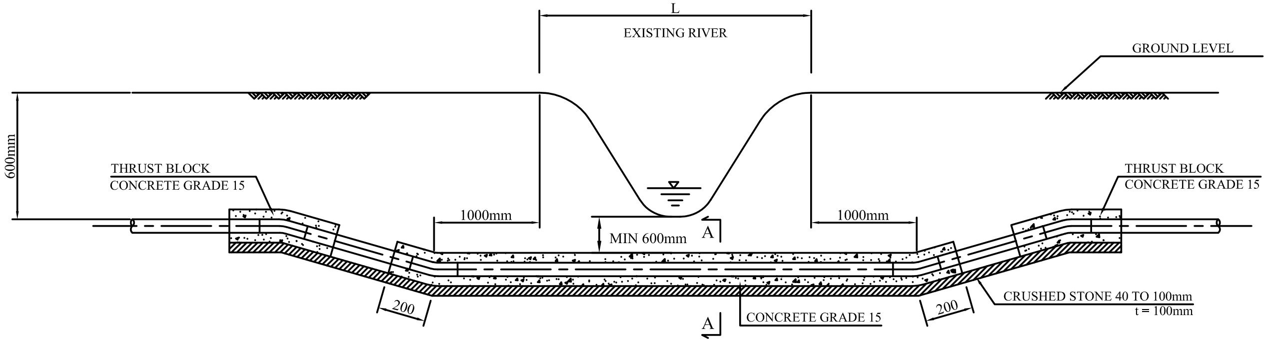
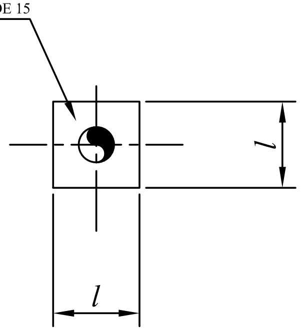


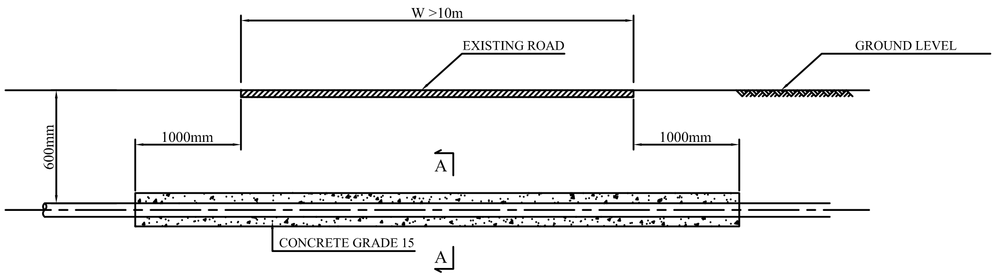
RIVER CROSSING

Note: "L" should be indicated in Profile Drawings.



SECTION A-A

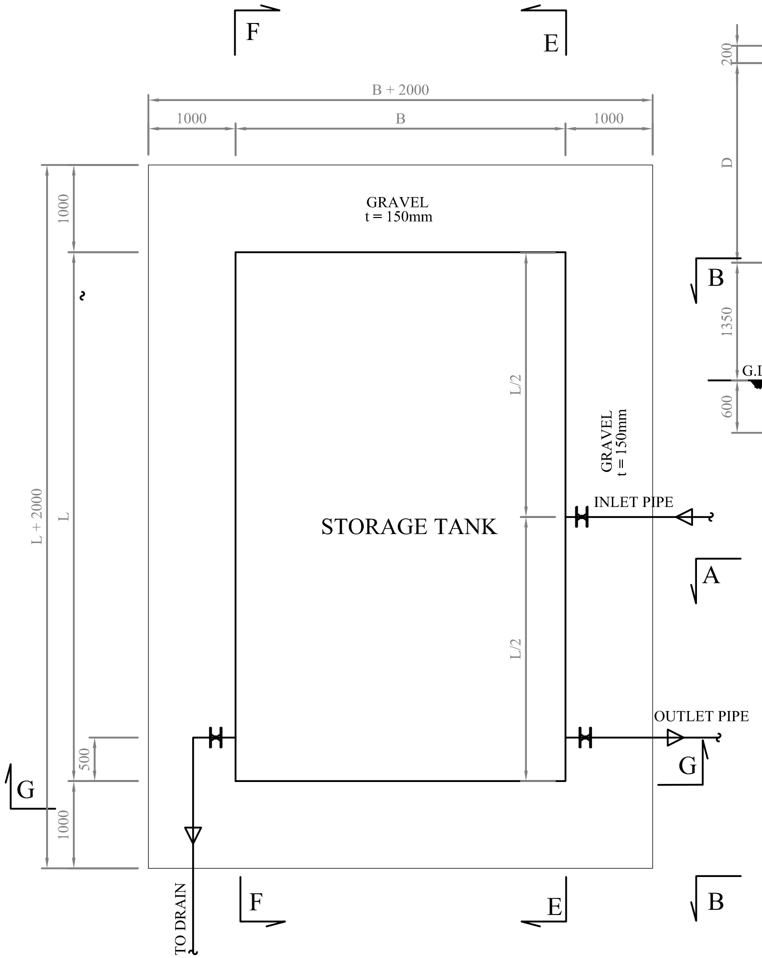
PIPE DIAMETER (mm)	l (mm)
50≤	260
50>	300



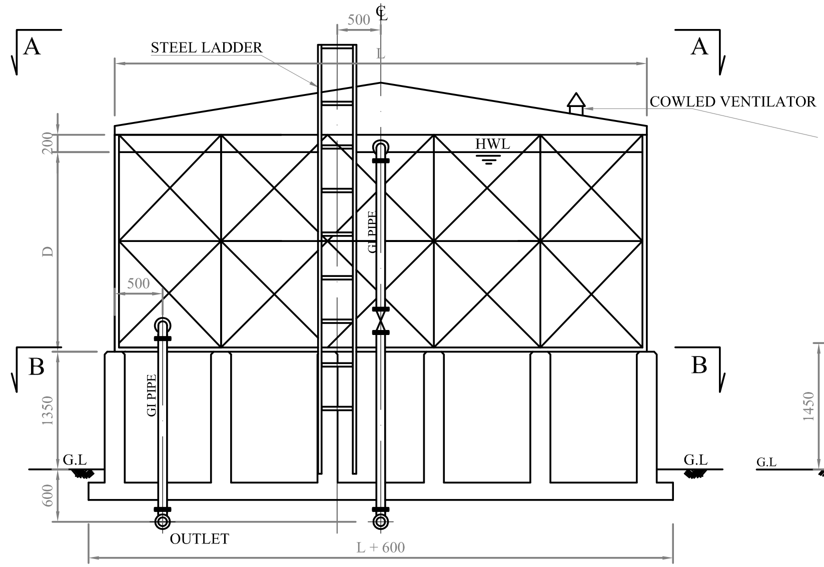
ROAD CROSSING

Note: "W" should be indicated in Profile Drawings.

OWNER:	PROJECT NAME:	CONSULTING ENGINEERS:	TITLE:
THE MINISTRY OF WATER AND IRRIGATION THE REPUBLIC OF KENYA	THE PROJECT FOR RURAL WATER SUPPLY	NIPPON KOEI CO.,LTD.	CONSTRUCTION OF WATER SUPPLY FACILITIES BY SUBMERSIBLE PUMP RIVER AND ROAD CROSSING OF PIPE SCALE: NONE DATE: NOV 2007 DRAWING NO.: SP-052

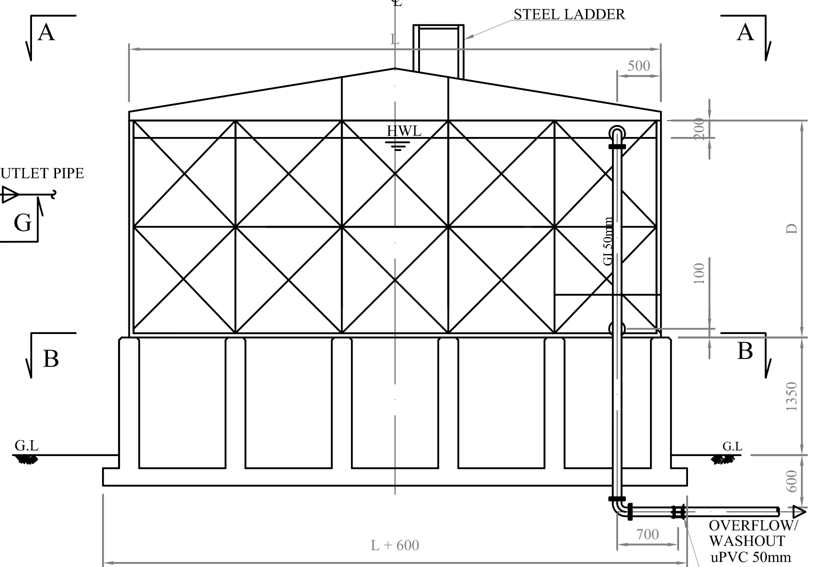


TYPICAL LAYOUT PLAN OF STORAGE TANK
Scale 1:60

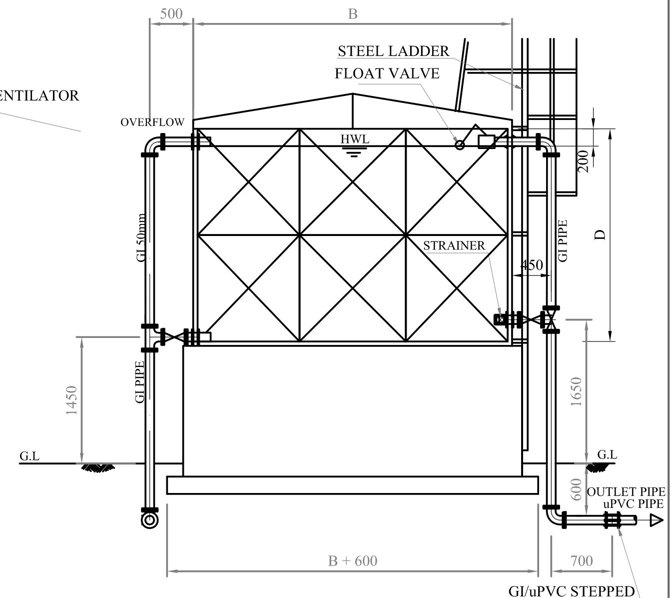


VIEW E-E
Scale 1:60

COUPLING



VIEW F-F
Scale 1:60



VIEW G-G
Scale 1:60

PIPE	DIMENSION (mm)	SERIAL NO.
INLET	GI40	123,142,162,173A,186, 200
	GI50	96A,98A,121,127A,133, 137A,151,156A,163,172, 195A,197
OUTLET	GI40	123,142,162,173A, 186, 200,
	GI50	96A,98A,121,127A,133, 137A,151,156A,163,172, 195A,197

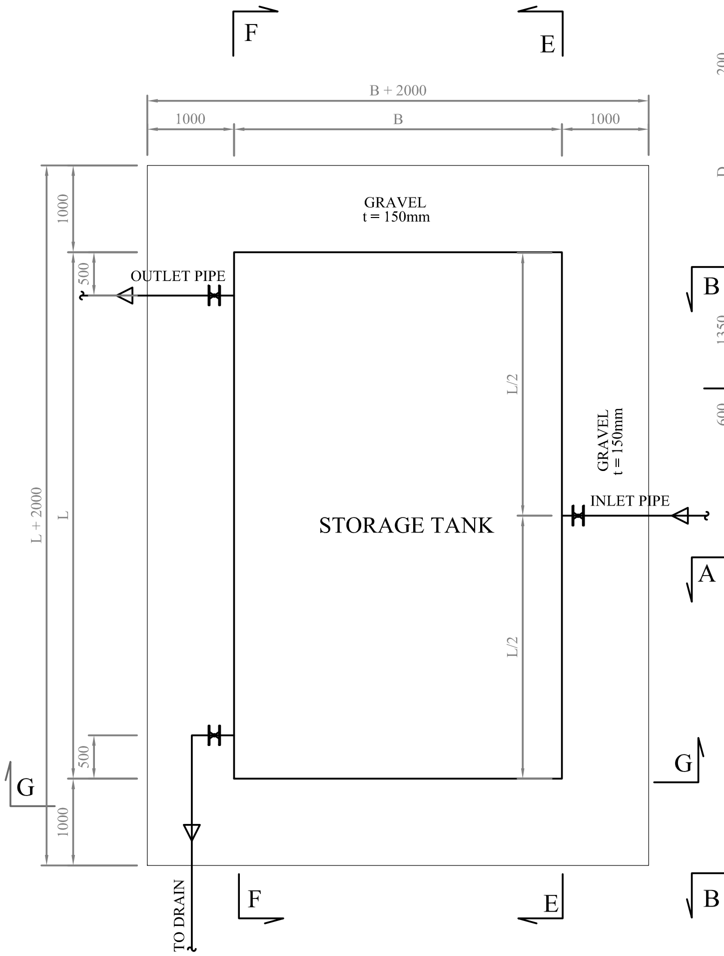
NOTE: FOR LADDER DETAILS SEE DRAWING NO SP-063

LEGEND:

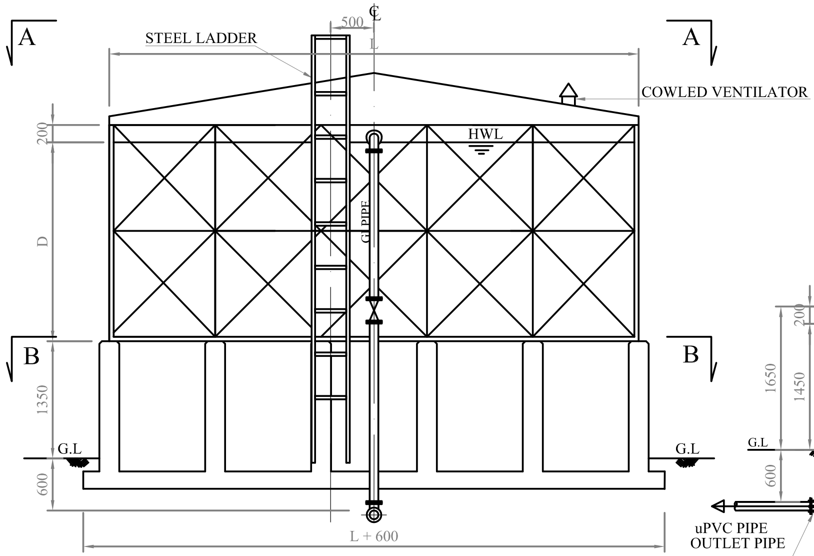
- GI/uPVC STEPPED COUPLING
- GATE VALVE
- PIPELINE

OWNER:	PROJECT NAME:	CONSULTING ENGINEERS:	TITLE:
THE MINISTRY OF WATER AND IRRIGATION THE REPUBLIC OF KENYA	THE PROJECT FOR RURAL WATER SUPPLY	NIPPON KOEI CO.,LTD.	CONSTRUCTION OF WATER SUPPLY FACILITIES BY SUBMERSIBLE PUMP STORAGE TANK (1/2)

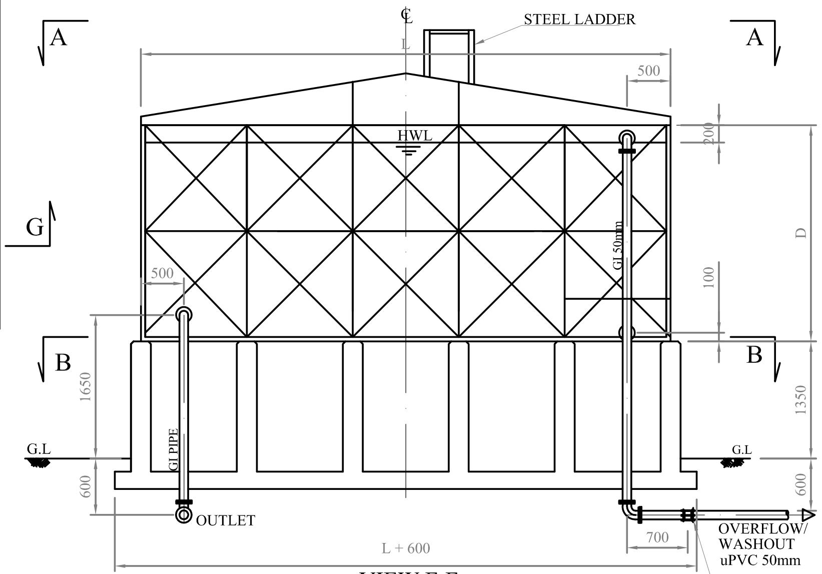
SCALE	DATE	DRAWING NO.
1:60	OCT 2010	SP-062A



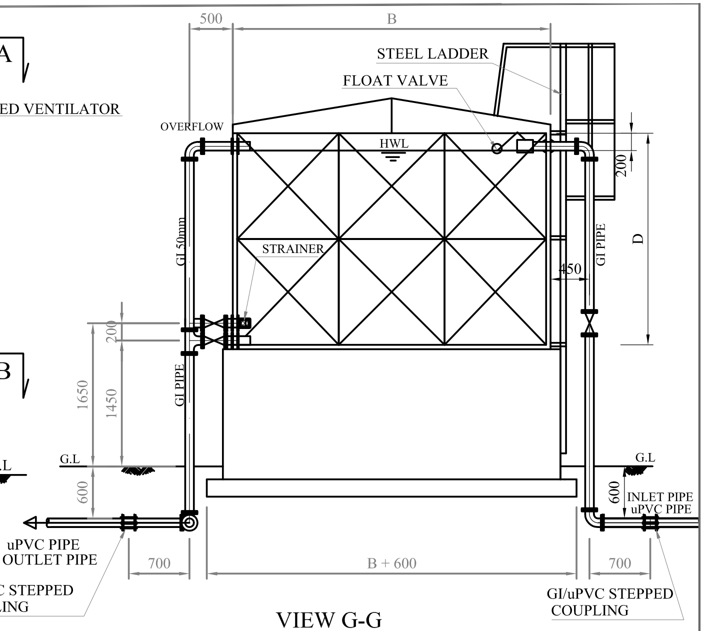
TYPICAL LAYOUT PLAN OF STORAGE TANK
Scale 1:60



VIEW E-E
Scale 1:60



VIEW F-F
Scale 1:60



VIEW G-G
Scale 1:60

PIPE	DIMENSION (mm)	SERIAL NO.
INLET	GI40	102,188A
	GI50	167,177,178A,183,187A, 189,191A
	G163	158,180
OUTLET	GI40	102,158, 188A
	GI50	167,177,178A,180,183, 187A,189,191A

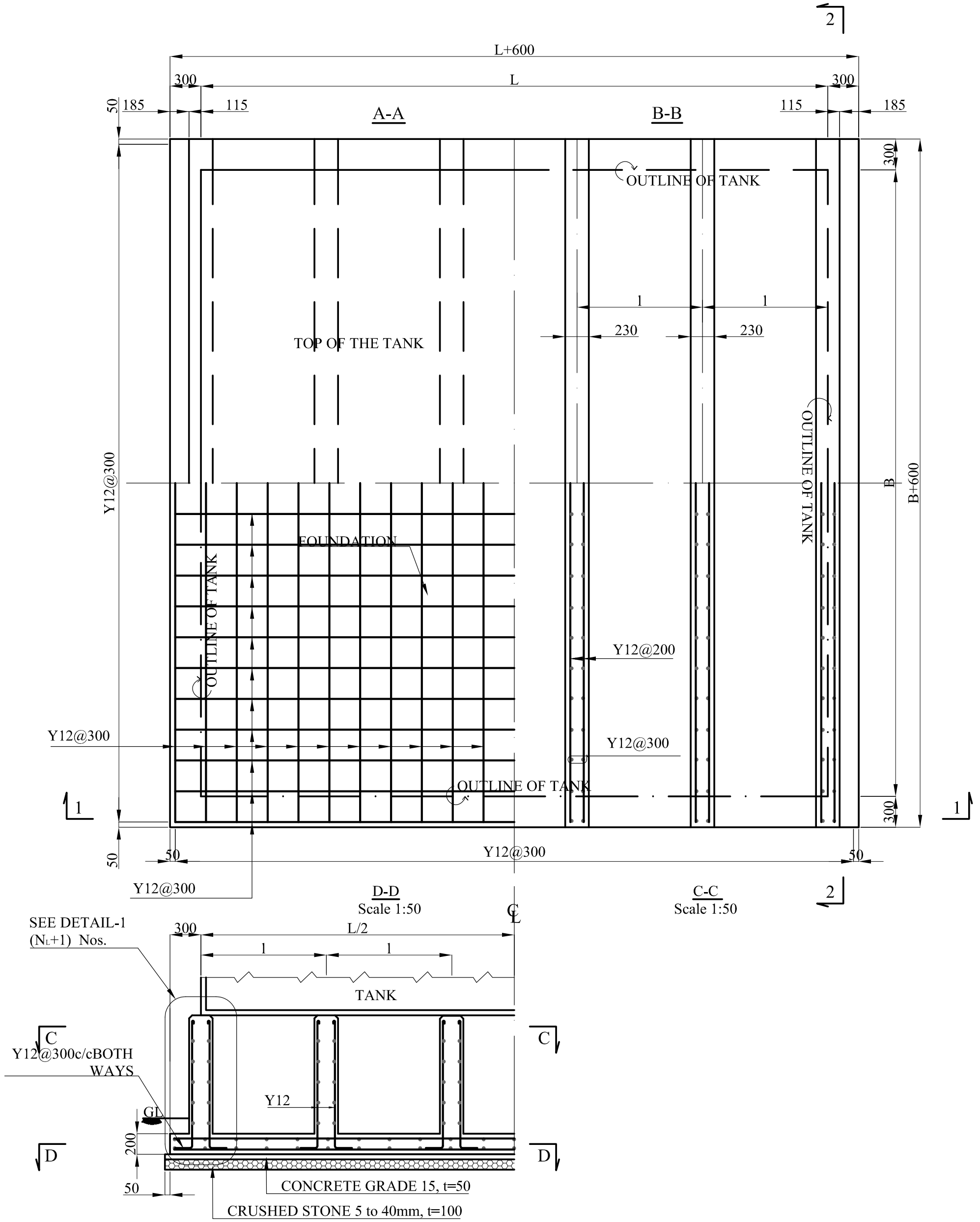
NOTE: FOR LADDER DETAILS SEE DRAWING NO SP-063

LEGEND:

- GI/uPVC STEPPED COUPLING
- GATE VALVE
- PIPELINE

OWNER:	PROJECT NAME:	CONSULTING ENGINEERS:	
THE MINISTRY OF WATER AND IRRIGATION THE REPUBLIC OF KENYA	THE PROJECT FOR RURAL WATER SUPPLY	NIPPON KOEI CO.,LTD.	CONSTRUCTION OF WATER SUPPLY FACILITIES BY SUBMERSIBLE PUMP STORAGE TANK (2/2) SCALE: 1:60 · DATE: OCT 2010 · DRAWING NO.: SP-062B

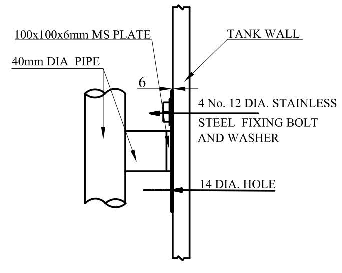
Tank Type	Storage Tank Vol Class (m^3)	Storage Tank Vol Actual (m^3)	Square Side l (m)	Dimension & Number L (m)	N_L (nos.)	B (m)	N_B (nos.)	D (m)	N_D (nos.)	Weight & Tank Empty (t)	Full Water (t)	Ladder Height h (m)	SERIAL No.
V4	4.0	4.0	1.00	2.00	2	2.00	2	1.00	1	0.7	4.7	0.70	200
V8	8.0	8.0	1.00	2.00	2	2.00	2	2.00	2	1.2	9.2	0.70	123,142,162,
V15	15.0	16.3	1.22	3.66	3	3.66	3	1.22	1	2.4	18.7	0.70	102
V24	24.0	29.1	1.22	4.88	4	2.44	2	2.44	2	3.1	32.2	1.20	98A,127A,151,163,167,177, 178A,186,187A,188A,191A, 195A
V50	50.0	54.5	1.22	6.10	5	3.66	3	2.44	2	4.8	59.2	1.20	96A,121,133,137A,156A,158, 172,180,183,189,197



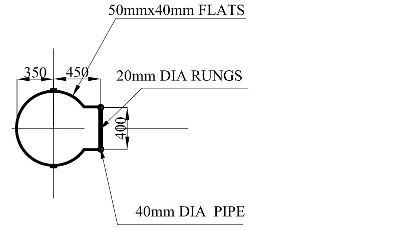
D-D
Scale 1:50

C-C
Scale 1:50

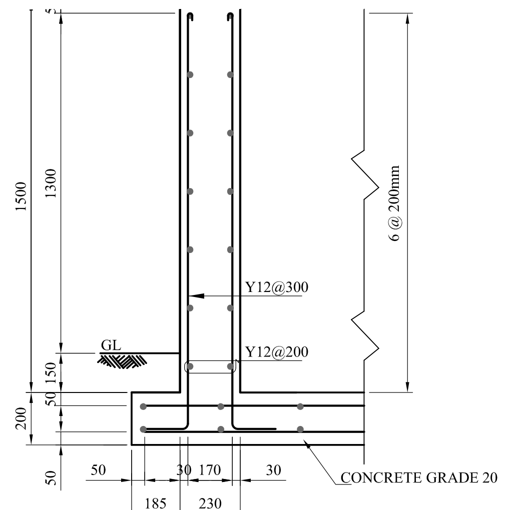
Scale 1:50



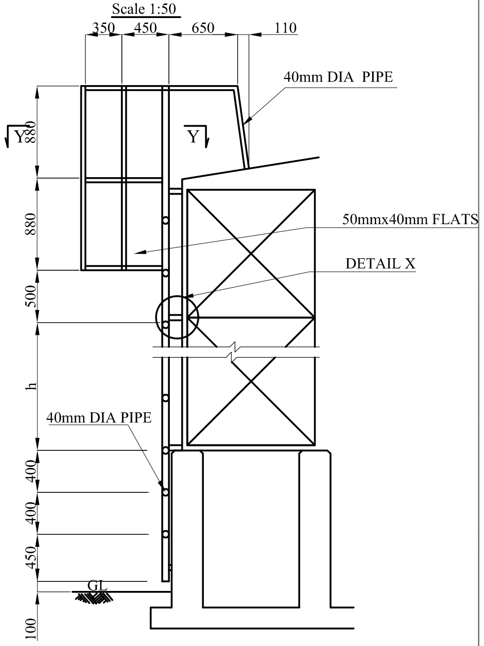
DETAIL X
Scale 1:5



SECTION Y-Y
Scale 1:50

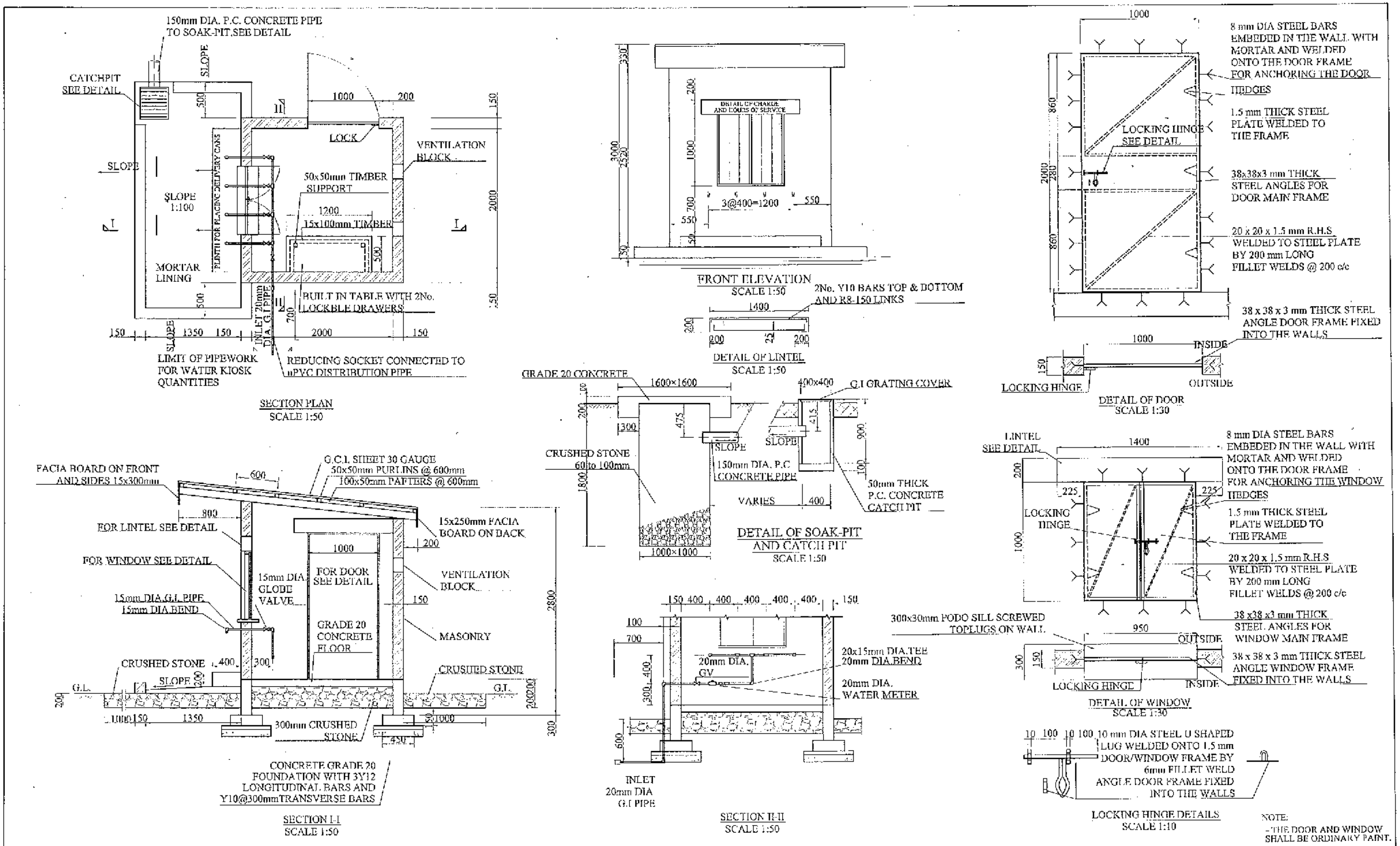


DETAIL 1
Scale 1:20



SECTION Y-Y
Scale 1:50
STEEL LADDER DETAILS

OWNER:	PROJECT NAME:	CONSULTING ENGINEERS:	
THE MINISTRY OF WATER AND IRRIGATION THE REPUBLIC OF KENYA	THE PROJECT FOR RURAL WATER SUPPLY	NIPPON KOEI CO.,LTD.	CONSTRUCTION OF WATER SUPPLY FACILITIES BY SUBMERSIBLE PUMP STORAGE TANK SCALE 1:5 1:20 1:50 — DATE OCT 2010 — DRAWING NO. SP-063

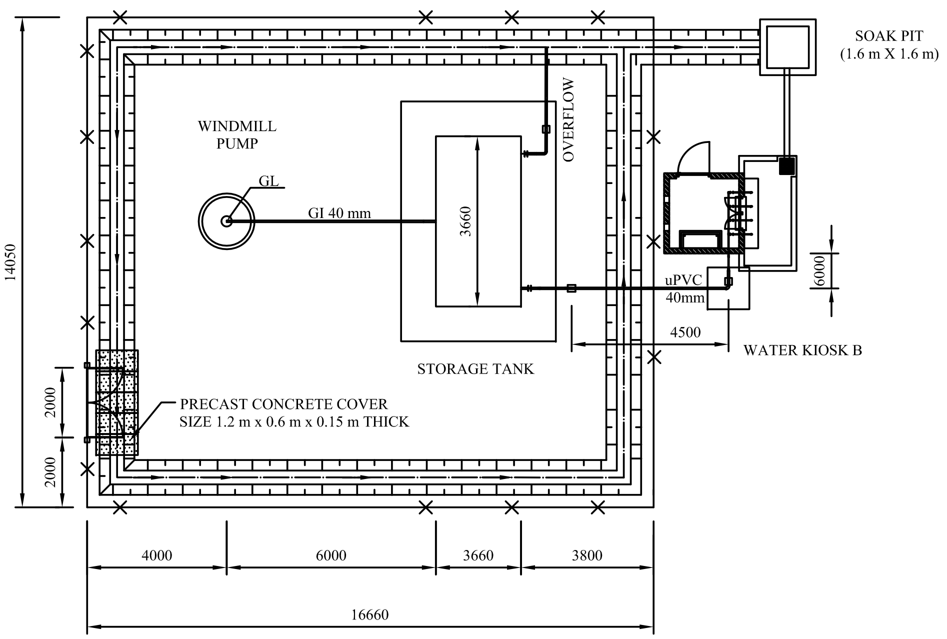


NOTE:
- THE DOOR AND WINDOW SHALL BE ORDINARY PAINT.

OWNER:	PROJECT NAME:	CONSULTING ENGINEERS:	TITLE:
THE MINISTRY OF WATER AND IRRIGATION THE REPUBLIC OF KENYA	THE PROJECT FOR RURAL WATER SUPPLY	NIPPON KOEI CO.,LTD.	CONSTRUCTION OF WATER SUPPLY FACILITIES BY SUBMERSIBLE PUMP WATER KIOSK B SCALE: 1:30 1:50 · DATE: JAN 2010 · DRAWING NO.: SP-068B

NOTES:

1. FOR DETAILS OF PRECAST CONCRETE COVER, GATE, FENCE AND DRAIN uPVC DITCH, SEE DRAWING No. SP-069 & 070.
2. FOR DETAILS OF STORAGE TANK, SEE DRAWING No. SP-062A TO 063.
3. CONSTRUCTION WORKS FOR 1) FENCES, AND 2) DRAIN OUTLETS ARE DONE BY RURAL COMMUNITY PARTICIPATION BASED ON THE UNDERTAKINGS OF THE GOVERNMENT OF KENYA.

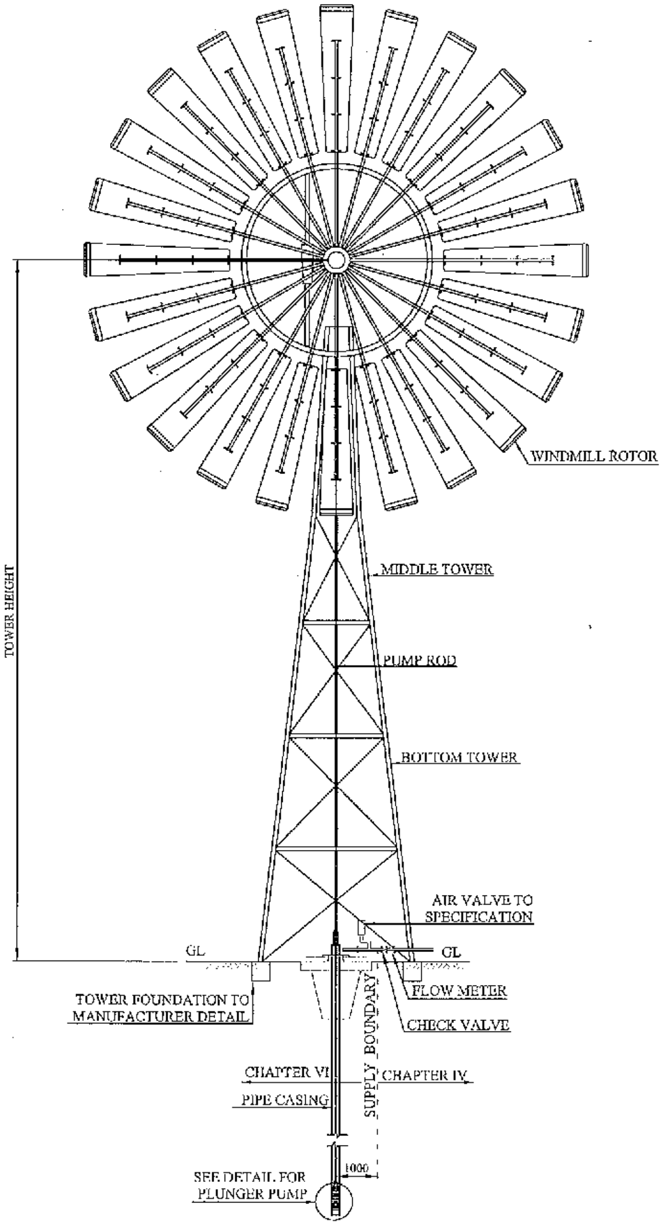


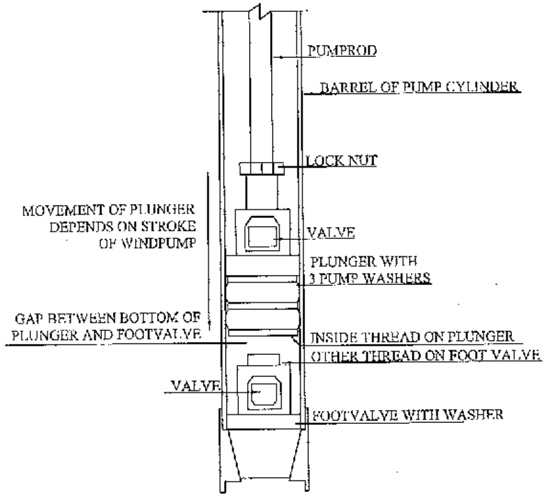
SERIAL. NO. 102

LEGEND:

- : PRECAST CONCRETE COVER
- : GATE
- : CHAIN-LINK FENCE
- : CUT SLOPE (GRADIENT 1:0.5)
- : WATER SUPPLY PIPE

OWNER:	PROJECT NAME:	CONSULTING ENGINEERS:	TITLE:		
THE MINISTRY OF WATER AND IRRIGATION THE REPUBLIC OF KENYA	THE PROJECT FOR RURAL WATER SUPPLY	NIPPON KOEI CO.,LTD.	CONSTRUCTION OF WATER SUPPLY FACILITIES BY WINDMILL PUMP LAYOUT PLAN OF TYPE W1		
			SCALE 1:150	DATE OCT 2010	DRAWING NO. WP-001





DETAIL OF PLUNGER PUMP

DESIGN DATA FOR PUMPS

Serial No.	Discharge (m3/d)	Well Depth (m)	Ground Elevation (m.)	Static Water level (GL-m.)	Low Water Level of Pump Well (GL-m.)	Water Level of Storage Tank (GL+m.)	Dia.of Windmill Rotor (m)	Height of Tower (m)
102	12.6	155	G.L.	5.00	137.0	2.37	7.9	9.1

OWNER:	PROJECT NAME:	CONSULTING ENGINEERS:	TITLE:
THE MINISTRY OF WATER AND IRRIGATION THE REPUBLIC OF KENYA	THE PROJECT FOR RURAL WATER SUPPLY	NIPPON KOEI CO.,LTD.	CONSTRUCTION OF WATER SUPPLY FACILITIES BY WINDMILL PUMP WINDMILL PUMP SCALE: NONE DATE: OCT 2010 DRAWING NO.: WP-002

8.12
「Practice Manual for Water Supply Services」抜粋

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 principal 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 l/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	l/head/ day	60	50	40	250	150	75
People without connections	l/head/ day	20	15	10	-	-	20
Livestock unit	l/head/ day	50			-		
Boarding schools	l/head/ day	50					
Day schools with WC without WC	l/head/ day	 25 5					
Hospitals Regional District other	l/bed/ day	 400 200 100 } + 20 l per outpatient and day (minimum 5000 l/day)					
Dispensary and Health Centre	l/day	5000					
Hotels High Class Medium Class Low Class	l/bed/ day	 600 300 50					
Administrative offices	l/head/ day	25					
Bars	l/day	500					
Shops	l/day	100					
Unspecified industry	l/ha/day				20,000		
Coffee pulping factories	l/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^{0}C 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 (•) 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/l	600	
Colour	TCU	50	
Copper	mg/l	1.5	
Iron	mg/l	1.0	
Manganese	mg/l	0.5	
pH	-	6.5 – 9.2	
Solids	mg/l	1500	
Turbidity	NTU	25	
Zinc	mg/l	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/l in standing or 100 mg/l 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_2SO_4/liter and $MgSO_4$.

- 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/l $CaCo_3$ and of an oxygen content of at least 6 mg/l of $v < 0.5$ m/s or 2 mg/l 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/l (respectively 2.0 mg/l 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/l 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

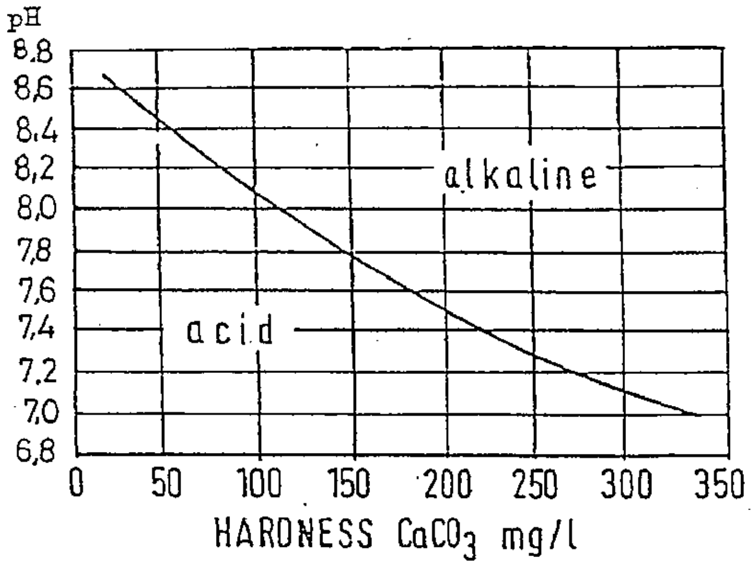


Figure 3.2 Aggressivity towards cement products (Concrete, mortar, AC pipes) depending on the hardness and the free CO2

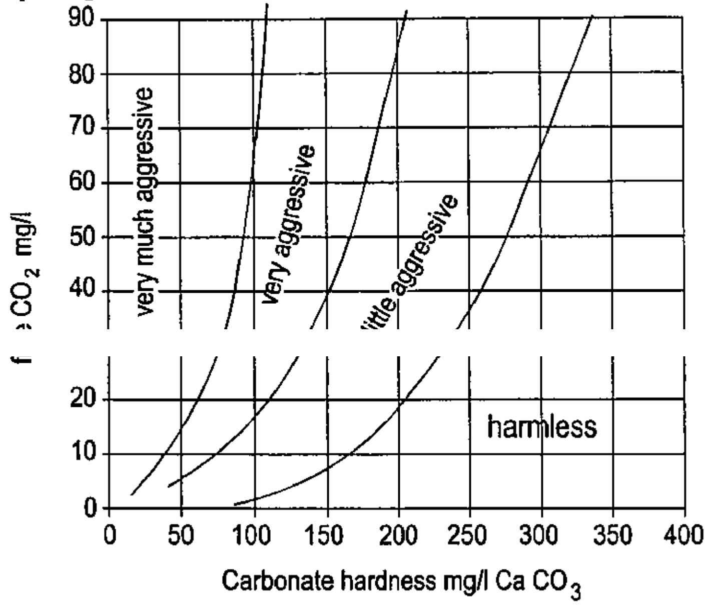
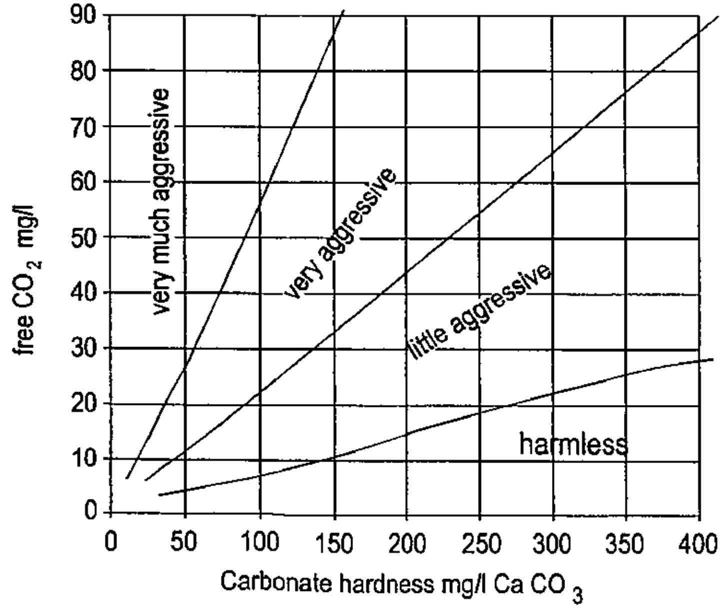


Figure 3.2 Aggressivity towards iron products (Steel pipes) depending on the hardness and the free CO2



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 Safe Yield for Principal 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 Safe 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

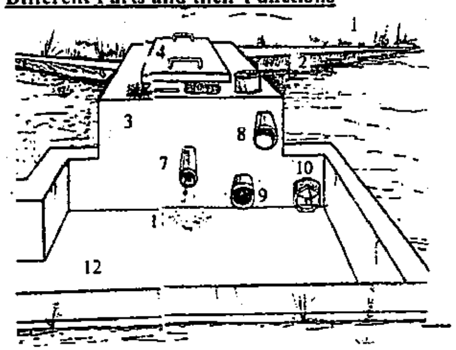
The best way to locate adequate springs and to get information about their reliability during dry spells is to interview people resident in the area.

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^{-4}	10^{-5} – 10^{-9}	10^{-9} – 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 months, 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:-

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

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:-

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

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:-

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

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.

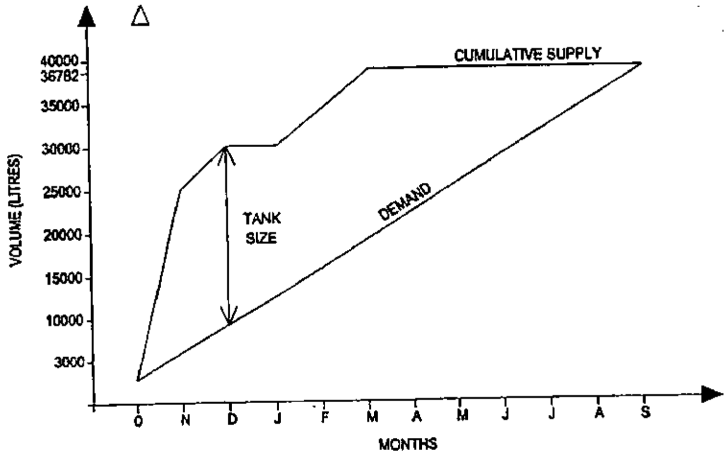


Figure 4.1: Graph of cumulative supply and demand

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:-

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

The head loss will be

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

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

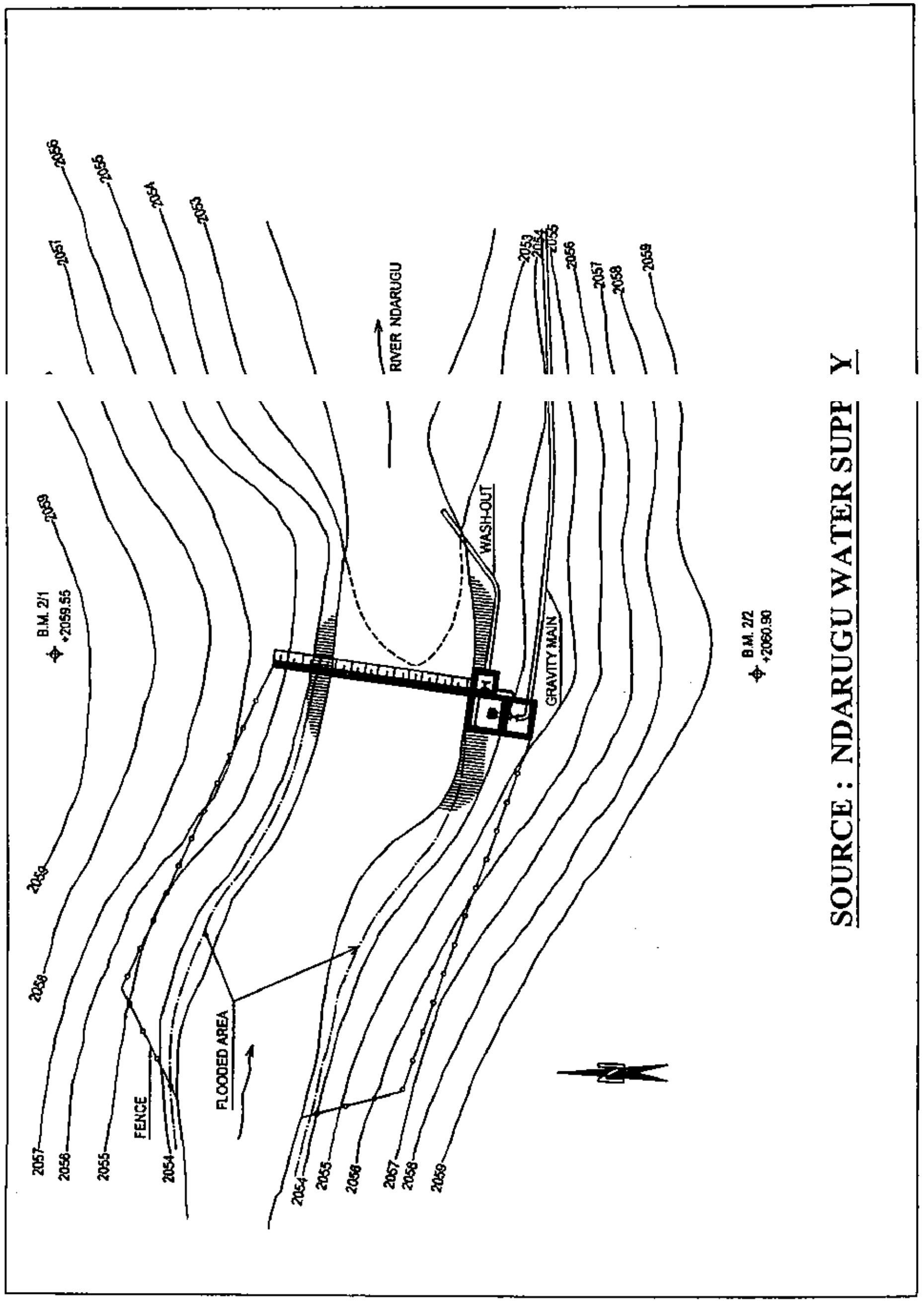
5.4.1 Protection

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.



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.

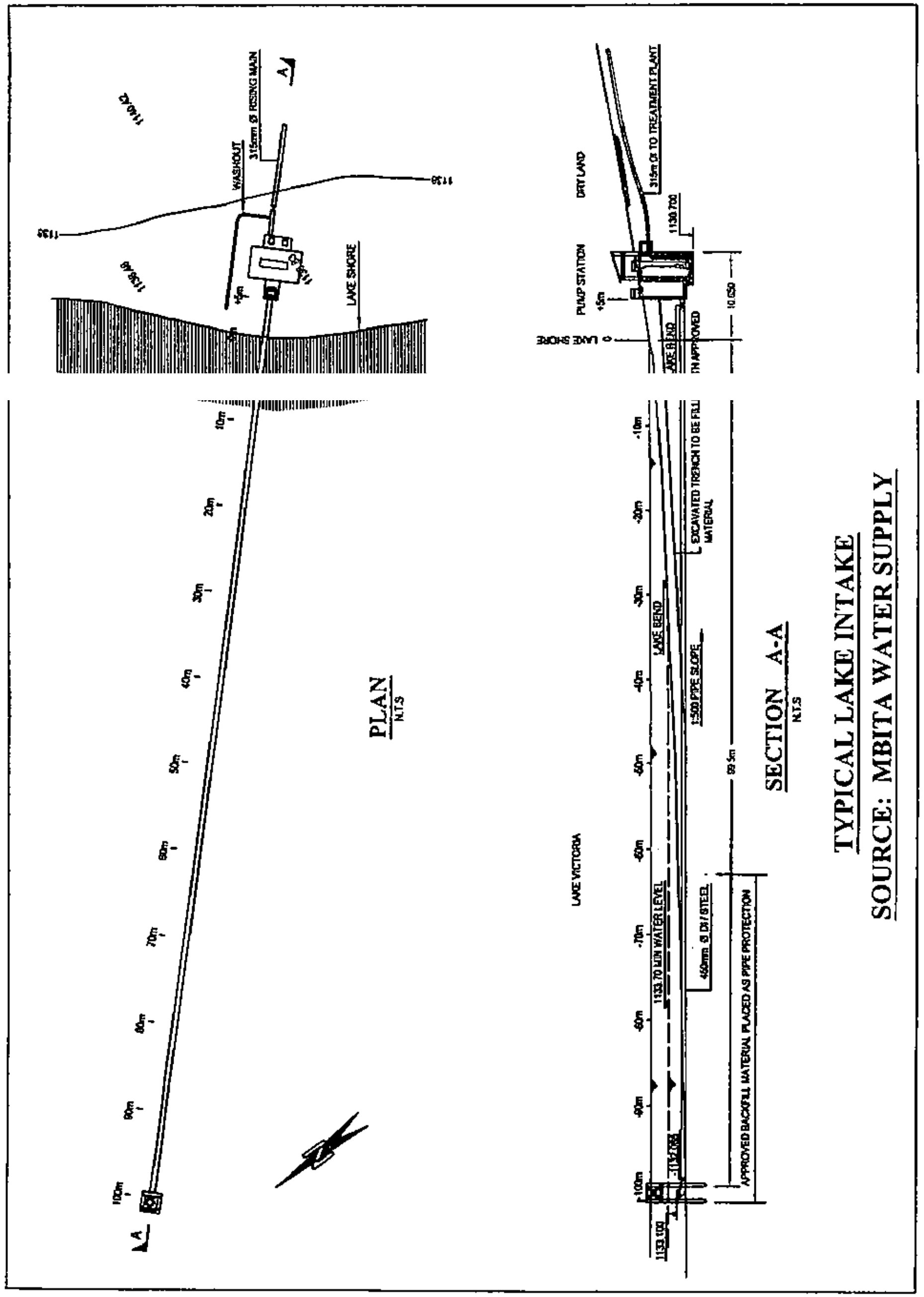
5.5.4 Foul Flush

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



TYPICAL LAKE INTAKE
SOURCE: MBITA WATER SUPPLY

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:

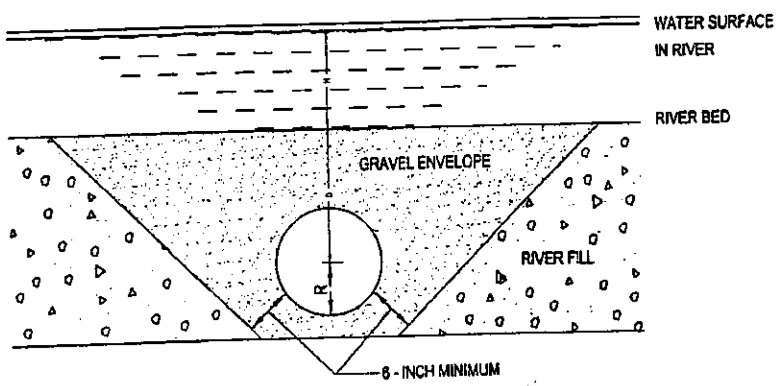
$$L = \frac{Q \ \ln \frac{2d}{r}}{2 \cdot KH}$$

Whereas:

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

ln = 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.

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.

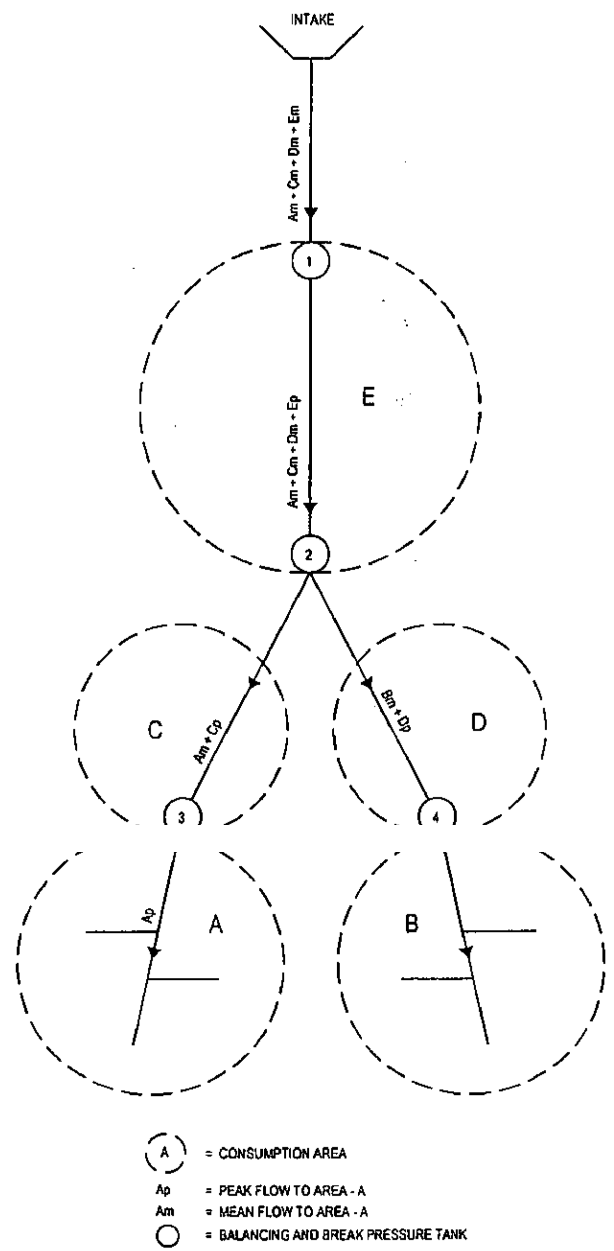
7.1.3 Pipeline Design

- 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 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.

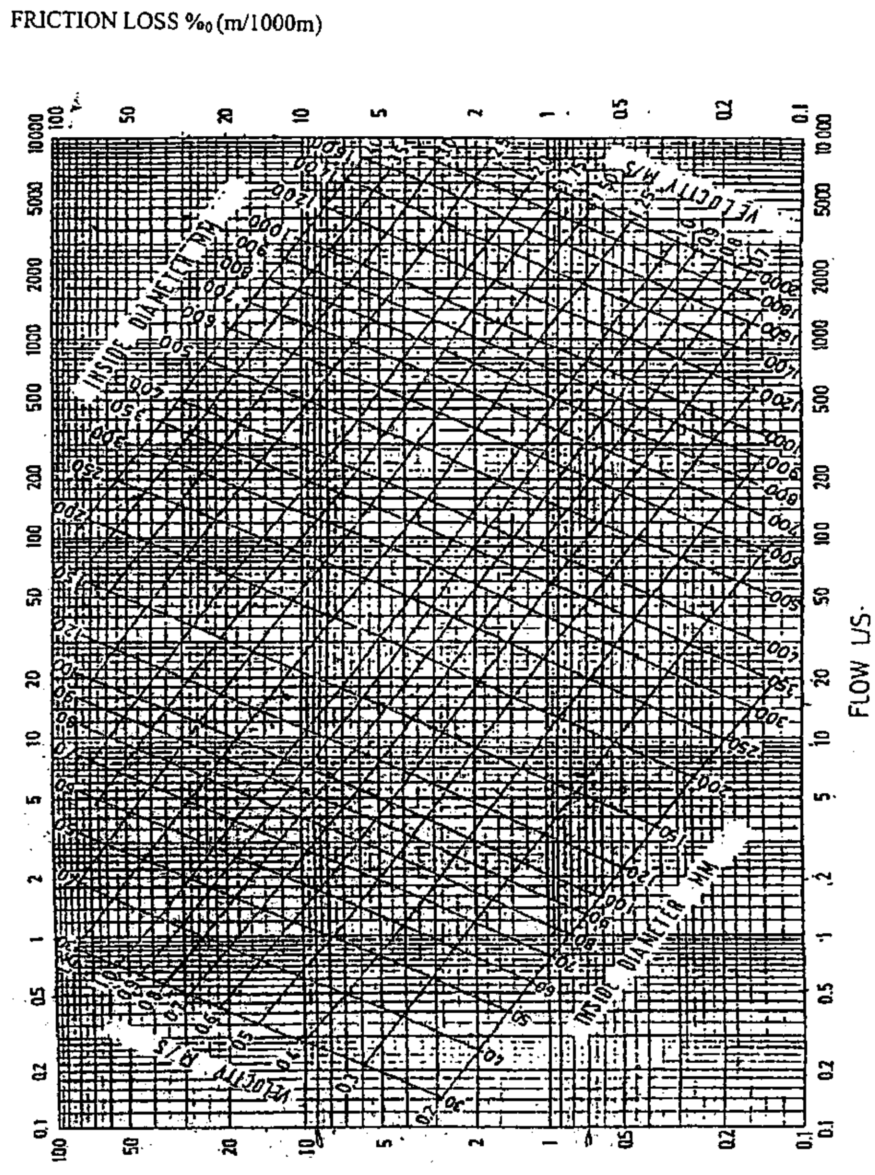


Fig 7.2: Loss of head for the flow of water in straight pipes, k = 0.1mm

FRICTION LOSS ‰ (m/1000m)

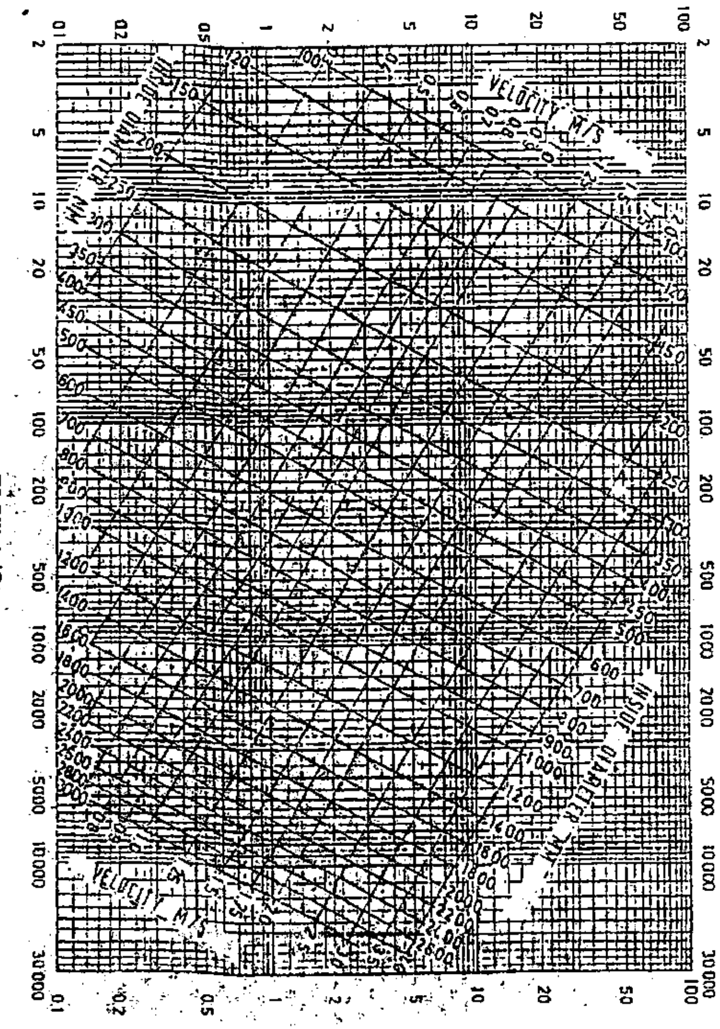


Fig 7.3: Loss of head for the flow of water in straight pipes, k = 1mm

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 = ± •V where also C = •g

(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 • 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 •
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 •	Celerity m/s	Factor •
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 l/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 Ox ide
2: When pipeline is coated with colored paint, epoxy paint should be app lied 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	Less than 700 ohm-cm	20
Depth or water saturated miller soil box)	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	5
	2 to 4	3
	4 to 6.5	0
	6.5 to 7.5	0*
	7.5 to 8.5	0
	More than 8.5	3
Redox potential	More than 100mv	0
	50 to 100 mv	3.5
	0 to 50 mv	4
	Less than 0 mv	5
Moisture content	Poor drainage and continuously wet:	2
	Fair drainage and generally moist:	1
	Good drainage and generally dry:	0
Sulphide content	Positive	3.5
	Trace	2
	Negative	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 $V = C \times R^{2/3} \times I^{1/2}$
(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 $10 N/m^2$ 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

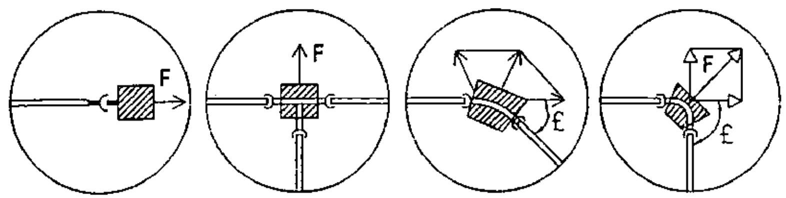
7.13.1 Location

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	0.31	0.06	0.12	0.16	0.24	0.31	0.44
32	0.80	0.16	0.31	0.41	0.61	0.80	1.13
50	1.96	0.38	0.76	1.01	1.50	1.96	2.77
63	3.12	0.61	1.22	1.62	2.39	3.12	4.41
90	6.36	1.25	2.48	3.29	4.87	6.36	8.99
110	9.50	1.86	3.71	4.92	7.27	9.50	13.44
160	20.11	3.94	7.85	10.41	15.39	20.11	28.44
225	39.76	7.79	15.51	20.58	30.43	39.76	56.23
280	61.58	12.07	24.03	31.88	47.13	61.58	87.09
315	77.93	15.28	30.41	40.34	59.65	77.93	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 l/h and a ¾ inch tap 1500 l/h at a pressure of 0.1 MPa (10 mhw). For other pressures the rate of delivery can be calculated with the formula:

$$q\,act = q\,nom \bullet (10H_{tap})$$

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:

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

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 $5\text{-}17m^3$ 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 l/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.

<u>Example</u>: 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

Principal 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 l/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 l/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 l/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 l/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 l/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°C. 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 l/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

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

Hence, select a pump, which have a NPSH of 4.15m or less for the capacity of 40 l/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: $NPSH_{req} \leq 7.9 + 3 - 0.75 = 10.10m$
Hence, select a pump, which has a NPSH of 10.15m or less for the capacity of 40 l/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:

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

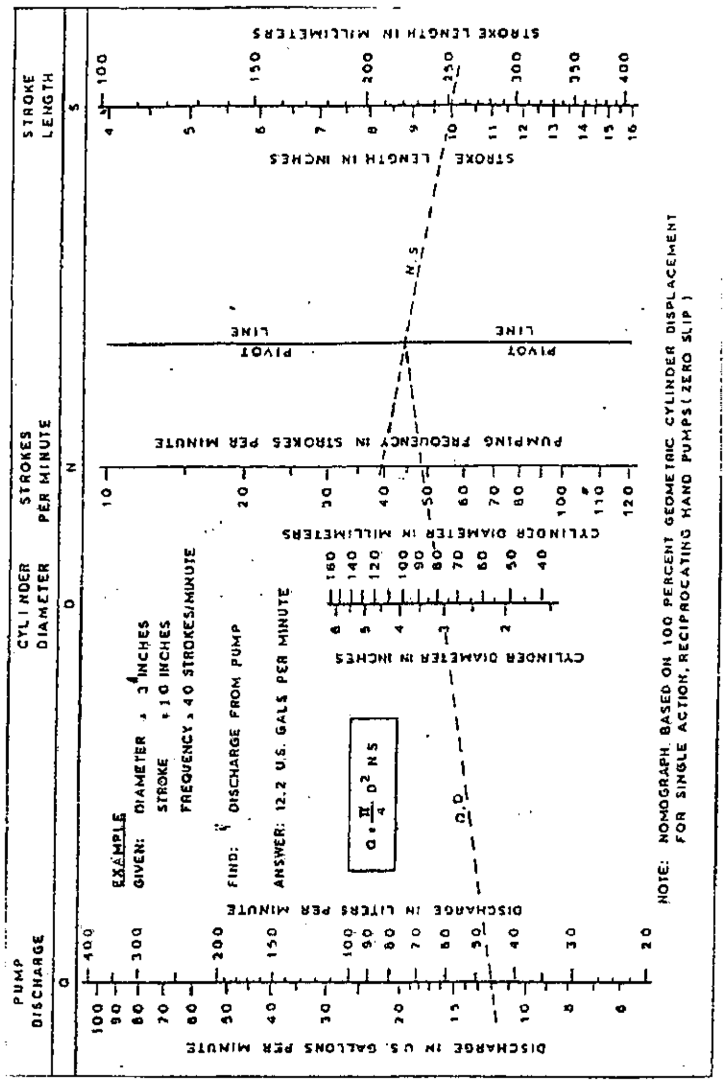
Where
H = head of water, metres
e = pumping efficiency (will have a value between 0 and 1)

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 l/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:

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

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 $5^{0}C$ that the air inlet temperature rises above $30^{0}C$. 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:-

  $$Q = \left\{ \frac{2.8 \times D^2 \times V^3 \times e}{H} \right\} \text{ l/min}$$

  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 calcu lated with the following formula:

$$P_T = \left(\frac{1.16 \times L}{I_D} \right) \text{ kWp}$$

Where P_T is the total power of the module at the operating temperature which in this case has been assumed to 60°C.

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. 10.4: Mean total 24-hour distribution of radiation (W/m^2) over Kenya in July (averaged over ten years or as shown in brackets)

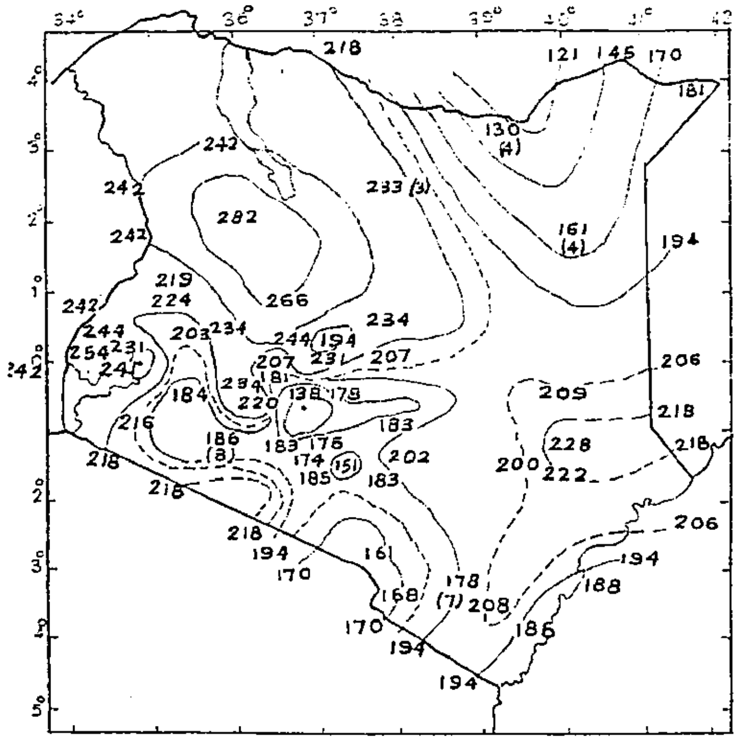


Figure No. 10.3: Mean total 24 hour distribution of radiation (W/m^2) over Kenya per year (average over ten years or as shown in brackets)

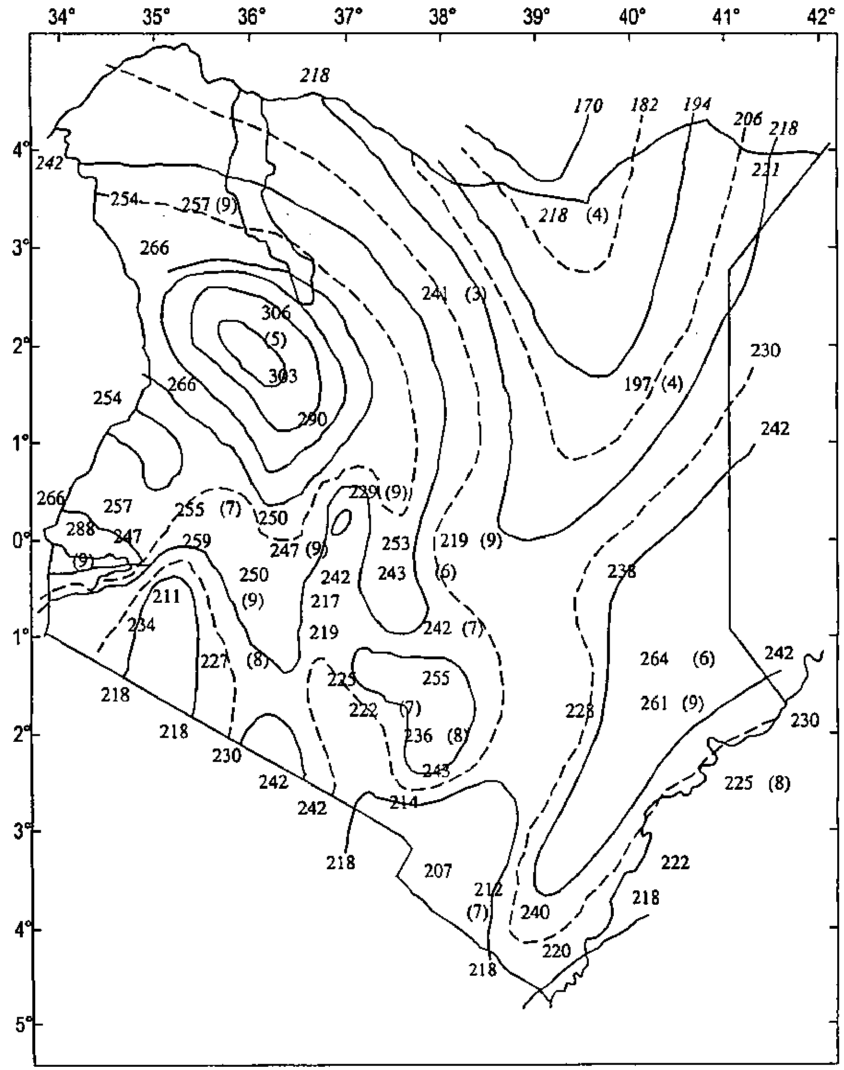
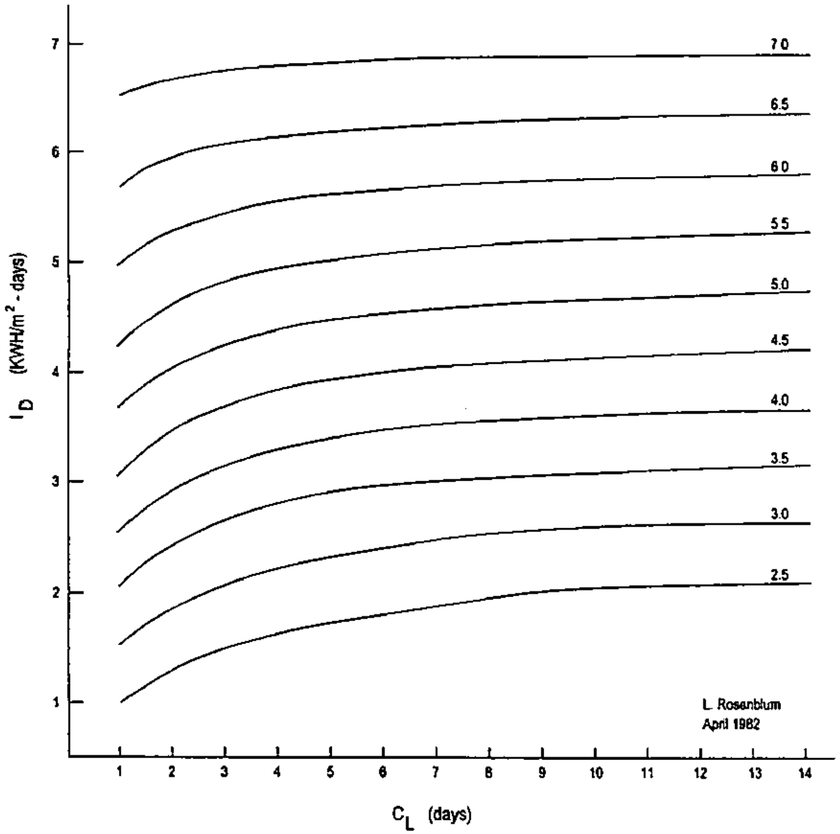


Figure No. 10.5: The 90% probability radiation, I_D, as a function of the average radiation in the worst month, I_{WM}, and the period C_L.



10.6 PUMP AND PIPE INSTALLATIONS IN THE PUMP HOUSE

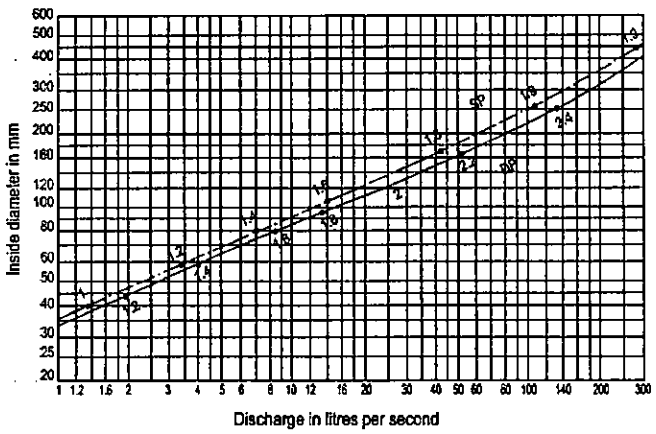
10.6.1 Pump Installation and Pump House

- 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.

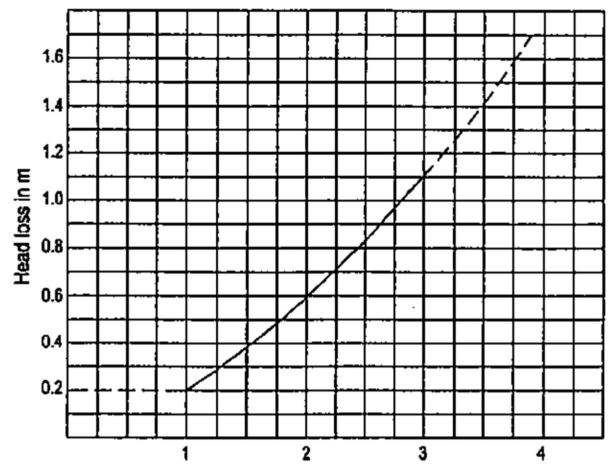
. Losses in the strainer and foot valve are shown in figure No. 10.7 below

. Losses in 90° 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

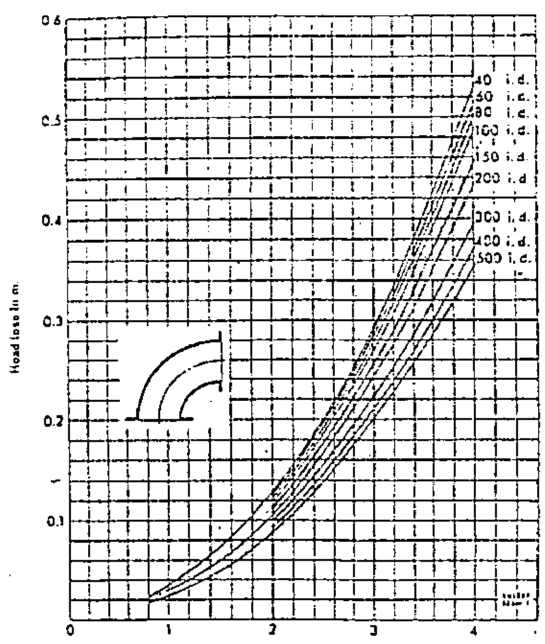


Velocity of liquid in m./sec/

Head losses due to friction in strainers with foot valves

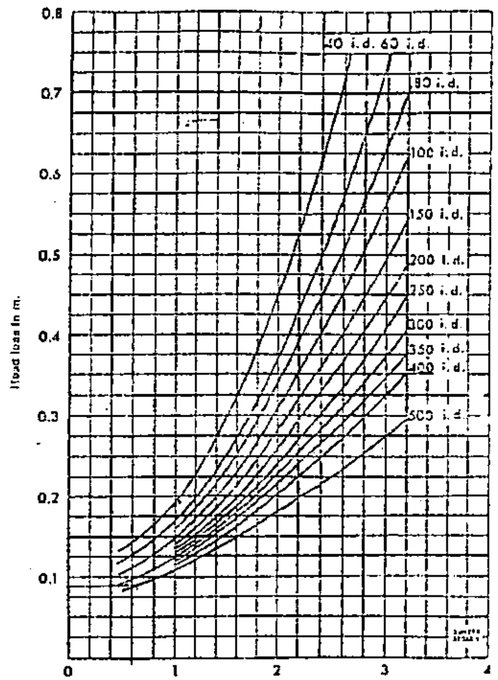
(valid up to 300mm diameter)

Figure No. 10.8: Head losses in 90 ° standard bends



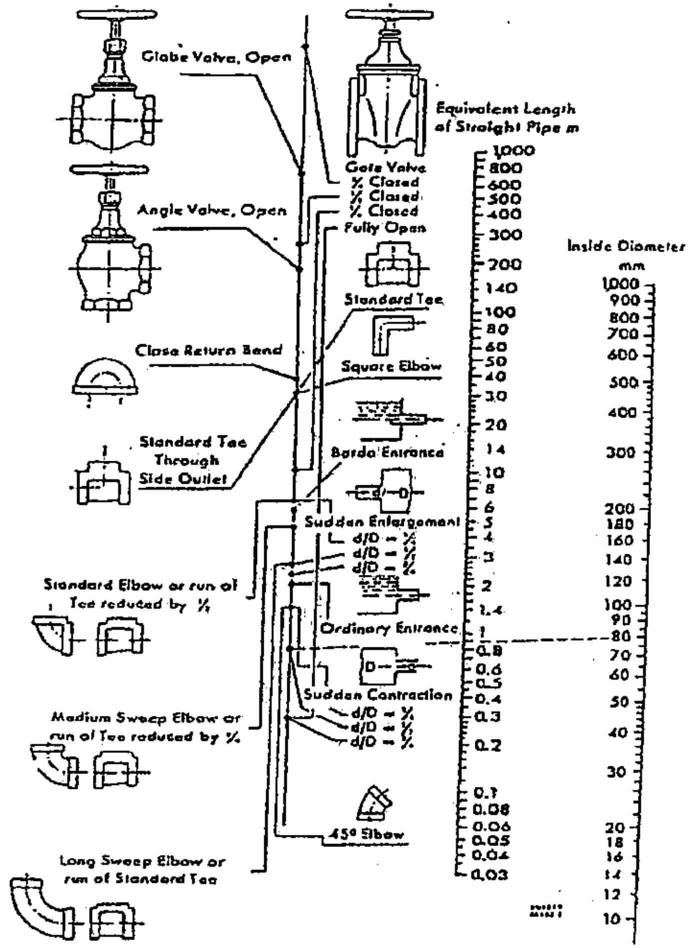
. Head losses in used 90° standard bends

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 STAND-BY 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:

$$P = \tfrac{1}{2} d A V^3$$

Where:

P = Power (KU)
d = Density of Air = $1.2 kg/m^3$ (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	107	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.

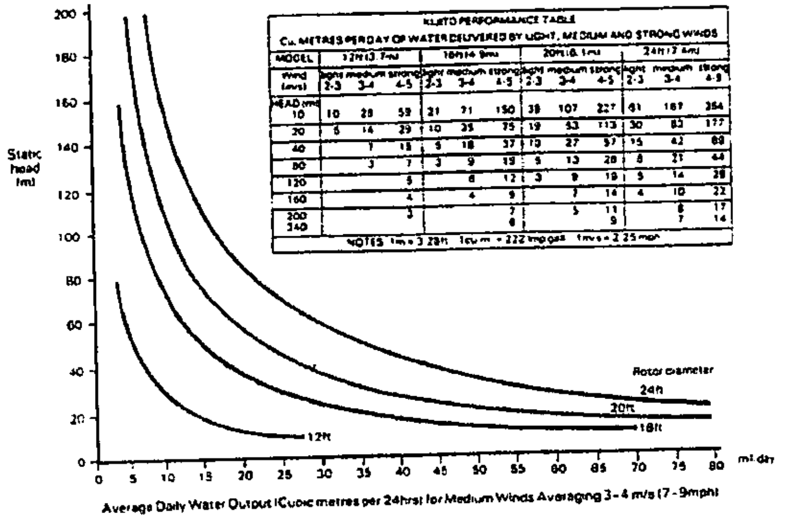


Fig. 10.11. Performance data for Kajito wind pump range based on the IT wind pumps (Fraenkel 1986)

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 Ø)	Annually
	4	Inspect/operate large valves (>250mm Ø) 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	Weekly
		• Check water levels	Weekly
		• Inspect general condition	Annually
		• Complete inspection including tank drainage and checking interior condition and for sedimentation build-up.	
		<5,000m^3	Every 5 years
		5000-50,000m^3	Every 10 years
		Over 50,1000m^3	Every 20 years.
	4	Pumps	
		• Check operation of routinely operating pumps	Daily
		• Check operation of standby operation pumps	Weekly
		• Check operation of standby generation equipment	Weekly
		• Pump effectiveness and performance testing	Annually
C		Water Quality	
	1.	Chlorine residual	Daily
	2.	Turbidity	
		• Surface water	Daily
		• Ground water	Every 2 years
	3	Bacteriological	Monthly
	4	Primary drinking water standards to KBS Standards	
		• Surface water	Annually
		• Ground water	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.

8.13
社会条件調査結果

井戸番号	A. 対象村落一般情報						B.人口とインフラ						
	村落名および行政区分						1.人口情報				2. 舗装道路からの距離(km)	3. 商用電力供給	
	大県 (Lager District)	県 (District)	郡 (Division)	Location	Sub-location	村落 (Village)	全体人口	男性数	女性数	世帯数		電化の有無	電化計画の有無
96	Makueni	Makueni	Wote	Kikumini	Kambimawe	Muambani	1533	667	867	120	2.5	no	no
98		Makueni	Wote	Muvau	Muvau	Nguumo	470	200	270	72	0.5	no	yes
98		Makueni	Wote	Muvau	Muvau	Nguumo	934	307	628	74	1	yes	-
100		Makueni	Wote	Kako	Kako	Kyaume	2010	810	1200	200	1	no	no
102		Makueni	Kaiti	Ukea	Kilala	Kithunzi	767	327	441	95	0.001	yes	-
107		Mbooni east	Kisau	Waia	Usalala	Kyang'ondu Primary	1467	600	867	167	0.25	yes	-
108		Mbooni east	Kisau	Kisau	Usalala	Kisau Health Centre	1867	693	1173	246	0.3	no	no
110		Nzaui	Matiliku	Kilili	Wee	Kanzili	400	150	250	66	1.5	no	no
111		Nzaui	Matiliku	Kilili	Kilili	Syaolwe	723	350	363	55	15	no	yes
112		Nzaui	Matiliku	Kilili	Mulenyu	Loyal turban	1259	814	1003	510	18	no	yes
113		Nzaui	Matiliko	Kilili	Mulenyu	Mboani	450	180	270	45	4	no	no
114		Nzaui	Mbitini	Mulala	Ng'ethe	Kitandi	2725	1075	1650	647	2	no	yes
118		Nzaui	Kalamba	Kithumba	Kithumba	Mathanguni	431	209	222	140	14	no	no
121		Mbooni east	Kalawa	Katengine	Ititu	Ititu	6000	2000	4000	600	1	no	no
123		Mbooni east	Kalawa	Kawala	Mbukoni	Ngunini	870	370	500	68	0.2	no	no
124		Mbooni east	Kalawa	Athi	Miangeni	Kyamutuku	775	260	515	123	1.5	no	no
127		Mukaa	Malili	Ngaamba	Itumbule	Kalembwani (Uvunye)	1550	650	900	144	0.5	no	yes
128		Mukaa	Kilome	Mukaa	Mukaa	Enzae-Maiani	1900	800	1100	230	1	yes	no
130		Mukaa	Kiou	Kwalee	Kwalee	Ndivo	1700	733	967	283	5	no	-
131		Mukaa	Kiou	Kiou	Lumu	Lumu	2100	900	1233	307	1	no	no
133		Mukaa	Kasikeu	Kasikeu	Wathini	Mangala	3500	1800	2367	340	0.25	yes	-
134		Mukaa	Kiou	Muani	Muani	Nguuni	1003	515	654	243	5	no	no
137		Nzaui	Nguu	Kikulumi	Ndunguni	Mbulutini	1000	407	593	300	3	no	yes
140		Nzaui	Nguu	Wolma	Wolma	Ilingoni	424	140	284	33	5	no	no
142		Kibwezi	Mtito Adei	Nthunguni	Nthingumi	Utu	413	200	213	53	15	no	yes
145		Kibwezi	Mtito Adei	Ngawate	Mukange	Yongoni	506	194	312	49	17	no	yes
146		Kibwezi	Mtito Andei	Kambu	Kitengei	Kietengei/Nguuswini	637	277	360	124	14	no	yes
148	Machakos	Masinga	Masinga	Kangonde	Kangonde	Kangonde	933	333	600	133	0.5	yes	yes
151		Masinga	Masinga	Kivaa	Kivaa	Kamunyu	633	233	400	200	3	no	no
152		Masinga	Masinga	Kivaa	Kivaa	City Cotton	317	133	183	50	0.3	no	no
156		Yatta	Kithimani	Kithimani	Kithimani	Nguumo	1433	633	800	277	1	no	no
158		Yatta	Yatta	Mavoloni	Kisiiki	Kisiiki Centre	1533	550	983	333	1	yes	no
162		Yatta	Katangi	Kyua	Kyua	Kikeneani	3000	1467	1533	417	1.5	no	no
163		Yatta	Katangi	Kyua	Syo Kisinga	Kiamani	1233	500	733	140	3	yes	no
164		Yatta	Katangi	Kyua	Syo Kisinga	Yumbuni	2833	1033	1800	483	0.5	no	yes
165		Yatta	Yatta	Ndalani	Ndalani	Ndalani Centre	2833	1400	1433	433	1	yes	-
166		Kathiani	Kathiani	Mitaboni	Miumbuni	Kwale	2600	1000	1600	453	0.02	no	-
167		Kangundo	Kakuyunin	Kaawethe	Kathaana	Mukukuni	2667	961	1367	324	1	no	-
172		Mwala	Masii	Makutano	Embui	Mumbuni	2833	1300	1533	467	3	yes	no
173		Machakos	Mwala	Kathama	Kwa Mutula	Katitu	2167	800	1367	250	0.5	no	yes
175		Mwala	Masii	Mango	Wetaa	Mango	3500	1467	2033	442	1.5	yes	-
177		Mwala	Mwala	Kathama	Muthwani	Masaua	1267	533	733	243	2	no	yes
178		Machakos	Mwala	Kyawango	Kyawango	Mutiuku	1300	500	800	93	1	yes	no
180		Kangundo	Kakuyuni	Kakuyuni	Kyevaluki	Kamwanyani	1533	600	933	257	1.5	no	yes
183		Mwala	Yathui	Miu	Makuhimo	Miu	2000	967	1200	267	1	yes	no
184		Mwala	Yathui	Yathui	Kyamatula	Kikaso	3000	1167	1833	160	3	yes	-
185		Mwala	Yathui	Miu	Kikulumi	Kikulumi	3167	1367	1800	633	3	no	-
186		Mwala	Yathui	Yathui	Kyamatula	Yathui	2000	833	1167	150	0.5	yes	-
187		Mwala	Yathui	Wamunyu	Kilembwa	Ilingile	1667	700	967	433	9	yes	-
188		Masinga	Ndithine	Muthesya	Kikule	Muambani	633	273	360	150	0.1	no	-
189		Masinga	Ndithine	Ndithini	Ndithini	Ndithini	1533	600	933	180	0.2	yes	-
190		Masinga	Muthesya	Muthesya	Muthesya	Munyiiki	597	367	350	133	0.1	no	-
191		Masinga	Ndithini	Mananja	Mananja	Mananja	2500	1000	1500	300	1.5	no	yes
195		Machakos	Ndithini	Mananja	Mananja	Ndela	1367	567	800	190	0.1	no	no
196		Masinga	Ndithini	Ndithini	Milaani	Milaani	1667	810	1190	493	0.01	no	no
197		Masinga	Ndithini	Ndithini	Milaani	Kamaimba	2267	1067	1200	367	0.007	no	no
198		Machakos	Kalama	Kombo	Muumandu	Kyawalia	1767	767	1000	250	1	no	yes
199		Machakos	Kalama	Kola	Iiyuni	Iiyuni	2500	1000	1500	200	0.03	no	no
200		Machakos	Central	Kahtekakai	Kitanga	Kyamutheke	2500	967	1533	377	0.5	no	no

井戸番号	C.村落の経済情報									D.給水状況	
	1. 世帯毎の平均収入月額	2. 世帯毎の平均支出月額	3. 村落の家畜数（頭数）			4. 世帯毎の平均家畜数（頭数）			5. 生産協同組合の有無	1. 世帯当たりの日平均水利用量*	
			ウシ	ヒツジ	ヤギ	ウシ	ヒツジ	ヤギ		雨季	乾季
96	2,500	2,500	150	180	170	2	2	2	no	4 jr/day	8 jr/day
98	3,330	3,330	190	50	1170	2	3	5	no	5 jr/day	3 jr/day
98	2,700	2,700	80	50	150	2	1	2	no	6 jr/day	8 jr/day
100	2,330	2,330	120	130	240	3	3	7	no	4 jr/day	3 jr/day
102	2,330	2,330	180	230	120	2	2	3	no	6 jr/day	9 jr/day
107	2,330	2,330	60	30	80	2	1	2	no	10 jr/day	8 jr/day
108	3,000	3,000	120	40	170	1	3	2	no	4 jr/day	6 jr/day
110	3,000	3,000	340	20	1500	5	1	21	no	6 jr/day	9 jr/day
111	3,000	3,000	450	970	1870	3	4	6	no	10 jr/day	7 jr/day
112	3,230	3,230	710	590	2000	2	2	8	no	3 jr/day	4 jr/day
113	3,000	3,000	140	50	200	3	1	4	no	8 jr/day	10 jr/day
114	3,670	3,670	800	340	1190	5	7	10	no	4 jr/day	6 jr/day
118	2,000	1,730	1070	1100	1600	4	3	2	no	4 jr/day	6 jr/day
121	3,000	3,000	1270	480	1930	2	1	3	no	10 jr/day	9 jr/day
123	2,500	2,500	30	50	70	1	1	3	no	4 jr/day	4 jr/day
124	2,670	2,500	70	70	280	2	3	15	no	3 jr/day	4 jr/day
127	2,330	2,830	260	300	570	2	2	4	no	4 jr/day	5 jr/day
128	1,770	1,770	190	190	240	1	2	4	no	6 jr/day	8 jr/day
130	2,500	3,170	1130	170	2470	4	3	5	no	4 jr/day	6 jr/day
131	2,170	2,670	1100	370	1570	3	1	5	no	12 jr/day	8 jr/day
133	3,030	3,030	480	430	380	3	4	6	no	6 jr/day	9 jr/day
134	2,500	2,670	940	2120	1600	3	6	9	no	6 jr/day	7 jr/day
137	3,000	2,100	970	530	2830	2	4	10	no	6 jr/day	10 jr/day
140	2,000	2,000	70	150	230	3	5	7	no	6 jr/day	8 jr/day
142	3,170	3,170	300	140	720	4	3	13	no	10 jr/day	8 jr/day
145	2,430	2,600	40	90	210	2	4	6	no	6 jr/day	6 jr/day
146	2,830	2,830	70	90	170	3	5	9	no	4 jr/day	5 jr/day
148	6,000	5,330	230	200	370	3	2	8	no	11 jr/day	5 jr/day
151	7,670	6,000	200	150	400	4	3	8	no	10 jr/day	5 jr/day
152	5,670	5,670	100	120	270	3	2	7	no	19 jr/day	9 jr/day
156	6,670	8,000	90	100	530	2	1	6	no	8 jr/day	4 jr/day
158	1,870	7,000	260	30	470	2	3	6	no	9 jr/day	4 jr/day
162	1,500	5,000	80	100	620	1	2	5	no	8 jr/day	5 jr/day
163	1,670	3,830	420	820	1830	3	3	10	no	13 jr/day	7 jr/day
164	1,670	3,500	1430	1570	4670	5	9	15	no	10 jr/day	6 jr/day
165	7,670	7,000	380	130	6330	3	1	5	no	12 jr/day	5 jr/day
166	4,330	7,330	1170	1430	2070	2	3	5	no	9 jr/day	4 jr/day
167	5,840	6,170	280	150	500	4	4	6	no	8 jr/day	4 jr/day
172	4,330	6,330	700	630	1770	2	1	6	no	14 jr/day	7 jr/day
173	3,500	6,000	300	230	450	2	1	4	no	13 jr/day	9 jr/day
175	5,000	4,670	920	350	5920	3	1	5	no	14 jr/day	12 jr/day
177	4,500	3,670	610	250	1430	3	1	6	no	14 jr/day	7 jr/day
178	3,170	3,500	900	120	600	5	1	12	no	16 jr/day	15 jr/day
180	6,670	6,000	250	200	350	1	1	2	no	13 jr/day	10 jr/day
183	10,670	7,000	150	150	420	1	1	4	no	12 jr/day	5 jr/day
184	5,330	7,000	300	510	830	1	3	4	no	15 jr/day	8 jr/day
185	2,500	6,000	1150	530	1700	2	1	3	yes	14 jr/day	5 jr/day
186	3,670	4,670	230	150	730	2	2	5	no	13 jr/day	6 jr/day
187	3,000	4,500	430	220	880	2	1	4	no	14 jr/day	6 jr/day
188	5,500	5,670	230	80	250	2	1	5	no	8 jr/day	4 jr/day
189	10,000	8,000	2100	600	1730	6	3	9	no	11 jr/day	6 jr/day
190	7,000	6,330	270	60	400	2	1	4	no	10 jr/day	5 jr/day
191	1,830	3,170	210	80	300	3	1	5	no	8 jr/day	4 jr/day
195	5,330	8,170	650	220	550	8	4	7	no	10 jr/day	6 jr/day
196	2,330	4,330	2450	700	1760	3	2	10	no	8 jr/day	7 jr/day
197	5,670	4,330	930	420	1670	3	1	4	no	8 jr/day	7 jr/day
198	3,670	6,670	60	20	730	2	1	5	no	10 jr/day	8 jr/day
199	4,000	6,000	280	120	500	3	1	5	no	5 jr/day	5 jr/day
200	3,500	5,670	30	120	320	1	3	4	no	10 jr/day	5 jr/day

* jr/day : jerrycan per day

井戸番号	D.給水状況													
	2. 給水水源						3. 水くみ				4. 水料金			
	雨季			乾季			雨季		乾季		雨季		乾季	
	水源	距離	水質***	水源	距離	水質***	水源までの時間	従事者****	水源までの時間	従事者****				
96	River	2m	M	Borehole/Earth dam	1, 1.5km	G/B	3minutes	W	3 hours	C		Free	10 Ksh	jerrycan
98	Shallowwell/Earth dam	1/1km	M/B	Borehole	2km	G	2hours	C	2hours	W	3Ksh	Jerry can	5Ksh	jerrycan
98	River/Shallow well	3.2/1km	M	River/Shallow well	4/3km	M/M	2hours	W/C	3hours	W/C		Free	3Ksh	jerrycan
100	River	200m	M	River/Shallow well	3km	M/B	20minutes	C	2hours	C		Free	3ksh	jerrycan
102	Borehole	0.5km	M	River	1km	B	15minutes	W	1hour	C	5ksh	jerrycan	5ksh	jerrycan
107	River/R.catchment	200m/10m	M/G	River/Earthdam	3/1km	M/B	15minutes	W	2hours	O/C	4ksh	jerrycan	5ksh	jerrycan
108	River/R.catchment	1km	B/M	River	1km	B	2hours	W	2.5hours	C		Free	5ksh	jerrycan
110	Pipedwater	2km	M	Shallowwell/Pipedwater	1.5/2km	M/M	1.5hours	W/C	2hours	W/C	3ksh	jerrycan	5ksh	jerrycan
111	River/Spring	3/3km	M/M	River/Spring	3/3km	M/M	1hour	buy	2hours	B	fixed 12,000ksh	house/year	fixed 12,000ksh	house/year
112	River	5km	M	River	8km	M	1hour	W	3hours	W		Free	3ksh	jerrycan
113	River	2km	M	River/Pipedwater	8/3km	M/G	1hour	W/C	4hours	S/W/C	3ksh	jerrycan	5ksh	jerrycan
114	River	3.5km	B	River	5.5km	B	1hour	W	2hours	W		Free		Free
118	R.catchment	0	M	River	5km	B	20minutes	W	2hours	W		Free	3ksh	jerrycan
121	River/R.catchment	3km	B/B	River	3km	B	2.5hours	W	2.5hours	W	5ksh	jerrycan	5ksh	jerrycan
123	Earthdam	2km	B	River	4km	B	2.5hours	C	2.5hours	C		Free	20ksh	jerrycan
124	River/R.catchment	1km	M/M	River	5km	B	30minutes	S/W	2hours	S/W		Free	40ksh	jerrycan
127	River/Water hole	1.5/1.5km	B/M	River/Waterhole	1.5/1.5km	M/M	1hour	S/W/C	1.5hours	S/W/C	2ksh	jerrycan	2ksh	jerrycan
128	River	3km	B	River	4km	B	1hour	W/C	3hours	W		Free	2ksh	jerrycan
130	River	2km	B	Borehole	4km	M	1hour	S/W/C	1hour	S/W/C/B	2ksh	jerrycan	2ksh	jerrycan
131	R.catchment	-	M	Kiosk	3km	M	30minutes	W/C	3hours	W/C	3ksh	jerrycan	3ksh	jerrycan
133	River/Pipedwater	2/3km	B/G	River	3km	B	1hour	W/C	2hours	W/C	2ksh	jerrycan	5ksh	jerrycan
134	River/Borehole/Shallow well	4/5/6km	B/M/B	Borehole/Shallow well	5/6km	M/B	3hours	S/W/C	6hours	S/W/C	3ksh	jerrycan	5ksh	jerrycan
137	River/Borehole/R.catch.	5/5/-km	B/B/G	River/Borehole	5/5km	G/G	2hours	W/C	8hours	W/C	2ksh	jerrycan	2ksh	jerrycan
140	River	5km	M	River	5km	M	3hours	W/C	3hours	W/C/B	20ksh	jerrycan	30ksh	jerrycan
142	Shallow well	2km	M	River	3km	B	2hours	S/C	3hours	C		Free	3ksh	jerrycan
145	Spring	3km	M	Kiosk	17km	G	5hours	W	7hours	O/B		Free	2ksh	jerrycan
146	Sshallow well	4km	M	River	5.5km	B	2hours	W	3.5km	W		Free	4ksh	jerrycan
148	River	0.5km	M	Dam	0.5km	G / M / B	0.5hours	S /W /C /O /B	0.5	S /W /C /O /B		Free		Free
151	River	7km	B	River	7km	B	3hours	W/C	3hours	W/C		Free	20Ksh	Lt /jr /Fix /Free
152	River	3km	B	River	3km	B	2hours	W/C	2hours	W/C		Free		Free
156	Yatta Canal	3km	B	Yatta Canal	3km	B	1.5hours	W/C	1.5hours	W/C		Free		Free
158	River	2km	M	River	2km	M	1hours	W/C	1hours	W/C		Free		Free
162	Dam	5km	B	Borehole	8km	G	3hours	W/C	5hours	W/C		Free	4Ksh	Fix
163	Dam	2km	B	Borehole	4km	g	1hours	W/C	3hours	W/C		Free	4Ksh	Fix
164	River	5km	M	River	5km	M	3hours	W/C	3hours	W/C		Free		Free
165	River	2km	B	River	2km	M	2hours	W/C	2hours	W/C		Free		Free
166	River	1.5km	B	River	1.5km	M	1hours	W/C	1hours	W/C	3Ksh	Fix	3Ksh	Fix
167	River	1km	B	River	1km	M	1hours	W/C	1hours	W/C		Free		Free
172	Dam	3km	B	Dam	3km	B	2hours	W/C	2hours	W/C		Free	10Ksh	Fix
173	River	0.5km	M	River	15km	M	1hours	W/C	4hours	W/C		Free		Free
175	River/Borehole/Spring	6km	M	River/Borehole	6km	M	3hours	W/C	6hours	W/C	2Ksh	Fix	3Ksh	Fix
177	River	1km	B	River	1km	B	2hours	W/C	3hours	W/C		Free		Free
178	Dam	2km	B	River	4km	M	1hours	W/C	5hours	W/C		Free	3Ksh	Fix
180	River	3km	B	River	3km	B	2hours	S/W/C	2hours	S/W/C		Free		Free
183	River	3km	B	River	3km	B	2hours	S/W/C	2hours	S/W/C		Free		Free
184	River	4km	M	River	4km	M	5hours	W/C	5hours	W/C		Free		Free
185	River	6km	B	River	6km	B	1hours	S/W/C	3hours	S/W/C		Free		Free
186	River	5km	B	River	5km	B	1.5hours	W/C	1.5hours	W/C		Free		Free
187	River	4km	M	River	4km	M	6hours	W/C	6hours	W/C		Free		Free
188	River	5km	M	River	5km	M	3hours	W/C	3hours	W/C		Free		Free
189	River	1km	M	River	3km	M	0.5hours	W/C	2hours	W/C		Free		Free
190	River	2km	M	Well	1km	M	1hours	W/C	1hours	W/C		Free		Free
191	River	0.5km	M	River	6km	M	0.5hours	W/C	3.5hours	W/C		Free		Free
195	Well	1km	M	River	2km	M	1hours	W/C	2hours	W/C		Free		Free
196	River	1km	M	River	3km	M	1hours	W/C	3hours	W/C		Free		Free
197	Dam	2km	B	Dam	2km	B	3hours	W/C	3hours	W/C		Free		Free
198	spring	10km	G	Spring	10km	G	4hours	W/C	4hours	W/C	20Ksh	Fix	20Ksh	Fix
199	Dam	0.5km	B	Dam	0.5km	B	0.5hours	W/C	0.5hours	W/C		Free		Free
200	Dam	0.5km	B	Borehole	2km	G	0.2hours	W/C	1hours	W/C		Free	3Ksh	Fix

*** G : Good
M : Medium
B : Bad

**** S : the said person
W : Wife
C : Children
O : Others
B : Buy brought water

井戸番号	E.衛生			F.給水施設の運営維持管理				Section-G. 掘削予定地周辺の対象人口			
	1. 村落で発生している水因性疾病の種類****	2. ヘルスセンターの所在		3. ジェリカンあたりの支払い可能額 (Ksh)	4. 世帯別月あたりの支払い可能額 (Ksh)	5. 水利用組合の有無	6. 水利用組合の設立方法*****	掘削サイトから2Km以内の人口、家畜数			
		名称	距離 (km)					世帯数	世帯構成者数	想定される人口	想定される家畜数
96	Ty	Makueni District Hospital	2	1.7	140	no	Vi	99	7	695	-
98	Ma /Ty	Kambi-mawe	3	1.8	233	no	Vi	72	7	504	-
98	Di /Ma /Ty /Wo	Makueni district hospital	13	1.7	203	no	Vi	38	9	351	-
100	Ty	kako	3	2.3	157	no	Vi	200	8	1667	-
102	Ty /Am	inlaak	0.5	2	183	yes	Vi	95	8	793	-
107	Ty	kiscu	9	1.7	127	yes	Vi	180	8	1440	-
108	Ma /Ty	kiscu	3	2	450	no	Vi	200	6	1200	-
110	Ma /Ty /Wo	kilili h. center	3	1	207	no	Vi	100	7	700	-
111	Ma /Ty /Am	kilili	3	2	367	no	Sc	42	8	333	1122
112	Ma /Ty	kilili h. center	6	1.7	233	no	Vi	247	6	1480	1413
113	Ma /Ty /Wo	kilili health center	3	1.3	183	no	Vi	98	7	688	1699
114	Di /Ma /Ty /Wo	matiliku	4.5	1.8	250	yes	Vi	174	8	1392	570
118	Ma /Ty	mutyambua	1	2	167	yes	Vi	533	8	4100	11700
121	Am	katangi/kalawa	0.1	2	383	no	Vi	200	8	1600	1563
123	Ty / Am / Br	kalawa	3	2	500	no	Vi	52	6	310	160
124	Di /Ty	kalawa	8	2.3	300	no	Vi	100	6	600	230
127	Ty /Wo /Am	Kiu AIC Dispensery	2	1.7	250	no	Vi	407	7	2980	10100
128	Ma /Ty	Mutiluni	9	1.7	250	yes	Vi	304	8	2435	950
130	Di /Ma /Ty /Wo /Am	Kwale Health Centre	4	1.7	217	no	Vi	363	6	2313	6667
131	Ma /Ty	Kwale Health Centre	8	1.3	207	no	Vi	283	5	1493	7564
133	Di /Ma /Ty /Am	kasikeu health center	6	2	283	no	Vi	333	7	1750	8550
134	Di /Dy /Ty /Am	sultan hamud, kasikeu	15	1.7	783	no	Vi	618	4	2473	11534
137	Di /Ma /Ty /Wo	Kikumini Health Centre	5	1	183	yes	Vi	400	4	1600	1920
140	Di /Ma /Ty /Wo	simba health center	5	1	233	no	Vi	45	5	225	1200
142	Ma /Ty	nzereni	2.5	2	117	no	Vi	39	8	312	930
145	Ma /Ty /Wo /Am	ngwata	17	1.7	200	no	Vi	34	8	275	248
146	Ty /Wo /Am	ngwata	20	2	187	no	Vi	60	6	340	240
148	Di / /Ma /Ty	Kangonde	0.8	2	140	no	Sc	167	7	1117	900
151	Di /Ma /Ty	Kivaa	11	2	200	no	Sc	200	7	1433	550
152	Di /Ch /Dy /Ma /Ty	Kivaa	9	2	200	no	Vol	83	7	553	460
156	Di /Ch /Dy /Ma /Ty /Wo	Kithimani	1.5	2	233	no	Sc	350	7	2767	750
158	Di /Ch /Dy /Ty	Kisiiki	3	1.7	217	no	Vol	300	9	2700	680
162	Di /Ch /Dy Ty	Katangi	4.8	1.7	170	no	Sc	433	7	2867	1050
163	Di /Ch /Dy /Ty	Katangi	3.8	1.7	203	no	Vol	200	7	1400	3000
164	Di /Ch /Dy /Ty	Katangi	6.7	2	187	no	Sc	387	8	2807	9900
165	Di /Ch /Dy Ty	Ndarani	1.2	1.8	217	no	Sc	367	6	2400	1500
166	Di /Ch /Dy /Ty	Mitaboni	4.7	1.5	157	no	Vol	400	6	2467	4000
167	Di /Ch /Dy /Ty	Kangundo	7.7	1.5	107	no	Vi	533	6	3200	1150
172	Di /Ch /Dy /Ty	Wamunyu	3.3	1.7	190	no	New	400	7	2633	3500
173	Di /Ch /Dy /Ty	Kathama	3	2	183	no	Vi	217	7	1583	900
175	Ch /Dy /Ty	Mango	4.3	1.7	207	no	Vi	233	8	1900	2400
177	Di /Ch /Dy /Ty	Kathama	4	1.7	140	no	Vi	217	8	1500	1000
178	Di /Ma /Ty	Kyawango	6.3	1.7	127	no	New	233	8	1867	2450
180	Di /Ch /Dy /Ma /Ty	Kathiani	6.7	2.3	203	no	Vol	350	6	2100	2200
183	Di /Ch /Dy Ty	Wamunyu	3.7	2.7	337	no	Sc	250	6	1583	1450
184	Dy /Ma /Ty /Wo	Miu	4	3	450	no	Vi	283	6	1700	2250
185	Di/Ma /Ty	Miu	2	2.7	373	no	Vi	483	7	3567	3200
186	Di/Ma /Ty	Wamunyu	3.7	2.7	223	no	Sc	170	8	1367	1480
187	Di/Ma /Ty	Wamunyu	7.7	2.7	207	no	Vi	183	8	1533	4000
188	Di/Ma /Ty	Kikule	10	3.3	233	no	Vi	173	6	1080	430
189	Di/Ma /Ty	Ndithini	3	3	383	no	Vi	883	7	2250	5300
190	Di/Ma /Ty	Mutesya	3.3	3	367	no	Vi	233	6	1500	900
191	Di/Ma /Ty	Mananja	5.3	2.3	350	no	Vi	400	7	2733	850
195	Di/Ma /Ty	Makuyu	17.7	2.3	250	no	Vi	237	7	1633	910
196	Di/Ma /Ty	Ekalakala	16.1	2	233	no	Sc	550	7	1100	650
197	Di/Ma /Ty	Ekalakala	8.7	2	233	no	Vi	383	7	2683	4000
198	No	-	0.8	2	200	no	Vi	300	7	2017	260
199	No	Kola	3	2	183	no	Vol	300	7	2000	550
200	No	-	-	2.3	233	no	Vi	333	7	2233	390

**** Di : Diarrhoea , Ch : Cholela
Dy : Dysentery, Ma : Malaria
Ty : Typhoid, W : Woems
Am : Amoebiasis,
Br : Brucellosis

***** Vi : Use village administration
Sc : Use school committee
Co : Use product coorperation
Vol : Use other voluntarily association
New : Establish new association
No : I have no idea abouti it