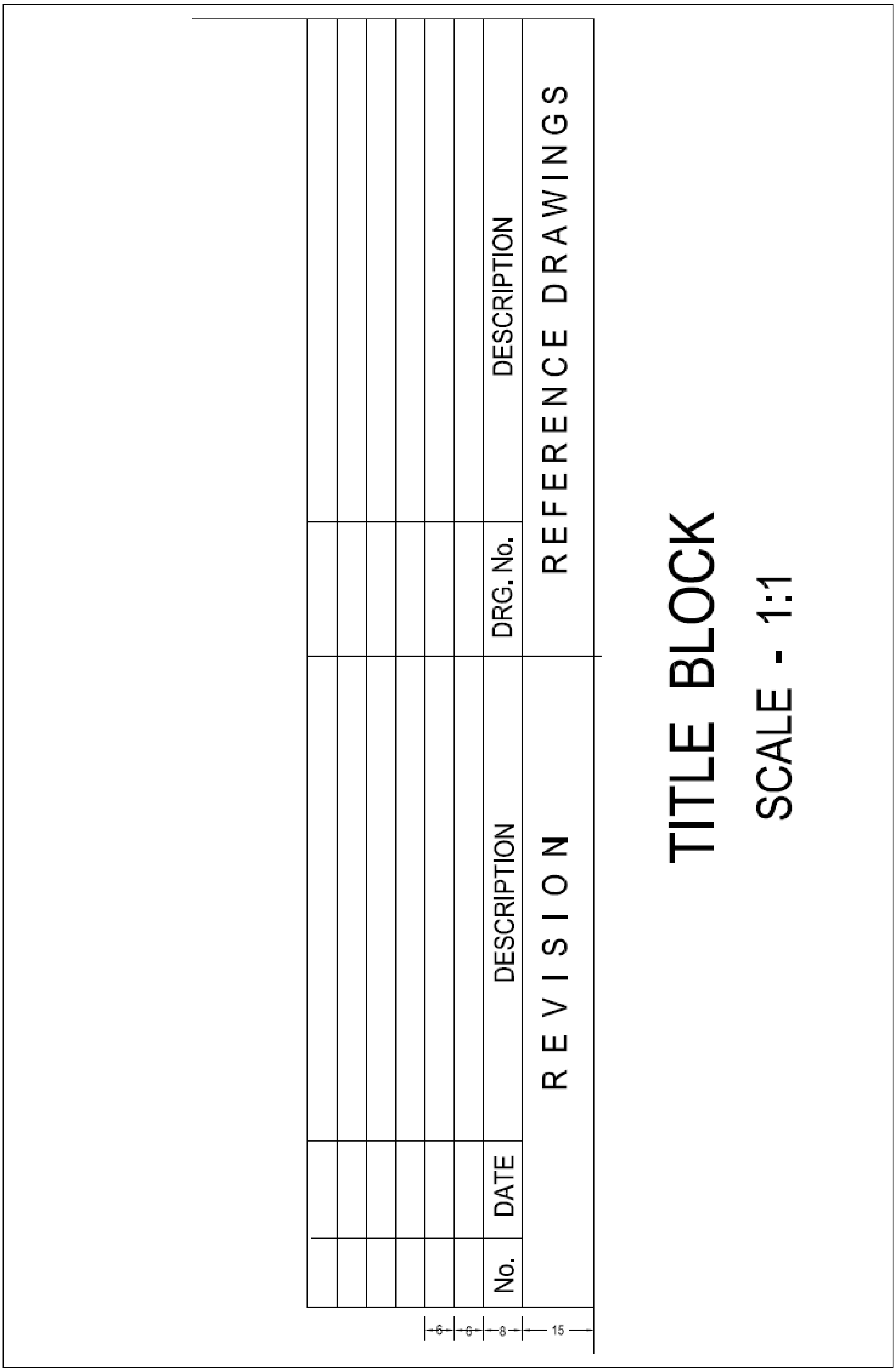
(a) Major items of plant
(b) Supervisory staff
(c) Construction programme
(d) Insurance (s)
(e) Sub-contractors
(f) Alternative standard (only in exceptional cases)
(g) works executed and references
(h) Any other schedule.

APPENDIX F:

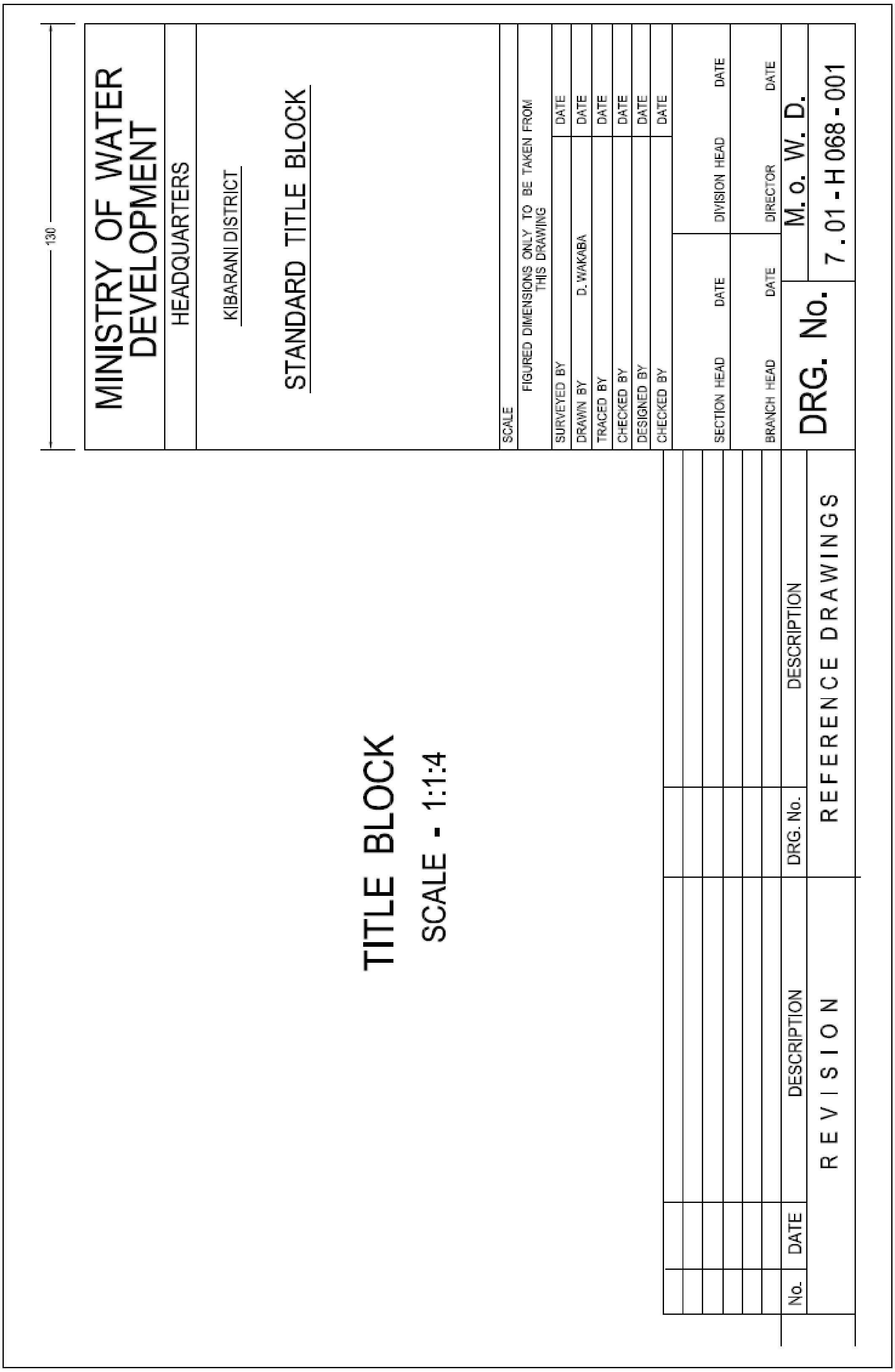
F.1.1: TITLE BLOCKS FOR DRAWINGS



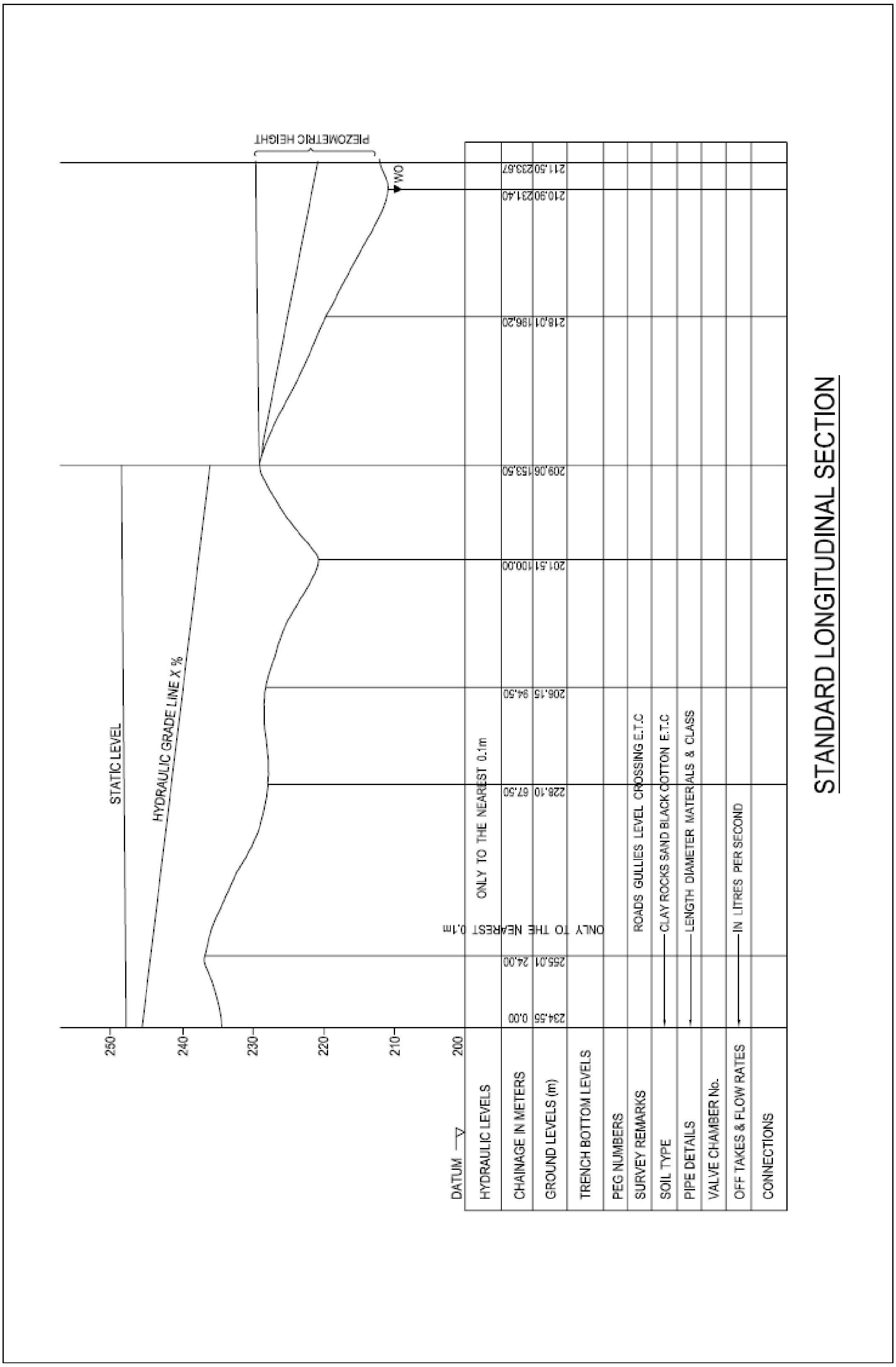
F.1.2: TITLE BLOCKS FOR DRAWINGS



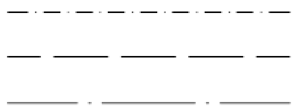
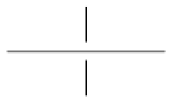
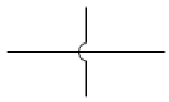
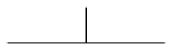
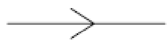
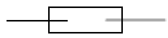
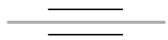
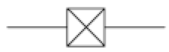
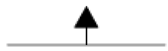
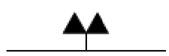
F.1.3: TITLE BLOCKS FOR DRAWINGS

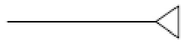
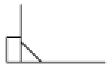
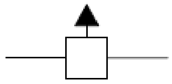
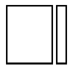
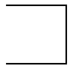
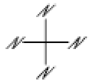
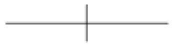
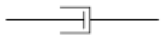
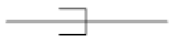


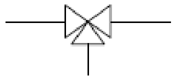
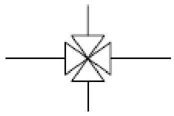
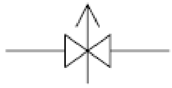
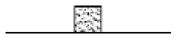
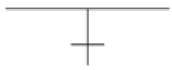
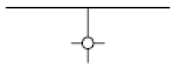
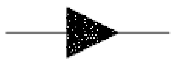
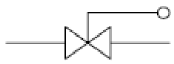
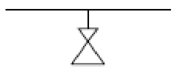
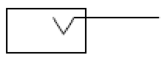
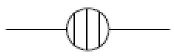
F.1.4: TITLE BLOCKS FOR DRAWINGS

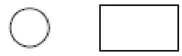
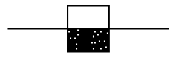
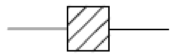
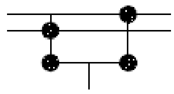
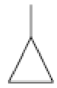
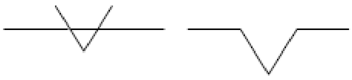
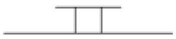
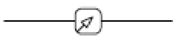
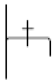
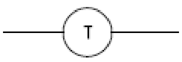
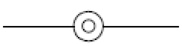


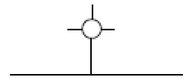
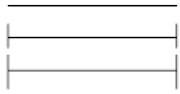
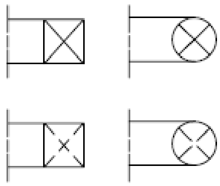
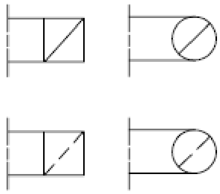
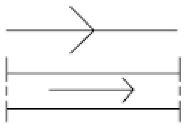
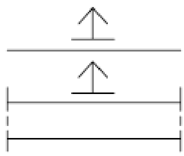
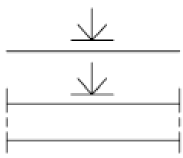
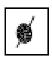
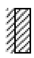
F.2: SYMBOLS FOR WATER SUPPLY, PLUMBING, HEATING VENTILATION AND DUCTING

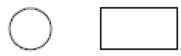
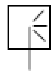
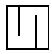
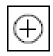
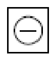
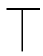
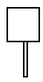
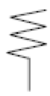
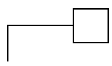
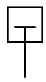
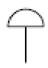
No.	DESCRIPTION	SYMBAL
1	Pipe, general symbol	
1.1	Method A: the symbols indicate the location of the pipe in ralation to the section visible at section concealed at section in front of or above section	
1.2	Method B: the symbols indicate the nature of fluids	
1.3	Crossing pipe, not connected	
1.4	Crossing pipe, not connected	
1.5	Tee – Reducing Tee – Equal	 
1.6	Flexible pipe; hose	
1.7	Direction of flow	
1.8	Direction of fall	
1.9	Expansion joint, general symbol	
1.10	Cap nut	
1.11	Sliding support	
1.12	Anchor point	
1.13	Air release device a) single air valve b) double air valve	 

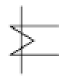
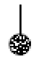
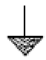
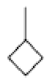
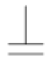
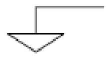
No.	DESCRIPTION	SYMBAL
1.14	End plug	
1.15	Reducer Taper	
1.16	Bends	
1.17	Clean - out bend	
1.18	Elbow	
1.19	Surge arrestor	
1.20	Union	
1.21	Bush - equal	
1.22	Nipple - equal	
2	JOINTS	
2.1	Joints, general symbol	
2.2	Spigot and socket	
2.3	Flange	
2.4	Collar	
2.5	Flexible socket	
2.6	Welded or cement joint	
2.7	Threaded joint male, female	
2.8	Stapped coupling joint	
3	VALVES:	
3.1	Valve, general symbol: also used for shut - off and regulating valve, two - way	

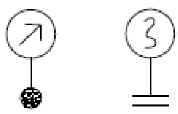
No.	DESCRIPTION	SYMBAL
3.2	Shut-off and regulating valve, three way	
3.3	Shut-off and regulating valve, four way	
3.4	Non - return valve (flow direction is from the non-shaded to shaded part of the symbol)	
3.5	Safety valve or pressure release valve	
3.6	Safety valve (small triangle: high pressure)	
3.7	Vacuum breaker	
3.8	Draw - off	
3.9	Hydrant, general symbol	
3.10	Sprinkler head	
3.11	Constant flow valve	
3.12	Float valve	
3,13	Soour valve or wash out	
4	GULLIES	
4.1	Gully, general symbol	
4.2	Gully, with trap	
4.3	Separator, general symbol	

No.	DESCRIPTION	SYMBAL
5.1	Apparatus, general symbol (It is preferred that the circular symbol be used for apparatus in which there are rotating parts: for other apparatus the rectangular symbol is used	
5.2	Pump	
5.3	Steam trap	
5.4	Strainer, general symbol	
5.5	Heater	
5.6	Mixing tap	
5.7	Shower	
5.8	Trap Views Section	 
5.9	Drain and Inspection opening	
5.10	Water meter	
5.11	Trap	
6	STRUCTURES FOR WATER SUPPLY	
6.1	Intakes	
6.2	Treatment works	
6.3	Pumping station	
6.4	Storage tanks	
6.5	Break pressure tanks	

No.	DESCRIPTION	SYMBAL
6,6	Communal water points	
6.7	Bore hole with ref. No.	
7	SYMBOL FOR VENTILATION AND AIR CONDITIONING INSTALLATIONS DUCTA	
7.1	Duct, general symbol	
7.2	Supply duct — Visible section Hidden section	
7.3	Exhaust duct — Visible section Hidden section	
7.4	Direction of flow	
8	APPLIANCES AND FITTINGS	
8,1	Air supply fittings	
8,2	Air exhaust fitting	
8,3	Damper	
8,4	Air grill	

No.	DESCRIPTION	SYMBAL
8.5	Apparatus, general symbol (it is preferred that the circular symbol be used for apparatus in which there are rotating parts; for other apparatus the rectangular symbol is used	
8.6	Pan	
8.7	Air filter	
8.8	Humidifier	
8.9	Silencer	
8.10	Air heater	
8.11	Air Cooler	
9	SYMBOLS FOR CONTROL AND REGULATION COMPONENTS	
9.1	Hand - operated, general symbol	
9.2	Automatic, general symbol	
9.3	Springs	
9.4	Weight	
9.5	Float	
9.6	Piston	
9.7	Diaphram	
9.8	Electric motor	

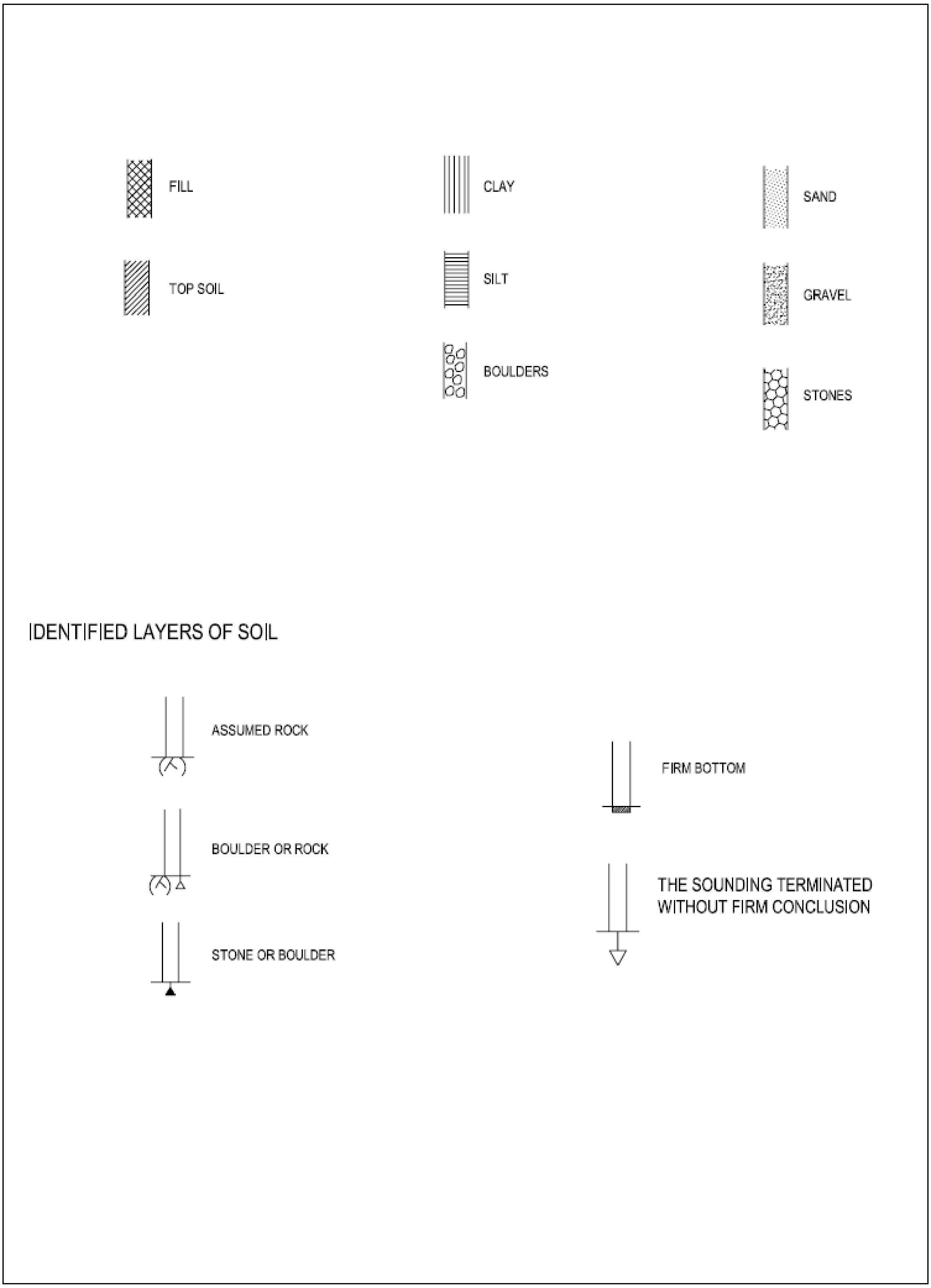
No.	DESCRIPTION	SYMBAL
9,9	Solonoid	
10	SYMBOL FOR SENSING ELEMENTS [1]	
10,1	Temperature	
10,2	Pressure	
10,3	Flow	
10,4	Humidity	
10,5	Level	
11	SYMBOL FOR INDICATING GAUGES, METERS AND RECORDERS [1]	
11,1	Indicating gauge or meter	
11,2	Recorder	



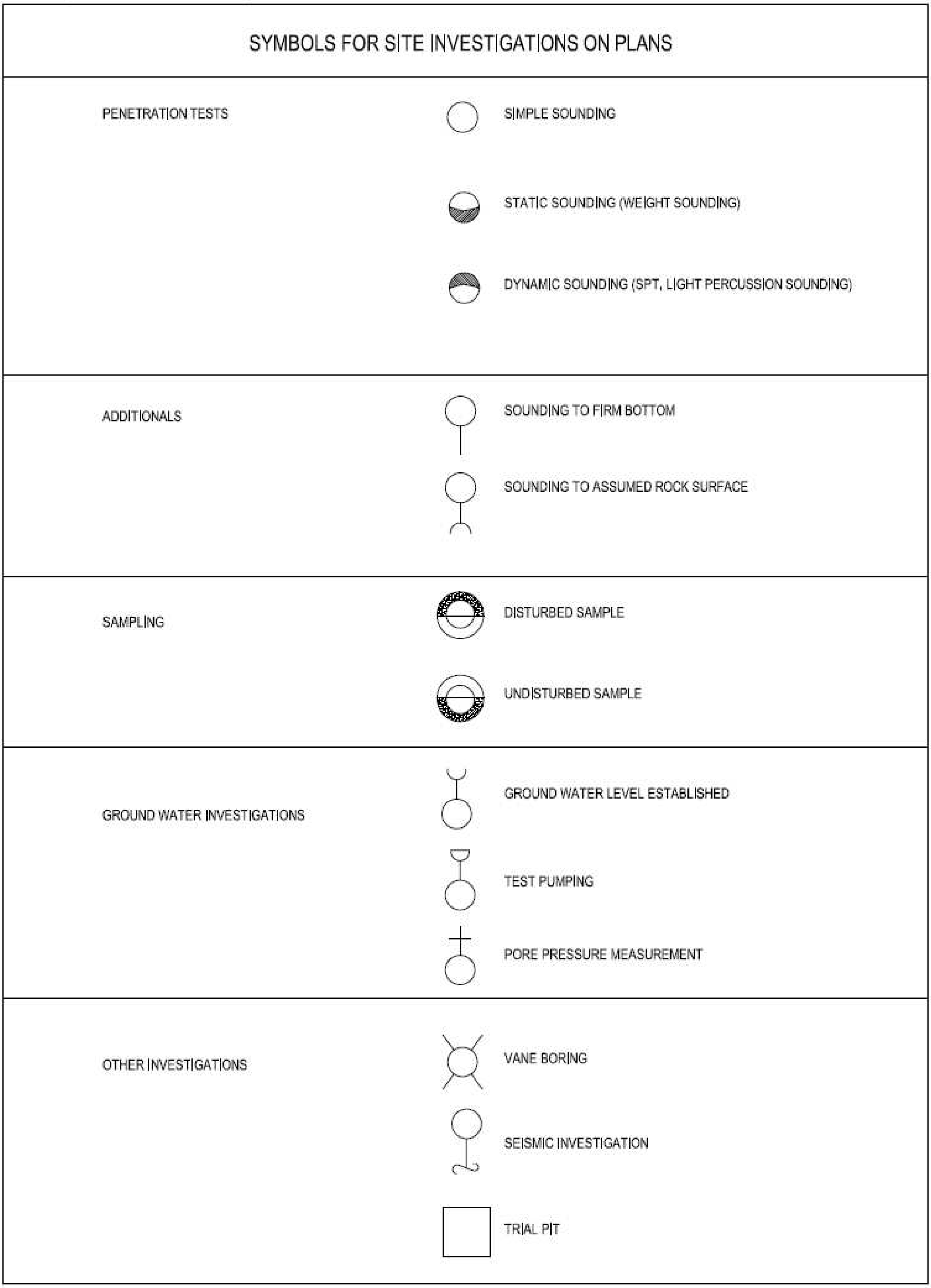
1) By combining the symbols of clauses 5 & 6, different types of indicating and recording instruments can be illustrated

F.3: SYMBOLS FOR GROUND INVESTIGATIONS

F.3.1: Identified layers of Soil



F.3.2: Symbols for Site Investigations on Plans



F.4: SYMBOLS FOR CONCRETE REINFORCEMENT

INTERNATIONAL STANDARD　　　　ISO 3766 - 1977 (E)

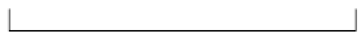
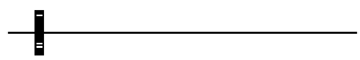
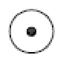
Building and civil engineering drawing - Symbols for concrete reinforcement

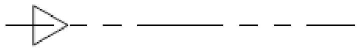
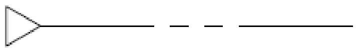
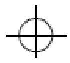
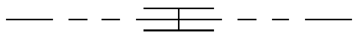
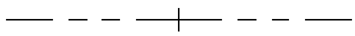
1, SCOPE AND FIELD OF APPLICATION

This International Standard esterblishes a system of symbols for use on drawings for reinforcement in reinforced concrete and in prestressed concrete

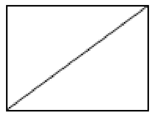
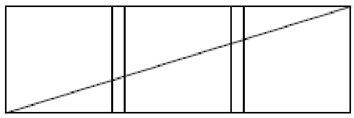
2, GRAPHICAL SYMBOLS

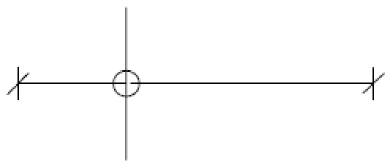
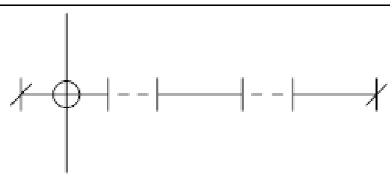
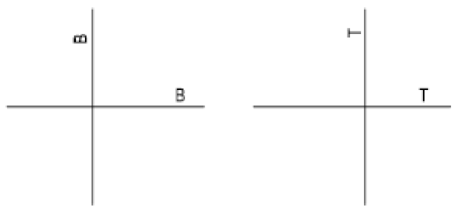
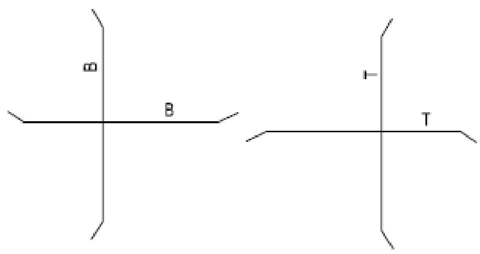
2.1, Ordinary reinforcement

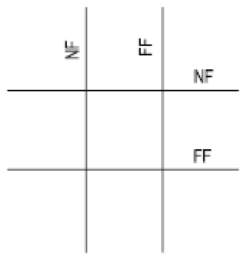
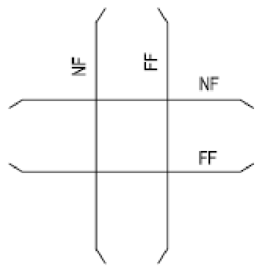
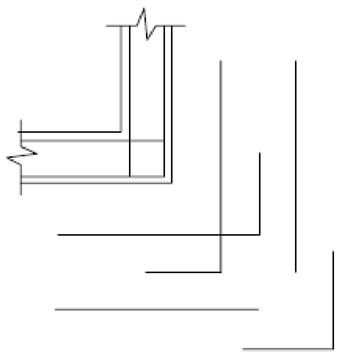
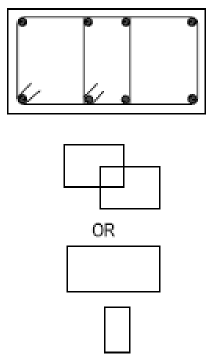
No.	DESIGNATION AND DESCRIPTION	SYMBAL
2.1.1	Reinforcing bar, continuous extra – thick line	
2.1.2	Section of reinforcement bar	
2.1.3	Bar with end anchorages a) with hooks b) with right angle bends	 
2.1.4	Bar without end anchorages If necessary to Indicate ends of the bar where bars are not separated on the drawing	 
2.1.5	Anchorage ring or plate	
2.1.6	End view of anchorage	
2.1.7	Bar bent at right angle away from the reader Alternatively for clarity where bars are very close and for microfilming	
2.1.8	Bar bent at right angle away from the reader Alternatively for clarity where bars are very close and for microfilming	

No.	DESIGNATION AND DESCRIPTION	SYMBAL
2.2.1	Prestressing bar or cable, long chain double - dashed extra thick line	
2.2.2	Section of post - tensioned reinforcement in pipes or conduits	
2.2.3	Section of prestressed reinforcement	
2.2.4	Anchorage at tensioning end [1)]	
2.2.5	Fixed anchorage [1)]	
2.2.6	End view anchorage [1)]	
2.2.7	Movable splice [1)]	
2.2.8	Fixed splice [1)]	

WELDED FABRICKS

No.	DESIGNATION	SYMBAL
2.3.1	One sheet of fabric, shown on plan	
2.3.2	Identical sheets of fabric in a row	

No.	CONVENTION	SYMBAL
3.1	Bends shall normally be drawn to scale Bends with the smallest permitted bend radius may be drawn with intersecting straight lines	 
3.2	A bundle of bars may be drawn with a single line, end markings indicating the number of bars in the bundle Example : Bundle with three Indentical bars	
3.3	Each set of identical bars, stirrups or links shall be indicated by one bar, stirrup or link drawn with continous extrathick lines, with a continous thin line across the set terminated by short oblique lines to mark the extreme bars, stirrups or links A circle drawn with a continuous thin line connects the "set line" with the correct bar, stirrups or link	
3.4	Bars placed in groups, each group spaced over the same distance and containing an identical number of identical bars, may be indicated as shown in the figure	
3.5	Two-way reinforcement shall be shown in section, or marked with text or symbol in order to show the direction of bars in the outside layer on each face of the construction in the plan or elevation	
3.6	On plan drawing for simple arrangements the top - layer and bottom - layer reinforcement shall have letters indicating the location of the layer added to the symbol If end marks are used, the end marks shall be drawn upwards or to the left for the bottom - layer and downwards or to the right for the top layer (B : bottom T : top)	 

No.	CONVENTION	SYMBAL
3.7	On elevations of walls with reinforcement on both faces, the reinforcement shall have letters added to the symbol, indicating the location of layer If end marks are used, the end marks shall be drawn upwards or to the left for far face reinforcement, and downwards or to the right for near face reinforcement (NF: near face FF: far face)	 
3.8	If the arrangement of the reinforcement is not clearly shown by the section, an additional sketch showing the reinforcement may be drawn outside the section	
3.9	All the types of stirrups or links present shall be indicated on the drawing. If the arrangement is complicated, it may be clarified by the aid of a sketch in connection with the notation	

F.5: BAR SCHEDULING

INTERNATIONAL STANDARD ISO 4066-1977 (E)

BUILDING AND CIVIL ENGINEERING DRAWINGS - BAR SCHEDULING

0 Introduction

The purpose of this International Standard is to ensure uniformity of practice in the scheduling of steel bars for the reinforcement of concrete. To establish a clear and unambiguous system for scheduling, it is necessary to specify the method of indicating dimensions to be used and the order in which the information is given on the bar schedule.

As the use of preferred shapes I considered to be very advantageous both for simplifying design and manufacture and for the use of computers, the opportunity has been taken to include a list of preferred shapes and a coding system; the layout of the bar schedule is based on the use of preferred shapes.

1. Scope

This International Standard establishes a system for the scheduling of reinforcing bars, and comprises:-

- The method of indicating dimensions
- A coding system for the bar shapes
- A list of preferred shapes
- The bar schedule

2. Field of application

This International Standard applies to all types of steel bar for the reinforcement of concrete.

Steel fabric and prestressing steel reinforcement are excluded.

3. Methods of indicating bending dimensions

The bending dimensions shall be indicated as showing figures 1 to 5.

Dimensions shall be outside dimensions, except for radius and the standard radius of bend shall be the smallest radius permitted by national standards or regulations.

The total length (cutting length) shall be calculated on the basis of the appropriate bending dimensions with connection for bends and allowances for anchorages.

4. Notations.

Items of information concerning reinforcement shall be written in the longitudinal direction of the bars or along reference lines indicating the bars in question.

4.1 The following information concerning reinforcing bars shall be given:

(a) number
(b) size
(c) quality
(d) length
(e) Spacing in millimeters
(f) bar reference number
(g) location in slab or wall

4.2 The following information concerning bundles of reinforcing bars shall be given:

(a) number of bundles
(b) number of bars in a bundle
(c) size
(d) quality
(e) length
(f) bar reference number
(g) spacing of bundles in millimeters
(h) location

4.3 Items of information for welded fabric shall be written along the diagonal line. The number of sheets for fabric shall be indicated together with the fabric type reference.

4. CODING SYSTEM FOR BAR SHAPES

The shape code number consists of two or, if essential, three or four characters, as defined in table 1.

Table 1 - Code number composition

First character	Second character	Third character	Fourth character
0 – No bends (optional)	0-Straight bars (optional	0-No end anchorage (optional)	S-Where a national standard specifies a special radius of bend (for example stirring links) this shall be indicated by use of the characters.)
1-1 bend	1-90^0 bend(s) of standard radius all bent in the same direction	1-end anchorage at one end, as defined in national standards	
2-2 bends	2-90^0 bend(s) of non-standard radius, all bent in the same direction	2-end anchorages at both ends, as defined in national standards	
3-3 bends	3-180^0 bend(s) of non-stranded radius, all bent in the same direction		
4-4 bends	4-90^0 bends of standard radius not all bent in the same direction		
5-5 bends	5-bends $<90^0$, all bent in the same direction		
6 – Arcs of circles	6-bends $<90^0$, not all bent in the same direction		
7-Helices	7-Arcs or helices		
81-89 – shapes defined in national standards			
99 - Special non standard shapes defined by a sketch. It is recommended that codes shapes 99 for all non-standard shapes be used. However, the numbers 91 to 99 are available for countries which require more than one number for special shapes.			

NOTE: This table explains the logic behind the numbering of the shapes in table 2. It is not to be used for making up codes of additional shapes.

5. List of preferred shapes

When a third character is used, the direction of the end anchorages shall be as shown by the dotted lines in the examples in table 2.

It is recognized that in some countries hooks are used to end anchorages.

The letter symbols refer to the dimensions which shall be given in the bar schedule.

6. Bar scheduling

The bar schedules is the document used to specify and identify reinforcing bars. The format specified below incorporates the use of preferred shapes.

6.1 Information content

A bar schedule shall contain the following information in the sequence listed below:

(a) member – identification of the structural member in which the bar is located;
(b) bar mark – unique reference of the bar;
(c) type of steel;
(d) diameter of bar;
(e) length of each bar (cutting length, allowing for loss or gain at bends, calculated from the dimensions and radii given in k); see clause 3);
(f) number of members;
(g) number of bars in each member;
(h) total number of bars (f) x (g);
(i) total length (e) x (h);
(j) shape code (as defined in clause 5);
(k) bending dimensions;
(I) revision letter
(m) title block

An example of a form of bar schedule is shown on page 7.

6.2 Special shapes

When special shapes are required, these shall be shown by a dimensioned sketch drawn in the space normally used for bending dimensions

6.3 Title block

The title block shall be placed below the schedule, and shall contain the following information:

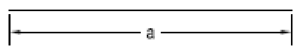
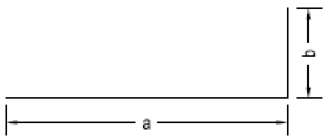
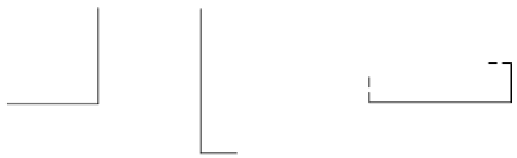
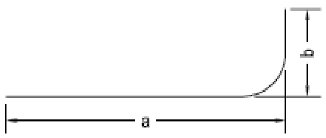
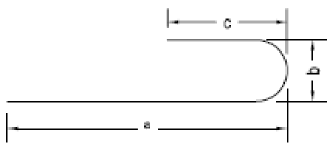
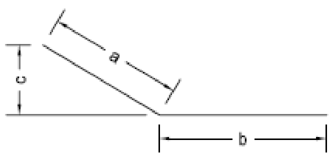
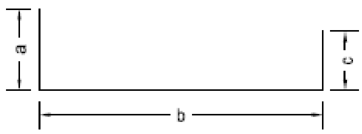
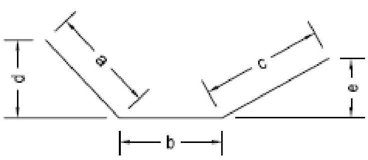
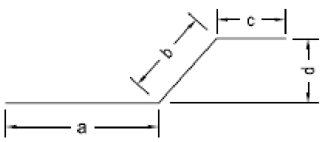
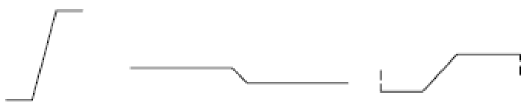
(a) name of the structural designer;
(b) title of the project;
(c) date prepared;
prepared by
checked by

(d) drawing number;
(e) bar schedule reference;
(f) revision letter and date of last revision;

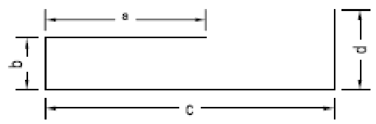
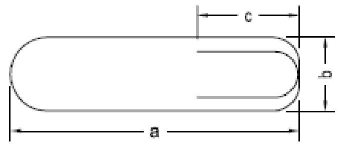
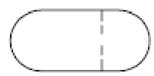
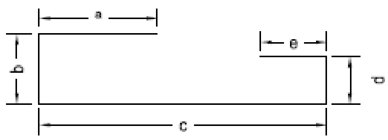
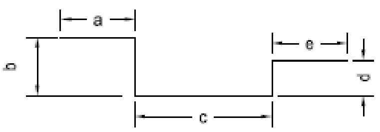
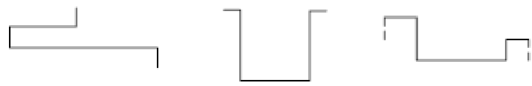
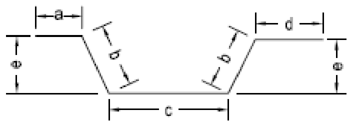
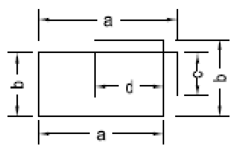
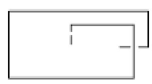
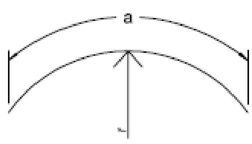
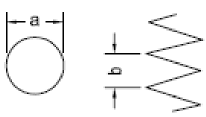
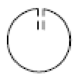
(g) a statement that the schedule has been prepared in accordance with the requirements of ISO 4066.

7. Summary sheet
If required, summary sheets may be used; separate sheets shall be used for each type of steel.

TABLE 2 – preferred shapes

SHAPES CODE	SHAPES	EXAMPLES
08		
11		
12		
13		
15		
14		
16		
17		

preferred shapes

SHAPES CODE	SHAPES	EXAMPLES
31		
33		
41		
44		
46		
51		
67		
77		

preferred shapes

ISO 4066 - 1977 (E)

BENDING DIMENSIONS

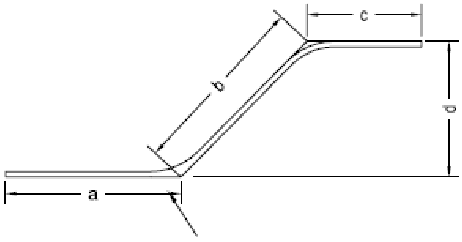


FIGURE 1

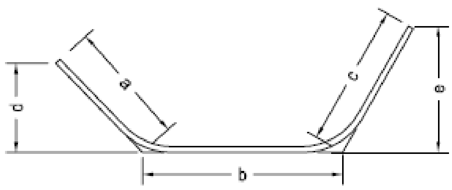


FIGURE 2

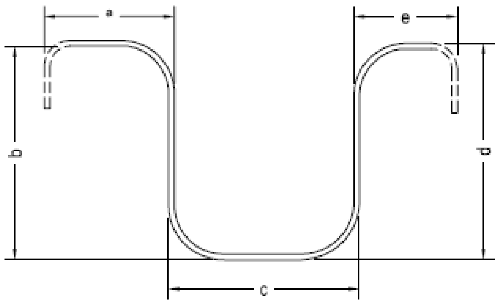


FIGURE 3

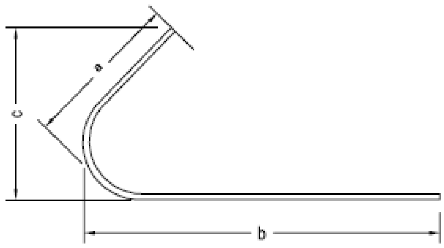


FIGURE 4

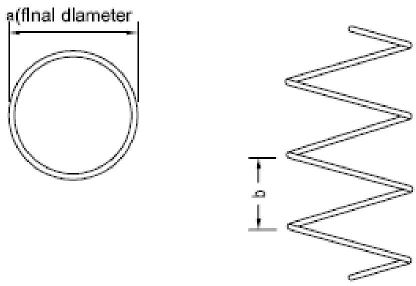


FIGURE 5

C: NUMBER OF COMPLETE TURNS

Example of ISO bar schedule - expression in millimeters

Member	Bar mark	Type of steel	Diameter	Length of bench bar	Number of members	Number of bars in each member	Total number	Total length	shape code				Rending dimensions					Revision letter
													a	b	c	d	e/f	

A, B, CED and Partners ZXY Street, London, WIA Phone : 01-000-0000	(TITLE OF PROJECT)	Preparation date Prepared by Checked by	Drawing number	Bar schedule reference	Revision date Revision letter

Schedule has been prepared in accordance with the requirements of ISO 4066.

APPENDIX G:

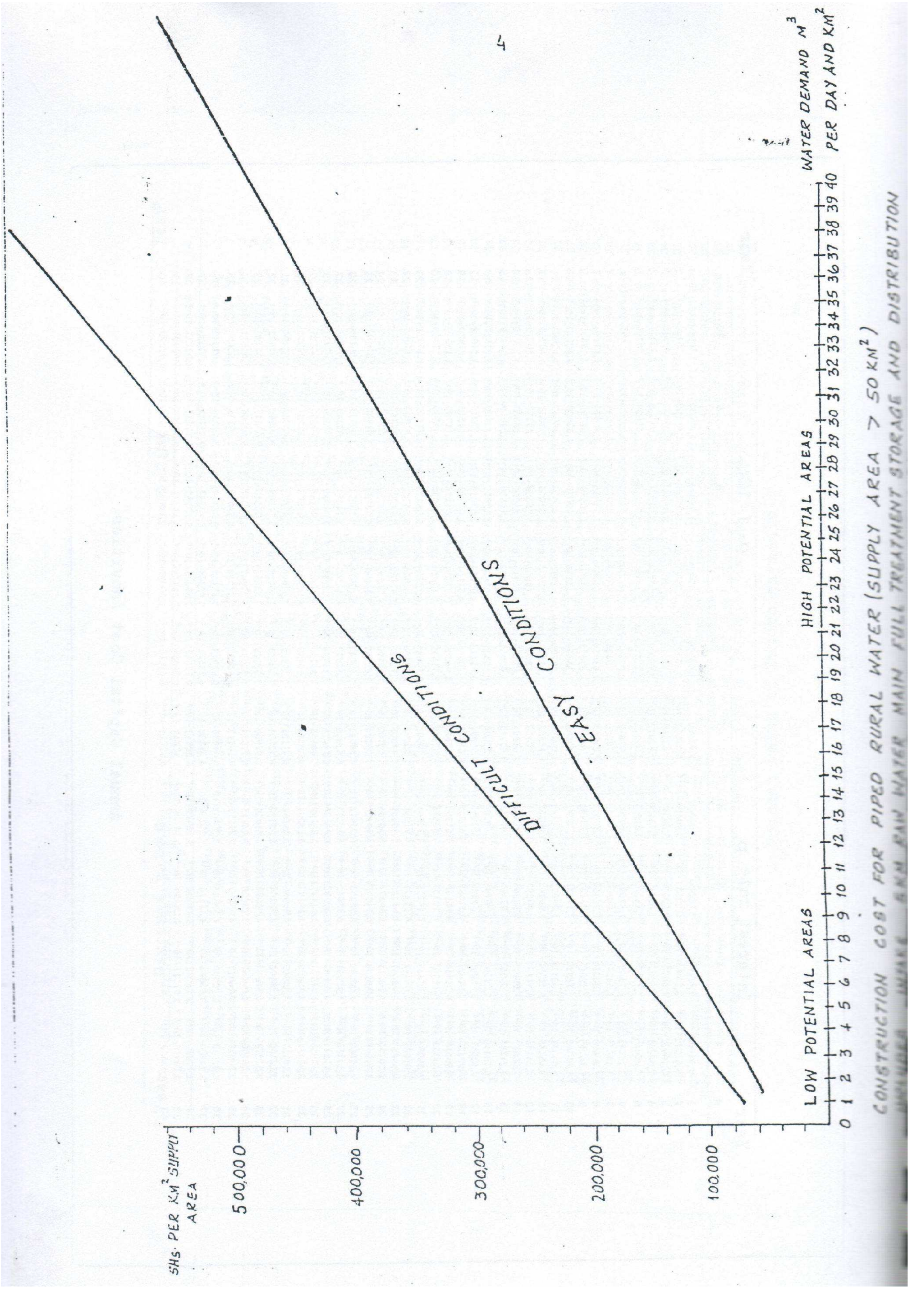
G.1 PRESENT VALUE OF FUTURE PAYMENTS

Year	Discount Rate P					P= 1 - 10%					Year
n	1	2	3	4	5	6	7	8	9	10	n
1	0,990099	0,980392	0,970874	0,961538	0,952381	0,943396	0,934579	0,925926	0,917431	0,909091	1
2	0,980296	0,961169	0,942596	0,924556	0,907029	0,889996	0,873439	0,857339	0,841680	0,826446	2
3	0,970590	0,942322	0,915142	0,888996	0,863838	0,839619	0,816298	0,793832	0,772183	0,751315	3
4	0,960980	0,923845	0,888487	0,854804	0,822702	0,792094	0,762895	0,735030	0,708425	0,683013	4
5	0,951466	0,905731	0,862609	0,821927	0,783526	0,747258	0,712986	0,680583	0,649931	0,620921	5
6	0,942045	0,887971	0,837484	0,790315	0,746215	0,704961	0,666342	0,630170	0,596267	0,564474	6
7	0,932718	0,870560	0,813092	0,759918	0,710681	0,665057	0,622750	0,583490	0,547034	0,513158	7
8	0,923483	0,853490	0,789409	0,730690	0,676839	0,627412	0,582009	0,540269	0,501866	0,466507	8
9	0,914340	0,836755	0,766417	0,702587	0,644609	0,591898	0,543934	0,500249	0,460428	0,424098	9
10	0,905287	0,820348	0,744094	0,675564	0,613913	0,558395	0,508349	0,463193	0,422411	0,385543	10
11	0,896324	0,804263	0,722421	0,649581	0,584679	0,526788	0,475093	0,428883	0,387533	0,350494	11
12	0,887449	0,788493	0,701380	0,624597	0,556837	0,496969	0,444012	0,397114	0,355535	0,318631	12
13	0,878663	0,773033	0,680951	0,600574	0,530321	0,468839	0,936208	0,367698	0,326179	0,289664	13
14	0,869963	0,757875	0,661118	0,577475	0,505068	0,442301	0,387817	0,340461	0,299246	0,263331	14
15	0,861349	0,743015	0,641862	0,555265	0,481017	0,417265	0,362446	0,315242	0,274538	0,239392	15
16	0,852821	0,728446	0,623167	0,533908	0,458112	0,393646	0,338735	0,291890	0,251870	0,217629	16
17	0,844377	0,714163	0,605016	0,513373	0,436297	0,371364	0,316574	0,270269	0,231073	0,197845	17
18	0,836017	0,700159	0,587395	0,493628	0,415521	0,350344	0,295864	0,250249	0,211994	0,179859	18
19	0,827740	0,686431	0,570286	0,474642	0,395734	0,330513	0,276508	0,231712	0,194490	0,163508	19
20	0,819544	0,672971	0,553676	0,456387	0,376889	0,311805	0,258419	0,214548	0,178431	0,148644	20
21	0,811430	0,659776	0,537549	0,438834	0,358942	0,294155	0,241513	0,198656	0,163698	0,135131	21
22	0,803396	0,646839	0,521893	0,421955	0,341850	0,277505	0,225713	0,183941	0,150182	0,122846	22
23	0,795442	0,634156	0,506692	0,405726	0,325571	0,261797	0,210947	0,170315	0,137781	0,111678	23
24	0,787566	0,621721	0,491934	0,390121	0,310068	0,246979	0,197147	0,157699	0,126405	0,101526	24
25	0,779768	0,609531	0,477606	0,375117	0,295303	0,232999	0,184249	0,146018	0,115968	0,092296	25
26	0,772048	0,597579	0,463695	0,360689	0,281241	0,219810	0,172195	0,135202	0,106393	0,083905	26
27	0,764404	0,585862	0,450189	0,346817	0,267848	0,207368	0,160930	0,125187	0,097608	0,076278	27
28	0,756836	0,574375	0,437077	0,333477	0,255094	0,195630	0,150402	0,115914	0,089548	0,069343	28
29	0,749342	0,563112	0,424346	0,320651	0,242946	0,184557	0,140563	0,107328	0,082155	0,063039	29
30	0,741923	0,552071	0,411987	0,308319	0,231377	0,174110	0,131367	0,099377	0,075371	0,057309	30
31	0,734577	0,541246	0,399987	0,296460	0,220359	0,164255	0,122773	0,092016	0,069148	0,052099	31
32	0,727304	0,530633	0,388337	0,285058	0,209866	0,154957	0,114741	0,085200	0,063438	0,047362	32
33	0,720103	0,520229	0,377026	0,274094	0,199873	0,146186	0,107235	0,078889	0,058200	0,043057	33
34	0,712973	0,510028	0,366045	0,263552	0,190355	0,137912	0,100219	0,073045	0,053395	0,039143	34
35	0,705914	0,500028	0,355383	0,253415	0,181290	0,130105	0,093663	0,067635	0,048986	0,035584	35
36	0,698925	0,490223	0,345032	0,243669	0,172657	0,122741	0,087535	0,062625	0,044941	0,032349	36
37	0,692005	0,480611	0,334983	0,234297	0,164436	0,115793	0,081809	0,057986	0,041231	0,029408	37
38	0,685153	0,471187	0,325226	0,225285	0,156605	0,109239	0,076457	0,053690	0,037826	0,026735	38
39	0,678370	0,461948	0,315754	0,216621	0,149148	0,103056	0,071455	0,049713	0,034703	0,024304	39
40	0,671653	0,452890	0,306557	0,208289	0,142046	0,097222	0,066780	0,046031	0,031838	0,022095	40

G.2: ANNUAL CAPITAL COST (ANNUATIES)

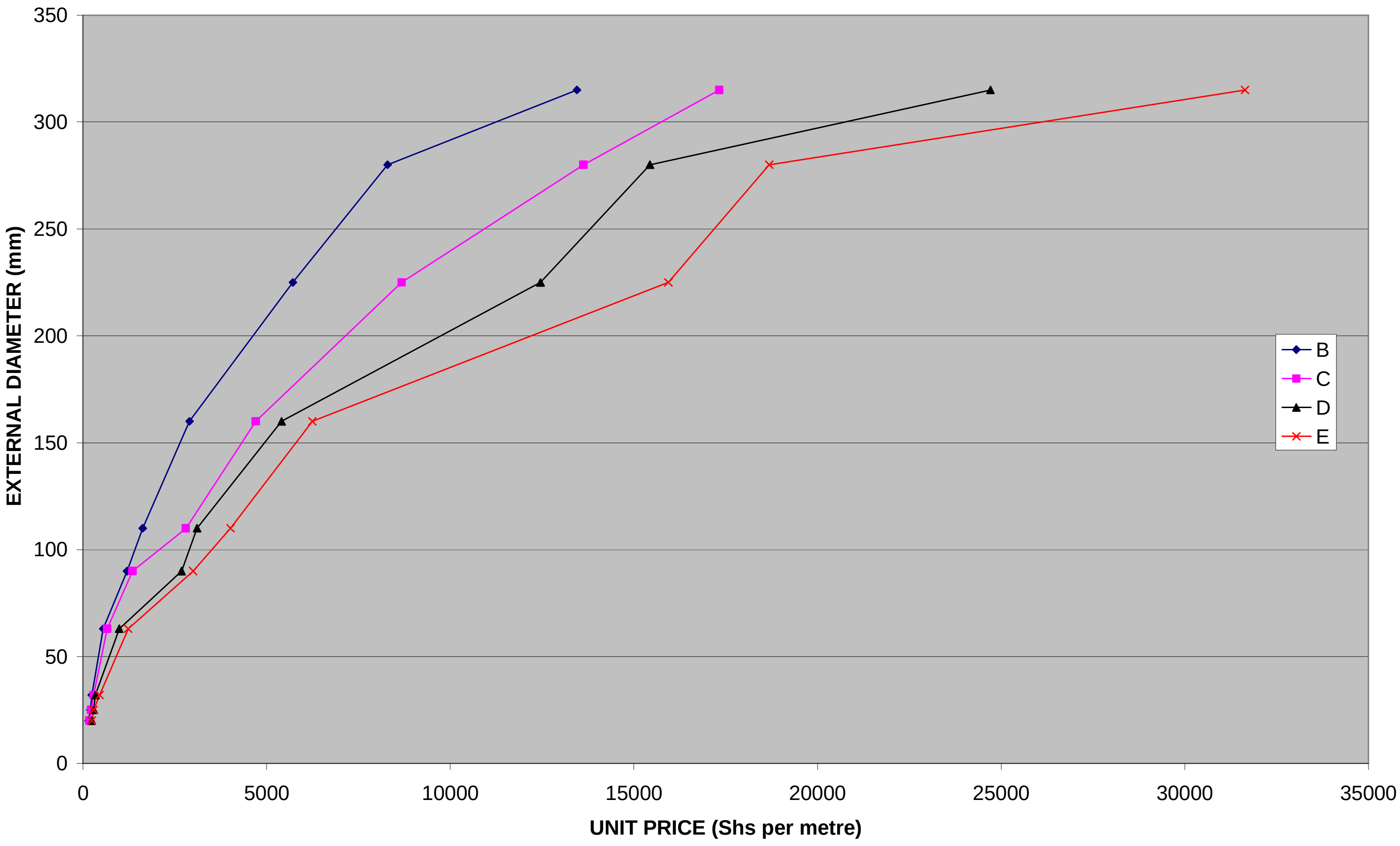
Year	Discount Rate P					P= 1 - 10%					Year
n	1	2	3	4	5	6	7	8	9	10	n
1	1,010000	1,020000	1,030000	1,040000	1,050000	1,060000	1,070000	1,080000	1,090000	1,100000	1
2	0,507512	0,515050	0,522611	0,530196	0,537805	0,545437	0,553092	0,560769	0,568469	0,576190	2
3	0,340022	0,346755	0,353530	0,360349	0,367209	0,374110	0,381052	0,388034	0,395055	0,402115	3
4	0,256281	0,262624	0,269027	0,275490	0,282012	0,288591	0,295228	0,301921	0,308669	0,315471	4
5	0,206040	0,212158	0,218355	0,224627	0,230975	0,237396	0,243891	0,250456	0,257092	0,263797	5
6	0,172548	0,178526	0,184598	0,190762	0,197017	0,203363	0,209796	0,216315	0,222920	0,229607	6
7	0,148628	0,154512	0,160506	0,166610	0,172820	0,179135	0,185553	0,192072	0,198691	0,205405	7
8	0,130690	0,136510	0,142456	0,148528	0,154722	0,161036	0,167468	0,174015	0,180674	0,187444	8
9	0,116740	0,122515	0,128434	0,134493	0,140690	0,147022	0,153486	0,160080	0,166799	0,173641	9
10	0,105582	0,111327	0,117231	0,123291	0,129505	0,135868	0,142378	0,149029	0,155820	0,162745	10
11	0,096454	0,102178	0,108077	0,114149	0,120389	0,126793	0,133357	0,140076	0,146947	0,153963	11
12	0,088849	0,094560	0,100462	0,106552	0,112825	0,119277	0,125902	0,132695	0,139651	0,146763	12
13	0,082415	0,088118	0,094030	0,100144	0,106456	0,112960	0,119651	0,126522	0,133567	0,140779	13
14	0,076901	0,082602	0,088526	0,094669	0,101024	0,107585	0,114345	0,121297	0,128433	0,135746	14
15	0,072124	0,077825	0,083767	0,089941	0,096342	0,102963	0,109795	0,116830	0,124059	0,131474	15
16	0,067945	0,073650	0,079611	0,085820	0,092270	0,098952	0,105858	0,112977	0,120300	0,127817	16
17	0,064258	0,069970	0,075953	0,082199	0,088699	0,095445	0,102425	0,109629	0,117046	0,124664	17
18	0,060982	0,066702	0,072709	0,078993	0,085546	0,092357	0,099413	0,106702	0,114212	0,121930	18
19	0,058052	0,063782	0,069814	0,076139	0,082745	0,089621	0,096753	0,104128	0,111730	0,119547	19
20	0,055415	0,061157	0,067216	0,073582	0,080243	0,087185	0,094393	0,101852	0,109546	0,117460	20
21	0,053031	0,058785	0,064872	0,071280	0,077996	0,085005	0,092289	0,099832	0,107617	0,115624	21
22	0,050864	0,056631	0,062747	0,069199	0,075971	0,083046	0,090406	0,098032	0,105905	0,114005	22
23	0,048886	0,054668	0,060814	0,067309	0,074137	0,081278	0,088714	0,096422	0,104382	0,112572	23
24	0,047073	0,052871	0,059047	0,065587	0,072471	0,079679	0,087189	0,094978	0,103023	0,111300	24
25	0,045407	0,051220	0,057428	0,064012	0,070952	0,078227	0,085811	0,093679	0,101806	0,110168	25
26	0,043869	0,049699	0,055938	0,062567	0,069564	0,076904	0,084561	0,092507	0,100715	0,109159	26
27	0,042446	0,048293	0,054564	0,061239	0,068292	0,075697	0,083426	0,091448	0,099735	0,108258	27
28	0,041124	0,046990	0,053293	0,060013	0,067123	0,074593	0,082392	0,090489	0,098852	0,107451	28
29	0,039895	0,045778	0,052115	0,058880	0,066046	0,073580	0,081449	0,089619	0,098056	0,106728	29
30	0,038748	0,044650	0,051019	0,057830	0,065051	0,072649	0,080586	0,088827	0,097336	0,106079	30
31	0,037676	0,043596	0,049999	0,056855	0,064132	0,071792	0,079797	0,088107	0,096686	0,105496	31
32	0,036671	0,042611	0,049047	0,055949	0,063280	0,071002	0,079073	0,087451	0,096096	0,104972	32
33	0,035727	0,041687	0,048156	0,055104	0,062490	0,070273	0,078408	0,086852	0,095562	0,104499	33
34	0,034840	0,040819	0,047322	0,054315	0,061755	0,069598	0,077797	0,086304	0,095077	0,104074	34
35	0,034004	0,040002	0,046539	0,053577	0,061072	0,068974	0,077234	0,085803	0,094636	0,103690	35
36	0,033214	0,039233	0,045804	0,052887	0,060434	0,068395	0,076715	0,085345	0,094235	0,103343	36
37	0,032468	0,038507	0,045112	0,052240	0,059840	0,067857	0,076237	0,084924	0,093870	0,103030	37
38	0,031761	0,037821	0,044459	0,051632	0,059284	0,067358	0,075795	0,084539	0,093538	0,102747	38
39	0,031092	0,037171	0,043844	0,051061	0,058765	0,066894	0,075387	0,084185	0,093236	0,102491	39
40	0,030456	0,036556	0,043262	0,050523	0,058278	0,066462	0,075009	0,083860	0,092960	0,102259	40

G.3: CONSTRUCTION COSTS FOR PIPED RURAL WATER SUPPLIES



G.4: UNIT PIPELINE COSTS (UPVC)

UNIT PIPELINE COSTS ROUGH ESTIMATES UPVC

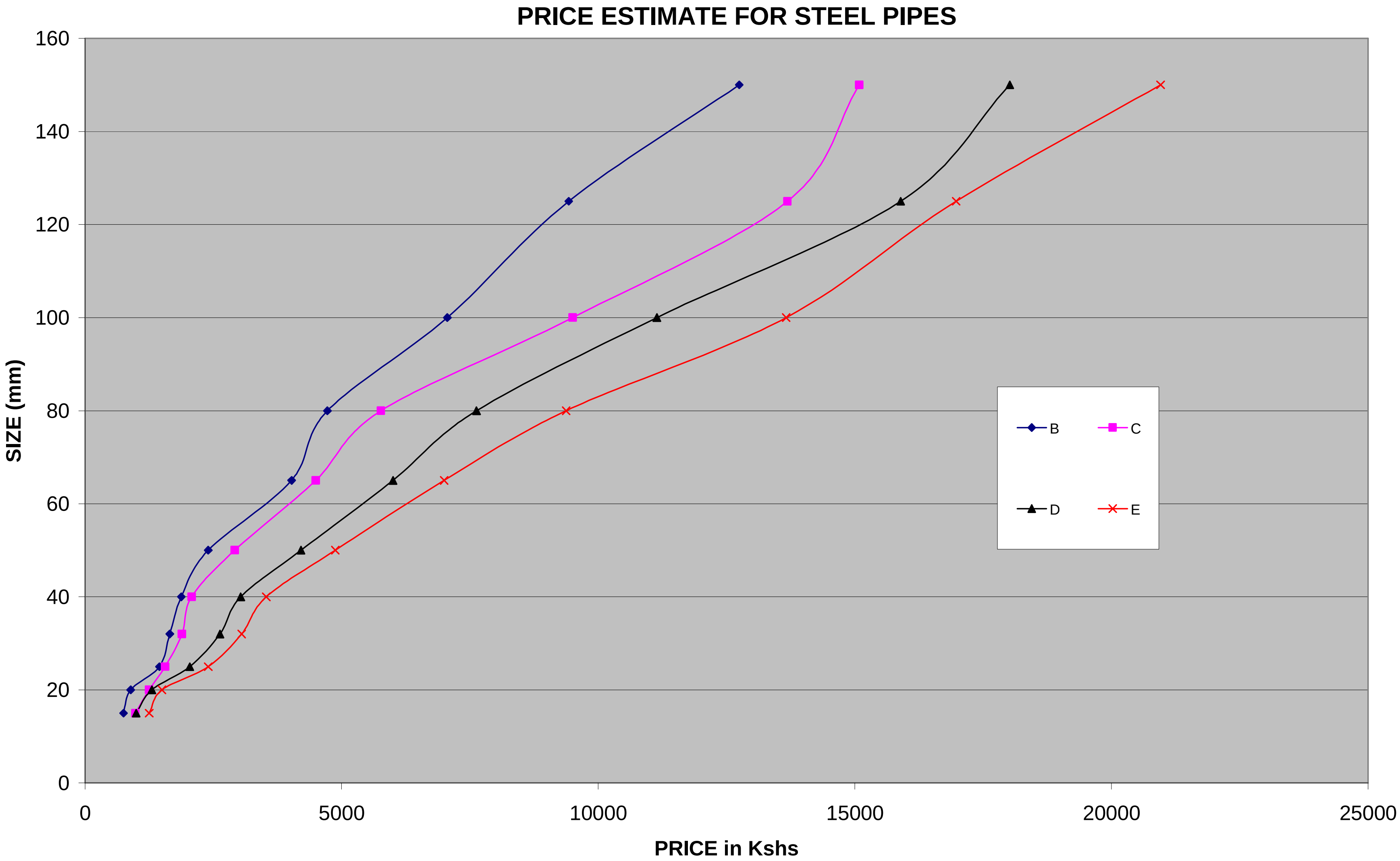


G5: UPVC PIPES EX-FACTORY PRICES

6 METRES EFFECTIVE LENGTH

	UNIT		RATE PER CLASS			
DESCRIPTION (SIZES OD)			B	C	D	E
20mm	M		150	180	240	240
25mm	M		200	225	290	285
32mm	M		240	290	340	450
40mm	M					
50mm	M					
63mm	M		550	660	986	1230
75mm	M					
90mm	M		1200	1350	2685	3000
110mm	M		1624	2800	3108	4018
140mm	M					
160mm	M		2900	4704	5410	6247
200mm	M					
225mm	M		5715	8680	12460	15939
280mm	M		8295	13629	15438	18685
315mm	M		13450	17325	24710	31640
355mm	M					
400mm	M					
450mm	M					

G.6: UNIT PIPELINE COSTS (Steel and DI)



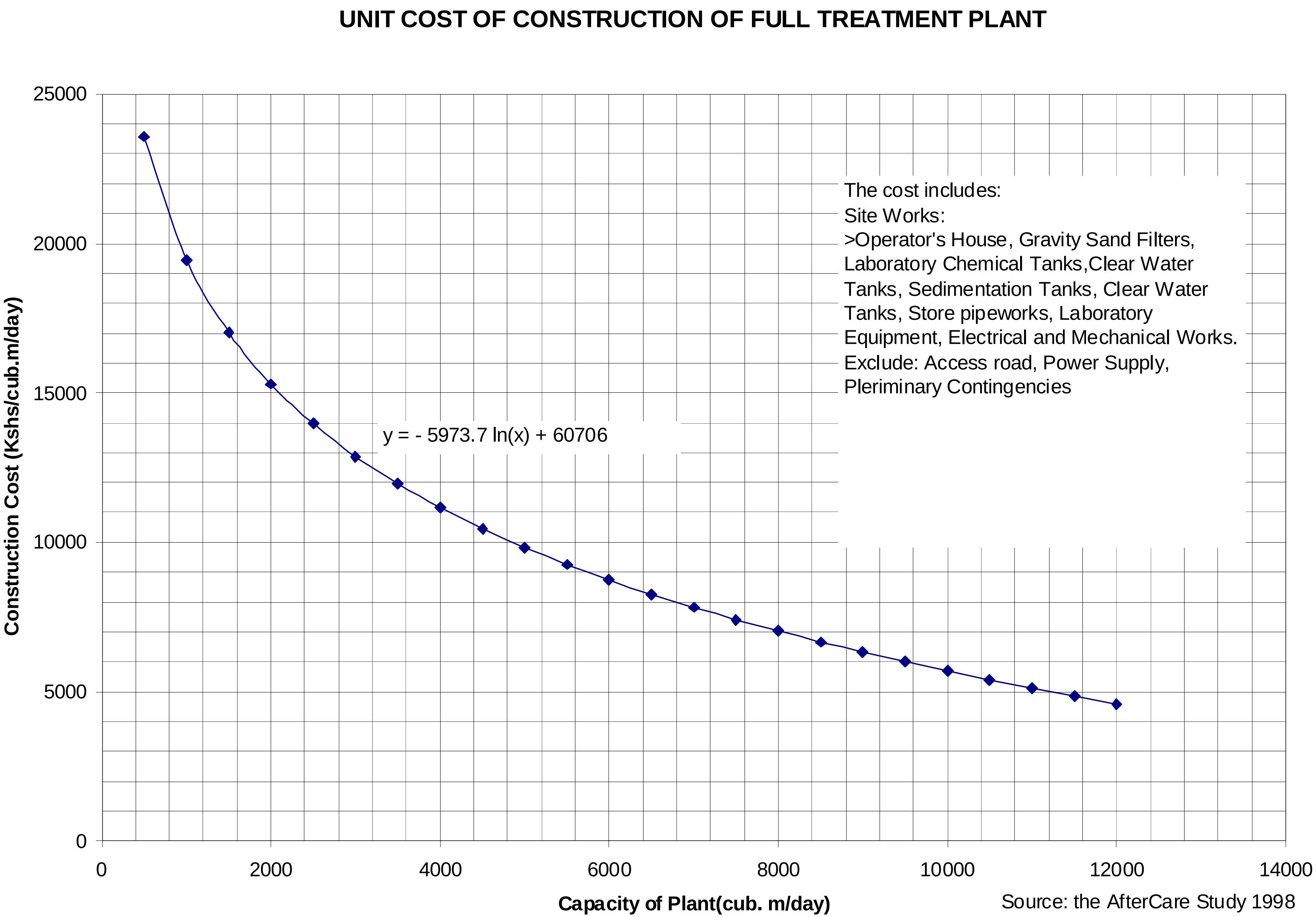
Steel Pipes to AWWA C200 Fusion bonded Epoxy to AWWA C213 with Cement mortar lined to AWWA C205 with Socket and Spigot pushfit joint to BSCP 2010

		US$	KShs
Description	Unit	Unit Rate	
80	m	16,56	1258,56
100	m	21,94	1667,44
150	m	28,79	2188,04
200	m	35,96	2732,96
250	m	44,44	3377,44
300	m	57,6	4377,6
350	m	62,83	4775,08
400	m	80,53	6120,28
450	m	85,08	6466,08
500	m	103,11	7836,36
600	m	142,57	10835,32

6 METRES EFFECTIVE LENGTH

	UNIT		RATE PER CLASS			
DESCRIPTION (SIZES OD)			B	C	D	E
15	m		750	980	999	1250
20	m		890	1250	1300	1500
25	m		1450	1560	2040	2400
32	m		1650	1890	2630	3053
40	m		1875	2076	3035	3533
50	m		2401	2917	4210	4877
65	m		4026	4496	6001	7000
80	m		4722	5766	7627	9376
100	m		7058	9500	11144	13663
125	m		9427	13686	15893	16973
150	m		12747	15085	18020	20956
200						
250						
300						
350						
400						
450						
500						
550						
600						
700						
800						
900						
1000						
1100						
1200						
1300						
1400						
1500						

G.7:



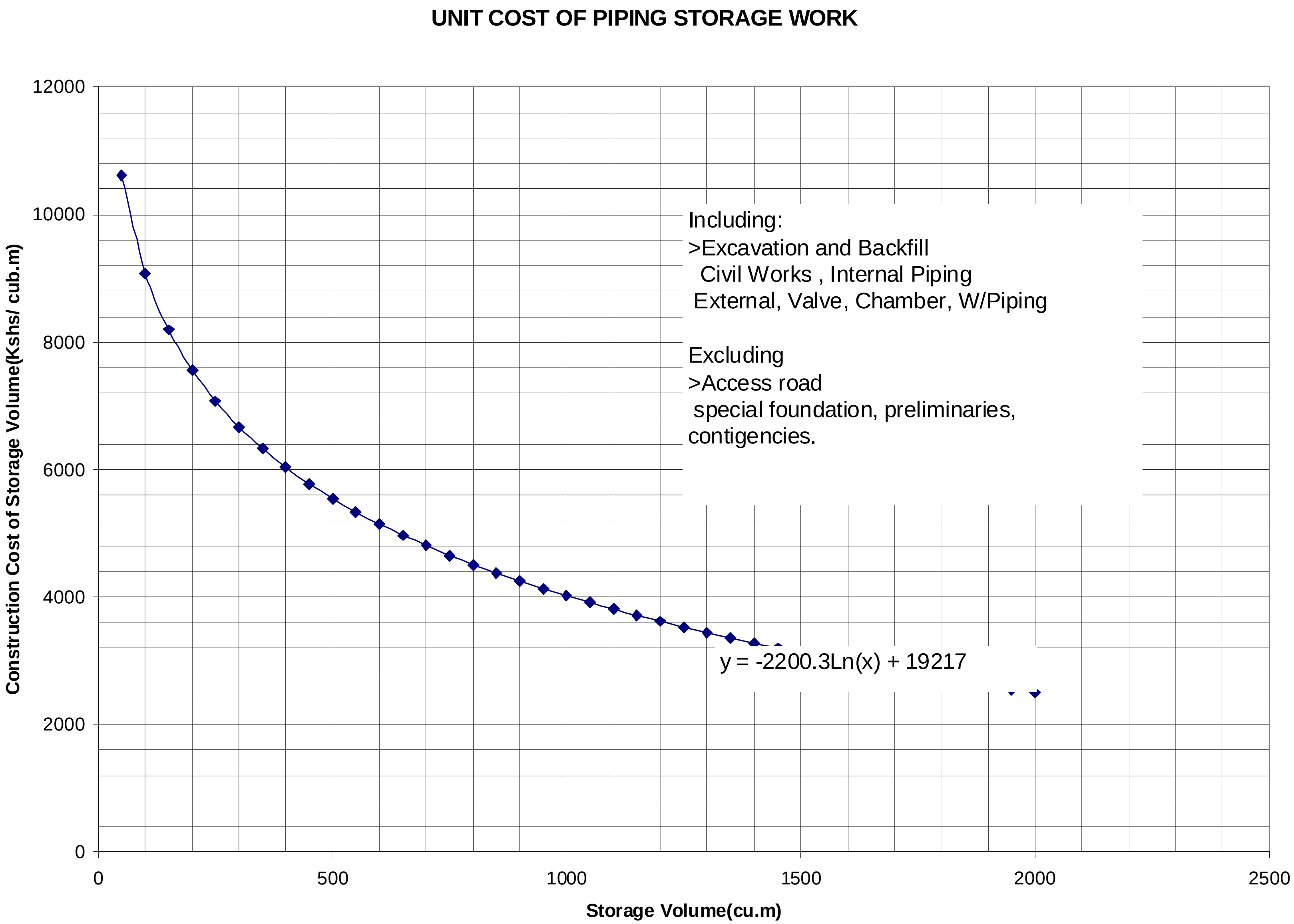
UNIT COST OF CONSTRUCTION OF FULL TREATMENT PLANT
Construction Cost (Kshs/cub.m/day)
25000
20000
15000
10000
5000
0
0
2000
4000
6000
8000
10000
12000
14000
Capacity of Plant(cub. m/day)
y = - 5973.7 ln(x) + 60706
The cost includes:
Site Works:
>Operator's House, Gravity Sand Filters, Laboratory Chemical Tanks,Clear Water Tanks, Sedimentation Tanks, Clear Water Tanks, Store pipeworks, Laboratory Equipment, Electrical and Mechanical Works.
Exclude: Access road, Power Supply, Pleriminary Contingencies
Source: the AfterCare Study 1998

G.7.1: CONSTRUCTION COSTS FOR TREATMENT WORKS

DATA SERIES

x	y		x	y		X			Y			X			Y		
0	#ZAHL!		500	23581,8		50	1500	2950	10609,38	3125,721	1637,57	50	1500	2950	10609,38	3125,721	1637,57
500	23581,7956		1000	19441,1		100	1550	3000	9084,244	3053,574	1600,589	100	1550	3000	9084,244	3053,574	1600,589
1000	19441,14229		1500	17019		150	1600	3050	8192,099	2983,717	1564,22	150	1600	3050	8192,099	2983,717	1564,22
1500	17019,01537		2000	15300,5		200	1650	3100	7559,112	2916,01	1528,442	200	1650	3100	7559,112	2916,01	1528,442
2000	15300,48898		2500	13967,5		250	1700	3150	7068,13	2850,325	1493,236	250	1700	3150	7068,13	2850,325	1493,236
2500	13967,49634		3000	12878,4		300	1750	3200	6666,967	2786,543	1458,585	300	1750	3200	6666,967	2786,543	1458,585
3000	12878,36206		3500	11957,5		350	1800	3250	6327,79	2724,559	1424,471	350	1800	3250	6327,79	2724,559	1424,471
3500	11957,51215		4000	11159,8		400	1850	3300	6033,981	2664,273	1390,878	400	1850	3300	6033,981	2664,273	1390,878
4000	11159,83566		4500	10456,2		450	1900	3350	5774,823	2605,595	1357,791	450	1900	3350	5774,823	2605,595	1357,791
4500	10456,23514		5000	9826,84		500	1950	3400	5542,998	2548,441	1325,193	500	1950	3400	5542,998	2548,441	1325,193
5000	9826,843032		5500	9257,49		550	2000	3450	5333,287	2492,734	1293,071	550	2000	3450	5333,287	2492,734	1293,071
5500	9257,488611		6000	8737,71		600	2050	3500	5141,836	2438,403	1261,412	600	2050	3500	5141,836	2438,403	1261,412
6000	8737,708749		6500	8259,56		650	2100		4965,718	2385,381		650	2100		4965,718	2385,381	
6500	8259,557626		7000	7816,86		700	2150		4802,658	2333,607		700	2150		4802,658	2333,607	
7000	7816,858833		7500	7404,72		750	2200		4650,853	2283,023		750	2200		4650,853	2283,023	
7500	7404,716116		8000	7019,18		800	2250		4508,849	2233,576		800	2250		4508,849	2233,576	
8000	7019,182352		8500	6657,03		850	2300		4375,456	2185,216		850	2300		4375,456	2185,216	
8500	6657,029049		9000	6315,58		900	2350		4249,691	2137,896		900	2350		4249,691	2137,896	
9000	6315,581832		9500	5992,6		950	2400		4130,727	2091,572		950	2400		4130,727	2091,572	
9500	5992,600473		10000	5686,19		1000	2450		4017,866	2046,204		1000	2450		4017,866	2046,204	
10000	5686,18972		10500	5394,73		1050	2500		3910,513	2001,752		1050	2500		3910,513	2001,752	
10500	5394,731916		11000	5116,84		1100	2550		3808,155	1958,18		1100	2550		3808,155	1958,18	
11000	5116,835299		11500	4851,29		1150	2600		3710,348	1915,454		1150	2600		3710,348	1915,454	
11500	4851,293805		12000	4597,06		1200	2650		3616,704	1873,542		1200	2650		3616,704	1873,542	
12000	4597,055436					1250	2700		3526,883	1832,414		1250	2700		3526,883	1832,414	
						1300	2750		3440,586	1792,041		1300	2750		3440,586	1792,041	
						1350	2800		3357,546	1752,394		1350	2800		3357,546	1752,394	
						1400	2850		3277,526	1713,45		1400	2850		3277,526	1713,45	
						1450	2900		3200,315	1675,183		1450	2900		3200,315	1675,183	

G.8


UNIT COST OF PIPING STORAGE WORK
12000
10000
8000
6000
4000
2000
0
0
500
1000
1500
2000
2500
Construction Cost of Storage Volume(Kshs/ cub.m)
Storage Volume(cu.m)
Including:
>Excavation and Backfill
Civil Works , Internal Piping
External, Valve, Chamber, W/Piping
Excluding
>Access road
special foundation, preliminaries,
contigencies.
y = -2200.3Ln(x) + 19217

G.9: UNIT COSTS FOR MISCELLANEOUS ITEMS

UNIT COSTS FOR MISCELLANEOUS ITEMS AT THE 2005 PRICE LEVEL

DESCRIPTION OF ITEM	COST KSHS.	
Hand drilled well with pvc lining depth e – 15m inclusive survey costs.	100,000.00	
Hand dug well with concrete ring lining depth 5-15m inclusive survey costs	120,000.00	
Borehole with lining depth 50-200m single	6,000.00	
Borehole with lining, depth approximate 50m	5,000.00	
Hand pumps for wells upto 20m -	30,000.00	- Afridev
Aluminum delivered to the District Hqs.	23.00	Per kg.
Soda Ash delivered to the District Hqs.	275.00	Per kg
T.C.L. delivered to the District Hqs.	45.30	Per kg
Diesel	60.00	
Electricity	9.50	KWH

The following is the survey costs for pipelines (including mobilization, clearance, beaconing, benchmarks, trial pits and presentation f drawings as per the Institution of Surveyors of Kenya.

Grid survey in KShs/Ha

	Flat KShs.	Medium KShs.	Rough (more) KShs.
Open	13,500	16,800	22,500
Low	16,800	22,500	30,000
Medium	22,500	30,000	42,000
High	30,000	42,000	57,000

The following is the cost for pipelines.(inclusive mobilisaiton, beacons, bench marks, trial pits and presentation of drawings).

Average – 7,500/km

Bush clearing for every 100m
Light bush - KShs.1, 800 / 100m
Medium bush - KShs.500 / 100m
Heavy - KShs.10, 000 / 100m
Profiling - KShs.2, 200 / hr

G10: STAFF REQUIRED FOR OPERATION OF WATER SUPPLIES

(maintenance is normally handled by ambulating teams but remote supplies may require extra staff)

Staff Category	Job group	Full treatment water production >500 m^3/day Operation 13-24 h/day	No treatment or only sed + chlorination > 500m3/dat operation 13-24 h/day
Water Operator Grade I Water Operator Grade IIA Water Operator Grade IIB Water Operator Grade III	E or F E D C	1 1 1 1 per 200 installed water meters	- 1 1 1 per 200 installed water meters
Chemical or general attendant Grade I	B	3	2
Pump attendant Grade I	B	1 per pumping station	1 per pumping station
Pipe fitter Grade I Pipe fitter Grade II	C B	1 1	1 1
Labourer	B	4	4
Line Patrollers	B	1 per 25 km of pipelines	1 per 25 km of pipelines.

For smaller supplies than 500 m^3 output per day and for supplies which will initially operate for less than 13 hours per day should the staff be reduced as decided by the O & M Branch.

G.11: CIVIL SERVICE SALARY SCALES

GROSS SALARIES FOR CIVIL SERVANTS AS PER MAY 2005

JOB GROUP	BASIC SALARY RANGE KSHS. P.M.			MEDICAL ALLOWANCE KSHS. P.M.	HOUSE ALLOWANCE RANGE KSHS.			AVERAGE GROSS PAY RANGE P.M.
	MINIMUM	AVERAGE	MAXMIMUM		MINIMUM	AVERAGE	MAXMIMUM	
A	4.985	2.508	5.015	375	1.800	2.400	3.000	5.285
B	5.015	5.045	5.075	375	1.800	2.400	3.000	7.820
C	5.085	5.300	5.515	375	1.800	2.400	3.000	8.075
D	5.155	5.625	6.095	375	1.900	2.600	3.300	8.600
E	5.515	6.135	6.755	495		2.600	2.900	9.230
F	5.875	6.625	7.375	495	2.100	2.500	2.900	9.620
G	7.665	9.375	11.085	750	2.300	3.650	5.000	13.775
H	8.965	10.475	11.985	990	2.300	3.650	5.000	15.115
J	10.045	11.865	13.685	990	3.000	4.500	6.000	17.355
K	15.235	17.935	20.635	1.245	5.000	7.500	10.000	26.680
L	17.235	20.135	23.035	1.500	10.000	15.000	20.000	36.635
M	20.135	20.585	27.035	1.500	10.000	15.000	20.000	40.085
N	22.435	25.860	29.285	1.740	11.000	17.500	24.000	42.825
P	48.935	52.085	55.235	1.740	12.000	26.000	40.000	79.825
Q	51.935	55.310	58.685	1.995	12.000	26.000	40.000	83.305

PART B - IRRIGATION

APPENDIX H:

H.1: AVERAGE IRRIGAITON REQUIREMENTS (mm)

	Station	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
1	Ahero	168	127	96	22	44	95	103	126	161	153	95	122	1312
2	Ainabkoi	126	126	99	11	3	8	9	0	16	34	8	61	501
3	Archers Post	252	253	236	169	223	261	263	284	290	238	137	183	2789
4	Bachuma	224	208	187	158	139	189	186	191	195	199	152	180	2208
5	Baricho	217	242	212	114	97	157	178	210	208	182	135	136	2088
6	Busia	129	113	66	10	0	8	42	49	71	43	30	83	644
7	Chebloch	215	202	175	94	92	111	64	87	155	178	143	181	1697
8	Eldoret	202	173	162	52	43	29	11	4	51	128	144	167	1166
9	Equator	180	177	141	27	19	10	2	3	19	66	81	149	874
10	Garissa	250	229	222	174	241	207	232	234	245	243	136	149	2562
11	Gede	242	202	191	65	12	15	39	91	172	160	137	181	1507
12	Habaswein	315	322	305	204	343	245	350	363	374	320	164	225	3530
13	Hola	246	264	235	155	202	180	198	227	229	215	149	189	2489
14	Isiolo	236	232	199	125	245	286	293	315	325	225	81	150	2712
15	Kabondori	200	184	99	27	69	107	159	146	213	125	26	87	1442
16	Kapenguria	163	163	119	27	9	15	2	4	21	40	82	144	789
17	Kapsabet	152	141	115	24	4	5	4	4	23	48	82	124	726
18	Kaputir	278	263	211	149	152	190	175	194	232	225	210	251	2530
19	Katumani	160	157	98	34	82	130	139	160	205	172	30	117	1484
20	Kedong	170	147	130	43	36	70	98	135	177	181	123	123	1433
21	Kericho	97	80	36	1	0	0	0	0	0	1	9	24	248
22	Kiambu	182	162	109	13	10	22	59	85	196	134	50	110	1132
23	Kibos	177	145	103	41	57	116	140	125	150	183	112	135	1484
24	Kimakia	43	59	31	0	0	0	0	0	15	13	4	15	180
25	Kipkabus	167	197	152	9	25	10	12	8	73	114	91	118	976
26	Kisumu	116	142	90	26	31	64	101	104	125	152	105	112	1168
27	Kitale	216	169	152	52	21	26	5	14	68	79	117	163	1082
28	Kitui	170	223	127	29	100	185	201	216	248	184	13	41	1737
29	Koru	115	103	47	10	0	12	24	24	70	71	83	86	645
30	Lamu	310	275	265	96	23	28	80	185	215	238	229	236	2180
31	Lamuria	102	109	73	53	134	140	137	122	197	92	30	49	1238
32	Lodwar	275	250	254	186	270	260	247	271	305	306	240	266	3130
33	Likichokio	262	244	175	123	216	197	166	184	244	241	197	231	2480
34	Likitaung	310	289	270	155	237	271	255	288	322	289	242	269	3197
35	Machakos	161	155	96	26	47	114	151	176	223	164	31	68	1412
36	Magadi	257	250	238	138	172	238	277	281	302	297	238	236	2924
37	Makindu	174	208	155	93	149	184	200	219	260	222	56	72	1992
38	Malindi	274	261	223	84	15	18	64	128	204	185	163	227	1846
39	Mandera	305	291	283	153	231	288	282	299	307	182	190	272	3083
40	Maralal	192	182	161	76	102	84	62	59	154	147	104	160	1483
41	Marigat	241	238	210	142	149	158	113	140	218	215	183	213	2220
42	Marsabit	177	188	126	20	55	134	168	189	212	86	41	76	1472
43	Masara	177	150	94	21	25	92	160	169	182	152	80	127	1429
44	Meru	38	105	92	7	17	57	104	193	178	47	1	7	846
45	Molo	130	137	117	23	16	15	10	6	25	58	45	103	685
46	Mombasa	237	257	220	76	25	66	100	143	170	154	154	175	1777
47	Moyale	292	256	211	37	48	144	167	204	209	100	114	200	1982
48	Muguga	138	157	139	14	2	26	40	70	160	164	65	107	1082
49	Mwea Tebere	204	210	149	36	33	94	119	189	214	147	57	155	1607
50	Mwingi	192	213	150	64	165	188	182	239	251	203	31	81	1959
51	Nairobi Kab.	155	169	109	19	8	34	58	86	163	140	41	104	1086
52	Nairobi Sth	191	180	113	26	30	57	90	139	204	171	81	131	1413
53	Naivasha	182	163	142	55	74	97	121	120	150	144	104	151	1503
54	Nakuru	139	174	130	35	50	63	64	52	94	92	66	150	1109
55	Nanyuki	180	168	118	31	50	96	92	104	131	87	44	112	1213
56	Narok	108	96	85	33	41	76	106	136	166	170	110	106	1233
57	Ngao	251	251	243	121	101	121	165	200	207	217	154	191	2222
58	Nyeri	191	165	130	31	10	38	37	106	178	111	39	88	1124
59	Ol Joro Orok	117	132	140	29	30	24	6	5	36	51	48	69	687
60	Oloitokitok	77	53	20	24	67	111	98	136	168	187	18	27	986
61	P. Victoria	160	145	89	35	24	89	141	135	169	139	99	138	1363
62	Ruiru	143	137	109	10	0	15	52	68	146	167	37	47	931
63	Rumuruti	199	196	185	96	127	121	89	98	182	179	118	180	1770
64	Sigor	162	172	152	30	59	91	39	51	97	89	108	192	1242
65	Sth Kinangop	51	51	27	1	0	0	1	2	9	17	10	21	190
66	Subukia	124	155	134	24	6	12	9	12	37	31	51	98	693
67	Taveta	176	193	107	52	81	158	169	193	229	225	126	148	1857
68	Thika	195	210	111	17	44	78	97	125	194	157	48	116	1392
69	Voi	193	224	172	119	170	206	208	209	219	222	135	95	2172
70	Wajir	294	274	245	164	215	249	246	257	265	211	172	231	2823
71	Wayu	262	252	213	157	215	191	199	263	235	210	148	158	2503
72	Yatta	200	207	129	64	110	147	143	206	230	185	66	140	1827

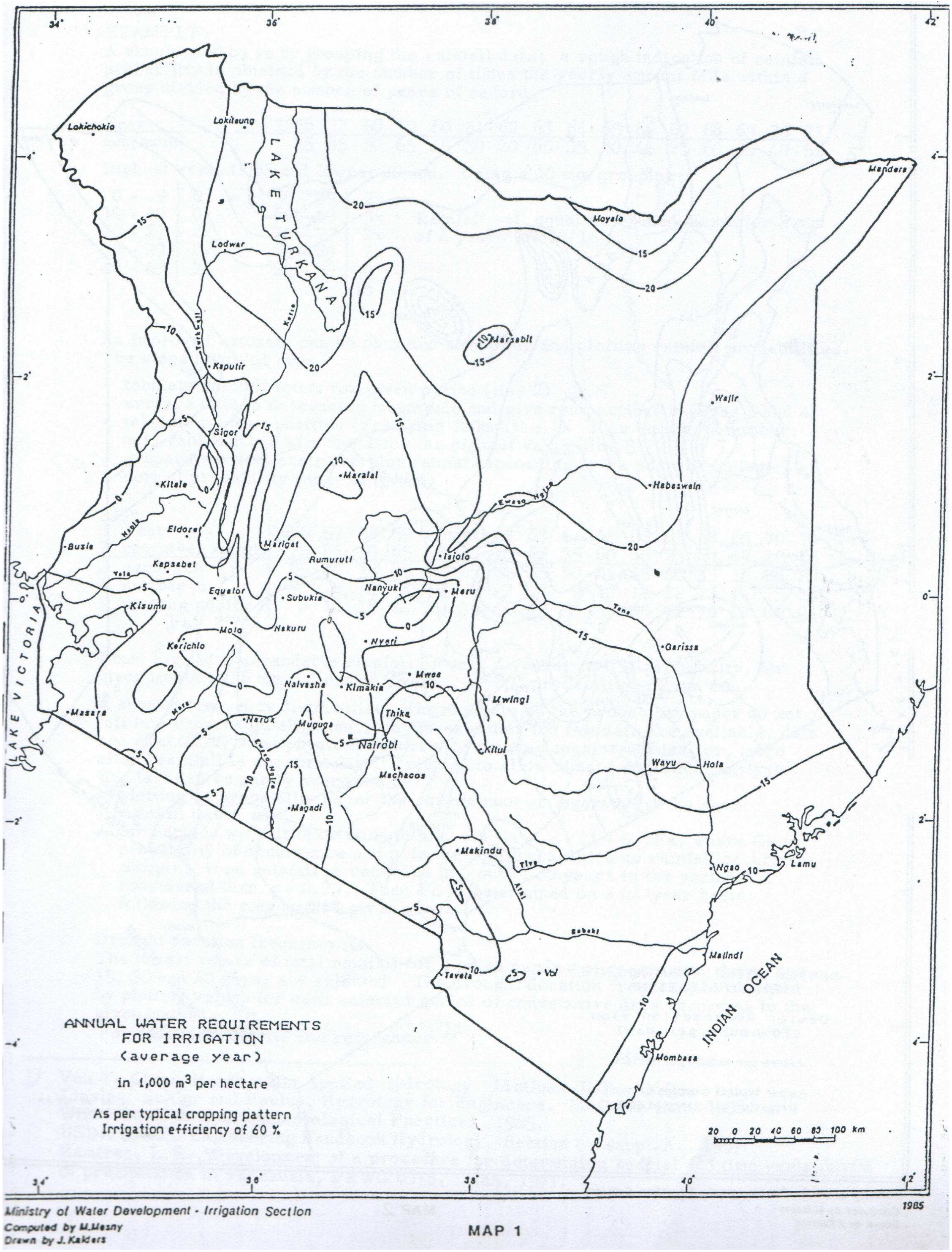
Note: Irrigation application efficiency of 60%
Crop and landuse coefficient of 100%
Data from "Water Requirements for Irrigation in Kenya", M. Mesny & J/ Kalders

H.2: IRRIGATION REQUIREMENT - DRY CONDITION (1 IN 5) (mm)

	Station	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1	Ahero	297	262	239	71	107	129	192	197	256	231	216	218
2	Ainabkoi	211	191	190	0	0	0	0	0	29	85	6	136
3	Archers Post	276	280	293	219	258	275	289	307	326	292	208	248
4	Bachuma	307	275	276	214	204	233	231	241	264	278	235	263
5	Baricho	299	287	281	207	182	200	242	265	277	274	244	181
6	Busia	222	176	158	14	0	0	92	85	181	98	84	1455
7	Chebloch	281	257	266	195	167	185	106	174	237	251	228	256
8	Eldoret	262	239	247	107	78	56	0	0	107	194	203	238
9	Equator	245	240	250	65	49	4	0	0	0	137	153	231
10	Garissa	286	259	278	250	278	235	262	263	279	292	211	226
11	Gede	292	242	278	153	2	13	89	157	256	270	256	284
12	Habaswein	348	342	355	293	412	360	368	385	392	374	258	290
13	Hola	310	311	315	243	264	223	255	281	300	282	237	291
14	Isiolo	305	295	273	192	308	303	321	348	359	295	151	240
15	Kabondori	264	226	194	71	153	129	183	163	245	209	73	135
16	Kapenguria	218	221	193	66	0	27	0	0	17	66	135	194
17	Kapsabet	233	221	208	63	0	0	0	0	36	94	146	186
18	Kaputir	311	286	277	227	246	234	245	256	289	296	270	284
19	Katumani	258	219	179	92	149	164	166	177	242	244	74	193
20	Kedong	219	189	220	117	80	123	132	164	220	239	196	208
21	Kericho	170	169	62	0	0	0	0	0	0	0	0	50
22	Kiambu	284	243	190	12	13	62	129	141	246	221	116	184
23	Kibos	273	237	204	88	101	170	219	194	236	261	218	221
24	Kimakia	111	128	72	0	0	0	0	0	25	23	0	27
25	Kipkabus	218	232	243	35	71	8	21	6	206	193	193	229
26	Kisumu	255	222	178	62	63	114	138	148	202	220	193	196
27	Kitale	271	235	231	124	27	48	0	32	147	118	201	234
28	Kitui	259	283	225	82	190	221	228	251	285	293	24	790
29	Koru	188	188	172	0	0	27	51	76	127	128	137	181
30	Lamu	341	293	316	193	40	69	153	236	286	318	312	307
31	Lamuria	187	157	138	131	211	178	208	180	247	161	91	133
32	Lodwar	312	281	299	254	323	290	297	306	333	340	290	313
33	Likichokio	304	295	270	218	266	250	262	253	308	307	332	307
34	Likitaung	349	337	344	239	304	306	315	324	356	336	315	330
35	Machakos	234	225	184	41	98	155	191	197	265	243	73	122
36	Magadi	344	321	330	238	258	276	309	303	344	349	318	335
37	Makindu	275	276	253	182	196	204	217	226	284	283	134	125
38	Malindi	322	292	303	199	6	43	124	182	288	289	290	310
39	Mandera	324	308	333	221	283	298	301	316	326	255	257	305
40	Maralal	230	233	226	145	168	146	112	123	224	193	162	219
41	Marigat	302	286	293	215	226	222	200	204	283	271	247	277
42	Marsabit	253	244	214	34	131	196	213	224	252	194	111	149
43	Masara	268	238	167	37	56	161	223	223	260	222	178	205
44	Meru	97	187	192	0	14	127	156	229	225	132	0	0
45	Molo	214	205	199	33	32	36	21	0	64	106	74	149
46	Mombasa	314	302	305	163	70	115	146	211	246	258	258	226
47	Moyale	341	299	186	81	87	186	207	243	265	189	206	247
48	Muguga	250	240	221	28	0	64	87	123	194	244	150	225
49	Mwea Tebere	280	275	228	89	92	153	164	224	264	235	110	198
50	Mwingi	284	276	255	138	228	210	205	268	278	281	84	159
51	Nairobi Kab.	218	243	207	32	6	82	110	139	210	202	90	165
52	Nairobi Sth	278	263	213	42	61	114	134	178	264	244	159	235
53	Naivasha	238	216	205	107	116	126	177	168	206	198	166	208
54	Nakuru	185	232	201	93	87	113	117	97	174	160	108	187
55	Nanyuki	231	216	189	71	85	129	158	158	182	148	100	150
56	Narok	198	159	167	61	90	123	140	176	211	218	190	184
57	Ngao	299	283	320	225	196	177	235	264	291	300	225	300
58	Nyeri	278	229	185	63	0	88	66	165	215	224	107	139
59	Ol Joro Orok	175	191	209	80	66	45	7	0	105	115	92	144
60	Oloitokitok	159	97	47	66	116	149	123	152	200	256	75	67
61	P. Victoria	234	227	175	106	43	145	208	204	211	208	183	198
62	Ruiru	254	209	194	0	0	41	16	123	197	258	99	82
63	Rumuruti	249	243	260	166	174	155	156	163	242	238	198	222
64	Sigor	214	238	226	66	115	167	108	87	129	157	196	247
65	Sth Kinangop	115	112	72	0	0	0	0	0	0	44	0	43
66	Subukia	190	227	217	85	0	22	7	20	93	50	89	193
67	Taveta	257	259	191	114	135	194	202	225	268	279	220	202
68	Thika	283	280	208	40	114	132	133	151	242	227	116	185
69	Voi	240	278	258	202	213	233	239	238	266	278	227	157
70	Wajir	326	302	306	223	263	263	271	277	294	269	244	282
71	Wayu	312	284	279	250	263	238	262	285	296	281	220	261
72	Yatta	293	267	234	156	185	185	172	231	268	277	161	216

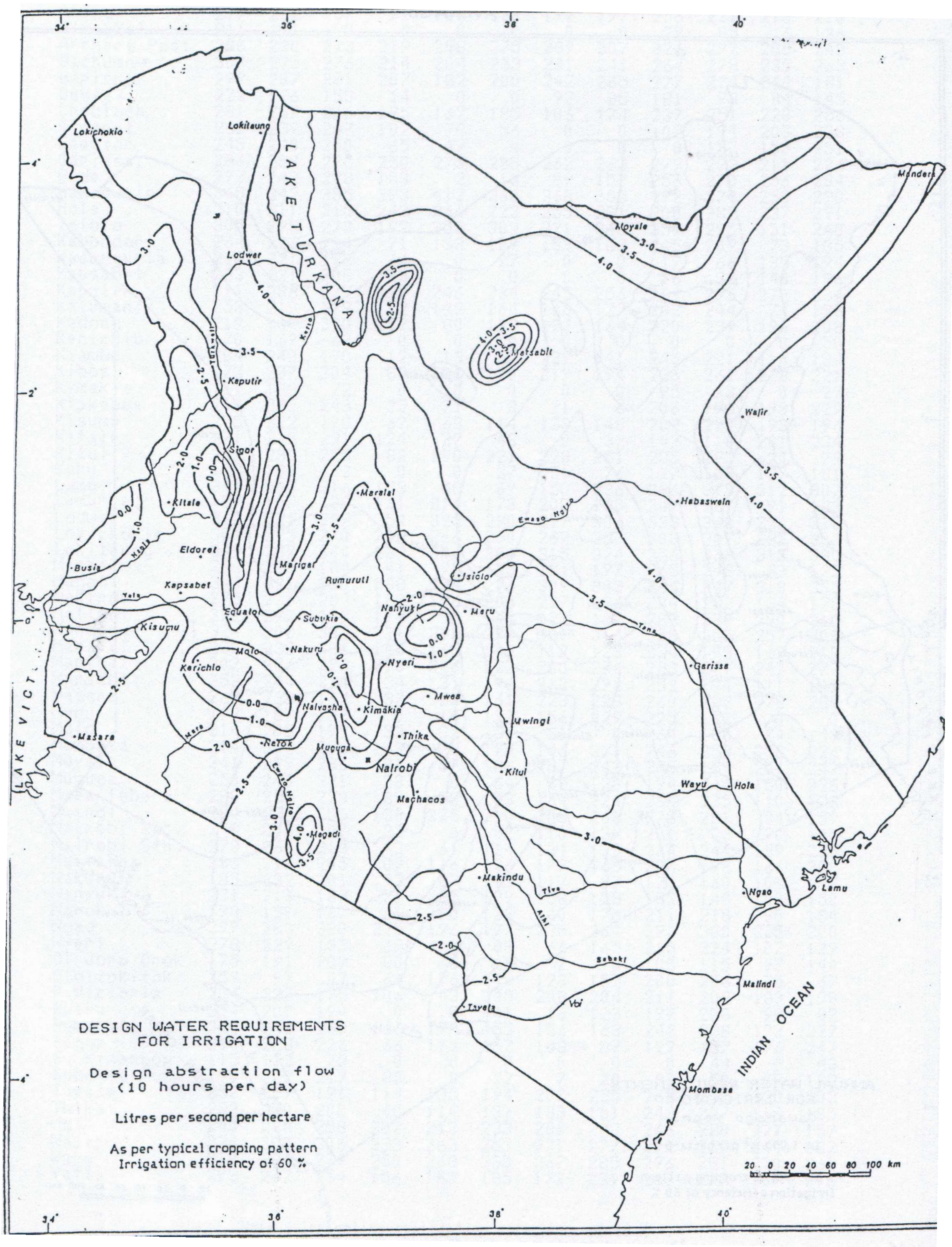
Note: Irrigation application efficiency of 60%
Crop and landuse coefficient of 100%
Data from "Water Requirements for Irrigation", M.Mesny & J. Kalders

.3: ANNUAL WATER REQUIREMENTS FOR IRRIGATION AVERAGE.



MAP 1

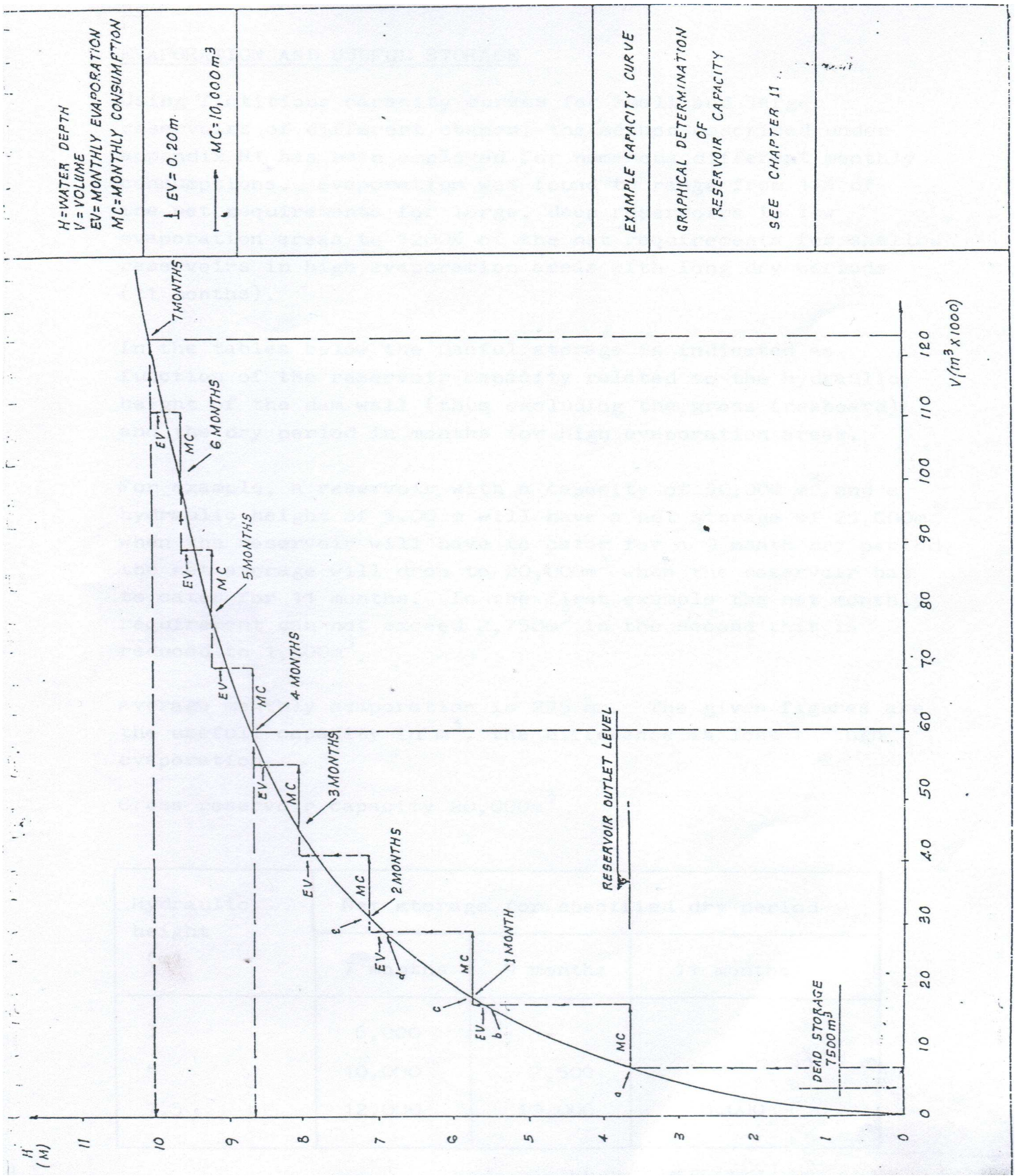
H.4: DESIGN WATER REQUIREMENTS FOR IRRIGATION



PART C – SMALL DAMS:

APPENDIX I:

I.1: Graphical determination of reservoir capacity



I.2: EVAPORATION AND USEFUL STORAGE

EVAPORATION AND USEFUL STORAGE

Using fictitious capacity curves for small and larger reservoirs of different shapes, the method described under appendix H1 has been employed for numerous different monthly consumptions. Evaporation was found to range from 11% of the net requirements for large, deep reservoir in low evaporation areas to 1200% of the net requirements for shallow reservoirs in high evaporation areas with long dry periods (11 months).

In the tables below the useful storage is indicated as function of the reservoir capacity related to the hydraulic height of the dam wall (thus excluding the gross freeboard) and the dry period in months for high evaporation areas.

For example, the reservoir with a capacity of 50,000 m^3 and a hydraulic height of 5.00 m will have a net storage of 25,000m^3 when the reservoir will have to cater for a 9 month dry period, the net storage will drop to 20,000m^3 when the reservoir has to cater for 11 months. In the first example the net monthly requirement can not exceed 2,750m^3 in the second this is reduced to 1,800m^3.

Average monthly evaporation is 225 mm. The given figures are the usefull capacity in m^3, the difference is lost through evaporation.

Gross reservoir capacity 20,000m^3.

Hydraulic height (m)	Net storage for specified dry period		
	7 months	9 months	11 months
3	6,000	-	-
5	10,000	7,500	-
7.5	12,000	10,000	8,000

Gross reservoir capacity 50,000 m^3

Hydraulic height (m)	Net storage for specified dry period		
	7 months	9 months	11 months
3	14,000	11,000	-
5	27,000	25,000	20,000
7.5	31,500	27,000	24,000
10	36,000	32,000	29,000

Gross reservoir capacity 100,000m^3

Hydraulic height (m)	Net storage for specified dry period		
	7 months	9 months	11 months
3	28,000	22,000	9,000
5	47,000	36,000	28,000
7.5	62,000	53,000	43,000
10	80,000	74,000	67,000

Gross reservoir capacity 200,000 m^3

Hydraulic height (m)	Net storage for specific dry period		
	7 months	9 months	11 months
4	75,000	55,000	35,000
5	85,000	73,000	50,000
7.5	120,000	98,000	90,000
10	150,000	135,000	120,000
15	165,000	150,000	145,000

I.3: SPILLWAY CALCULATIONS

GENERAL SPILLWAY CALCULATION

The calculations below are an earth channel spillway, to enable quick reading of the required spillway dimensions for different channel widths. This method allows quick economical comparisons for different dimensioned spillways for the same return flood.

Included in the calculations is the required freeboard above the spillway crest as safety against overtopping of the earth wall.

The calculations are based on the principals that rapidly varied flows and eddies are not allowed due to high erosion risks. Equally the water velocities will have to be limited to a maximum of 2.50m/s which is thought the maximum admissible on the very solid soils in situ – in Kenya. Anywhere where higher velocities will occur channel protection will be required.

The calculations are done for a pre-selected spillway width, bottom slope and maximum admissible velocities in the channel.

The assumptions are that the spillway crest is fixed through means of a concrete sill at level with the channel bottom and that the calculated channel bottom slope will gradually lead the water back into the original riverbed.

SPILLWAY CHANNEL – OUTFLOW SECTION

For this section the Manning Strickler formula is used, assuming that a uniform flow has developed after the crest:

$$Q = K_S . R^{2/3} . S^{1/2} . A \qquad (1)$$

In which:

- Q = Discharge in m3/s for adopted return flood
- K_s = Strickler's coefficient
- R = Hydraulic Radius in m
- S = Slope in decimals
- A = Wet surface in m^2

As spillway channels are near rectangular channels, with the width far larger than the water depth, the hydraulic radius is approximately equal to the water depth (h) in which case formula (1) can be written as:

$$Q = K_S . h^{2/3} . S^{1/2} . A \qquad (2)$$

For d the width of the channel,

A = d h and q d = Q, hence the formula becomes per metre width of the channel:

$$q = K_s \cdot h^{5/3} \cdot S^{½} \qquad (3)$$

In which:

Q = $(m^3/s)/m$, discharge per metre width

Ks = Strickler's coefficient, throughout assumed to be 40 (metric system) as we are dealing with non maintained earth channels.

For any known Q (return flood) and preselected width (d) q is known. As per definition:

$$Q = v\ a \qquad (4)$$

In which:

V = the mean velocity in the channel using above quoted definitions for Q and A formula (4) becomes:

$$q = v\ h \qquad (5)$$

As q is known and v has a maximum of 2.50 m/s, formula (5) gives the minimum admissible water depth (h). With h known formula (3) gives the maximum admissible channel slope.

In case the assumed Ks – value will be too high (when rougher channel conditions are expected) and a lower value should have been adopted, it can be seen from formulas (3) and (5) that where q has out of necessity to remain the same (return flood condition) the water height will increase (formula (3)) and hence velocities will decrease (formula (5)), consequently a decreased erosion irks, but an increased overtopping risk of the wall for the subcritical flows.

SPILLWAY CREST

The calculation of the flow over the spillway crest is important for the determination of the required freeboard for the dam wall.

Two flow conditions are possible:

(a) Sub-critical flow in the outflow section, under which condition the flow pattern in the outflow section has an influence on the reservoir water level, hence the free-board.

(b) Supercritical flow in the outflow section and no influence of this section on the reservoir level.

(a) Subcritical flow in outflow section

When the water depth in the outflow section is higher than the critical depth, the subcritical flow condition will revert.

The following formula has been derived from the Bernouilli-energy equation and is applicable for free entrance into a channel:

$$H_A = h_1 + h_e - a\frac{V_A^2}{2g} \qquad (6)$$

in which: H_A is now the water height above the crest in the reservoir and is called the NET-FREEBOARD.

H_A = Water depth above the crest level in m in the inflow part of the channel.
V_A = Water velocity at the same point, (m/s)
h_1 = Water depth in the channel calculated under the outflow section, (m)
h_e = Head loss due to friction expressed in terms of velocity at the section of h_1 for the subcritical flows, (m) plus velocity head at the same section.
a = Coefficient depending on water velocities and smoothness of walls.

As $a\frac{V_A^2}{2g}$ is the velocity head of the approaching flow, which is usually very small and is about zero in the middle of the reservoir this factor can be ignored, hence equation (6) becomes:-

$$H_A = h_1 + h_e \qquad (7)$$

For subcritical flow h_e is expressed as:

$$h_e = C_e\frac{V_1^2}{2g} \qquad (8)$$

in which:

C_e = a coefficient which has an average value of 1.25 for a well rounded entrance, which is assumed to be the case.
V_1 = Water velocity at the section of h_1 and also calculated under the outflow section.

Substituting formula (8) in formula (7) gives :

$$H_A = h_1 + C_e\frac{V_1^2}{2g} \qquad (9)$$

making the Net Freeboard a direct function of the water velocity and height in the outflow section. After substituting equations (5) and (3) in formula (9) an exponential equation is derived which makes HA (the net freeboard) a function of a q and the outlet channel slope.

It should be noted that the calculations showed that for slopes of around 0.0075 or steeper for the outlet section the flow become supercritical, hence slopes weaker than 0.0075 are presented only as for the supercritical flows the slopes do not have an influence on H_A, consequently a relation between q and H_A is independent of the slope as explained in the next sub-chapter.

(b) Super critical flow in the outflow section

In this case the crest sill will act as a weir, with water depth over the sill at critical depth.

The critical depth equation is:

$$h_c = \sqrt[3]{q^2/g} \qquad (10)$$

in which: h_c = critical water depth (m) on top of the crest sill. As the critical depth is two third of the original depth : $h_c = 2/3\ H_A$, formula (10) becomes:

$$H_A = \frac{3}{2} \sqrt[3]{q^2/g} \qquad (11)$$

Formula (11) is basically the same as the weir formula, the latter being more refined through the introduction of a coefficient (f) to account for entrance conditions. The weir formula is:

$$Q = 0.385 \text{ x f x b x } \sqrt{2g} \text{ x } H_A^{3/2} \qquad (12)$$

or per unit width (m):

$$q = 0.385 \text{ x f x } \sqrt{2g} \text{ x } H_A^{3/2} \qquad (13)$$

The factor f ranges from 0.9 for bad conditions to 1.1 for good conditions, the average of 1.0 has been adopted for the calculations.

Equation (13) gives a relation between H_A (the Net Freeboard) and q,

GROSS FREEBOARD

The gross freeboard is calculated from the net freeboard plus a safety factor, as overtopping of an earth wall is under no circumstances allowed.

All reservoirs will have wave development and set up (water being piled up on one side of the reservoir due to wind forces) which will cause the water to rise higher than the average lake level (H_A) at the windward shores. Whenever the shore is the embankment additional safety is required.

As pointed out in chapter 2 most reservoirs will be 'small' and significant set up is not expected and therefore not included in the calculations. Wave development however can still be significant.

Most formulas require knowledge about the local predominant wind directions and velocities and as those are usually unknown factors, the Mallet and Pacquant formula, which considers wave height and wave velocity, both as a function of the FETCH has been used:

$$\text{GROSS FREEBOARD} = H_A + h_w + \frac{V_w^2}{2g} \qquad (14)$$

In which:

H_A = Net Freeboard (m)
h_w = Wave height (m)
V_w = Wave Velocity (m/s)

And $h_w = 0.5 + 0.33\sqrt{L}$

$V_w = 1.5 + 0.66.\, h_w$

In which L =Fetch or reservoir length in wind direction (towards the Dam) in Km.

Formula (14) is an empirical formula and valid for smaller reservoirs, the fetch not exceeding 3km. For larger fetches a safety factor of 1.10 will have to be introduced.

General

Net freeboard of over 1.50 m is for the size of dams under consideration generally uneconomical.

Generally water velocities should be kept down to 2.50 m/s in earth lined channels. Velocities are however shown up till 6.00 m/s to allow for design of the spillway on more solid materials, mainly solid rock or riprap lining.

The topography might require the design of two spillways, one on either side of the dam. The total width of the spillway however remains the same as required.

I.4: INFORMATION TO BE INCLUDED IN DESIGN REPORT

INFORMATION TO BE INCLUDED IN DESIGN REPORT

Dam …………………………………………………………………….
District …………………………………………………………………
Location ………………………………………………………………
Designed by …………………………………………………………….
Date……………………………………………………………………….

1. SALIENT FEATURES

1.1 S.K. Map Sheet No. ……………………. Scale ……………………………………
Grid Ref ……………………………………………………………………………..

1.2 PURPOSE:

1.3 REFERENCE DRAWINGS:

	SUBJECT	NO.
i)	…………………………………………	…………………………………………
ii)	…………………………………………	…………………………………………
iii)	…………………………………………	…………………………………………
iv)	…………………………………………	…………………………………………
v)	…………………………………………	…………………………………………
vi)	…………………………………………	…………………………………………

1.4 SITE AND CATCHMENT SITUATION:

- Catchment area ………………………………………………………………… Km^2
- Length catchment area ………………………………………………………. Km
- Maximum catchment altitude ………………………………………………….. m
- Altitude at dam site ………………………………………………………….. m
- Mean annual potential evaporation …………………………………………….. mm
- Mean estimated annual runoff …………………………………………………….. MCM
- Q25 years flood ……………………………………………………………… s M^3/
- Q100 years flood ……………………………………………………………… M^3/s
- Mean annual rainfall ………………………………………………………… mm
- Topography ……………………………………………………………………..
  ……………………………………………………………………….
- Geology ……………………………………………………………………...
  ……………………………………………………………………………..
  …………………………………………………………………………….
- Soil …………………………………………………………………………….
  …………………………………………………………………………….
- Vegetation ……………………………………………………………………….
- Erosion ………………………………………………………………………..

1.5 Reservoir

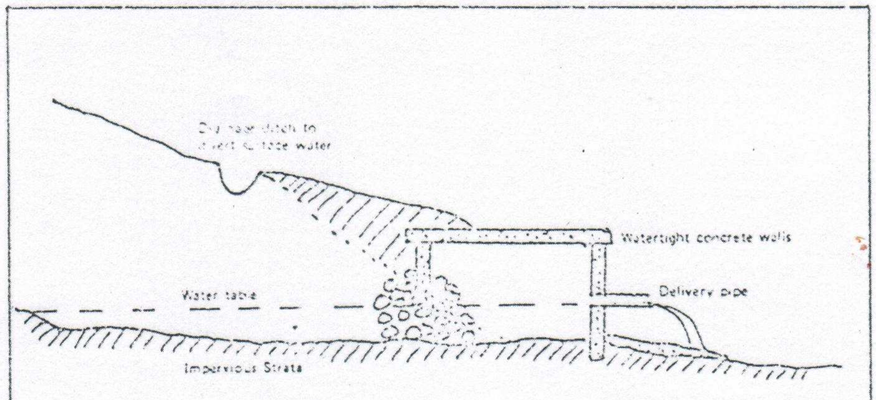
- Area ………………………………………………………………………….. ha
- Capacity ………………………………………………………………………… m^3
- Mean Annual Evaporation ……………………………………………………… m^3
- Estimated other losses ………………………………………………………… m^3
- Dead Storage …………………………………………………………………. m^3
- Live or net storage ……………………………………………………………… m^3
- Fetch ……………………………………………………………………………. m
- Maximum depth ………………………………………………………………… m

1.6 Dam wall

- Structural height …………………………………………………………..…. m
- Height above ground level ……………………………………………………… m
- Hydraulic heights ……………………………………………………………… m
- Net free board …………………………………………………………………. m
- Gross freeboard ………………………………………………………………….. m
- Crest width …………………………………………………………………… m

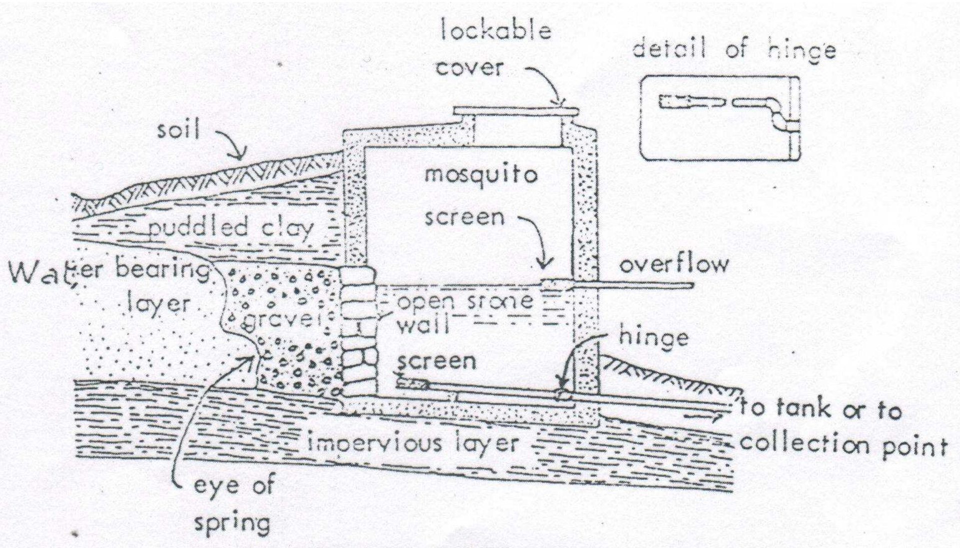
APPENDIX J

J.1 PROTECTION OF NATURAL SPRINGS TO PREVENT POLLUTION BY SURFACE RUN-OFF



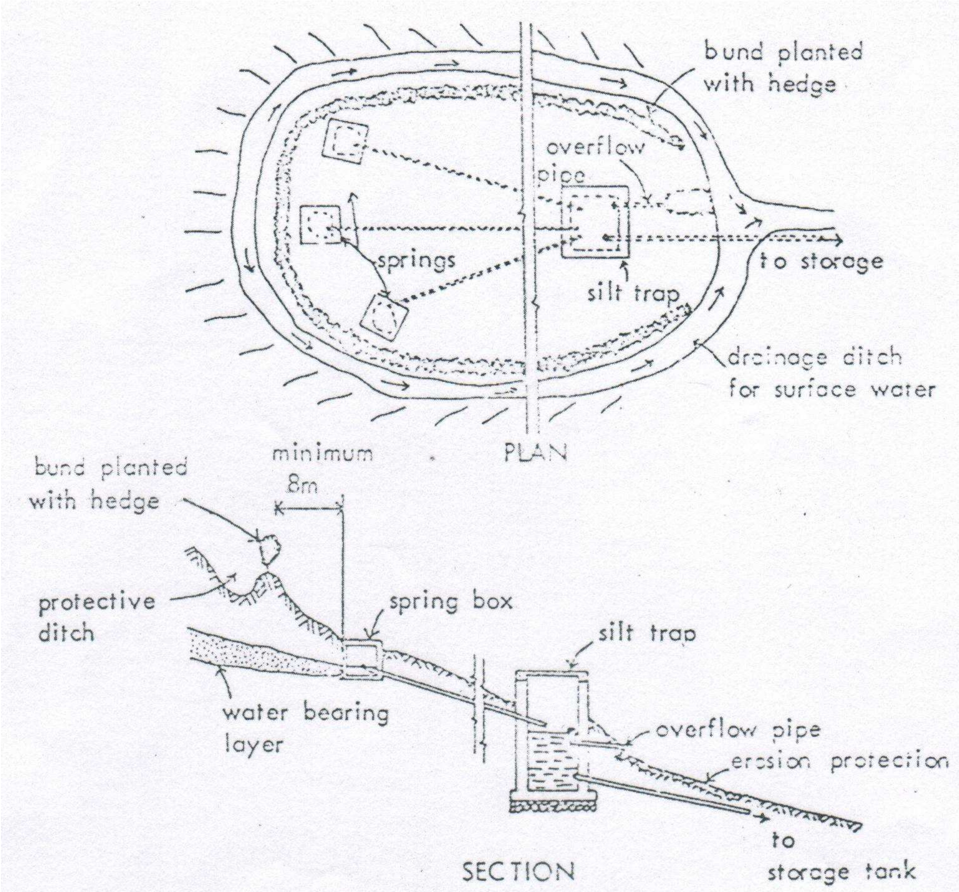
PROTECTION OF NATURAL SPRINGS TO PREVENT POLLUTION BY SURFACE RUN OFF

J.2 A SPRING BOX



A spring box; the inset detail shows a hingemode with two pipe bends, enabling the screen to be lifted above the water for cleaning.

J.3 THREE PROTECTED SPRINGS CONNECTED TO A SILT TRAP



Three protected springs connected to a silt trap.

J.4 ARTESIAN CONTACT SPRING OF LARGE LATERAL WIDTH