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The Federal Democratic Republic of Ethiopia
Ministry of Water Resources

Water Resources Administration
Urban Water Supply and Sanitation Department

Urban Water Supply Design Criteria

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1 INTRODUCTION

This document presents the planning and design criteria proposed for the Urban Water Supply projects Feasibility and Detail Design.

These criteria are developed for water supply feasibility studies, designs and execution for urban situations of the scale relevant to the National level.
The user shall modify the parameters taking in to account the particular nature of the town or the area on which the project is to be implemented.

Some criteria herein are presented as guidelines. Such guideline criteria will be reviewed, after analysis, in the field, of socio-economic conditions, water resource situations, operation and maintenance needs, wastewater implications and environmental conditions. Such criteria will thus be confirmed or varied as required by the particular circumstances of the individual scheme to which they are to be applied.

2 GENERAL

2.1 Planning Horizons

Two planning horizons will be considered:

- Stage 1 – For 10 years
- Stage 2 – For 20 years

2.2 Potable Water Quality

The MoWR was prepared guidelines on drinking water quality. In the meantime, the recommendations of the World Health Organization (WHO) for drinking water should be adopted. These recommendations state that potable water shall contain no concentration of any substance or organism high enough to impair potability.

Harmful and uneconomically treatable substances will render a source as unsuitable and unusable (for potable water purposes).

2.3 Population Projections

Population figures are available from the 1994 population and housing census of Ethiopia, published by the Central Statistical Authority (CSA), Office of Population and Housing Census Commission. The CSA makes population projections for towns (urban) and rural areas by region. Population growth rates will vary from town to town, depending on numerous factors. The basic growth rates for domestic water demand calculation will be those available or implied from the above-mentioned CSA publications for the corresponding population. Was it not being due to the unpredictable occurrences disrupting population trends, it has been an established fact that the human population kinetics would trace an s-shaped growth curve. Short-term estimates of future populations (1-20 years) are generally made by arithmetic progression, geometric progression, decreasing rate of increase, or graphical extension. Each of the first of these three procedures is based upon the "S" shaped growth curve. In forecasting future population estimates the past population trends shall be analysed to find out which of the three methods closely fits the trend. The method which depicts the past population growth trend shall be adopted for making future population forecasts. Further, the social structure of the town will be taken into account in establishing the base population and projection of the same for the design period.

For the future population prediction the above mentioned method (S curve) will be used if found suitable otherwise the geometric growth method with CSA growth rates established at national level for every 5 years interval will be used as in the table below.

CSA's Country Level Population Growth Rates

Year	Urban Growth rate %
1995-2000	4.3
2000-2005	4.1
2005-2010	4.06
2010-2015	3.88
2015-2020	3.69
2020-2025	3.51
2025-2030	3.35

2.4 Water Sources

Water sources will be selected to meet the expected maximum day demand for the relevant design period. A 24-hour basis is not mandatory given the need for consideration of maintenance and operation activities when pumping is involved. Gravity spring sources, however, should operate 24 hours per day.

2.4.1 Springs

Where quantity is not a limitation, springs will be developed to satisfy at least the maximum day demand of the 20 Years design horizon.

2.4.2 Tubewells

Staged development of tube wells will be considered where more than one borehole is required to meet the anticipated demand. Where access to the wellfield for future mobilization is not a problem, the number of tubewells adequate to satisfy the maximum day demand for 5 years will be considered if two-stage development is more economical than constructing all tubewells at once. The remaining infrastructure of the wellfield shall, in any case, be designed for 10 years.
Location, depth, diameter and screen position, discharge with the pump depth and specification shall be determined based on the revelation of the hydro geological investigations.

The issue of stand-by tubewells is dealt with in paragraph 5.3.

2.4.3 Storage Dams

For surface water schemes requiring a storage dam, a design period as long as the design life of the dam is advisable, i.e. 50 years.
The design flood is the flood, which can be passed with normal freeboard allowance. Normal freeboard allowance: Minimum 0.5 m, but increase to 1.0 m if the open water fetch exceeds 1.0 km.
The peak flood should not overtop the dam.
Dams constructed for the main purpose of water supply, is recommended to be of type earth fill or rock fill.

2.4.4 Intakes

Intake structures shall be designed for a planning horizon commensurate with the criteria for related works (transmission main, treatment plant, etc). The intake structure for surface water supply must satisfy the maximum day demand for the 20 years period. In streams where there is adequate flow that is more than two times the maximum day demand during low flow periods, the submerged through type intake with steel bars cover is to be laid across the river bed. Water in excess than the maximum day demand will be drawn into the trough and this is carried in a pipe at a velocity of more than 2 m/s. The pipe enters in to a sand-removing tank which functions

as a grit chamber. Intake structure which may incorporate desiltation facility, perforated gravity collector pipe, suction pipe, screens and trash rack shall be designed for surface water sources and springs to satisfy maximum day demand. The intake structure to be designed is with a consideration of its simplicity for operation and maintenance.

2.4.5 Protection of Water Sources

Surface water sources should be monitored for pollution and siltation in order to make appropriate designs. Catchment areas should ideally be protected, but for large catchments this is not a practicable step. Therefore careful monitoring of raw water quality must be undertaken after construction of the scheme. Within the vicinity of the source works there should be a sanitary zone with restricted access.

2.4.6 Wellfield protection

Protection of groundwater against pollution is recommended for all the existing and proposed wellfields and tubewells included in the detailed designs undertaken by the Project.

2.4.7 Protection Zones

Three protection zones are recommended around wellfield boreholes. The zone sizes should be based upon criteria of either time-of-travel or distance. For aquifers that can be readily modelled the time of travel criteria should be used. Where aquifers cannot be readily modelled (e.g. fractured rock) the distance criteria should be used.

- Zone 1 Inner Source Protection Zone
  - modelled 100 day travel time or radius of 100 m, whichever is the larger.
- Zone 2 Flow Protection Zone
  - for non-modelled aquifers, a zone of 1 km fixed radius
  - for modelled aquifers, 1000 day travel time.
- Zone 3 Outer Source Protection Zone
  - for modelled aquifers
  - for non-modelled aquifers, a zone of 25 km fixed radius
    - 100 year travel time zone.

In all cases, operational courtyards should be constructed in the immediate vicinity of all boreholes. Activities which pose a threat to groundwater quality, and may pollute boreholes, fall into two categories:

- point source discharges,
- diffuse polluting activities.

2.4.8 Point Source Discharges

The main groups of organic and hazardous chemical point source pollution risks are listed below:

- Discharges from private septic tanks and pit latrines.
- Solid and liquid sanitation waste disposal.
- Sewage works, oxidation ponds and process effluents via soak-aways.
- Storage of agricultural wastes, intensive livestock feedlots and housing.
- Burial sites.
- Production, storage and use of hazardous chemicals (agricultural, commercial and industrial).
- Oil and petroleum storage and disposal of waste oils.
- Mining and associated activities, mine waste and quarrying.
- Major infrastructural developments including new roads, industrial areas and residential areas.

2.4.9 Diffuse Polluting Activities

Diffuse pollution is pollution spread over space and time which is not caused by specific discharges or events. It is caused by aerial spread of pollutants or by the cumulative effects of many ill-defined events.

Potentially hazardous activities include the leaching to groundwater of fertilisers, pesticides, herbicides and organic and possibly chemical and heavy metal contamination from the spreading of sewage and other sludges.

The spreading of manure on agricultural land for the purpose of fertilising or otherwise beneficially conditioning the land, is seen to be an acceptable risk in Zones 2 and 3, unless there is a particular local concern, when controls may be advisable.

Many small pollution sources, such as a high density of pit latrines over a large area or high livestock concentration, may cumulatively have a significant impact upon water quality and may be considered as a diffuse pollution source. High concentrations of free ranging livestock can only be controlled through watering-point location control.

2.4.10 Measures in Operational Courtyards at New Boreholes

The physical measures against pollution within operational courtyards for new boreholes to be designed under the Project are proposed to be:

- perimeter fence (cattle-proof) with personnel and vehicle access gates,
- effective capping of any adjacent exploration and observation boreholes,
- cement-mortar or concrete seal at least 500 mm deep from ground level, around borehole casing,
- concrete surface apron, 2.5 m square, around the casing and connected to the 500 mm deep seal,
- Septic tank for operators' sanitation facilities which should discharge to a water course or a soakaway as far as possible from the borehole,
- Secure storage for diesel fuel, with sealed collection tanks for spillage,

2.4.11 Measures at Existing Boreholes

To exclude even minor pollution, the following measures are recommended for the existing boreholes:

- to erect, or repair (if necessary) fencing around production and standby borehole sites,
- to remove any contaminated soil around wellheads and to replace it with a concrete apron,
- provide secure storage for diesel fuel with sealed collection pit for spillage,
- effectively cap any unused boreholes,
- provide suitable sanitation facilities for operating staff (septic tank which should discharge to a water course or a soakaway as far as possible from the borehole).

Should pollution of groundwater be present before a water supply scheme is planned, then alternative sources should be sought. Should pollution of groundwater occur after a scheme has been built, or there are no feasible alternatives to polluted groundwater, then treatment and/or chlorination should be considered to bring the groundwater up to potable water standards.

2.4 Raw Water Quality

Raw water quality should be tested for turbidity, chemical content (including iron, manganese and fluorides) and bacteriologic contamination, to ensure its suitability and to determine the type of treatment process required. Toxic substances and heavy metals should not be present in raw water. The pH value should be between 8.5 and 9.2 (unless correction is anticipated in the treatment process).

2.5 Treatment

2.5.1 General

The use of surface water treatment plants would be avoided by using boreholes or springs wherever possible. However, when they are deemed to be absolutely necessary, the following design criteria will be adopted:

- Systems with simple installation as well as those which are durable and robust, constructed out of local materials, skills and labour will be given priority
- Use of chemicals will be minimized where ever possible
- Mechanical equipments with moving parts should be kept to a minimum.
- Costs, especially those for O&M should be kept as low as possible.

The following steps should be adopted in the choice of treatment plants

- The use of plain sedimentation prior to chemically aided sedimentation tanks wherever applicable
- Filtration with roughening filters and slow sand filters before going into rapid sand filtration proceeded by coagulation / flocculation
- Post chlorination instead of pre and post chlorination together.
- Cascade aerators where needed,
- Mechanical operating system rather than computer aided sophisticated systems will be given priority.
- A treatment plant shall be hydraulically dimensioned to cater for the maximum day water demand.

2.5.2 Groundwater

Ground water treatment may include:

- Aeration, when the iron and/or manganese content exceeds the maximum recommended level, or when a low pH-value is caused by high CO_2 content.
- pH correction, when the final value does not fall within the recommended range.

Chlorination of groundwater (springs or tubewells) will be recommended where there is pollution, or risk of pollution, and the source is not, or cannot, be excluded on pollution grounds.

2.5.3 Ground water intake

Based on the hydraulic parameters obtained from pump tastes, i.e safe yield, aquifer depth and characteristics, the well design, the number of wells required, the size of pumps e.t.c. Will be determined. Proper wellheads to safeguard the well from pollution and an inspection system shall be incorporated

2.5.4 Surface Water

Surface water should receive conventional treatment that is technologically appropriate and proven, reliable and energy efficient. The basic elements should be as follows (though some processes may be omitted when using slow sand filtration):

- coagulation,
- flocculation,
- clarification,
- filtration,
- disinfection (chlorination).

Further guidelines on conventional treatment are given in paragraph 4.2.

2.5.5 Fluorides and Nitrates

Care will be taken to avoid (or treat) high fluoride content. Water with high nitrate content will be avoided unless wellfield or spring protection measures can be relied on to reduce nitrate levels to an acceptable value (see wellfield protection guidelines).

2.5.6 Iron and Manganese

Where water with high iron and manganese content is the only feasible water source option, then treatment by aeration and rapid sand filtration should be considered.

2.6 Units and Standards

2.6.1 Units

The metric system of units shall be used throughout for feasibility studies and designs, except where imperial units are acceptable and in common usage (e.g. for galvanized mild steel pipes).

2.6.2 Standards

For design and construction, the relevant Ethiopian National Standard shall be used or referred to.

If no National Standard exists, then the relevant ISO Standard shall be used. If no Ethiopian National or ISO Standard exists then one of the following national standards shall be used:

- British
- German

2.6.3 Earthquake Design

Design of structures shall be for earthquake resistance in accordance with the relevant Ethiopian Building Code Standard (EBCS-8) of the Ministry of Urban Works and Development, 1995 edition.

3 WATER DEMAND

3.1 Specific Water Demands

3.1.1 General

Domestic water consumption varies according to the mode of services, climatic conditions, socio-economic condition and other related factors. After reviewing previous design criteria in the country we have arrived to the following per capita water consumption.

3.1.2 Domestic Demand

Domestic water demand for the following categories of consumer:

		Stage 1	Stage 2
-	House connection (HC).	50 l/c/day	70 l/c/day
-	Yard connection, own (YCO).	25 l/c/day	30 l/c/day
-	Yard connection, shared (YCS).	30 l/c/day	40 l/c/day
-	Public tap supplies (PT).	20 l/c/day	25 l/c/day

Users of PTs are or may also be users of traditional water sources. A low value of per-capita consumption for PT users allows for this.

3.1.3 Institutional and Commercial Demand

This refers to the water demand of facilities such as schools, hospitals, hotels, etc. and small commercial enterprises, and also public demand where appropriate.

The review will assess the extent and development of the institutional and commercial base in each town and vary the likely daily demand, if necessary, based on the following consumptions:

The daily demands of different consumptions is as follows

Restaurants	10 l/seat
Boarding school	60 l/pupil
Day schools	5 l/pupil
Public offices	5 l/employee
Workshop/shops	5 l/employee
Mosques & Church	5 1/worshipper
Cinema house	4 l/seat

Abattoir	150 l/cow
Hospitals	50 - 75 l/bed
Hotels	25 -50 l/bed
Public Bath	30 l/visitor
Railway & Bus station	5 l/user
Military Camps	60 l/person
Public latrines (with water facility connection)	20 litres/seat

3.1.3 Industrial Demand

Small-scale industrial enterprises will not be categorized separately but should be included in the allowance for institutional and commercial demand. However, water demand for larger industries will be considered separately as point supplies from the system. The actual demand shall be quantified for each individual case.

3.1.4 Domestic Animal Demand

The demand for livestock watering from the public water supply system shall be assessed for each town individually during the socio-economic survey.

When animal watering is to be allowed for, the following specific demands will be adopted:

Cattle, donkeys, horses, etc:	50 l/head/day
Goats/sheep:	10 l/head/day
Camel	150 l/head/month

3.1.5 Fire Fighting

Water demand for fire fighting purposes shall be assessed on a town-by-town basis, depending on the existence of equipment and the capacity of any fire fighting service.
Fire hydrants shall be installed at public and municipality interest such as schools, shops, hospitals, fuel stations and at salient points of distribution network. This demand is taken of by increasing the volume of the storage tanks by 10 %.

3.2 Unaccounted-for (or non-revenue) Water

Unaccounted-for water (UFW) is expressed as a percentage of the total water produced for the system.

UFW arises from system leakage, water taken by illegal connections, inaccuracies in metering, overflowing of reservoirs, and legitimate unmetered use such as fire fighting, flushing, etc.

UFW cannot be assessed easily without adequate and reliable metering. Others have estimated that in Addis Ababa some 25% to 30% of the water produced might be unaccounted for. The situation in the other towns may be even worse given the generally lower levels of maintenance. A figure of 50% would not be without precedent. A figure of 15% is generally regarded as good, and uneconomical to try and reduce.

Losses as % of Production				
Start year	5 years	10 years	15 years	20years
40%	35%	30%	27.5%	25%

The above figures may be varied according to the individual circumstances in each town, if required and justified.

3.3 Demand Variations

3.3.1 Seasonal Peak

Towns in Ethiopia are characterised by widely varying climatic conditions and so the variations in consumption during the year, reflected by a peak seasonal factor, will similarly vary. Some consultants have adopted a seasonal peak factor of 1.1. The seasonal peak factor adopted for any particular scheme shall be selected according to the particular climatic conditions and existing consumption records (if reliable and unsuppressed). It is expected that seasonal peak factors will vary between 1.0 and 1.2, representing the relative increase in the average daily demand during the dry and/or hot season months compared with the average annual demand.

3.3.2 Peak Day Factor

Many communities exhibit a demand cycle that is higher in one day of the week than in others. This situation shall be taken into account by the use of a peak day factor. Some consultants have used peak day demand factors of between 1.0 and 1.3. The value adopted for the design of each individual scheme shall be selected according to judicious observance of the habits of consumers and the knowledge of the community and system operators. It is expected that any value selected for the peak day factor would not fall outside the above range.

3.3.3 Peak Hour Factor

Water demand varies greatly during the day. The distribution system must be designed to cope with the peak demand, which is taken into account by the use of a peak hour factor.

This peak hour factor is expressed as a multiple of the annual average daily demand and applied additionally to the seasonal and peak day factors. The peak hour factor varies inversely with the size of the consumer base.

Population Range	Peak hour factor
< 20000	2
20001 to 50000	1.9
50001 to 100000	1.8
> 100000	1.6

Adjustment due to socio-economic factors

Socio-economic factors determine the degree of development towns.

We can use the following,

Group A towns enjoying high living standards and with very high potential for development factor of 1.10
Group B towns having a veryhigh potential for development but now lower living standards factor of 1.05
Group C towns under normal Ethiopian conditions a factor of 1.00

4 SYSTEM COMPONENTS

4.1 Source Capacity

A water source capacity needs to be sized for the total water demand including unaccounted-for water. However, where treatment losses are significant (e.g. at conventional treatment plants), such losses must also be taken into account.

The source capacity will be sized for peak seasonal demand, including the peak day factor.

4.2 Conventional Treatment Plants

Treatment plants for water from surface sources shall be designed to meet the specific conditions of the raw water. Methods and configurations of plant components will vary depending on circumstances. The following will be used as guideline criteria.

4.2.1 Pre-chlorination

Due to the formation of carcinogenic substances (usually trihalomethane) when chlorine comes in contact with humic or fluvic acids, the practise of pre-chlorination is not recommended. Only if humic acids or fluvics acids can be confirmed as not being present in the raw water, and only then if essential, should pre-chlorination be carried out.

Disinfection of the water should be achieved by post-chlorination.

Should algae growth be a problem, and pre-chlorination is not carried out, then the following precautions should be given consideration against growth of algae:

- shading from sunlight,
- careful intake design to avoid floating algae,
- micro-straining as pre-treatment,
- regular cleaning,
- chemical treatment during maintenance of structures.

4.2.2 Coagulation

Utilising a rapid mixing device for the coagulating agent (usually alum sulphate), prepared in solution tanks at a concentration of around 5%; two tanks of 12 hours' working time each should be used to achieve a final dosage range of 40 to 150 mg/1.

4.2.3 Horizontal flow sedimentation

The Horizontal flow sedimentation shall be designed for the maximum day demands of stage II and is implemented in two stages. Rectangular horizontal flow clarifiers without mechanical sludge removal are advantageous for communities in developing countries because of their simplicity and ability to adopt to various raw water conditions, such as sudden changes in turbidity, flow increases, or two high flow rates.

The design guidelines for Horizontal flow setting basins are shown below.

Surface loading rate (m/day)	20 - 60
Detention period (hr)	1½ - 4
L/W Ratio	> 3
Horizontal flow velocity	4 - 36 m/hr

4.2.4 Aeration

In cases of high iron or CO_2 concentration in the raw water, in conjunction with coagulation mixing. A cascade aerator would usually be considered.

4.2.5 Flocculation

The Baffled Channel flocculator shall be designed for the maximum day demand of stage II and will be implemented in two stages. The following parameters will be used for design.

- Detention time (se) (t) - 15 - 30 min
- Velocity gradient (S^{-1}) (G) - 100 to 10 S^{-1}
- GT Value - 23,000 - 210,000
- Water Velocity m/s - 0.3 - 0.1 m/sec

In addition to the above design criteria, the practical guidelines enumerated below will also be adopted.

For Horizontal Flow

- Distance between baffles should not be less than 45 cm to permit cleaning
- Clear distance between the end of each baffle and the wall is about 1.5 times the distance between baffles and should not be less than 60 cm.
- Depth of water should not be less than 1 m.

For Vertical Flow

- Distance between baffles should not be less than 45 cm.
- Depth should be two to three times the distance between baffles.
- Clear space between the upper edge of a baffle and the water surface, or the lower edge of a baffle and the basin bottom, should be about 1.5 times the distance between baffles.
- Weep holes should be provide for drainage.

Hydraulic Jet-Action Flocculators

The Hydraulic Jet-action flocculator shall be designed for the maximum day demand of stage II and is implemented in two stages.
The parameter used to design Hydraulic Jet-action flocculators are shown below.

- Inlet velocities range from 0.5 m/s to 0.7 m/s for the first chamber to 0.1 to 0.2 m/s for the latter chamber
- The outlet should be placed at a depth of about 2.5 m below the water level
- Rated capacity per unit chamber 25 to 50 l/s per m^2
- Velocity at turns 0.40 to 0.60 m/s

▪ Length of unit chamber	0.75 to 1.50 m
▪ Width (W)	0.50 to 1.25 m
▪ Depth (H)	1.50 to 3.0 m
▪ Detention time (t)	15 to 25 min
▪ Velocity gradients	40 to 50 S^{-1}

4.2.6 Plain sedimentation

The plain sedimentation shall be designed for the maximum day demand of Stage II but implemented in two stages. The parameters used to design plain sedimentation are shown below.

▪ Detention time (hr)	0.5 to 3
▪ Surface loading (m/day)	20 to 80
▪ Depth of the basin (m)	1.5 to 2.5
▪ Length/width ratio	4:1 to 6:1
▪ Length/depth ratio	5:1 to 20:1

4.2.7 Clarification

To precipitate the flock. The type of clarifier would be selected according to local conditions, convenience and economic considerations. In the case of horizontal basins, the velocity rate would be around 1.0 to 1.2 m/h; in case of vertical flow devices, the upward velocity would be around 2.0 to 2.5 m/h.

4.2.7 Rapid Sand Filtration (NTU>100)

To remove the remaining flocks after clarification. The filtration rate for rapid sand filters is usually around 6 to 8 $m^3/h/m^2$. The accumulated impurities are periodically removed by backwashing with treated water and compressed air. Usual backwash rates are 18 to 50 $m^3/h/m^2$ with an air flow rate of about 15 to 25 m/sec. Grain sizes for rapid sand filters need to be 0.15 mm to 0.35 mm with a uniformity coefficient of 2.5.

4.2.8 Slow Sand Filtration (NTU 25-100)

Where turbidity of the raw water is less than 25 NTU the use of slow sand filters can be considered. The filtration rate will be 0.1 to 0.2 $m^3/h/m^2$.

Slow sand filters required larger areas of land, but have the following benefits.

- technologically simpler than rapid sand filters,
- very efficient at removing bacteriological contamination, reducing the required level of post-chlorination.

With higher turbidities, between 25 NTU and 100 NTU, slow sand filters may still be preferable if pre-treatment of the raw is carried out (e.g. roughing filters) to reduced the turbidity level.

Grain sizes for slow sand filters need to be 0.5 mm to 2.0 mm with a uniformity coefficient of 2.0.

4.2.9 pH-correction

By adding lime slurry or soda ash solution; it may be applied in the mixing chamber of the coagulating agent, in the clear water tank or in both places. In the case of lime, slurry concentration is usually about 5% to 10% in order to achieve final dosage of 25 to 50 mg/1.

4.2.10 Disinfection

All schemes shall have disinfection facilities for water entering the supply system, to prevent contamination in the distribution system or storage reservoirs. By using commercial grade hypochlorite in concentrations as for pre-chlorination. Dosage rates should be as appropriate to meet specific site conditions in order to reach a residual concentration range of 0.2 to 0.5-mg/l chlorine in water in the distribution system.

Although there are several types of disinfection methods, disinfection by chlorination is the most common method of disinfection due its lasting effect after the initial dose has been administered. Chlorination as a water treatment process is used principally to kill pathogenic organisms in the water. Water intended for human consumption must be free from harmful bacteria and pathogens. This is achieved by disinfecting the water by the addition of chlorine, either in the treated water tank or, if the system is pressurized, in the rising main to the storage reservoir. In both cases, means must be provided for the efficient mixing together of the chlorine solution and the water being treated. A contact time of at least 30 minutes is required. The quantity of chlorine to be added in order to disinfect the water depends on composition, temperature and retention time.

Chlorination of water supply systems can be effected:

i) by chlorine gas
ii) by using calcium hypochlorite compound with a commercial strength of 65-70%.

Chlorine is not manufactured in Ethiopia, either in gas or hypochlorite form. It is imported, and thus requires hard currency. So, depending upon the size of the treatment plant and the availability of chlorine, there are cost considerations for both types of chlorination systems.

However, chlorination by gas is more effective than calcium hypochlorite and other chlorine derivatives. For larger towns, with more sophisticated treatment plants, gas chlorine is being utilized, e.g. in Awassa, Jimma, Addis Abeba. As chlorine gas is extremely poisonous, necessary precautions during transportation, storage and dosing have to be taken. Safety devices such as chlorine gas detectors, sprinklers, respirators and ventilation systems have to be installed along with chlorine gas cylinders and injection equipment. However, due to higher sophistication and requirement for more E & M works, gas chlorination systems better to replace by calcium hypochlorite systems.

Chlorination with calcium hypochlorite is presently being used in most water supply systems in Ethiopia due its easy handling and operation. The disadvantage of this system is that conveyance systems are easily clogged. However this could be improved through proper supervision and operation and maintenance.

Dosing with calcium hypochlorite requires E & M units such as dosing pumps, stirrers and a control system. It requires also mixing tanks, clear solution tanks and conveyance systems of the required capacities.

For dosing, a membrane pump, or similar, that delivers a fixed amount of solution with each stroke is recommended. The capacity of each unit shall be determined based on the flow of water.

Dosage rates for the two methods of measuring chlorine in water are given in the table below:

	Combined Chlorine	Free Chlorine
Dosage	5-10 mg/l	1-2.0 mg/l
Residual	1-2 mg/l	0.1 – 0.2 mg/l

The applicability of the two methods of measuring chlorine in water are given in the table below:

Combined Chlorine	Free Chlorine
1. If water contains sufficient ammonia	1. Where raw water is poor.
2. If ammonia content is very low, ammonia has to be added externally	2. Where oxidation of iron and manganese is required. 3. Where contact time is insufficient.
	4. To destroy objectionable tastes and odour produced by substances.
	5. Diminish chlorine resistant bacteria and other growths in the system

4.2.11 Stand-by capacity

Treatment plant facilities should have standby capacity sufficient to prevent reduced output during cleaning and maintenance of the various components.

4.3 Pipelines

4.3.1 System Type

There are four basic arrangements of water supply systems for delivery of water from the source to the consumer. These types are listed below, with their salient characteristics:

Type		**SOURCE TO STORAGE RESERVOIR**	Storage Reservoir to Consumer
Type 1	Pure gravity system	**GRAVITY**	Gravity
Type 2	**MIXED PUMPING/GRAVITY SYSTEM**	Fixed rate pumping	Gravity
Type 3	**MIXED PUMPING/GRAVITY SYSTEM**	Variable rate pumping	**PART PUMPING/PART GRAVITY**
Type 4	Pure pumping system	**VARIABLE RATE PUMPING ONLY - NO STORAGE RESERVOIR**	

Type 1 systems, pure gravity, will be given first consideration where suitable water resources are available. As these systems have low operation (no energy) costs then, where such schemes are within technical norms, such schemes will be analysed carefully at feasibility stage.

The remaining schemes, Types 2 to 4, all rely on pumping to a lesser or greater degree. Technical arguments for and against each of these systems are given in the table below.

Scheme	Pros	Cons
Type 2	**ALLOWS UNIFORM CONDITIONS FOR SUPPLY PUMPS, THUS ALLOWING SIMPLIFIED PUMPING ARRANGEMENTS AND ENHANCED WORKING LIFE AND EFFICIENCY OF THE PUMPS.** **ALLOWS EASIER MONITORING OF**	Requires storage big enough to even-out hourly fluctuations in demand.

	FLOWS FROM THE SOURCE WORKS FOR DETECTING ABNORMALITIES. Allows adequate contact time for post-chlorination before water reaches the distribution system.	
Type 3	May allow a smaller reservoir capacity, depending on the installed pumps. Allows a reducing-diameter rising main to be installed.	Requires a high level of accuracy in setting pump heads and flows (i.e. water demands and related criteria) so that actual operating pressures are such that the reservoir functions correctly (or even at all). Requires multiple pumps for variable pumping with higher operation and maintenance costs. May not be suitable for wellfield supplies without intermediate collector/booster station.
Type 4	Allows the possibility of dispensing with distribution storage.	Pumping equipment has to be designed for peak flow conditions, meaning multiple pumps. Multiple pumps and related equipment mean high operating and maintenance costs. Storage at the source works will be necessary if the source is designed for average flow. Will not be suitable for wellfield supplies unless intermediate collection/booster is provided. May not allow sufficient post-chlorination contact time before water reaches the consumer.

The type of system for each town will be determined according to the following criteria:

- Selection of pure gravity scheme over any scheme that requires pumping, providing fixed costs (mainly distance of source to the consumers) are not prohibitive.
- Otherwise selection of Types 2 to 4 depending on:
  - Engineering considerations as to suitability.
  - Overall economic cost.

4.3.2 Transmission Mains

Rising and gravity transmission mains from source to distribution should be designed for the maximum day demand, based on the design hours of water source operation. Storage facilities at the termination of the transmission main(s) should cater for the peak hourly flow in the distribution system. The number and diameters of transmission pipes should be determined primarily on the basis of economic considerations, comprising either a single large diameter pipe of sufficient capacity for the final planning horizon or several parallel pipes of smaller diameter, installed at various intermediate horizons. The economic analysis should take into account the cost of pipe and energy to determine its optional diameter (which should normally be selected from the standard range diameters). However, engineering considerations should also be taken into account if important.

Where transmission or gravity mains involve working or static pressures that are higher than advisable in relation to pump capacities or pipe pressure ratings, then break pressure tanks and/or booster stations will be considered.

No house connections should be made to transmission mains.

4.3.3 Distribution Systems

The distribution network will be designed for the peak hourly demand. The minimum pipe size to be considered for primary and secondary networks should be DN 2”. Tertiary pipes may be below DN 2”, but not below DN 1”. Large scale networks may conceivably have a larger minimum diameter for primary and secondary pipes.

Distribution systems should be planned with either one large diameter pipe suitable for the final planning horizon, or multiple smaller diameter pipes installed at various intermediate-planning horizons. An economic analysis should be carried out to determine the cheapest solution.

The operating pressures in the distribution network shall be as follows:

	Normal Conditions	Exceptional Conditions	
Minimum Maximum	15 m water head 60 m water head	10 m water head 70 m water head	* (1) * (2)

*(1) Envisaged where distribution pipes are close to reservoirs in terms of perhaps both location and elevation, and in small sections of the distribution system that would require a PRV or BPTor otherwise mean raising pressures generally to achieve a 15 m minimum pressure.

*(2) Envisaged in small section(s) of the distribution system which would otherwise require separate pressure zone(s).

4.3.4 Velocity and Headloss

Water velocities shall be maintained at less than 2 m/sec, except in short sections (see also paragraph 5.2 for pumps). Velocities in small diameter pipes (<DN100) may need even lower limiting velocities.

A minimum velocity of 0.6 m/sec can be taken, but for looped systems there will be pipelines with sections of zero velocity.
Headloss is related to velocity and pipe roughness. The maximum headloss with therefore be governed by the maximum velocity criterion.

4.3.5 Hydraulic Computation

Any internationally recognized formula may used in the hydraulic computations, with coefficients taken as follows:

For Hazen-Williams (C-value):

Type of Pipe	uPVC	Steel	DCI/GI
New	130	110	120
Existing	100-110 *	90-110 *	100-110*

For Colebrooke-White and Darcy-Weisbach (value in mm):

Type of Pipe	uPVC	Steel	DCI/GI
New	0.25	0.85	0.55
Existing	1.35 – 0.85*	2.60 – 0.85*	1.35 – 0.85*

* Depending on age and condition.

The above C- and mm- values are applicable to transmission mains and similar lengths of pipelines with few appurtenances. For distribution systems, it is generally recognized that a C-value of 100, or 1mm for Colebrooke-White/Darcy-Weisbach, be universally used.

Experience shows that a pipe designed to flow at a velocity between 0.6 and 1.5 m/sec, depending on diameter, is usually at optimum condition (head loss versus cost). Short sections, particularly at special cases, e.g. at inlet and outlet of pumps, may be designed for higher velocities (see paragraph 5.2).

The static state pressures in pipelines must be less than the pipe nominal pressure rating. In the case of long mains where water hammer risk is expected, due attention must be given to the pipe material and a proper water hammer analysis carried out.

4.3.6 Selection of Pipe Material and Type

The following materials will normally be selected, taking into consideration useful lifetime, leakage levels and maintenance requirements.

For transmission and rising mains:

- DCI for DN 100 and above,
- GI for DN 80 and lower.

For distribution systems:

- DCI for pipes DN 250 and above.
- uPVC for pipes DN 200 to DN 80.
- GI for pipes for tertiary systems and pipes of DN 3" to 1".

The type of pipe material to be selected shall depend on:

- Characteristics of the soil
- Chemical nature of the water
- Cost of the pipe
- Types of Crossings/fittings

The physical characteristics (crushing strength, resistance to corrosion, etc) of the pipes should suit the actual service conditions arising in the system with respect to internal and external loads and soil conditions.

The pressure rating of pipes used in distribution systems should be 6 bar minimum. However, where 6 bar uPVC pipes are used, strict adherence to stringent installation produces should be mandatory, including sand bedding and a minimum cover of 90 cm.

The pressure rating of pipes used for transmission mains should be selected from standard pressure ratings (according to the most economic solution).

4.3.7 Alignment of Mains

The following considerations should govern the alignment of mains within a supply area:

- mains to follow the shortest route between the headworks and the supply area, with deviations only where necessitated by topographical conditions,
- wherever possible, mains should be laid at road sides or verges, footpaths or green strips. Mains to be laid along roads should be located at a minimum distance of 1 m from the edge of the road or the roadside drain.
- distribution systems forming part of the main grid should follow the existing or planned roads, while observing the necessary requirements for hydraulic efficiency and economy.

Marker posts shall be erected to identify mains alignments.

4.3.8 Depth of Mains in the Ground

The criteria governing the depth at which pipes are laid are, on the one hand, protection and safety of the pipeline, and on the other hand, easy maintenance and avoidance of excessive earth pressure and live load due to traffic. In these respects, the following criteria should be adopted:

- mains laid in trenches in soil should have a normal minimum cover as shown in the table below.
- mains laid in rocky conditions may have a minimum cover of 60 cm, or could be surface laid if security and anchorage concerns are properly addressed,
- where the minimum cover cannot be achieved, a buried pipeline will be encased in a concrete surround,
- mains unavoidably laid under carriageways or at road verges should have a minimum cover of 100 cm,

Normal cover for mains laid in the ground

Pipe Material	Depth of cover cm
Ductile iron (DCI)	80
Galvanized mild steel (GS)	80
uPVC	90

4.3.9 Washouts and Air Vents

Air vents of the double orifice kinetic type DN 80 should be installed on mains of diameter DN 250 and larger. DN 50 single orifice air vents should be installed on pipelines of smaller diameter and for larger pipes where only accumulated air has to be expelled (i.e. emptying and filling of the pipeline is not to be catered for).

Air vents in the distribution system will not be installed, except at up-and-over crossings and at large diameter pipe dead-ends.

Air vents should be provided generally at the highest point in mains DN 150 and above and near isolating valves on downhill slopes. In general, air vents will be installed as follows:

- between water source and pump (where circumstances require);
- downstream of pumps;
- at high points of vertical bends and over-crossings;
- on both sides of sharply dipping pipelines, e.g. at river crossings;
- every 500 m to 1000 m on long pipeline sections with mild slope.

Washouts will be provided taking into account an emptying of the respective pipeline section in 3 to 4 hours. On mains of DN 250 and above, washouts will normally be of DN 100 or DN 80; on smaller mains and pipelines, a washout of DN 50 should be installed. Two types of washouts may be considered:

- Vertical, installed on connection branch, possibly including a DN 80 cast iron
  hydrant.
- Horizontal, installed on connection branch and valve, with or without a drainpipe.

Washouts should be located at the lowest point of the pipeline, or next to a section valve, and where possible, discharge to drains, streams, etc.

A flap check-valve is to be installed and secured at the ends of washout drainpipes, where they discharge to the drainage system.

4.3.10 Valves

The provision of valves should be based on the following considerations:

- Spacing: Isolating valves on mains should be installed at intervals as required, their spacing being dictated by factors such as washout requirements, connections to consumers and connections to other mains. In any event, the maximum spacing should be 500 m.
- Mandatory locations: Isolating valves should be provided at interconnecting pipes, by-pass pipe connections, hydrant connections, washouts and air vents.
- Diameter: Isolating valves on mains DN 400 and smaller and those installed on branches
  for air vents, hydrants, washouts and bypasses should be of the same size as the main or respective branch pipe. Pipelines larger than DN 400 may have valves smaller than the pipe diameter if the cost savings (including the necessary tapers) of a smaller size outweigh head loss considerations.
- Gearing of valves for operation should be carefully considered, especially for larger diameters and higher pressures.

4.3.11 Fittings

Pipeline fittings (bends, tees, couplings, flanges, branches, elbows, etc.) should be as follows:

- Appropriate for the pipeline configuration; normally they will be of ductile cast iron or of uPVC to match the pipeline material installed, the same diameter and the same or higher pressure class of the pipeline in which they are installed.
- Wherever fittings for assembling steel pipes are required, they are to be of the same design strength as that of the pipe.
- Fittings for GI pipes should be in GI material.

4.3.11 Public Taps

Public taps should be installed to provide a maximum walking distance of 500 m in any direction. The definitive spacing and location of public taps should be determined in collaboration with the served community taking into consideration the operating hours and the number of faucets per installation. Locations should be fixed during the design stage, or during the construction stage if such details are left open during the design.

Supply pressures at public taps should be limited to a range of 2 to 5 metres using a suitable pressure reducing valve.

4.3.12 Construction Materials

Reinforced concrete will be used as the major construction material for the intake structures, sand-removing tanks, clarifiers rapid sand filter, equalizing reservoirs and collecting chambers with masonry works being incorporated where necessary

4.4 Reservoirs

4.4.1 General

Operational reservoir(s) should be provided to command a distribution system, located at elevation(s) providing the required pressure for water flow within the system. They should have sufficient storage to cover the difference between hourly peak demand and actual supply from the source, fire fighting demands if to be allowed for, and for a limited emergency volume in case of power breakdown, repairs or O&M activities.

4.4.2 Types of Reservoirs

The two main types of reservoir are the ground level type (GLR) and elevated water tank type (EWT). Whenever the local topographical conditions permit, ground level reservoirs are preferable.
Ground level reservoirs will be usually be of solid block masonry or reinforced concrete, cylindrical or rectangular but under special circumstances may be of glass reinforced plastic (GRP).
Elevated water tanks will be cylindrical or conical in reinforced concrete.

4.4.3 Reservoir Location

A reservoir location should maintain the desired pressure range in the supply network. Possible future extension of the storage capacity should be taken into consideration when selecting a site.

4.4.4 Reservoir Equipments

Reservoirs should be provided with inlet, outlet, drainpipe, overflow pipe, water level indicator, manhole
Ladder, ventilation pipe, lightening conductor.

4.4.5 Total Storage Requirements

In order to provide for security of supplies above the need for balancing purposes it is recommended that the minimum total reservoir storage capacity be in the range of 30% to 50% of the average daily demand.

When determining the level of storage, within the above range, the following criteria will be considered:

- reliability of pumping arrangements at the source works,
- reliability of electricity supplies if 100% standby generation capacity is not provided,

- socio-economic status of the town (or an indicator of the level of qualified staff available for system operation),
- accessibility of the woreda & zonal water office for maintenance work outside the scope/capacity of the town water office.

4.5 Metering

Provision for metering should be made according to the following criteria:

- at the outlets of springs,
- for individual tubewells,
- at the outlets of treatment works or wellfield pumping stations,
- at the outlets of distribution reservoirs,
- for all consumers and pubic taps.

For installation of bulk meters, the recommendations of the manufacturer regarding the conditions for achieving laminar flow through the meter will be observed.

Consideration will be given to designing, or arranging, distribution networks into supply zones that can be readily isolated so that they are served by a single distribution main. Provision (either temporary or permanent) can be then be made for flow metering on this single distribution main for:

- establishing minimum night flow,
- carrying out a water balance (against consumers' meters).

5 POWER SUPPLY AND PUMPS

5.1 Power Source

Pumps may be driven by an electric motor or by a diesel engine. Considering convenience, operational uniformity and pollution aspects, electric drive for pumps is preferable. Electric power may be provided from the ELPCO grid, or locally provided by diesel generator(s). Where the driving power is electricity, it should preferably be available from two sources; from the grid, as prime source, and from a diesel generator as a standby source. If no generator is provided because electricity supplies are deemed to be reliable, then a suitable voltage regulator should be installed if low-voltage periods in the grid are excessive.

Generating sets should be capable of driving all the electric motors of the plant to full design capacity plus any expected overload (e.g. at start-up).

5.2 Pump Types and Borehole Equipment

Deep boreholes should be provided with submersible pumps. While maintaining this practice, the possibility of equipping medium depth boreholes with shaft driven turbines should not be precluded. Shaft-driven pumps may be more expensive in terms of initial investment, but could well be cheaper in the long run through ease of maintenance, longer service life span and more efficient operation. Shaft driven turbine pumps will usually be driven by an electric motor with power supplied from the grid or locally provided by diesel generator. Only under special circumstances may such pumps be directly driven by diesel engine through a coupling and suitable gear drive.

Booster pumps will usually comprise standard horizontal units driven by an electric motor. Standardisation of equipment should be favoured to allow convenient maintenance and reduced stocks of spare parts.
Power factors of electric motors should be maintained at 0.95 or above by suitable equipment, and starting arrangements should ensure a maximum start-up load of 3 times the normal load.

In selecting pumps and motors, the following factors should be considered:

- required de-rating factors for altitude,
- water quality,
- hydraulic characteristics of the system,
- draw-down (in boreholes) or suction head (in surface horizontal pumps),
- mechanical/electrical efficiency,
- economic considerations,

- availability of maintenance services and spare parts,
- standardisation of equipment and familiarity by the operators,
- operating speed.

The design working capacity of pumps (duty point) will be determined taking into account the system requirement and the number of units working simultaneously. The pump characteristics should normally allow working in a range between $^{2}/_{3}$ and $1^{1}/_{2}$ times the nominal discharge at the design duty point (unless a pump with such characteristics is not a standard item). The duty point of pumps expected to work in parallel with others will take into account the effect of the combined pumps' duty on the resistance head of the system.

The available system NPSH at the maximum flow rate should exceed by at least 1.0 m the pump manufacturer's required NPSH. However, consideration will be given to pump systems of a design which avoids the problems of low or negative suction heads.

Maximum flow velocities for pumping systems will be as follows:

- through the pump's discharge flange: unlimited,
- at inlet branch: 2m/sec,
- at outlet branch: 3.5 m/sec,
- at inlet manifold; 1.2 m/sec,
- at outlet manifold: 3 m/sec,
- in riser pipe from submersible pump to borehole head: 2 m/sec.

Pumps will be provided with a discharge isolating valve, non-return valve, air vent and pressure gauge(s). In addition, pumps will be provided with an inlet valve where the suction head is positive. A mechanical flow meter (water meter) should be installed on the outlet of a pumping station (after the manifold, see also paragraph 4.5).

For water quality monitoring, a convenient sampling point should be provided on the discharge pipe.

Borehole pump installation should have an arrangement for measuring the water level in the tubewell (dip tube). There must be a low water level protection device for the pump-motor set.

Electric motor ratings will take into account the actual working range, and be over-rated at least 20% of the maximum calculated power requirement at any point in the actual working range, in addition to any de-rating factor for altitude.

5.3 Standby Capacities

Boreholes are to be provided with a power source standby diesel generator, as stated above, if the electricity grid is the prime source. The standby generator may serve as a stand-by for several boreholes if ratings and mobility allow.

Where multiple boreholes are needed for any particular scheme, consideration will be given to providing 50% stand-by borehole capacity, fully equipped, depending on the vulnerability of the scheme.

Booster and pumping stations are to be provided with both pumping capacity standby and power source standby, as follows:

- the required discharge capacity should be divided between at least two equal units, with a further, similar, unit installed as stand-by.
- full electrical power standby capacity is to be provided, for the nominal capacity of all duty pumps operating simultaneously plus starting conditions.

6 ECONOMIC LIVES

The following service lives for system components will be adopted for economic analysis calculations:

- Boreholes in hard rock: 25 years
- Boreholes in limestone: 15 years
- Electromechanical equipment of pumping stations and boreholes: 10 years
- Ductile iron pipes: 40 years
- PVC pipes: 25 years
- Steel pipes: 30 years
- Masonry/solid block water tanks: 25 years
- Concrete works: 50 years
- Concrete water tanks: 50 years
- Civil engineering building works (general): 40 years
- Treatment plants: 50 years
- Chemical dosing: 10 years

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Well head protection guideline

Objective

The objective of these guidelines is to create the awareness to the people, especially to those concerned professional; the imperative needs to draft guideline to protect the existing and designed well fields and tube wells from possible contamination. The final goal of well head protection, which is further referred as ground water protection, is to protect the ground water resources of the nation so that these resources can support their identified beneficial uses and values in an economically, socially, and environmentally sustainable and acceptable manner.

Need for guideline

Many reported cases related to serious pollution of ground water sources due to the facts that measures were not taken in wellhead (well fields) protection. Dire Dawa Water supply is a very good example. The ever-increasing high nitrate concentration in the communal water supply system is from on site sanitation pollution sources. Currently the Sabian well field is highly populated by settlers and the underlined sandy aquifer is vulnerable to pollution from on site sanitation (septic tanks and latrines). Doubtful weather appropriate measures are now being taken to protect the Akaki well field of Adiss Ababa water supply. Therefore formulation of these guidelines is decisive.

Scope of guidelines

These guidelines focus on specific part of ground water protection related to well field (existing and future design in the water supply system). Though there are many contaminant sources special emphasis is given to the widespread form of pollution, i.e. from on site sanitation. Risk assessment of pollution from on site sanitation is discussed in detail in Chapter 4. The acceptability of different sources such as, industrial process effluent, agricultural wastes, and e.t.c in protection zones is highlighted in table form Table 2.2, 2.3 and 2.4. Their detail risk assessment is out of the scope of this guideline. However, some topics related to these kinds of pollutions are discussed in chapter 3.

Approaches to wellhead protection

Forms of intervention

In our situation intervention by the responsible governmental institution to improve the wellhead protection activities can take place in two main ways, by direct command and by raising social awareness through community participation and education. There can be, of course, some other ways of intervention, but this needs further discussions and research by the concerned party.

The first way of intervention can be by government regulations which directly controls activities. The question rises, "How strong are our regulations?" Is the public submissive to them?" "What kinds of measures are being taken against those who violate them?

Yes! We are strong enough to protect the civil law. To pollute this scarce resource should be seen as killing the nation. The legislative bodies are surprisingly fast to bring an ordinary thief to a court. Why not this happens in a case of pollution control or any kind of environmental protection? We have to be reasonably serious to keep our priceless resource.

Types of measures

There are various types of ground water protection measures, which are used in different part of the world. In Ethiopia we have at least have to adopt the following measures:

- Ground water Management
- Land use Planning

Ground water Management

The management has to focus on issues related to the allocation and utilization of the resource. Ground water management may be integrated with surface water management on a catchment basis. The current effort, which is now being made by the Ministry, to license the utilization of the nation resources, has to be highly encouraged and supported. This category of measures includes controls on ground water extraction rates, the gathering of hydrogeological information, and monitoring of critical over-draw situations. Controls can cover aquifer contamination issues through the reservation of special areas, well construction and abandonment measures, and driller and to have a well drilled licensing.

Though there are too many actions included in ground water management measures, in these guidelines only a very short portion is highlighted. Ground water management plans which are discussed above should be followed by serious of actions. In our case, since the ground water resource is not adequately studied and a reasonable ground water hydrogeological map at an acceptable scale has not yet been produced, the overall aquifer classification and vulnerability mapping is not possible at this stage. However, for localized areas where groundwater resources are developed and sufficient geological and other information are available the aforesaid approach can even has to be adopted.

Wellhead protection plan is one of ground water management measures, and so far these guidelines are focusing on wellhead protection it is presented as follows.

Wellhead Protection Plans

The wellhead protection plan is a system of ground water protection which involves the following components:

⇒ **Well Integrity Assurance**: A set of actions to assure that the well is properly designed and constructed to achieve protection objectives. This includes provision of adequate protection of the well collar against physical damage and seepage of contaminants.

⇒ **Wellhead protection zones**: The delineation of protection zones around the well head aims at protecting that part of ground water flow system which contributes to the discharges of the well. This will include the nearby cone of influence defined by drawdown of the water table and the flow field up gradient not affected by drawdown. The size and location of these zones needs to be defined on a site specific basis to allow for effective designs of the monitoring system. These may need to account for non-advective particle motion since density, dispersion and diffusion effects can predominate in many circumstances.

⇒ **Monitoring system** : Water quality within the aquifer and in the pumping well needs to be monitored to protect against contamination. Monitoring the pumping well alone is in adequate as no warning of an imminent contamination incident is provided . Monitoring is needed with in the aquifer at positions at sufficient distance up-gradient to allow time for preventative and if necessary, remedial action to be implemented in the event of contamination being detected. Ground water near the contamination sources within the zone also needs to be monitored.

⇒ **Contamination sources/Land-use control**: The location and nature of existing contamination sources within the protection zones needs to be documented and controls placed upon these or potential land-uses within the zones.

Wellhead Protection Zones

Delineation of a wellhead protection area is typically done through the use of computer modeling efforts based on travel time and pollutant transport. These models can be complex and can create large areas where many land uses are prohibited.

TABLE 2.1 WELLHEAD PROTECTION ZONE FOR

PUBLIC WATER SUPPLIES

Zone	**Boundaries**	**Restrictions**
3	50 m radius (or with in the wellfield)	No storage of hazardous or tainting material, implementation of an a integrity assurance plan.
I	10 years residence time	No waste dumps, implementation of a monitoring system, including monitoring for response to a contaminated plume within the boundary.
II	Greater than 10 years residence time	No restriction beyond those imposed by a regional plan and monitoring system to provide warning of deterioration in water quality.

The approach is based on the definition of concentric protection zones around the well head. The nearest zone (Zone 1) would commonly encompass the water authority compound around the wellhead but could also include adjacent properties. The most stringent controls on land use and materials handling would apply in this zone. The second zone (Zone II) is arbitrarily defined as the maximum distance would have traveled if it took 10 years to reach the well. The controls on land use in this zone would require a monitoring system for all potentially polluting activities with in the area. Monitoring of ground water with in this area, particularly up -gradient or with in the capture zone of a well would give early warning of contamination.

The third zone (Zone III) would correspond with the regional protection area where greater than 10 year residence time is available. This is usually the "catchment area" of the contributing aquifer. Definition of the size in outer protection zone(s) would require adoption of a standardized methodology for estimation of the area with in the specified residence time zones(s) . The approach to management within this area would be largely affected by the nature of the aquifer being protected, particularly its degree of confinement.

When wellhead protection zones are declared for existing bores supplying town and cities, careful thought needs to be given to the implementation strategy. The zoning will only protect against future contamination threats and so the strategy must look at all existing threats and take remedial action.

Land use planning

As mentioned above a competitive ground water protection can be achieved in integrated basin management approach.
The planning process for land and resource utilization is complex, and usually has many tires of authority.
In Ethiopia there is no such strong coordination of act in resources utilization. The municipalities are less concerned about environmental issues. In town planning and any kind of development programs, hardly ever implemented delineation of conservation areas, i.e. an area or zone to be protected from over extraction, or reserved specifically for water supply.
The problem is severing when every thing is finished and all areas are occupied. Delineation of areas should be made prior to the final stage of the development.

The water resources planner has to make the due concern for the environmental aspect of any water resources projects. In any water supply design and construction every existing, and possible future pollution sources have to be investigated and mapped. The water supply sources selection and system design must be accomplished with delineation of reserved area, and notification of the adopted measure, underlining the required cooperation acts, to the whole concerned parties.

Table 2.2 Point Sources Risks Principally of Micro-Biological Nature

Risk	Zone 1	Zone 2	Zone 3
Discharges from septic tanks and pit latrines:			
Discharge < 2 m^3 / day	Not acceptable	Acceptable subject to regulated discharges	Acceptable
Discharge > 2 m^3 / day	Not acceptable	Not acceptable	Acceptable
Pit latrines	Not acceptable. Existing pit latrines in this zone should be sealed or replaced by conservation tanks	Acceptable but subject to site specific density loading regulations and construction operation controls	Acceptable
Sewage works oxidation ponds, sewage effluent disposal through soak ways or irrigation schemes	Not acceptable	Not acceptable	Acceptable only in low vulnerability zones with engineered containment. Rigorous site investigation. Discharge according to guidelines and with waste discharge permit. No waste disposal below water table.
Sanitation sludge lagoons or dumps	Not acceptable	Not acceptable	
Landfills, domestic	Not acceptable	Not acceptable	Acceptable subject to site investigation and site-specific constraint. No waste disposal below water table.
Industrial process effluent disposal via soakways or irrigation schemes.	Not acceptable	Not acceptable	Acceptable after evaluation and subject to site investigation and construction/operation constraint. Discharge according to guidelines and with waste discharge permit.
Storage, disposal of agricultural wastes and intensive livestocks feedlots and housing	Not acceptable	Not acceptable	Acceptable after evaluation and subject to site investigation and construction/operation constraint. Location in areas with low aquifer vulnerability desirable with Zone 3.
Burial sites	Not acceptable	Not acceptable	Acceptable

Table 2.3 Point Sources Risks Principally of Chemical Nature

Risk	Zone 1	Zone 2	Zone 3
Production, storage, disposal and use of hazardous chemicals. Industrial landfill operations	Not acceptable	Not acceptable	Acceptable subject to rigorous site investigation and installation regulations. Development in areas of low aquifer vulnerability encouraged. No disposal through soakways or permeable lagoons. Trade waste agreement necessary for disposal to sewer. All sites should be sealed to prevent contamination in the event of leakage. Authorization for hazardous chemicals should take place through licensing. Regular site inspection necessary , groundwater and soil monitoring recommended . Health and safety guidelines for the use and storage of hazardous chemicals and wastes should be formulated and incorporated into legislation.
Oil and petroleum storage Disposal of waste oils	Not acceptable	Not acceptable. Existing installation should conform to specification and be made of all existing hydrocarbon storage sites and risks assessed. Leaking installation must be closed or repaired. All sites should be sealed to prevent contamination of groundwater in the event of leakage.	Existing installation should be assessed for zone 2. New storage facilities should conform to strict design and operational rules.
Industrial process effluent disposal through soak ways, permeable lagoons not involving hazardous chemicals	Not acceptable	Not acceptable	Acceptable subject to site investigation and site-specific and construction/operation constraint and with waste discharge permit.
Mining associated activities, mine waste sites, quarrying	Not acceptable	Not acceptable	Presumption against depending on nature of activity. Each case assessed individually in relation to risks posed. Aquifer vulnerability in zone 3 should be considered.
Major infrastructural developments new roads, industrial area residential areas	Not acceptable	Presumption against	Acceptable

Table 2.4 Acceptability of Diffuse Polluting Activity

Activity	Zone 1	Zone 2	Zone 3
Pesticide usage Herbicide usage Artificial fertilizer usage	Not acceptable	Not acceptable	Acceptable but subject to investigation concerning level of risk, acceptability criteria and persistent nature of the substance.
Sewage or organic wastes sludge spreading Sanitation facility loading (pit latrines, septic tanks)	Not acceptable Control as specified in Tables 1 and 2	Not acceptable (not applicable to manure spreading for fertilizing) Control as specified in Tables 1 and 2	Acceptable Not regulated
Livestock watering points	Not acceptable	Acceptable	Acceptable

Contaminant sources

Sources of contamination are most often referred to as either **point sources** or **diffuse sources.** Point **sources** refer to cases where contamination is localized and often is center on one or more identifiable structures or surface facilities. Diffuse sources are broad scale and cannot be ascribed to a sole source, but are caused by either a widespread land use practice (e.g. use of agricultural pesticides) or by a widespread collection of small point sources (e.g. septic tanks in an unsewered area).

There are many types of contaminant and may vary from country to country. The most common sources are solid waste dumps and refuse pits, house hold sullage and storm water drains as well as animals.

The following ways of contamination can be attributed to the wellhead.

- Incompatible land-use practices within well recharge areas , e.g. .septic tanks;
- leakage of contaminants in to the well or around the outside of the casing, if not properly sealed or poorly operated and maintained;
- Aquifer contamination by leakage of poor quality or contaminated ground water from one aquifer to another via improperly constructed or corroded wells;
- Interaquifer leakage in well holes drilled for other purposes which are not properly abandoned

Guidelines for assessing the risk of pollution of ground water from on site sanitation

The purpose of this chapter is to provide guidance on how to assess and reduce the risk of contamination of ground water supplies from on-site sanitation systems and is aimed at those responsible for planning water supply and sanitation schemes.

Specific objectives include providing:

- guidance where water supply and/or sanitation system are planned
- confidence that existing wells /boreholes are properly constructed (pollution risk is assessed as low and monitoring confirms good quality water)
- help to identify the likely source(s) and pathway(s) of pollution where pollution is observed
- guidance on the planning of monitoring programmes.

The chapter dose not seek to provide a set of perspective rules instead to provide the framework for arriving at a decision based on an evaluation of the risks posed by on site sanitation system to ground water drinking water supplies.

Types of sanitation systems

Sanitation systems can be divided into two principal categories:

1. Off-site methods – these are different forms of sewerage where faecal and household wastes are carried away from the household.
2. On-site methods – including septic tanks and all forms of pit latrines. In these systems the wastes are stored at the point of disposal and usually undergo some degree of decomposition on site.

Risked-based approach

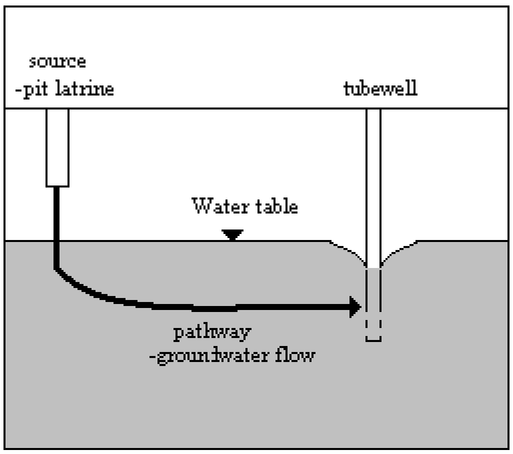
The risk-based approach to contamination uses the concept of source- pathway – receptor, as shown in figure 4.1. For a risk to a receptor (in this case a ground water supply) to exist both a source of contamination and a path way must be present.

In the natural environment, sources of contamination are always present and usually widespread, including on site sanitation.
Pathways that allow water to move from these sources to the receptor can be subdivided into:

- Pathways that occur naturally in the subsurface due to opening and cracks in the soil and rock
- Man-made pathways that occur as a consequence of the design and construction of the receptor, as seen in figure 4.2

Reducing the risk (the receptor) can be achieved by :

- removing the source of contamination or reducing the levels of contaminants that are produced;
- increasing the time for water to travel from the source to the receptor; and
- minimizing man made pathways



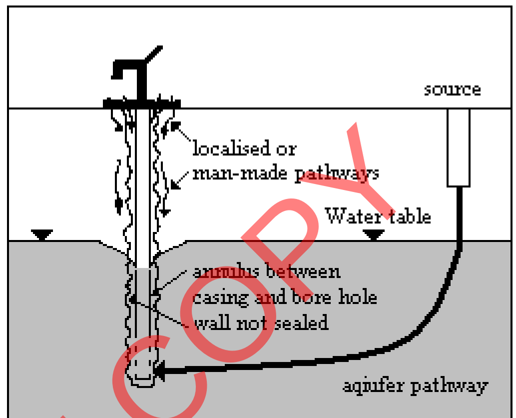


Fig 4.1 Aquifer pathway for contamination pollution pathways

Fig 4.2 Natural & man made from pit latrine to borehole

Aquifer vulnerability to pollution and risk to ground water supplies

The term aquifer pollution vulnerability is used to represent the intrinsic characteristics of the aquifer which determine whether it is likely to be affected by an imposed contaminant load.

Vulnerability assessment is based on the likely travel time for water to move from the ground surface to the water table – the greater the time for travel the greater the opportunity for contaminant attenuation (process of reducing contaminant). Aquifer vulnerability can be subdivided into four broad classes which are defined in Table 4.1. Extreme vulnerabilities are associated with highly fractured aquifers of shallow water table which offer little chance for contaminant attenuation.

Table 4.1 Vulnerability classes

Vulnerability class	Definition
Extreme	Vulnerable to most water pollutants with relatively rapid impact in many pollution scenarios
High	Vulnerable to many pollutants except those highly absorbed and/or readily transformed
Low	Only vulnerable to many persistent pollutants in the very long-term
Negligible	Confining beds present with no significant ground water flow

Hydrogeological environments

The ground water vulnerability to pollution is dependent on the geological structure and the depth to ground water table.

The broad classification of aquifer vulnerabilities for the major hydrogeological environments is summarized in Table 4.2

Table 4.2 Hydrogeological environments and their associated pollution vulnerability

Hydrogeological environments		Natural travel time to saturated zone	Attenuation potential	Pollution vulnerable
Major alluvial formations	Unconfined	Weeks-months	Low-high	High
	Semi-confined	Years-decades	High	Low
Recent costal limestone	Unconfined	Days-weeks	Low-high	High
Intermountain valley infill	Unconfined	Months-years	Low-high	Low-high
	Semi-confined	Days-weeks	Low-high	Low-high

Consolidated sedimentary aquifers	Porous sandstones	Months-years	Low-high	Low-high
	Karstic limestones	Days-week	Low	Extreme
Weathered bedrock terrain	Unconfined	Days-week	Low-high	Extreme

Contaminant associated with on site sanitation

Microbiological

The principal microbiological pathogens found in human faeces can be classified into four broad groups according to their chemical, physical and physiological characteristics. Listed in order of increasing functional complexity the groups are viruses, bacteria, protozoa and helminthes (worms). In general helminthes transmission is unlikely to be related to protected groundwater sources, but would be related to the use of surface waters like ponds or unprotected wells.

Chemical

The chemical contaminant of principal importance that derived from on site sanitation are nitrate and chloride. Nitrate is a health concern and WHO have set a guideline value of 50 mg/l as the save level of where the likelihood of methaemaglobinamenia will be low. Chloride is of less concern for health, but affects the acceptability of the water and thus may result in use of alternative more microbiologically contaminated water. In both cases, environmental protection concern also need to addressed, as remediation of contamination is difficult. Nitrate and chloride are generally stable and, especially in aerobic environments and therefore contamination is likely to be build up and persist in the long term.

Attenuation of contaminants

Attenuation is a process of removal or reduction of contaminant concentration as a result of flow through pathways created by normal porosity and the permeability of the rocks.

Attenuation is generally most effective in the unsaturated zone and in particular in the upper soil layers where biological activity is greatest. The soil layer represents the greatest opportunity for attenuation as both microbiological, and to lesser extent key chemical contaminants, are removed, retarded or transformed as a result of biological activity. At deeper layers in the unsaturated zone, attenuation still occurs, although the processes tend to be less effective as biological activity decreases.

Microbiological contamination

The key processes in the attenuation of microbiological contaminants are

- Die-of and predation
- Adsorption
- Filtration
- Dilution /dispersion

Microorganisms have a limited life span. Die-off rates vary enormously from a few hours up to several months. In ground water, some viruses are known to survive for up-to 150 days. In the case of indicator bacteria, an estimated half-life (i.e. the time taken for a 50% reduction in numbers) in temperature ground water has been noted as being as high as 10 – 12 days, with survival of high numbers up to 32 days.

The removal of microorganisms through predation by other microorganisms may occur readily in biological active layers that develop around the filled sections of pit latrines and these may represent the most effective barrier to breakthrough.

Empirical evidence from limited number of limited number field studies has shown that a separation between the pollution source and the water supply equivalent to 25 days travel time is usually sufficient to reduce concentration of faecal indicator bacteria to levels where detection within most samples is unlikely. However, the studies did not analyse for other pathogens such as viruses that are expected to survive for longer travel times in the subsurface. The generally 1accepted minimum separation for contaminant source and groundwater supply in Western Europe, which aims to bring water quality within guidelines is that equivalent to 50 days travel time. This 50 days travel time is based on survival times of viruses from laboratory and field experments. Therefore with in the guidelines three level of risk are defined:

- Significant risk – less than 25 day travel time
- Low risk-greater than 25 day travel time
- Very low risk- greater than 50 day travel time

The 'low risk' category provides confidence, but no guarantee, that travel time between contaminant source and groundwater supply would result in level of microorganisms which are unlikely to present a major risk to health.

In this chapter we suggest that a water supply is acceptable where the risk assessment is considered low or very low and the monitored water quality meets the guideline value.

Chemical attenuation

Biological uptake of nitrate occurs with in the soil. However, this may be overwhelmed during recharge periods, when rapid leaching of nitrate held in the soil may occur. In the case of nitrate sources such as on site sanitation and solid waste dumps, leaching is expected, partly because there is limited ability for uptake of nitrate either because plants are not present or their roots do not normally extend to the base of the latrine. Under aerobic condition nitrate is mobile and not retarded. In the saturated zone and where ground water conditions are anerobic, denitrification can occur. Denitrification is a microbiological process in which bacteria consume nitrate(in the absence of oxygen)

Risk assessment

There are two principal routes by which boreholes, wells and spring may become contaminated by on site sanitation system.

⇒ the first relates to the natural vulnerability of the aquifer to pollution. This pathways exists naturally because the subsurface is permeable and water and contamination can percolate from on site sanitation systems to the water table and from there migrate into the ground water supply.

⇒ the second route is where a pathway is created by the poor design or construction of the ground water supply.

In these guidelines three types of contamination are considered:

1. Microbiological contamination of ground water supply due to the natural vulnerability of the aquifer.
2. Microbiological contamination of ground water supply due to the poor design or construction of the ground water supply. (Construction failures)
3. Nitrate contamination, especially where the population density is very high.

1.1 **Risk assessment of microbiological contamination due to the aquifer vulnerability** If the

❖ hydraulic load of on -site sanitation is low, i.e. dry pit latrines or pour flash (low usage) sanitation system is used.
❖ unsaturated zone is composed of fine sand, silt sand and clay; soft weathered basement and medium clean sand.
❖ water table is greater than 5 m for fine sand, silt sand and clay; greater than 10 m for soft weathered basement.

If the

❖ hydraulic load of on-site sanitation is high, i.e. septic tanks and the depth to the screen is greater than 30 m
❖ sediments are unconsolidated and not coarse grained.

<u>Then ***10 m*** nominal horizontal separation is sufficient.</u>

If the above conditions are not satisfied, then: -

➢ select horizontal separation between potential pollution source and water supply using table 5.1.

The horizontal separation is calculated by the following simple equation.

$$\text{separation} = \text{velocity} \times \text{time}$$

$$t = 25 \text{ or } 50 \text{ days}$$

$$\text{where velocity} = \frac{KI}{n}$$

K= hydraulic conductivity (permeability) m/d
I= hydraulic gradient
n=porosity(effective)

Value of parameter can be obtained from table 5.1. For safety take higher permeability value and minimum porosity.

Table 5.1 Aquifer properties and feasibility of using horizontal separation

Lithology	Typical porosity	Typical Kh:Kv ratio	Range of likely permeability (m/d)	Feasibility of using horizontal separation	Lateral separation to reduce faecal indicator bacteria to acceptable levels
Silt	0.1-0.2	10	0.01-0.1	Yes	Cms
Fine silty sand	0.1-0.2	10	0.1-10	Yes , should be generally acceptable	up to several meters
Unconsolidated weathered basement	0.05-0.2	1	0.01-10	Yes	Cms-meters
Clean sand	0.2-0.3	1	10-100	uncertain, needs site specific testing and monitoring	tens-hundred meters
Gravel	0.2-0.3	1	100-1000	not feasible	
Fractured rocks	0.01	1	Difficult to jeneralize, velocities of tens and hundreds of m/d possible	not feasible	

Risk assessment of microbiological contamination due to construction failures

- If an adequate sanitary seal 2-3m cement grouting is installed
- If the aquifer is unconsolidated and not coarse grained and the screen is at a depth greater than 30 m.

Then well design /construction meets ***acceptable criteria***.

If the well design /construction never meets acceptable criteria, then.
- ✓ Find the means to seal the well.
- ✓ Select drilling method/design which permits installation of sanitary seal, if the cost is acceptable.

Risk assessment of nitrate contamination

Risk of nitrate contamination of ground water supply from on-site sanitation always exists; the question rises whether the amount of leached nitrate is low or high.
Two things have to be considered:

1. The amount of leached nitrate from on-site sanitation to the ground water. If the amount is high, then high risk; and low risk for low amount.

The amount of leached nitrate depends on the:
- ➢ the hydrogeological environment , which affects groundwater recharge.
- ➢ the size of on-site sanitation system, which is proportional to the population
- ➢ the climatical condition, which governs amount of nitrate concentration. In humid area low concentration and in arid very high.

Is NO_3 concentration of infiltration is greater than 50 mg/l, i.e., the recharge rate less than 500 mm/a and the population density is greater than 30 p/ha or, the recharge rate is less than 200 mm/a and the population density greater than 10 p/ha, the recharge rate less than 100 mm/a and the population density greater than 5 p/ha.

The time delay to reach deep ground water.

The delay may be important and allow other measures (different sources of water or sewered sanitation) to be installed in the longer term.
The risk assessment is to determine if the risk is in short term or in long term.

Significant delay can be anticipated where the screened interval of the borehole is deep (>30m) and the aquifer possesses considerable porosity (e.g. unconsolidated).

Recommended design and construction of ground water supply

The good design and construction of ground water supplies is critical to the prevention of contamination
The following are the basic or essential components for design and construction that are required to limit the risk of contamination

Borehole

- Sound cement seal around the casing to depth 3-5m from the surface.
- Where the top of the gravel pack is at a depth greater than 5m the annular section between the casing and the top of the gravel pack should be backfilled to a depth of 5m below ground surface with suitable materials such as the drilled cuttings.
- The seal can be mixed in the ratio of 30 liters of water to 50 kg Portland cement.
- Placing the screen as deep as possible.
- Concrete collar (or concrete plinth) a square or circle shape with at least 2 m in diameter and 0.5 m in height (Whitfield, 1995: pers.comm.). Approximately 0.2 m of the collar should be below ground surface and 0.3 m above it. The height above the ground can increase according to site condition (if the area is low land and exposed for flood water).

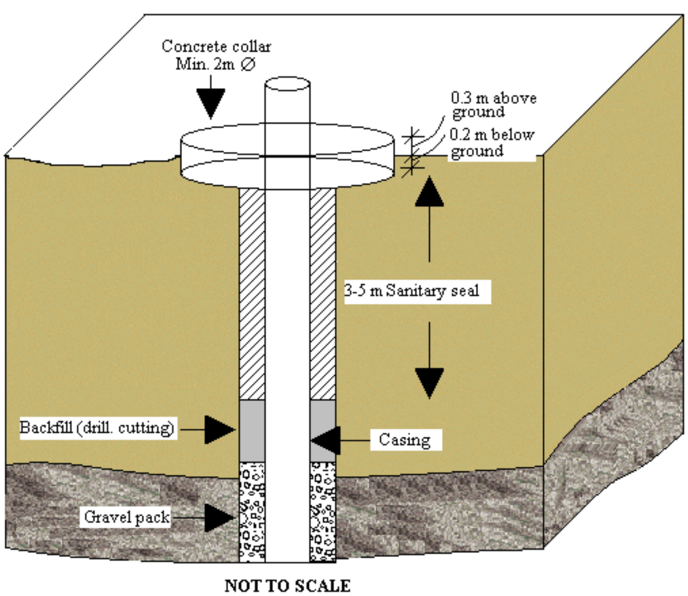
Protected spring

- A filter media will be laid in the back fill area and should be sufficiently fine to provide reasonable filtration and attenuation (a pea gravel of nominal diameter of at most 25 mm is sufficient, although finer may be required in urban or more intensely polluted areas.)
- The filter should be overlain by a clay layer to reduce infiltration by surface water, with above this a sand layer to remove cysts and finally a soil layer.
- The backfill should have a full grass cover and be protected by a fence and diversion ditch.

Dug well

- Plaster seal on the lining wall with cement mix (1:2:4 or 1:3:6)
- The lining should extend at least 0.3 m above the level of the ground as headwall and covered by slab.

Fig 6.1 Sanitary standard for borehole construction



Minimum sanitation siting requirements

Pit latrine

- The base of the pit should be at least 2 m above the water table of the underlying aquifer
- Pit depth should be restricted to a minimum, thereby increasing the attenuation zone between the effluent and the ground water table.
- Surface water runoff diverting structures such as berms and shallow cut trenches should be constructed around the latrines.

Septic tank

- The base of the unit should be at least 2m above the water table of the underlying aquifer.
- Septic tank should not be constructed within reach of the normal storm water or floods of rivers and streams
- Septic tank depth should be restricted to a minimum, thereby increasing the attenuation zone between the effluent and the ground water table.
- Surface water runoff diverting structures such as berms and shallow cut trenches should be constructed around the latrines.

Solid waste disposal site

- Geological impervious strata should be selected for a waste-dumping site.
- It is preferable to construct a waste dumping site on a confined aquifer.
- The water table should be at least 10 meter below the ground surface.
- The disposal sites are not to be constructed within reach of the normal storm water or floods of river and streams.
- For higher risk waste sites and near by located ground water development, there have to be monitoring boreholes to detect any sign of pollution penetrating into groundwater system.