MINISTRY OF LOCAL GOVERNMENT, HOUSING AND CONSTRUCTION

NATIONAL WATER SUPPLY AND DRAINAGE BOARD

SRI LANKA

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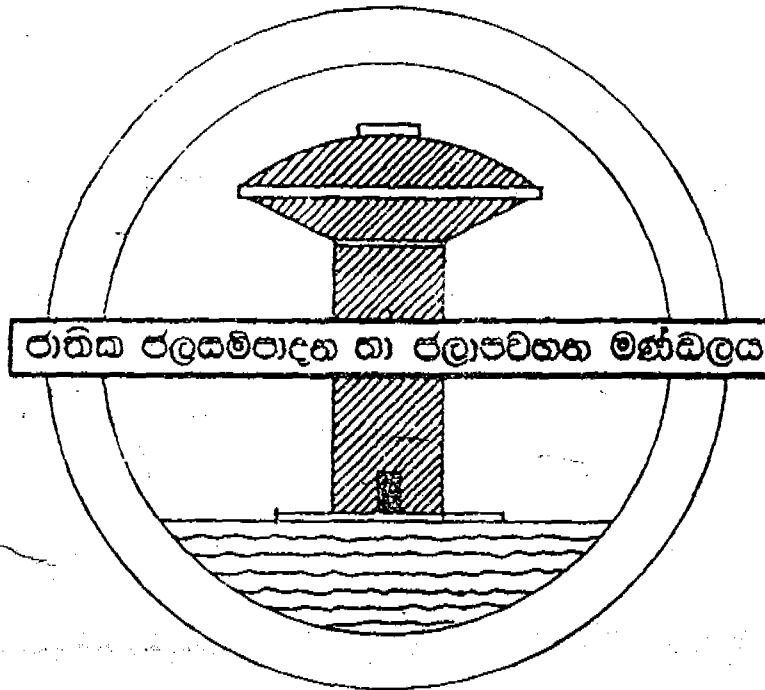
DESIGN MANUAL D3

WATER QUALITY AND TREATMENT

MARCH, 1989

WATER SUPPLY AND SANITATION SECTOR PROJECT

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DESIGN MANUAL   D3

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March 1989

MANUAL D 3

WATER QUALITY AND TREATMENT

Table of Contents

A N N E X E S

A.   Design Example — Cascade Aerator

B.   Design Example — Hydraulic Flocculator

C.   Design Example — Orifice Plate for Metering/Control of Rapid Gravity Filter with Declining Rate Flow.

D   Filter Paper Test

E   Preparation of Sand for Filters

F   Hydraulics of Under-Drain Systems

G   Practical Guide for Dosage of Bleaching Powder in Disinfection of Public Drinking Water Supplies

H   Information to be Included in the Tender Specifications for a Water Treatment Works

List of Figures

1. INTRODUCTION

This Manual is for the use of NWSDB Engineers in assessing water quality, identifying the need for water treatment, and in planning, designing, rehabilitating and, to some extent, operating water treatment facilities. It is a development of the Design Manual on Small Community Water Supplies, prepared for NWSDB with WHO assistance in 1982 (Ref.1), and includes all relevant material from this earlier manual, which in some cases has been reviewed or updated.

The Manual is substantially more comprehensive than the earlier manual, and includes sections on water quality characteristics and standards, water borne diseases, sampling and testing, interpretation of chemical analyses, and other specific unit processes not covered previously.

The manual was put together by G. A. Bridger, Environmental/ Sanitary Engineer, USAID Project from a variety of source materials, the most important of which are listed in the References.

2. WATER QUALITY CHARACTERISTICS
AND STANDARDS

2.1 DRINKING WATER QUALITY

A potable water supply is essential for the prevention of waterborne diseases. Water that is contaminated by pathogenic (disease causing) organisms or that is unacceptable aesthetically will need some kind of treatment before being used. It is the engineer's responsibility to provide an ample supply of pure and wholesome water, free from:

o visible suspended matter;
o colour, taste and odour;
o objectionable dissolved matter;
o bacteria indicative of pollution; or
o aggressive constituents.

In assessing potential water supplies it is the role of the engineer to be able to identify and measure any water quality problems which exist or may occur under certain conditions, and to understand the impact of these problems on the potability of the water. Subsequently, suitable treatment methods must be provided, if necessary, to ensure that a potable supply is delivered at all times. Such treatment must be carefully designed to an appropriate technological level to match the operating and maintenance skills available, or the quality of water produced may fall below acceptable standards.

2.2 WATER QUALITY CHARACTERISTICS

Physical Characteristics

o Temperature – important for its effect on other properties, such as speeding up chemical reactions, changing solubility of gases, amplifying tastes and odours, etc.

o Taste and
  odour – due to dissolved impurities, often organic in nature, e.g. phenols and chlorophenols; difficult to measure.

o Colour – true colour is due to material in solution, apparent colour due to suspended matter. Natural yellow colour in upland catchment water is due to organic acids and is harmless. Measured in degrees Hazen.

o   Turbidity –   colloidal solids give water a cloudy appearance, aesthetically unattractive and possibly harmful. May be due to to clay and silt particles, sewage or industrial waste, or micro-organisms.

o   Solids –   may be organic/inorganic matter in suspension and/or solution. Total dissolved solids (TDS) are due to soluble materials; suspended solids (SS) are discrete particles which may be filtered out. Settleable solids are those which settle out in a graduated cone after standing for 2 hours (useful for assessing sedimentation tank performance).

o   Electrical conductivity–   depends on quantity of dissolved salts, and approximately proportional to TDS content.

Chemical Characteristics

o   pH –   the intensity of acidity or alkalinity of a sample, actually the concentration of hydrogen ions present.

o   Alkalinity –   due to the presence of bicarbonate, carbonate or hydroxide. It is useful as it provides buffering to resist changes in pH. Expressed in mg/L of $CaCO_3$.

o   Acidity – CO_2 acidity is in the pH range 8.2–4.5, and most natural waters are in this range. Mineral acidity, below pH 4.5 due to H_2SO_4, HCl or HNO_3 would normally be due to industrial wastes. Expressed in mg/L of $CaCO_3$.

o   Hardness –   the property which prevents lather formation with soap and produces scale in hot water systems. Due to metallic ions of Ca, Mg, Fe, Sr.

There are two forms –
    i)   Carbonate hardness (metals associated with HCO_3) and

    ii)   Non-carbonate hardness (metals associated with SO_4, Cl, NO_3).

Total hardness – alkalinity = non carbonate hardness

Expressed in mg/L of $CaCO_3$.

-3-

o   Dissolved
    Oxygen(DO) –   required for maintaining higher forms of
                   biological life in water; the effect of
                   a waste discharge on a river/stream is
                   largely determined by oxygen balance of
                   system. Solubility of oxygen varies
                   with temperature:

                   °C   10   20   30
                   DO mg/L   11.3   9.1   7.6

o   Nitrogen –   exists in 4 main forms:
                   i)   Organic nitrogen (in the form of
                        proteins, amino acids and urea).

                   ii) Ammonia nitrogen (as ammonium
                        salts or free ammonia).

                   iii) Nitrite nitrogen (final oxidation
                        product). Analyses can give useful
                        indication of pollution of sample,
                        but bacteriological analysis is
                        more reliable.

                   iv) Nitrate nitrogen – (an
                        intermediate oxidation stage)

o   Chloride –   gives a brackish taste and may be
                   indicator of sewage pollution due to
                   chloride content of urine. Detectable
                   at about 250 mg/L upwards.

Biological Characteristics

Bacteriological analysis of water is the most sensitive
water quality parameter, since almost all organic wastes
contain large numbers of micro-organisms. Microorganisms
are important in water quality control in terms of self-
purification of rivers and stabilization of organic
wastes. They are, however, responsible for waterborne
diseases, tastes and odours, corrosion and microbial
growths in bodies of water. They may be broadly classified
as either animals or plants.

Plants –   require soluble food which may be organic or
           inorganic.

o   Viruses – the smallest plants less than 0.3 um in
              diameter, and can only live and reproduce
              inside a specific host cell. Diseases such
              as influenza and poliomyelitis are caused by
              virus infections. Commonly found in large
              numbers in sewage effluents, they are
              difficult to remove and resistant to normal
              disinfection methods. Requires strong
              oxidation to kill them.

o   Bacteria –   basic units of plant life, being single cell organisms of various shapes ranging in size from 0.5 to 5 um. May be aerobic or anaerobic, and are responsible for waterborne diseases. Important for waste water stabilization.

o   Fungi –   multicellular aerobic plants, more tolerant of acid conditions and a drier environment than bacteria. Exist in polluted rivers, and responsible for tastes and odours.

o   Algae –   Multi-cellular photosynthetic (energy from sunlight) plants, incorporating chlorophyll. Are usually microscopic in freshwater, and utilize inorganic compounds such as CO_2, NH_3, PO_4 to produce more cells and oxygen. Growth of large algal blooms in lakes can cause severe taste and odour problems.

Animals –   require only organic food, usually in solid form.

o   Protozoa –   single cells 10-100 um long and live by eating large quantities of bacteria, and are important in biological waste treatment.

o   Rotifers –   multicellular animals which like stable conditions with high DO, therefore indicators of good quality water.

o   Crustaceans –   hardshelled multicellular animals, an important fish food, and found in stable conditions. Some may be seen with the naked eye.

o   Worms and
Larvae –   rotifiers which scavenge in bottom deposits.

Measurement of bacteriological quality

It is not practicable to test the water for all organisms it might contain, therefore bacteriological quality is measured using certain specific types of bacteria which are indicative of contamination. Coliform bacteria are the predominant organisms that are found in the intestinal tract of man and other animals. The coliform group includes faecal and non-faecal bacteria, such as are found in soil. Presence in drinking water of significant numbers of coliform bacteria indicates that the source is not adequately protected. E-coli (Escherchia Coli) is a specific type of coliform organism which indicates contamination by faecal matter. Generally, testing is done for the presence of coliforms and of E-coli.

2.3 WATER QUALITY STANDARDS

The standards for water quality in Sri Lanka are based on the SLS 614, 1983 - Part 1 Physical and Chemical Requirements, and Part 2, Bacteriological Requirements, which are similar to the WHO Standards (See Table 2.1, 2.2).

These standards should always be applied with common sense, particularly for small community and rural water supplies where the choice of source and the opportunities for treatment are limited.

The criteria should not in themselves be the basis for rejection of a groundwater source having somewhat higher values for iron, manganese, sulphates or nitrates than in the Table. Care must be exercised in respect of toxic substances such as heavy metals, which should be allowed only after expert opinion of the health authorities has been obtained.

For small supplies which frequently are to be provided from individual wells, boreholes or springs, the water quality criteria may have to be relaxed. Obviously, in all instances, everything possible should be done to limit the hazards of contamination of water. Using relatively simple measures such as the lining and covering of a well, it should be possible to reduce the bacterial content of water (measured as coliform count) to less than 10 per 100 mL, even for water from a shallow well. Persistent failure to achieve this (particularly if E. coli is repeatedly found) should as a general rule lead to condemnation of the supply.

Table 2.1

Specification for Potable Water

SLS 614 (1983)

PART 1 – PHYSICAL AND CHEMICAL REQUIREMENTS

Characteristic	Maximum Desirable Level	Maximum Permissible Level
pH	7.0 – 8.5 units	6.5 – 9.0 units
Colour	5 units	30 units
Odour	Unobjectionable	Unobjectionable
Taste	"	"
Turbidity	2–JTU	8–JTU
Elect.Conductivity	750 uS/cm	3500 uS/cm
Chloride (Cl)	200 mg/L	1200 mg/L
Chlorine–Free resid(Cl)	–	0.2 "
Alkalinity (as CaCO	200 "	400 "
Ammonia–Free	–	0.06 "
Ammonia–Albuminoid	–	0.15 "
Nitrate (as N)	–	10 "
Nitrite (as N)	–	0.01 "
Fluoride (as F)	0.6 "	1.5 "
Phosphates–Total (PO)	–	2.0 "
Total Solids	500 "	2000 "
Hardness Total (as Ca Co)	250 "	600 "
Iron–Total (as Fe)	0.3 "	1.0 "
Sulphate	200 "	400 "
Calcium	100 "	240 "
Magnesium	30 to 150 *	150 "
Copper	0.05	1.5 "
Manganese	0.05 "	0.5 "
Zinc	5.0 "	15.0 "
Aluminium	–	0.2 "
Arsenic	–	0.05 "
Cadmium	–	0.005 "
Cyanide	–	0.05 "
Lead	–	0.05 "
Mercury		0.001 "
Selenium	–	0.01 "
Chromium	–	0.05 "
Anionic Detergents (as MBAS–LAS)	0.2 mg/L	1.0 mg/L
Phenolic Compounds (as Phenolic OH)	0.001 "	0.002 "
Oil & Grease	–	1.0 "
Pesticide Residue	(Refer to WHO &	FAO requirements)
Chem. Oxygen Demand (COD)		10 mg/L

* Depending on sulphate content, i.e. for 250 mg/L sulphate, max Mg. is 30 mg/L; for less sulphate, more Mg is allowed

-7-

Table 2.2

Specification for Potable Water

SLS 614 (1983)

PART 2 — BACTERIOLOGICAL REQUIREMENTS

Requirements

1.   Pipe-borne water supplies:

   o   Throughout any year, 95 per cent of the samples shall not contain any coliform organisms in 100 mL.

   o   None of the samples examined shall contain more than 10 coliform organisms per 100 mL.

   o   Coliform organisms shall not be detectable in 100 mL of any two consecutive samples.

   o   None of the samples examined shall contain E.coli in 100 mL. (Faecal coliform).

2.   Individual or small community supplies:

   o   None of the samples examined shall contain more than 20 coliform organisms per 100 mL on repeated examination.

   o   No sample shall contain E. coli in 100 mL. (Faecal coliform)

NOTE: Individual or small community supplies include wells, bores and springs.

Frequency of Sampling for Pipe Borne Water

Population Served	Maximum Interval Between Successive Samples	Minimum Number of Samples to be Taken From Whole Distrib. System Each Month
Less than 20,000	1 Month	1 Sample per 5000 popn.
20,000 to 50,000	2 Weeks	– do –
50,000 to 100,000	4 Days	– do –
More than 100,000	1 Day	1 Sample per 10000 popn.

2.4   WATER RELATED DISEASES

Water related diseases are those traceable to water supply and excreta disposal. The most important of these diseases may be classified as follows:

Category	Transmission	Specific Diseases
Water-site-insect carried	Disease-carrying insects breed in or near water.	African trypanosomiasis (sleeping sickness) Onchocerciasis (river blindness) Malaria Arboviruses (yellow fever and dengue) Filariasis (including elephantiasis) Japanese encephalitis
Water contact	Disease transmitted by direct contact with water.	Schistosomiasis (bilharziasis)
Water quality microbiological	Disease transmitted by consumption of microbiologically contaminated water.	Cholera Typhoid fever Diarrhea Dysentery (amoebic, bacillary) Guinea worm (dracontiasis)
Sanitation-related/water hygiene	Disease transmitted by inadequate use of water.	Shigellosis (bacillary dysentery). Trachoma and conjuctivitis Ascariasis (roundworm) Scabies

The various categories are:

o   Water-site-insect carried – where the source or supply site provides a breeding ground for insects such as mosquitos, which transmit diseases by biting people – it is not necessary for people to come into contact with the water. Transmission is complex, involving a host (man or animal), a parasite that causes the disease (protozoa or worm) and a carrier (insect, fly or mosquito). Malaria, filariasis, dengue and Japanese encephalitis carried by mosquitos. Piped water supplies, and elimination of mosquito breeding places, would help to control these diseases.

o   Water contact – where contact with water such as swimming is sufficient to produce infection in people, usually through a disease-causing organism penetrating the feet, legs, hands or other part of the body.

o   Water quality – microbiological – where consumption of water causes disease by ingestion of a germ, or toxic substance. These diseases are endemic in areas where excreta disposal is haphazard and can achieve epidemic proportions in serious cases of contamination.

o   Sanitation-related water hygiene – infection is most common due to faecal contamination of water, food, and fingers and might have been eliminated through proper washing or cleaning.

3. SAMPLING AND TESTING

3.1 SANITARY SURVEY

A sanitary survey is essential for the adequate interpretation of chemical and bacteriological analyses of drinking water supplies. It consists of an extensive field inspection and evaluation of local environmental and health conditions by assessing current and potential hazards to the water supply or existing water system.

Sanitary surveys are important as backup information when interpreting the results of water analyses. If analyses indicate contamination of the water supply, a sanitary survey should be conducted to identify the source of the contamination.

Sanitary surveys should be conducted for:

o   development of new water sources;

o   rehabilitation/inspection of existing schemes.

Simple observation is the major part of all sanitary surveys. Inspections of existing water supply systems involve checking facilities and operational practices for signs of contamination. For example, water from a well should be considered as possibly contaminated if the well is uncovered or unprotected, or when the collection vessel is not kept clean. Leaking pipes may draw in sewage or other contamination from the soil particularly in intermittent systems.

If a sanitary survey reveals possible bacteriological contamination, further investigation should should be done to determine the source and the level of contamination and determine the treatment needed, or steps taken to eliminate the source of contamination.

Sanitary surveys of existing systems can reveal physical and chemical impurities. Water that is turbid may not be safe or aesthetically acceptable. If pipelines, pumps or other system components are corroded, the water supply may be chemically contaminated. Hard, scaly deposits on pipes or pumps indicate water with high mineral content which may be unsuitable for many domestic purposes. Minerals can encrust pipes and pots. If red stains appear on plumbing the water may be high in iron. If the community believes a water source is harmful even though contamination is not evident, the water should be tested for harmful toxic chemicals.

If a sanitary survey reveals possible physical or chemical contamination, a complete laboratory analysis should be done to verify conditions. If analysis reveals that the water system is seriously contaminated either physically or chemically, the problem should be solved immediately or another water source should be developed.

3.2 TYPES OF ANALYSIS

The following types of analysis are listed in Reference 4 as being carried out by NWSDB Central Laboratory.

o Partial Analysis (Short)

The parameters assessed for a partial analysis include: temperature (°C), residual chlorine (RCl), hardness (EDTA), alkalinity (ALK), chloride (Cl), hydrogen ion concentration (pH), electrical conductivity (EC), turbidity (NTU), and sometimes colour and ion. Other parameters may be included when demanded – such as copper, sulphides, sulphates, and nitrogen as ammonia and albuminoid.

o Complete Analysis (Major Constituents)

The traditional "Complete" analysis, in addition to the parameters measured for partial analysis, includes: silica (SiO_2), calcium, (Ca), magnesium (Mg), sodium (Na), potassium (K), sulphate (SO_4), chloride (Cl), nitrate (NO_3), fluoride (F), boron (B), and total filterable residue (TFR). Values to be calculated from these measured values include carbonates (CO_3), bicarbonates (HCO_3), free carbon dioxide (CO_2), percent sodium (%Na), total dissolved solids (TDS), and the sulphate to chloride ratio (SO_4:Cl). Ammonia nitrogen and albuminoid nitrogen may require analysis as demanded by the nature of the source water.

o Minor and Trace Inorganic Analysis
   (Metals and Nonmetals)

The analyses in this group are scheduled on a rotating basis. Metal minor and trace inorganic constituents include: aluminium (Al), arsenic (As), barium (Ba), beryllium (Be), cadmium (Cd), chromium (Cr), cobalt (Co), copper (Cu), iron (Fe), lithium (Li), lead (Pb), manganese (Mn), mercury (Hg), molybdenum (Mo), nickel (Ni), selenium (Se), silver (Ag), strontium (Sr), and zinc (Zn).

Nonmetal minor and trace inorganic constituents include sulphate (H_2S), phosphates (PO_4), nitrates (NO_2-N), ammonia nitrogen (NH_4-N), and organic nitrogen (Org.N).

-12-

o   Organic Analysis

Chemical oxygen demand (COD), biological oxygen demand (BOD), extractable oil and grease, and total organic carbon (TOC) are included in this group.

o   Microbiological Analysis

This analysis of water includes tests for coliform and faecal coliform organisms. In addition, special straining techniques such as the gram stain are used when applicable.

o   Biological Analysis

This analysis of water from surface sources and reservoirs includes the identification and enumeration of observable organisms. Included in the grouping are the green and blue-green algae, and other floral forms such as submerged aquatics, and animate forms such as the aquatic invertebrates.

o   Reservoir and source water

These waters are monitored for fluctuations in dissolved oxygen, pH, conductivity, and turbidity (or an equivalent) as a function of location, depth and time with the objective of obtaining improved water quality through better reservoir management.

Another analysis which is of interest in design and rehabilitation of schemes is:

o   Filter Media Cleanliness Determination.

Designed to provide a quantitative evaluation of filter media condition (cleanliness) and/or backwash efficiency. This procedure can be used for cleanliness determination on any type of filter media. Media samples can be taken in depth increments for the determination of accumulated solids at any particular layer, or a standard 300 mm core sample can be taken for the determination of the total accumulated solids remaining per volume of filter media.

The results in gm/L are compared to those of a new media placement or specially washed used media. A filter bed should be investigated at least twice per year to determine the condition of the media in order to advise on maintenance or to evaluate filtration effectiveness. The need for this type of determination can be observed in the monitoring of filter effluent turbidities.

-13-

In addition, it should be noted that the Laboratory
conducts analyses to measure the characteristics of
treatment chemicals and the characteristics of sludge.

3.3   SAMPLING PROCEDURES

Accurate analysis of a water sample depends on proper
collection of the sample. Samples must be taken from
several locations at different times so that they are
representative of the entire water supply. Samples for
specific analysis are those collected when a new water
source is being developed, when an outbreak of water-
related disease occurs, when a scheme is being rehabil-
itated, or when water pollution is suspected. Samples
collected regularly to monitor an existing water system
are for routine analysis.

Methods of sample collection depend on whether the
analysis to be made is bacteriological or physical and
chemical, and on whether the sampling and analysis are
routine or specific. The main considerations in sampling
are to collect representative samples of the water and
transport them quickly to the laboratory. Where the
equipment is available, field testing may also be carried
out.

Samples must be collected in the correct volume, with the
right device and in the right container. They must be
marked properly showing the date, time and exact location
of the sampling point, and be accompanied by an analysis
request form showing the required tests. In addition,
samples should be adequately preserved, if required, and
transported to the laboratory with a minimum of delay.
Analyses only represent the water quality at the time of
sampling, and it will often be necessary to take a series
of samples at different times or seasons in order to
obtain a more complete picture of the water quality
variations. This is particularly true for bacteriological
samples — analysis of a single sample is not reliable.
For routine bacteriological sampling, the frequency of
sampling is established in SLS 614. (See Table 2.2)

Samples for bacteriological analysis must be collected in
sterile glass bottles with sterile stoppers. A paper or
foil hood over the stopper and bottle neck is necessary.
Stoppers can be made of ground glass or rubber. Normally,
pre-sterilised sample bottles are obtained from the
Central Laboratory. In an emergency, well-washed bottles
can be sterilized in the field by boiling them for five
minutes. If sample bottles are not sterile, any bacteria
present on them will make the analysis useless. The
sample bottle must be kept sealed, and the stoppers and
bottle neck should be covered with parchment paper or thin
aluminium foil. The covering should be kept in place
after sampling. Each sample should be at least 100 mL in
order to perform all tests required for bacteriological
contamination.

-14-

Samples for physical and chemical analysis must be collected in chemically clean plastic or glass bottles. At least 2 litres of water need to be collected for a complete physical and chemical analysis.

Samples drawn from a stream or river that is being developed as a new source should not be collected too close to the bank nor too deep under the surface. Areas of stagnation should be avoided. Samples should not be drawn from the same spot every time, but should be taken from different points.

In piped water systems, samples should be collected at all points where water enters the distribution system. In addition, samples should be drawn from taps connected directly to the water main and not from storage tanks.

Special samples need to be drawn from known problem areas, such as areas with low pressure, areas with high leakage, and areas far from the treatment plant. When collecting bacteriological samples, great care must be taken to avoid contamination of the sample by touching the inside of the stopper or bottle neck or contamination from other sources apart from the water being tested. Bottles should be labelled immediately after sampling.

The following procedures should be used for sampling from different sources:

o   Stream or river – collect from midstream, if possible, holding base of bottle and scoop water in an upstream direction beneath the surface.

o   Lake or tank – collect from alternative intake locations, plunging bottle neck down into the water and filling the bottle horizontally, beneath the surface.

o   Shallow well – use a weighted bottle and lower to required depth, until full.

o   Handpump well – pump to waste for at least 1 minute before sampling.

o   Tap – allow to run to waste for at least 1 minute before sampling.

Sample labels should give the following information:

o   Sample number;
o   Reason for sampling;
o   Exact place of sampling and type of source;
o   Temperature of sample source;
o   Date and time of sampling;
o   Sample depth (if required);

-15-

Additional information, as shown in Table 3.1 should be kept for each sample, to aid in the interpretation of results. It is also useful to sketch on the reverse side of the form the example location.

Storage of samples Changes occur in the bacteriological content of water when it is stored. To get an accurate analysis, bacteriological testing of the water sample is best begun within one hour of collection. Samples for bacteriological analysis must be tested within 24 hours of collection. When a field kit is used to analyse samples, testing within one hour of collection is possible. If samples must be transported to another site, they must be carefully stored and transported so they remain representative of the water supply at the time the sample was collected. Temperature of samples during storage should remain as close as possible to the temperature of the source from which they were drawn. The length and temperature of storage of all samples should be recorded and considered in interpreting the analysis.

For physical and chemical samples, analysis should be done as soon as possible, and should not be delayed over 72 hours. Whenever possible, samples should be kept between 0°C and 10°C during storage and transportation.

3.4 INTERPRETATION OF CHEMICAL ANALYSIS

Interpreting the results of a chemical analysis requires a knowledge of three factors:

a) What is the water to be used for?

b) What are the values of the test results?

c) What are the consequences of using the water for its intended use, taking into account the test results?

Some substances, although dangerous in excessive amounts, may be tolerable in trace amounts. These among others include lead, arsenic, fluoride, selenium, and hexavalent chromium. They are not normally determined unless there is reason to believe the elements are present, since they seldom occur in a water supply.

Engineers involved with interpretation of chemical analysis of water should know, for instance, that nitrate and sodium make water unfit only in specific cases; chlorides and sulphates over 500 mg/L will give water a salty taste; sulphates have a laxative effect; iron causes staining; and calcium and magnesium cause heavy use of soap and leave residues.

-16-

Table 3.1   Additional Sampling Data

Name of person requesting sample: _____ Sample No: _____

Reason for sample: ☐ Routine   ☐ for bacteriological analysis

                ☐ Specific (explain) _____   ☐ or physical/chemical analysis

Date and hour of sampling _____   ☐ for other analysis

Sample location:

Scheme/Project: _____

Sample source: ☐ Tap   ☐ Cistern   ☐ Stream   ☐ Pond   ☐ Well

               ☐ Spring   ☐ Rain catchment   ☐ Other (specify) _____

Source name, if any   _____

Exact spot from where sample was drawn: _____

Is water treated? ☐ No ☐ Yes   (If yes, specify type of treatment) _____

Does water quality change after heavy rains? ☐ No ☐ Yes   If yes, explain (odour, color, taste, turbidity)_____

For well sample, specify:

    Depth of sampling   _____
    Depth of well   _____
    Distance from water surface to ground level   _____
    Whether well is covered or uncovered   _____
    Type of cover, materials, condition of cover _____
    Whether well is newly constructed or recently altered or repaired (explain) _____
    Method of raising water (pump, rope and bucket, etc.) _____
    Whether the well has a protective apron _____
    Type, material, size and condition of apron: _____
    Well lining material _____
    Possibly sources of contamination _____

For spring sample, specify:

    Whether sample is drawn directly from spring or collection box _____

    Construction material and condition of collection box _____

For lake or tank sample, specify:

    How lake is supplied   _____
    Nature of catchment – conserved or not   _____
    Nature and extent of weed growth or algae   _____
    Possible sources of contamination   _____

For stream or river sample, specify:

    Depth at which sample was drawn   _____
    Whether sample was drawn from boat   _____
    Possible sources of contamination   _____
    Whether water level was above or below normal   _____

Length of time sample was stored before analysis   _____

Temperature at which sample was stored   _____
Remarks _____
    (eg. weather conditions or recent rainfall which may affect sample)

Completed by _____   Date _____

When a chemical analysis is required, engineers should be aware of the objectives and the available tests, (refer to Section 3.2) and be selective regarding tests to be carried out. A guide to intepretation of chemical analyses is given in Table 3.2.

The "partial analysis" is designed to give enough basic information about a proposed water supply to be able to determine whether the supply is suitable as a potential water supply. The tests are not too cumbersome, time consuming or costly and include the following:

pH -

Values are given in units varying from 0 to 14. All pH values below 7.0 denote acidity. Values above 7.0 denote alkalinity. The closer the value is to 7.0 (neutral) the more neutral is the substance, thus a pH value of 5.5 indicates a greater degree of acidity than a value of 6.7. Similarly 7.6 is slightly alkaline while 9.5 indicates a considerable degree of alkalinity. The further one gets from 7.0 the more severe is the acidity and alkalinity.

A pH of 6 is ten times the acidity of neutral water, a pH of 5 is 100 times, a pH of 4 is 1000 times, and a pH of 3 is 10,000 times the acidity of neutral water. Below pH 6.5, corrosion and objectionable tastes may occur.

Colour -

Colour in water is generally derived from leaves, peat, logs and other similar organic substances. Sometimes it is due to iron or manganese that has combined with organic matter. Swamp water is the source of most coloured water. Colour is not injurious to health but is aesthetically objectionable.

True colour of water is due to material in solution while the apparent colour is due to the effect of particles of suspension. Only true colours are important, therefore substances in suspension must be removed before colour determination is made. Water that is highly turbid is seldom highly coloured because materials causing turbidity tend to absorb those causing colour.

The unit of colour used as a standard is that produced by 1 mg of platinum in a litre of water. Colour should not exceed 30 and preferably be less than 5. Colour of less than 3 units will not be noticed, even in a filled bath tub, whereas colour of 5 units, although scarcely perceptible, will be noted by many.

Table 3.2   Guide to Characteristics of Water Quality

Name of Test	Results (mg/L)	Rating	Remarks
pH	7.0 to 8.5 units 5.0 to 7.0 and 8.5 to 9.0 below 5.0 above 9.0	Good Fair Bad	
Colour	0 - 5 units 5 - 30 units above 30 units	Good Fair Bad	Appearance
Turbidity	less than 1 unit 1 - 5	Good Fair	
Electrical Conductivity	0 - 750 um/cm 750 -3500 um/cm	Good Fair	For most industries
Chloride	0 - 50 50 - 250 above 250	Good Fair Bad	Salty taste
Alkalinity	does not include hardness by more than 35	Fair	
Carbonate Alkalinity	120	Fair	
Nitrates	0 to 10 10 to 30 above 30	Good Fair	Danger to Babies
Nitrites	0 - 0.1	Fair	
Fluoride	0 to 0.6 0.6 to 1.5 above 1.5	Good Fair Bad	
Total Solids	0- 500 500-2000 over 2000	Good Fair Bad	Taste, industrial problems
Hardness	above 600	Fair	
Iron (Fe)	0 to 0.3 0.3 to 1.0 over 1.0	Good Fair Bad	
Sulphates	0 to 200 200 to 400 above 400	Good Fair Bad	
Calcium/Magnesium	0 - 100	Good	
Copper	0 to 0.05 0.05 to 1.5	Good Fair	
Zinc	0 to 5 5 to 15	Good Fair	
Arsenic	0 to 0.05	Good	
Lead	0 to 0.05 0.05 to 0.1	Good Fair	
Sodium	above 100	Bad	For sodium-free diet
Silica	20 to 30	High	
ABS	0 to 0.2 0.2 to 1.0	Good Fair	
Phenols	0 to 0.001 above 0.001	Good Bad	
BOD	up to 250 250 25000	Good Bad	

Turbidity –

Water is turbid when it carries in suspension so many finely divided particles of clay, loam, sand or occasionally micro organisms that it presents a muddy appearance. There is evidence that freedom from disease organisms is associated with freedom from turbidity. Turbidity may cause gastro-intestinal irritation, but is primarily objectionable from a standpoint of appearance.

Turbidity is measured photometrically by determining the percentage of light of a given intensity that is either absorbed or scattered. Most turbidity meters work on the scattering principle, although turbidity caused by dark substances that absorb rather than reflect light should be measured by the absorption technique. Formazin is the standard reference chemical with meter readings expressed as formazin turbidity units (FTUs). The term nephelometry turbidity units (NTUs) is used to indicate that the test was carried out on the scattering principle. Turbidity of a good water supply should not exceed 5 and it is preferable to maintain it at 1 or less. At 10, slight cloudiness can be observed in a glass tumbler. At 500, water is practically opaque.

Conductance –

The specific conductance of a water is a measure of the ability of the water to conduct an electric current. It provides an accurate measurement of stream purity; pure water is highly resistant to the passage of an electric current and therefore has a low specific resistance.

Chlorides –

Chlorides are combinations of chlorine with other elements and are usually found in natural waters. Where the normal chloride content of water is known, the determination of chlorides can be of value in judging the sanitary quality of water. Chlorides in excess of the normal content may be considered an indication of contamination by sewage, since chloride is a constituent of urine, or intrusion of salt water if the supply is located near the coast. Chlorides increase the corrosive character of water.

Where contamination is suspected, the source of chlorides may be due to sewage, manufacturing wastes, mineral deposits, oil fuel wastes or salt water. The chloride upper limit of 250 mg/L is not set for health reasons, and water containing greater concentrations can be used without harm.

Chlorides will impart a salty or brackish taste to water which will be noticeable around 250 mg/L. At 500 to 1,000 mg/L content, water will be unpalatable. Water containing more than 4,000 mg/L is unsafe. Chloride tests are not essential in determining safe water supplies, but may be used as a general indication and guide.

-20-

Alkalinity -

Water can be acid, alkaline or neutral. Alkalinity is usually caused by the presence of alkaline salts in water. Various degrees of alkalinity are typical with different forms of the salts.

o   Bicarbonate alkalinity is derived from bicarbonates of calcium and magnesium.

o   Carbonate alkalinity is derived from carbonates of sodium, potassium, calcium and magnesium.

o   Hydroxide or caustic alkalinity is derived from hydroxides of sodium, potassium, calcium and magnesium.

Although some of these compounds can cause hardness, the total amounts of alkalinity and hardness are the same only when calcium and magnesium carbonates are present alone. A criterion for minimum and maximum alkalinity in a public water supply is related to the relative amounts of bicarbonate, carbonate and hydroxide alkalinity. Generally alkalinity should not be less than 30 mg/L nor higher than 400 mg/L.

Ammonia (N) -

Albuminoid Nitrogen (called albuminoid ammonia) is nitrogen present in organic matter before any decomposition occurs. The formation of ammonia nitrogen or free nitrogen marks the first step in the decomposition of organic matter. Ammonia combines with oxygen to form nitrates ultimately.

Nitrites - Nitrates -

Nitrates in water should be regarded with suspicion, as they may be due to pollution by organic wastes or due to mineral deposits. The Nitrite ion is the substance responsible for causing methemoglobinemia, a blood disease in babies. Having considered this aspect, nitrates over 10 mg/L may be safe providing they are bacteriologically safe.

Spot tests for nitrates show whether or not there are nitrates in the sample and give only a rough approximation of the content. The purpose of this test, which is very simple to perform, is to indicate the presence or absence of nitrates, thus indicating whether further tests should be carried out.

-21-

The process of forming nitrates is briefly as follows:

Organic matter — (albuminoid nitrogen, albuminoid ammonia) decomposes to ammonia nitrogen (free ammonia, free nitrogen) then decomposes to unstable nitrites (characteristic of fresh sewage) then oxidizes to nitrates — a relatively stable form.

The relative amounts of various nitrogen compounds rather than the actual amounts indicate the condition of the contaminating material. When ammonia nitrogen is greater than the albuminoid nitrogen, it is a fairly definite indication that the water has been contaminated by sewage and that decomposition has already commenced. If the organic matter in the water is old, nitrites will have been changed to nitrates. It is seen therefore that where nitrites are found continuously, it may be concluded that there is continuous contamination in close proximity to the sampling point.

Additional tests may be required to give further specific information for specific problems.

Fluoride —

Fluoride occurs naturally in groundwaters in certain areas of Sri Lanka. The element fluorine may be found in volcanic gases, fluorite or fluorspar in sedimentary rocks, or triolite in igneous rocks. Disfigurement or mottling of teeth occurs when the fluoride content of a potable water exceeds approximately 1.5 mg/L and becomes pronounced when the content exceeds 3 to 6 mg/L.

However, fluoride in the range of 0.6 to 1.5 mg/L is beneficial in prevention of tooth decay in children, and is often added to water supplies when naturally absent.

Total Solids — Fixed Solids — Dissolved Solids —

Total solids in water is the weight of all mineral and organic matter in the water — either in solution or suspension. It is not a measure of any particular ingredient but rather the sum of all the various elements and compounds dissolved and suspended in the water. It includes, therefore, iron and other chemical constituents.

The SLSI recommends the total solids content should not exceed 2,000 mg/L and preferably should be below 500 mg/L.

The suitability of a water for drinking purposes is indicated in a general way by the solids content. High solids content indicates that a water is highly contaminated or contains excessive amounts of mineral matter.

Filtering a sample of water will remove all suspended solids and a sample that is evaporated will leave behind all total solids. The difference between total solids and suspended solids is the dissolved solids. If the total solids are burned and the resulting ash weighed, this is an indication of the fixed solids.

Hardness -

Evidence of hardness in water can easily be detected in the field. Water is hard if it requires much soap to produce a lather and soft if it lathers freely. Hardness is due to salts of calcium and magnesium, and is expressed in mg/L of $CaCO_3$. Hardness is not dangerous to health.

The following are examples of mineral compounds related to hardness in water:

o   Calcium bicarbonate - solution of $CaCO_3$ in waters containing CO_2.

o   Magnesium bicarbonate - together with above are the main constituents of temporary or carbonate hardness.

o   Calcium sulphate.

o   Magnesium sulphate - permanent hardness or non-carbonate hardness.

Also present sometimes are calcium and magnesium chloride, and calcium and magnesium nitrate.

The objections to hardness are:

o   It wastes soap and affects the skin.

o   Soaps and salt form a precipitate or curd which clings to fabrics in laundry work and textile manufacturing.

o   Hardness modifies colours in dying work.

o Carbonate hardness forms boiler scale when heated that adheres to the boiler shell reducing the capacity of the boiler and causes tubes to leak or burn out.

o Calcium deposits choke water heaters.

Since hardness may be due to several different compounds, results are normally expressed so that we can see at a glance the relationship between different waters. Hardness ratings are:

        0 - 50 mg/L - soft
       50 - 100   - moderately soft
      100 - 200   - moderaly hard
      200 - 400   - hard
      400 + over   - very hard

Water can contain up to 800 mg/L and be satisfactory, though over 200 mg/L softening may be considered. Some wells, however, containing 5,000 mg/L hardness have been used without harm.

Iron and Manganese -

Iron, although it has no bearing on health, is important with respect to appearance, palatability and suitability for domestic and industrial use. It stains glass, porcelain fixtures, clothes when laundered, and unites with tanic acid in tea and coffee changing their flavour and appearance. It may impart an inky or metallic taste to water. Water which contains excessive amounts of iron will develop a brown precipitate or fine brown suspension due to iron oxide (iron rust) following exposure to air. An iron content exceeding 0.05 mg/L may develop some odour, stain fixtures or form precipitates.

The SLSI set limits of iron content at 0.3 mg/L. . This limit was not set for reasons of health but rather taste, odour, laundering, etc. Condemning a water because of high iron content is therefore not justified for health reasons although a water with higher iron content is not readily accepted because of a reddish appearance and staining characteristics.

Manganese in concentrations of a few hundredths milligrams per litre, will cause a build up of coatings in distribution piping which slough off. It can cause laundry staining or formation of precipitates.

Sulphates

Sulphate content is not too significant when assessing the potability of a drinking water.

Excessive sulphates may:

o   Cause bad taste
o   Have a laxative effect when used for the first time.

The SLSI specifies that a potable water should not exceed 400 mg/L sulphates. Elsewhere, water with sulphates up to 6,000 mg/L is being consumed without any apparent harm.

Calcium and Magnesium -

Calcium and magnesium are the constituents which produce hardness in water. They waste soap, affect skin, are less satisfactory for cooking and washing and form scales in boilers and pipes. Calcium and magnesium bicarbonates are the main constituents of temporary hard water. The sulphates and chlorides constitute permanent hardness, and removal is more difficult.

Calcium compounds cause hardness, alkalinity and salinity in water and although their presence in most natural waters has no sanitary significance, they may have a detrimental effect on the use of water for domestic and industrial purposes.

Copper -

Except for traces, copper salts do not occur in natural waters. Their presence in potable water is attributable to corrosive action on piping.

Concentrations large enough to be significant from a health standpoint would render water supplies completely unpalatable. However, there is little danger of copper poisoning from this source. Tastes in the water supply will occur with concentrations of 3 to 5 mg/L. Concetrations of 0.5 mg/L or less in some soft waters will cause staining of porcelain.

The SLSI specifies a limit of 1.5 mg/L.

Zinc -

Zinc has been found in concentrations up to 50 mg/L in zinc mining areas. However, zinc content is normally limited to amounts incidental to corrosion of brass, galvanized piping, and those amounts are not of practical significance until the corrosion is sufficient to cause metallic tastes in water. The limit set by the SLSI is 15 mg/L. However 5 mg/L may cause a disagreeable taste and may aid corrosion.

Arsenic –

Arsenic may be present in water from hot springs or in the wastes from some industries. The maximum limit set by the SLSI is 0.05 mg/L.

Lead –

Lead is not present in natural waters and would not be present if not for direct contamination.

Lead contamination results when corrosive waters are allowed to stand for a length of time in lead piping or to come in contact with lead bearing jointing compounds, and the problem is considered limited. Lead poisoning usually results from the cumulative toxic effect of lead after protracted consumption of water containing significant concentrations, rather than from occasional presence of concentrations in excess of 0.1 mg/L.

The limit set by SSI is 0.5 mg/L.

Sodium – Potassium –

Sodium content of water may be deleterious to those suffering from high blood pressure, where 200 mg/L of sodium may be significant.

Sodium and potassium chloride can be tested. Sodium content around 200 mg/L is reasonable except for sodium free diet consumers where content above 100 mg/L is bad.

Silica –

Silica in water can be present in colloidal form or in solution. Its presence has advantages such as assisting coagulation with aluminium sulphate and helping build up a good coating that protects iron from corrosion. Its disadvantage is in high pressure boilers and turbines where it forms a scale which is hard and troublesome.

ABS –

ABS is short for Aklyl Benzene Sulphonate and it is a surface active agent or surfactant, as they are called. They are organic substances which, at usage concentrations, significantly lower the surface tension of water.

The ease with which a surfactant is degraded or discharged by biochemical means is called biodegradability. Those resistant to biodegradability are called hard detergents and conversely those detergents that contain surfactants that are easily degraded are called soft detergents.

When wash waters and waste which contain synthetic
detergents go down the drain they will contain
approximately 0.1% to 0.5% of finished detergent product.
This is 1,000 mg/L to 5,000 mg/L respectively and is
equivalent to 200 mg/L to 1,000 mg/L of either ABS, LAS,
or other surfactant. This LAS is a linear alkylate which
when sulphonated becomes linear alkylate sulphonate hence
LAS.

When ABS goes down the drain into the sewer it becomes
diluted by other wastes 100 times or so, so that raw
sewage contains typically about 10 mg/L ABS.

Traces of ABS which find their way into drinking water,
almost always come from sewage. ABS therefore is an index
of pollution of sewage origin.

While a level of 5 mg/L ABS might be encountered as an
unusually rare circumstance on a temporary basis, ABS
levels in drinking water are almost always below 1.0 mg/L
which is the recommended limit and are usually well under
0.2 mg/L.

Phenols —

Phenols result from oily or tarry wastes from coke ovens
and wood installation plants. The limit of .001 mg/L will
produce tastes with the addition of chlorine.

The amounts of phenols would not be significant if not for
the fact that objectionable tastes and odours result when
phenol bearing wastes are subjected to chlorination.

Biochemical Oxygen Demand (BOD) —

Biodegradeable organic matter in waste discharges is
broken down by aerobic microorganisms to provide food and
energy. The dissolved oxygen used by the microorganisms
in this metabolic process is referred to as the bio-
chemical oxygen demand (BOD).

The test for BOD is an important one for determining the
degree of pollution of streams and the oxygen requirement
of sewage and trade wastes.

The BOD is a measure of the amount of organic matter
present or rather of the oxygen required to oxidize the
organic matter. In raw sewage the five day BOD is
generally above 100 mg/L and for industrial wastes the BOD
may range from 1,000 to 50,000 mg/L.

-27-

Pesticides -

This test should be included where there is the possibility of industrial pollution or agricultural runoff, since pesticides may create health problems. Refer to local WHO or FAO office for guidelines.

Mineral Oils -

May cause tastes and odours after chlorination.

3.5   INTERPRETATION OF BACTERIOLOGICAL ANALYSES

If the water samples taken do not meet the prescribed standard, the source should be considered unsuitable for use without treatment. If any coliform organisms are found, further investigations may be necessary to determine their source, if this is not evident. Disinfected water supplies should be completely free of any coliforms, however polluted the raw water may have been.

4. SELECTION OF TREATMENT PROCESSES

The object of water treatment is to obtain water of approved quality standards. Treatment must be both economical and reliable, requiring the least capital investment and operation and maintenance. The relative merits of sources requiring treatment against the cost of long pipelines from a distant source not receiving treatment should be weighed carefully in view of the constant attention which would be required for the operation and maintenance of a treatment plant.

The methods of treatment depend on the nature of the source and its quality. Treatment may comprise only chlorination for disinfection where raw water quality is within the recommended limits or it may involve certain specific or full treatment including aeration, sedimentation (either plain or flocculation and coagulation), rapid or slow sand filtration followed by chlorination. Ground waters generally require the least amount of treatment, chlorination usually being sufficient. In the case of surface water and iron-bearing ground waters, part or all of the full treatment processes may be required. It is important, therefore, that water quality of the source be thoroughly evaluated to select, first of all, the source which requires the least treatment and secondly, the most economical treatment process (es). Data on design of the various unit processes are given in Chapter 5.

For small community water supplies, complicated treatment schemes are not suited and in such cases a better solution may be to develop an alternative unpolluted source of water, even when this can only be found at a greater distance.

Important design considerations are:

- o Low cost
- o Minimum use of mechanical equipment
- o Avoidance of use of chemicals, when possible
- o Ease of operation and maintenance.

The possible treatment stages including unit processes, chemical dosing points, washwater recovery and sludge disposal are shown in Figure 4.1. The effectiveness of treatment processes in removal of various impurities is shown in Table 4.1.

Table 4.2 presents treatment options in a slightly different format with recommended processes for various levels of undesirable characteristics. Both Tables 4.1 & 4.2 should be used with caution since treatment effectiveness usually depends on a combination of unit processes.

Figure 4.2 shows the approximate operational ranges of treatment processes with respect to removal of particles of different sizes, and Table 4.3 shows unit processes required for various typical types of source waters.

-29-

Table 4.1
**Effectiveness of Water Treatment Processes in
Removing Various Impurities**

Water Quality Characteristic	Aeration	Chemical Coagulation and Flocculation	Sedimentation	Rapid Filtration	Slow Sand Filtration	Chlorination
Dissolved Oxygen Content	+	o	o	—	— —	+
Carbon Dioxide Removal	+	o	o	+	++	+
Turbidity Reduction	o	+++	+	+++	+++	o
Colour Reduction	o	++	+	+	++	++
Taste and Odour Removal	++	+	+	++	++	+
Bacteria Removal	o	+	++	++	++++	++++
Iron and Manganese Removal	++	+	+	++++	++++	o
Organic Matter Removal	+	+	++	+++	++++	+++
Hardness	+	— —	— —	— —	o	o
Corrosiveness	+++	— —	— —	— —	o	o

Legend

+++ increasing positive effect
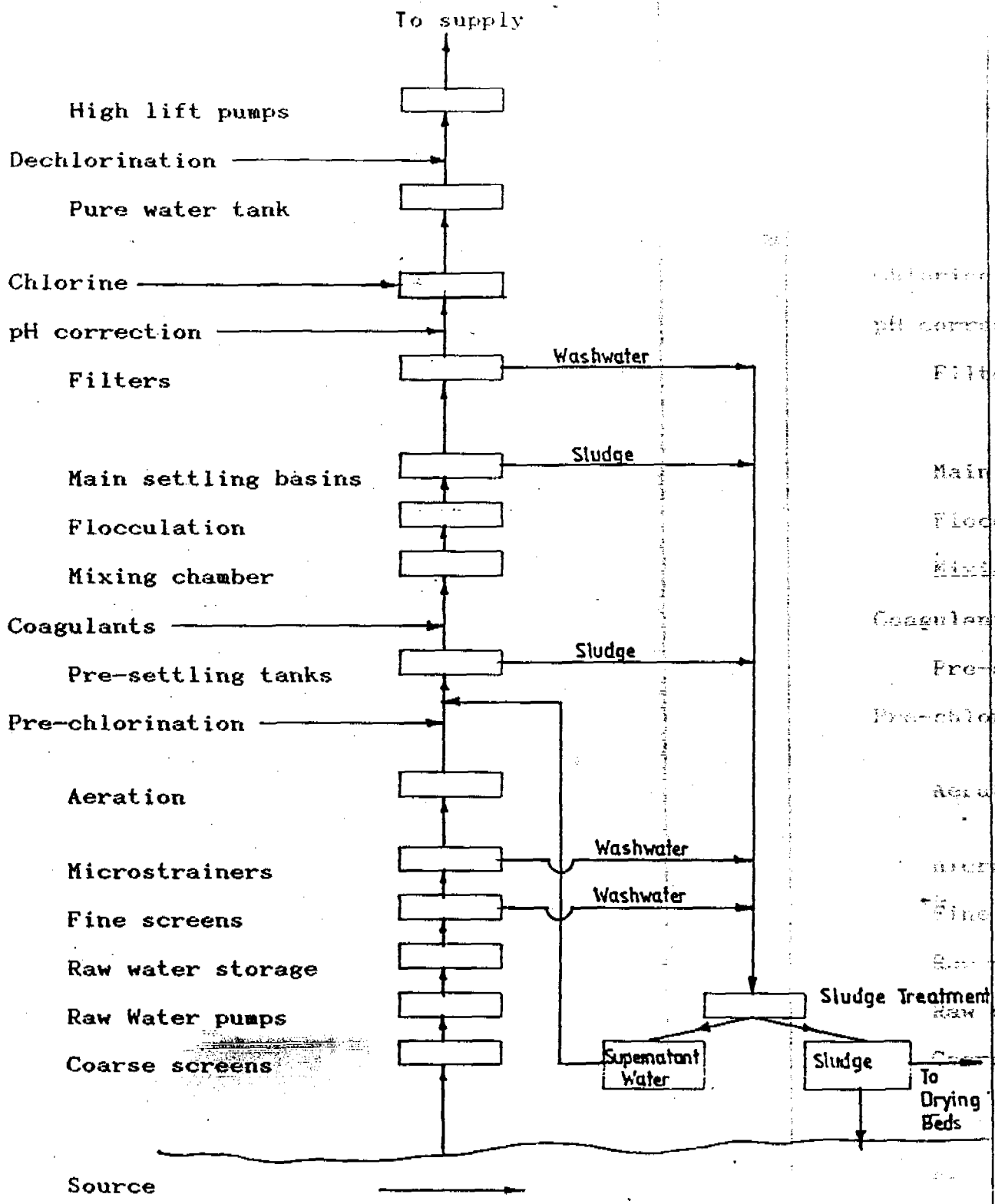o   no effect
—   negative effect

-30-

Table 4.2   Recommended Treatment Processes

Water Quality Characteristic to be Reduced	Raw Water storage	Screening	Micro-strainers	Aeration	Pre-Chlorination	Preliminary Settlement	Flocculation	Coagulants and settling	Rapid filtration	Slow sand filters	Post-chlorination	Superchlorination and dechlorination	Lime (-soda softening)	Activated carbon	Desalting
Floating debris	-	E	R	-	-	-	-	-	-	-	-	-	-	-	-
Algae	-	-	R	-	R	-	-	E	E	-	-	-	-	-	-
Turbidity:															
0- 5 TU		R	-	-	-	-	-	R	R	E	E	-	-	-	-
5- 30 TU		R	-	-	-	-	R	E	E	R	E	-	-	-	-
30- 100 TU		R	-	-	-	-	R	E	E	R	E	-	-	-	-
100 -750 TU		R	-	-	-	R	R	E	E	R	E	-	-	-	-
750-1000 TU	R	R	-	-	-	R	R	E	E	-	E	-	-	-	-
> 1000 TU	R	R	-	-	-	E	R	E	E	-	E	-	-	-	-
Colour:															
< 30 Hazen	-	-	-	-	-	-				-	-	-	-	-	-
> 30 Hazen	-	-	-	-	-	-	E	E	E	-	-	-	-	-	-
Tastes & odours	-	-	-	R	R	-	-	-	-	R	-	R	-	R	-
Calcium carbonate >200 mg/L	-	-	-	-	-	-	-	-	-	-	-	-	E	-	-
Iron and manganese:															
<0.3 mg/L	-	-	-	R	R	-	-	-	-	-	-	-	-	-	-
0.3-1 mg/L	-	-	-	R	-	-	-	E	E	R	-	-	-	-	-
> 1 mg/L	-	-	-	E	E	-	-	E	E	R	-	-	-	-	-
Chlorides:															
0-500 mg/L	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
> 500 mg/L	-	-	-	-	-	-	-	-	-	-	-	-	-	-	E
Coliform bacteria, MPN per 100 mL:															
0- 20	-	-	-	-	-	-	-	-	-	R	E	-	-	-	-
20- 100	-	-	-	-	-	-	-	E	E	R	E	-	-	-	-
100-5000	-	-	-	-	R	-	-	E	E	R	E	-	-	-	-
> 5000	R	-	-	-	E	R	-	E	E	R	E	-	-	-	-
Toxic Chemicals								E	E						

Legend   E = Essential,   R = Recommended

Table 4.3 Treatment Systems for Water from Various Sources

Source	Aeration for Increasing O_2	Aeration for Reducing CO_2	Pre-Chlorination	Infiltration Gallery	Plain Sedimentation/ Roughing Filter	Coagulation and Flocculation	Sedimentation	Rapid Filtration	Slow Sand Filtration	Post Chlorination
Groundwater										
Aerobic, not corrosive										▭→
Aerobic, corrosive		▭								▭→
Anaerobic, not corrosive; no Fe and Mn	▭									▭→
Anaerobic, not corrosive, with Fe and Mn	▭				▭ (BYPASS OPTIONS)				▭ (bypass)	▭→
Anaerobic, corrosive no Fe and Mn		▭			▭ (BYPASS OPTIONS)				▭ (bypass)	▭→
Anaerobic, corrosive with Fe & Mn		▭				▭	▭	▭		▭→
Surface Waters										
Protected hill country streams Turbidity <10										▭→
Streams, slightly polluted turbidity occasionally >10					▭ (BYPASS OPTIONS)				▭	▭→
				▭						▭→
Streams polluted, frequent high turbidity and colour						▭ (BYPASS OPTIONS)	▭	▭		▭→
Tanks or lakes, canals slightly polluted, low turbidity and algae					▭ (BYPASS OPTIONS)				▭	▭→
			▭							▭→
Tanks or lakes, canals, lowland rivers, polluted high turbidity and algae tastes & odours	▭ (BYPASS OPTIONS)		▭ (BYPASS OPTIONS)		▭	▭	▭	▭		▭→

To supply

High lift pumps

Dechlorination ──────────→

Pure water tank

Chlorine ──────────────→

pH correction ────────→

    Filters ──────────────── Washwater ────→

Main settling basins ──── Sludge ──────→

    Flocculation

    Mixing chamber

Coagulants ─────────────→

Pre-settling tanks ──── Sludge ──────→

Pre-chlorination ──────→

    Aeration

Microstrainers ──── Washwater ────→

Fine screens ──── Washwater ────→

Raw water storage

Raw Water pumps   Sludge Treatment

Coarse screens   Supernatant Water   Sludge   To Drying Beds

Source ──────────→

Flow Diagram Showing Possible Treatment Stages

Source: Smethurst,G., 1979 "Basic water treatment for application worldwide". Thomas Telford, London.

Figure 4.1

APPROXIMATE OPERATIONAL RANGES FOR TREATMENT PROCESSES
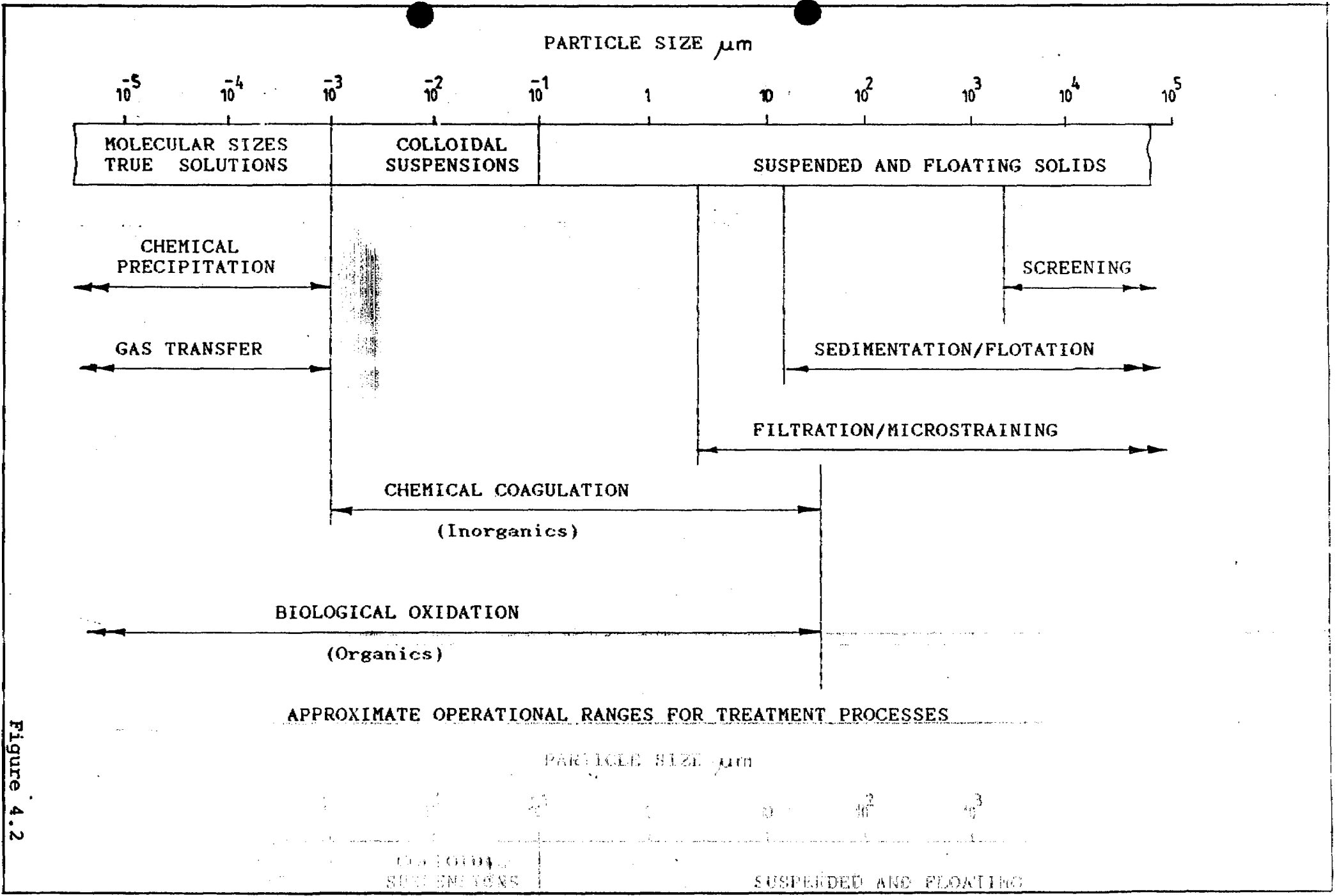
Figure 4.2

5.   UNIT PROCESSES

5.1   AERATION

Aeration may be used on surface waters to help remove
tastes and odours due to dissolved gases, or on ground
water supplies to reduce or remove objectionable amounts
of carbon dioxide, hydrogen sulphide or methane and to
introduce oxygen to assist in iron and manganese removal.
In surface waters with a high organic content (such as
Kalatuwawa) aeration may also assist in colour removal.

There has in the past been over-use of aeration in surface
water schemes in Sri Lanka. Where aeration is not
required for the above purposes, it adds to the cost of
pumping (usually 1-3 m head) and increases the oxygen
content making the water more corrosive. This adds to the
cost of treatment and makes for a more aggressive water
in the distribution system. Aeration should not,
therefore, be used unless required for:

o   addition of oxygen to anaerobic waters;

o   precipitation of iron and manganese in solution; or

o   removal of carbon dioxide, hydrogen sulphide or other
    soluble organic and inorganic substances causing
    tastes, odours and sometimes colour.

Natural surface waters often contain dissolved complex
organic and inorganic substances which volatilise on
aeration and some oxidation of dissolved organic matter
also occurs. This exchange has the effect of removing or
reducing obnoxious tastes and odours and, with it, the
ultimate chlorine demand of the natural water. However,
aeration may not remove all tastes and odours caused by
less volatile substances.

Dissolved ferrous and manganous compounds are oxidised by
aeration, and precipitated as insoluble ferric and
manganic oxide hydrates which may then be removed by
sedimentation or filtration. This process is likely to be
less effective in the presence of organic matter, which
may combine with the iron and manganese to form complex
compounds which do not precipitate well. In such cases,
moderate rather than vigorous aeration may work better.

Groundwaters will release a part of any dissolved gases
and aerate with atmospheric oxygen thus acquiring a more
desirable taste. The release of carbon dioxide increases
the pH and reduces the corrosivity of the water.

Reducing the carbon dioxide content, however, may shift
the carbonate-bicarbonate equilibrium in the water so that
deposits of calcium carbonate are formed which may cause
problems.

The rate of aeration is governed by the area of interface between the air and liquid, the thickness of the interlayers and the time of contact.

The type of aerators commonly used are:

Spray Aerators

These consist of stationery nozzles, through which water is sprayed into the air, at about 5-7m/sec. The nozzle design is important in achieving optimum dispersion. Spray aerators are appealing, but are no more efficient than other types which require less space.

Typical design criteria are:

Nozzle dia: 10-40 mm @ 0.5 - 3.0 m spacing

Nozzles to be tilted $3^\circ - 5^\circ$ to the vertical to avoid interference due to falling water. They should be enclosed by suitable windshields to prevent loss of water;

Output per nozzle: 5-10L/s

Head required at nozzle: 2-9 m of water

Velocity of water in the aerator pipe: 1.0-1.5 m/sec.

Aerator area: $0.72 - 2.16$ m^2/m^3/day of flow.

The time of exposure of the droplets, the head required and the flow from each nozzle can be calculated from the following formulae:

$$V = C_v \sqrt{2\,gh}$$

$$q = C_d\, a \sqrt{2\,gh}$$

$$t = 2\,C_v \sin x\ \frac{2h}{g}$$

h = total head of water at nozzle
g = acceleration due to gravity
V = initial velocity of drop emerging from nozzle
C = coefficient of velocity (0.9)
C = coefficient of discharge (0.9)
q = discharge per nozzle
a = cross section area of nozzle opening
t = time of travel or exposure
x = angle of inclination of spray to horizontal

Multiple Tray Aerators

These are simple, inexpensive and occupy little space (See Fig 5.1(a). Water is dispersed evenly through perforated pipes over the upper tray from where it trickles down.

Typical design criteria are:

    Number of perforated trays = 3-5
    Tray perforations = 10 mm dia. @ 50 mm centres.
    Spacing between trays = 30-50 cm
    Height of aerator   = 2-3   m
    Size of media in trays = 4-7   cm
    Depth of media   = 10-25 cm
    Flow rate   = 0.01-0.02 m^3/s per m^2 of
                           tray surface
    Tray area   = 1.0-2.0 $m^2/m^3/min.$
                        (34-68 ft^2/mgd)

Trays can be of any suitable non-corrosive material, and for better dispersion may be filled with coarse gravel. Coke may also be used, which acts as a catalyst to promote the precipitation of iron from the water.
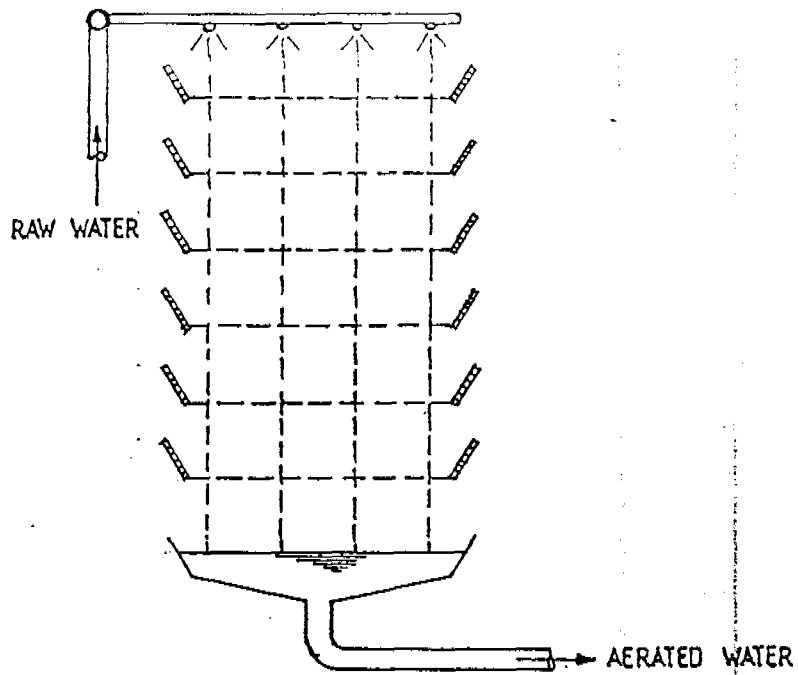
Cascade Aerators

The principle is to spread the water as much as possible and let it flow over obstructions to produce turbulence. These are similar to tray aerators, but with a series of steps or platforms over which the water cascades (see Fig.5.1(b),(c)). Obstacles may be placed on the edge of each step. Cascades aerators generally take up more space than tray aerators, but the overall head loss is lower, and maintenance is minimal. Where space permits, therefore, they are the preferred type of aerator.
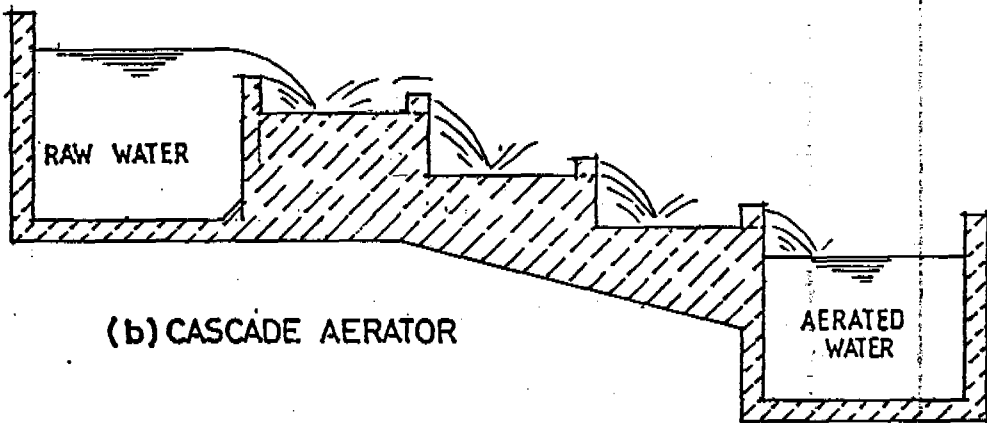
Typical design criteria are:

    Number of drops   = 4-6
    Height of drops   = 30-60 cm
    Overflow rate   = 0.01 m^3/s over m width of step.
    Height of aerator = 2-3 m
    Cascade area   = 1.5-2.0 $m^2/m^3/min$ of flow
                (51-68 ft^2/mgd)

An example of cascade aerator design is given in Annex A.

(a) MULTIPLE TRAY AERATOR

RAW WATER

AERATED WATER

(b) CASCADE AERATOR

RAW WATER

AERATED WATER

PLATFORMS

WATER BASIN

PLAN

VIEW

SECTION
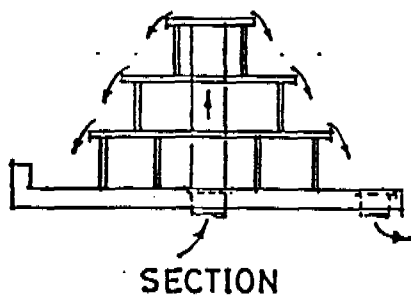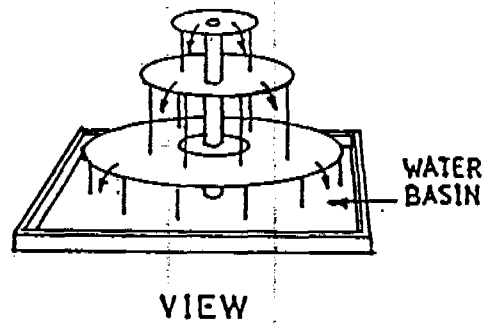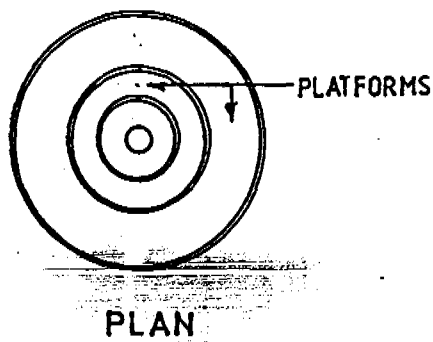
(c) MULTIPLE PLATFORM AERATOR

AERATORS
FIGURE 5.1

5.2   SCREENING

For river and lake intakes some form of protective boom or coarse screen, about 75 mm opening, should be used to prevent large floating objects entering the intake. The velocity of water through the screen opening should not exceed 0.5 m/s.

Fine screens are usually 5-20 mm opening, duplicate screens being provided to allow the removal of one screen for cleaning manually. Alternatively, provision may be made for cleaning by raking or water jetting. Maximum velocity through the screening medium should not exceed 1 m/s. Mechanical screens may be necessary on larger schemes, operated continuously or intermittently as controlled by the loss of head across the unit. For a belt type screen 15 mm/s would be a typical belt speed for continuous operation and drum and disc type units may rotate continuously at 0.2-2 rev/min.

The microstrainer is a development of the drum screen using a fine stainless steel mesh supported on a coarser material. Because of the small aperture size of the fabric, clogging occurs rapidly and the fabric must be continually cleaned by washwater jets. The speed of rotation of the drum depends on the diameter but in general the maximum peripheral speed should not exceed 0.5 m/s. Filtering head is normally 40-140 mm and maximum operating head should not exceed 0.25 m. Three types of fabric are in general use: Mark 0 23 micron aperture, Mark I 35 micron aperture and Mark II 65 micron aperture.

Straining rates are usually in the range 700-2300 m^3/m^2d and backwashing requires about 2% of the influent water. Typical sizes and capacities are shown below:

MACHINE SIZE		MOTOR (kW)	CAPACITY RANGE (m^3/d)
DIA (m)	EFFECTIVE WIDTH (m)		
1.5	0.15 or 0.3	0.4	230-2300
1.5	0.9	0.6	1360-6800
2.2	1.5	1.5	3620-18100
3.0	3.0	3.0	13600-45360
3.0	3.0	4.0	23000-68000

5.3 COAGULATION

Colloidal particles and fine suspended material cannot be removed by plain sedimentation because of their extremely low settling velocities. It is therefore not normally possible to remove particles of normal specific gravity with diameters less than 50 micron by sedimentation in conventional tanks. Suitable chemicals can be added to water to form bulky floc particles which will settle readily removing most of the colloidal and suspended constituents of the water.

The most commonly used chemical for coagulation of water is aluminium sulphate (alum) $Al_2(SO_4)_3.14H_2O$ but the following chemicals may sometimes be used:

Ferrous sulphate (copperas) $FeSO_4.7H_2O$

Chlorinated copperas $FeSO_4.7H_2O + Cl_2$

Ferric sulphate $Fe_2(SO_4)_3$

Ferric chloride $FeCl_3$

The substances that frequently are to be removed by coagulation and flocculation are those that cause turbidity and colour. Surface waters in Sri Lanka are often turbid and contain colouring material. Turbidity may result from soil erosion, algal growth or animal debris carried by surface runoff. Colour may be imparted by substances leached from decomposed organic matter, leaves, or soil such as peat. Both turbidity and colour are mostly present as colloidal particles.

Generally, water treatment processes involving the use of chemicals are not so suitable for small community water supplies and should be avoided whenever possible. Chemical coagulation and flocculation should only be used when the required water quality cannot be achieved with another treatment process using no chemicals. If the turbidity and colour of the raw water are not much higher than is permissible for drinking water, it should be possible to avoid chemical coagulation in the treatment of the water. A process such as slow sand filtration would serve both to reduce the turbidity and colour to acceptable levels, and to improve the other water quality characteristics, in a single unit. A roughing filter can serve to reduce the turbidity load on the slow sand filter, if necessary.

Alum is by far the most widely used coagulant but iron salts have advantages over alum in some circumstances. One advantage is the broader pH range for good coagulation. (For alum the pH zone for optimum coagulation is quite narrow, ranging from about 6.5 to 7.5 whereas for ferric sulphate it is considerably broader, about 5.5 to 9.0). Thus, in the treatment of soft coloured waters where colour removal is best obtained at low pH, iron salts may be preferred as coagulants. Iron salts should also be considered for coagulation at high pH since ferric hydroxide is highly insoluble in contrast to aluminium salts which form soluble aluminate irons at high pH. Synthetic organic polyelectrolytes have become available as coagulants and coagulants aids, but are generally not economical for small water supply systems, nor are they readily available in Sri Lanka.

The coagulant must be added in the correct dose and mixed thoroughly, for optimal coagulation. The optimal dose should be determined periodically by carrying out a jar test.
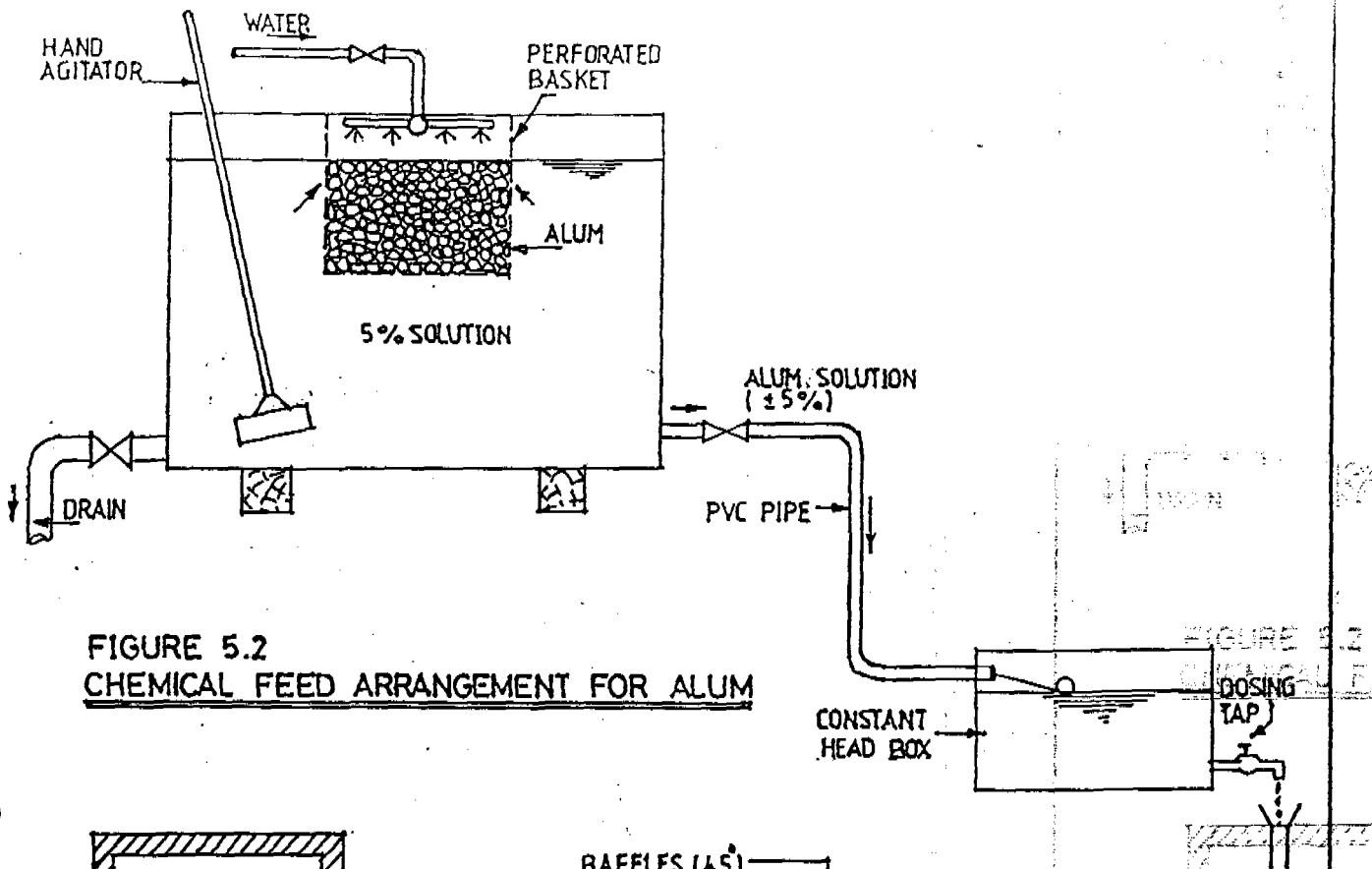
Dosing

Chemicals are normally added to water as a solution or suspension so that rapid dispersion throughout the main body of water becomes possible. For accurate continuous dosing the chemical must be carefully measured and this may be done by either dry or solution feeders.

Dry feeders draw finely ground chemical from agitated storage hoppers, measurement being gravimetric or volumetric. The measured chemical is usually delivered to a solution pot for injection into the main supply as a solution or a slurry.
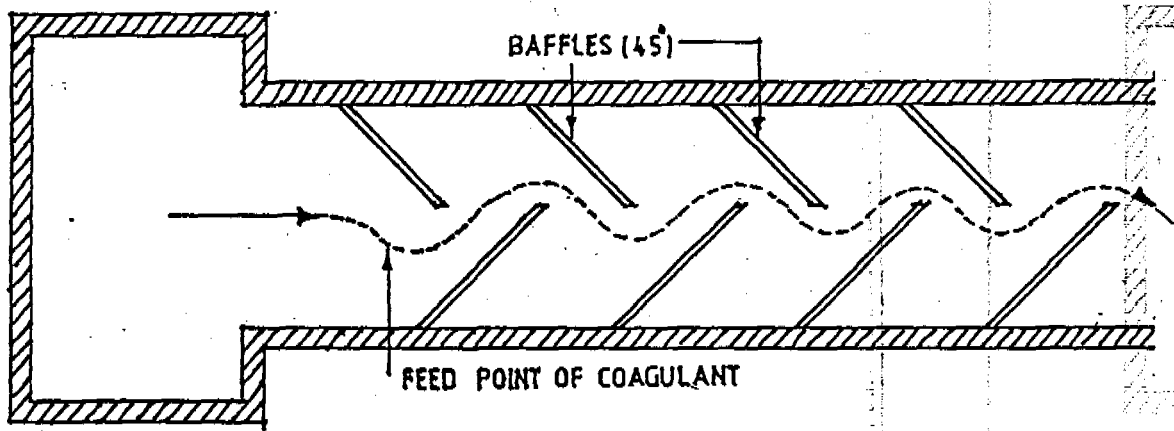
With solution feeders the chemical is dissolved in large storage vats holding 12-24 hours supply of solution with stirrers to a standard concentration (usually 3-7%) which is then added via a constant head orifice or similar measuring device.

In either case duplicate feed capacity must be provided in case of breakdown and there should be at least one month's chemical storage capacity adjacent to the feeders. Since most of the concentrated chemical solutions are highly corrosive, solution lines should be of rubber or plastic and tanks should be lined with a plastic coating. A typical arrangement is shown in Figure 5.2.

Problems have been reported with lime and alum dosing equipment in treatment plants in Sri Lanka, from which the following conclusions may be drawn:

-41-

HAND AGITATOR
WATER
PERFORATED BASKET
5% SOLUTION
ALUM
DRAIN
ALUM SOLUTION (±5%)
PVC PIPE
CONSTANT HEAD BOX
DOSING TAP

FIGURE 5.2
CHEMICAL FEED ARRANGEMENT FOR ALUM

BAFFLES (45°)
FEED POINT OF COAGULANT

BAFFLED CHANNEL FOR RAPID MIXING.

FEED POINT OF COAGULANT
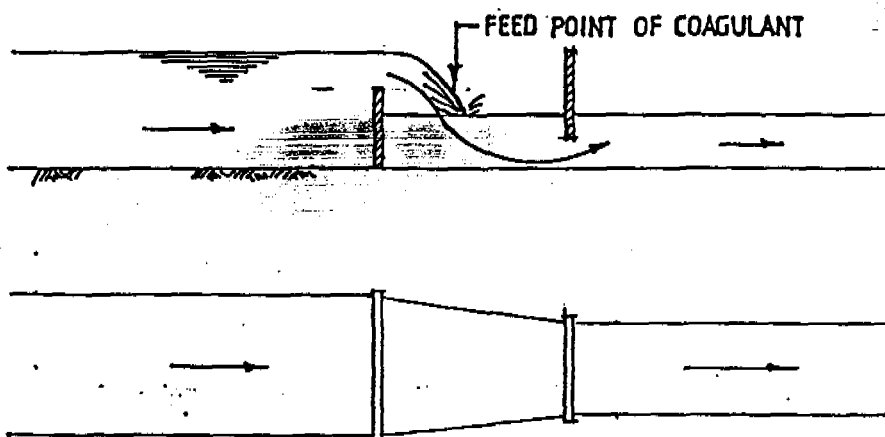
NOTE: THE WEIR CAN ALSO BE USED
TO MEASURE THE FLOW

COAGULANT MIXING
FIGURE 5.3

o   proportional feed dosing pumps have been commonly
     used, but are unreliable, and a gravity feed system
     through pipes of steady gradient should be utilized
     wherever possible;

o   the amount and concentration of the solution must be
     known for optimum coagulation and economy of coagulant
     use;

o   solution tanks are sometimes some distance from the
     point of chemical dosing, requiring long feeding
     pipes.   Solution preparation, dosing and flocculation
     should be as close together as possible.

Coagulant Dispersion

The initial dispersion of the coagulant in the raw water
is of utmost importance, and to ensure even dispersion of
the chemical throughout the water some form of mixing is
essential.   The desired degree of mixing may be
accomplished either by mechanical means or by utilising
hydraulic turbulence.

Hydraulic methods include overflow weirs and baffled
channels as shown in Figure 5.3 or in the discharge piping
of a pump.   5-10 sec. mixing time should be adequate, if
it is really vigorous and continuous.

If a hydraulic jump is used for mixing, the velocity of
flow in the flume should be 3-4 m/s and the water leaving
the flume should mix with water moving at about 0.7 m/s,
the total mixing time being about 5 sec. The head loss
for such mixing is 0.25-0.50 m.

Mechanical rapid mixing may be achieved by high speed
(300-900 rev/min) propellor type mixers installed in a
small chamber; criteria to follow are:

     Detention time of at least 30 s (commonly 60 s)
     Power requirement 2.5-5.0 kW/m^3
     Head loss 0.2-0.6 m
     Ratio of tank dia to height 1-3

However, experience indicates that mechanical rapid mixing
is not as effective as well designed hydraulic mixing, and
Parshall flumes or weirs are preferred.

Dilution of the coagulant, to a solution of 0.5 to 1.0%
should be carried out immediately prior to dosing the raw
water.   The dilution water should be filtered, but without
pH adjustment (before lime addition). It must be possible
to measure the volumes of both dilution water and
coagulant.   A suitable and simple type of diffuser system
is shown in Figure 5.4.   The feeding point should be
across the flow at the point of maximum agitation.

-43-
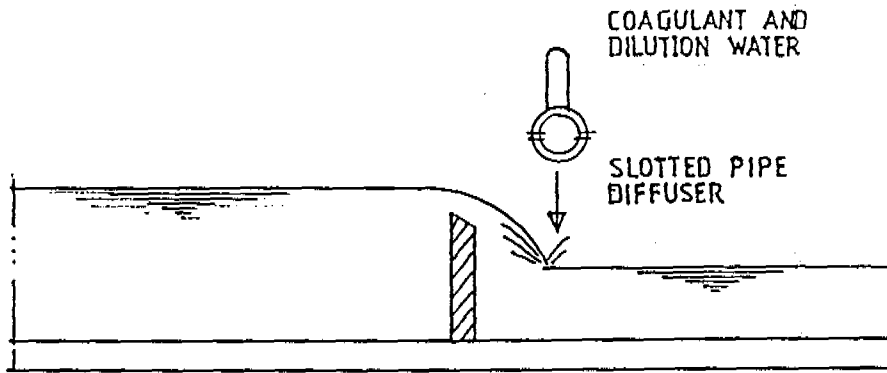
WEIR AND COAGULANT APPLICATION POINT
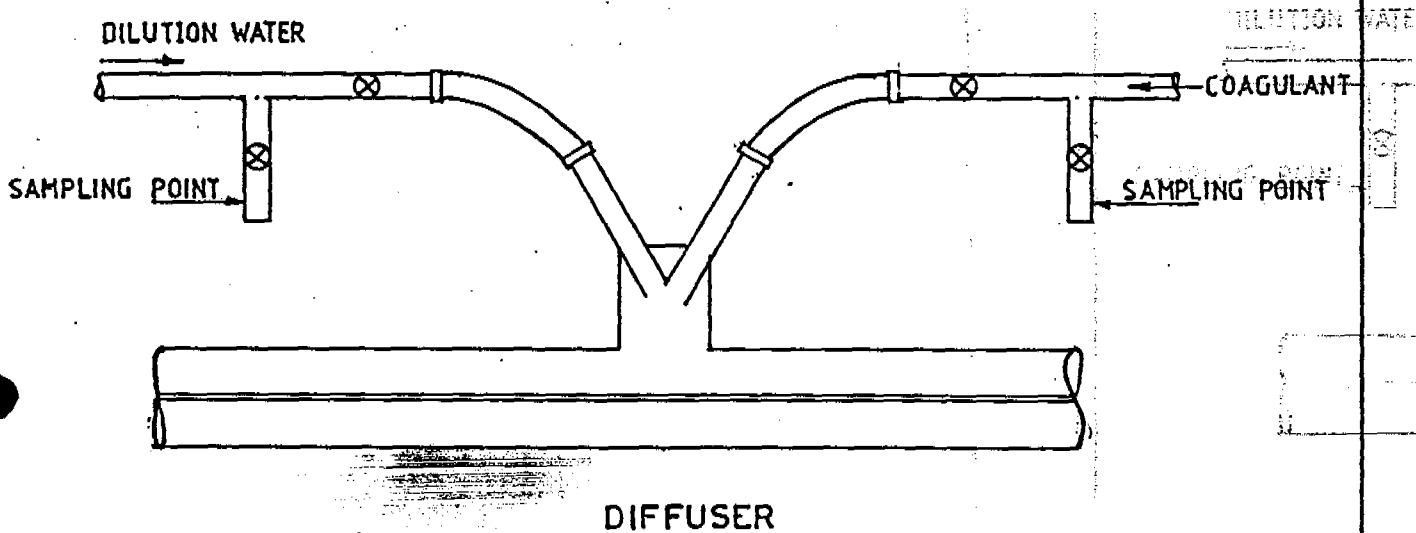
COAGULANT DIFFUSER WITH DILUTION LINE AND
SAMPLING TAP

FIGURE 5.4

Flocculation

The size of floc particles and their speed of formation can be improved by creating velocity gradients in the water to increase the chance of collision between particles. Such conditions can be provided by rectangular horizontal flow tanks with slowly rotating paddles or with baffles, or by vertical flow tanks with or without mechanical stirring.

a) **Mechanical Flocculators**

Flocculators are generally of mechanical type in larger plants and hydraulic or gravitional type in smaller plants. For mechanical type, use the following typical criteria:

Detention period	= 20 to 30 minutes
Avg. velocity of flow	= 0.3 m/s
Paddle area	= 10 to 25% of the area swept
Speed	= 2 rpm
Max. peripheral velocity	= 0.15-.0.75 m/s
Head loss	= 0.15 m
Depth of tank	= 1½-2 paddle diameters
Power requirement	= 2.5-5.0 $kW/m^3/s$. (design for starting torque)

Operate as many paddles as possible from one shaft and motor. Many methods of coagulation and flocculation can be effective as long as the concept of plug flow is maintained rather than uncontrolled or undirected flow through large open basins. The very simplest mechanically-driven flocculators, either horizontal or vertical, can be effective if the tank is divided into sections or cells requiring a series flow pattern. Even if not provided in the original design, this can often still be accomplished later by installing baffles in existing tanks to create cells or compartments.

b) **Solid Contact/Upflow Sludge Blanket Clarifiers**

Most existing plants in Sri Lanka use a combination of flocculation and settling within a single structure in upflow sludge blanket type clarifiers. (see Fig. 5.5.) These units operate best under heavy turbidity conditions so that a dense sludge blanket can be formed; this is not the case in most of the plants. Furthermore, these basins do not take overloads well and sharp changes in flow adversely affect their performance. The sludge rises and overflows at the slightest provocation.

-45-

Most unreliable of the existing units are the single compartment flocculators which allow short circuiting and short flocculation times. Since slow agitation is critical to the formation of settleable floc this unit is very important. The time of agitation and amount of energy input are important parameters. In upflow basins neither is well controlled.

For the most part the loading of the basins is at the limit of reasonable overflow rates. Add to this the fact that flocculation times are close to or below reasonable minimums, and it becomes clear that these clarifier-flocculator units are not easily modified and upgraded.
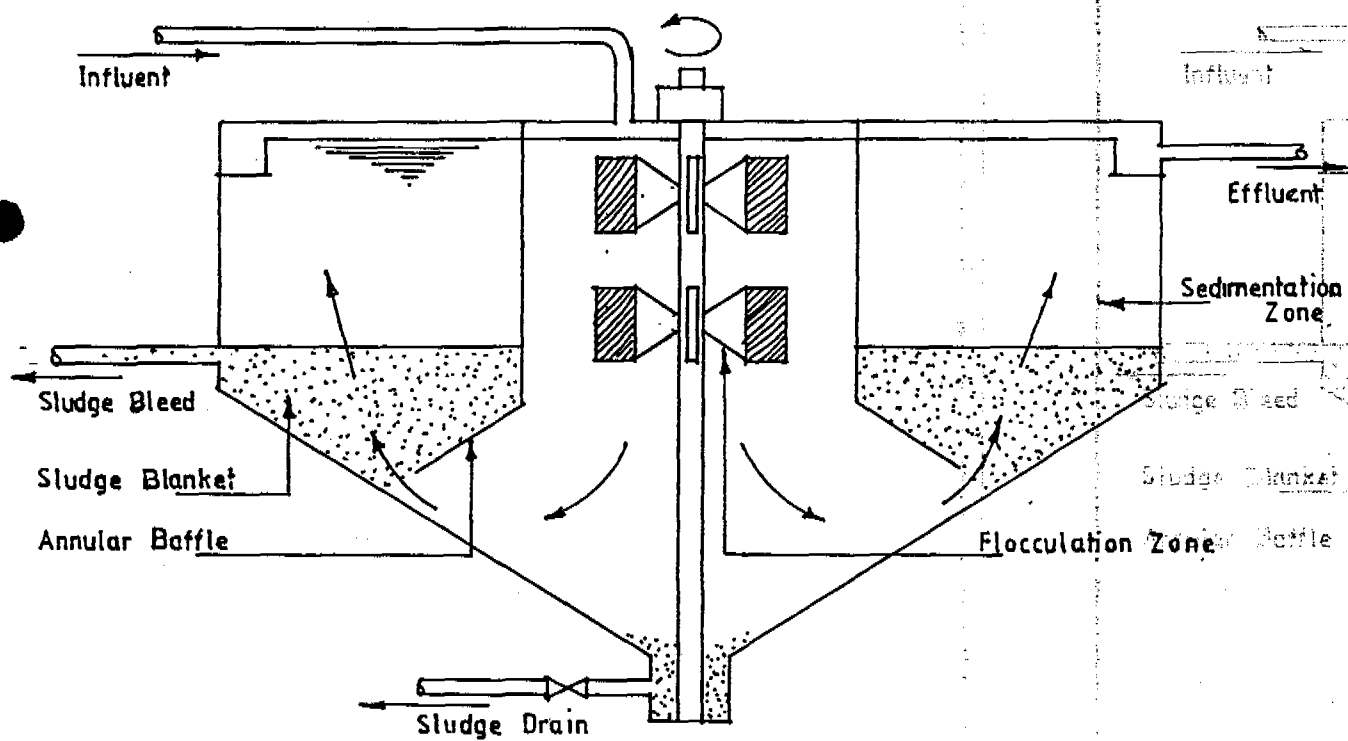
If more clarification capacity is needed in the future at these plants, new flocculation and settling basins will be required. There is, however, an alternative solution: a light coagulant dosage and direct filtration can probably produce excellent water quality for most of the year and possibly even cope with seasonally high turbidity waters (during the rainy season) by more frequent filter washing.

c) Hydraulic Flocculators

Baffled or hydraulic flocculation basins are simple, economical alternatives to mechanical flocculators. They have the distinct advantage of being free from mechanical equipment, O&M problems as well as short-circuiting. Their disadvantages are that mixing intensity is dependent on flow rate and that they require more area.

The velocity gradient G (sec^{-1}) for hydraulic flocculators can be controlled by the spacing of the baffles. G should be greater than 10 in order to promote flocculation but less than 75 if disintegration of the floc by shear forces is to be avoided. The value of G can be adjusted to floc size growth requirements by providing a higher initial value and decreasing it progressively. The initial G value may be as high as 100 with the final values dropped to as low as 10 (typical values would be in the range 45-90).

The minimum detention time in the basin should be at least 20 min (1200 sec). Care must be taken in introducing flocculated water into the sedimentation tank to minimize floc disintegration. The recommended velocity in the channel or pipe to the sedimentation basin should be between 0.1-0.3 m/s and emphasis must be placed on smoothness of flow.

COMBINED FLOCCULATION AND SEDIMENTATION TANK

Influent

Effluent

Sedimentation Zone

Sludge Bleed

Sludge Blanket

Annular Baffle

Flocculation Zone

Sludge Drain

FIGURE 5.5

Two methods of providing tapered G values are by either varying the baffle spacing in a basin having a fixed cross-section or by evenly spacing the baffles along a varying channel section. An example calculation for a fixed cross-section channel is given in Annex B.

Typical sketches of horizontal and vertical flow baffled channel flocculators are shown in Figure 5.6.

Typical design criteria are as follows:

Velocity gradient, G		$= 45-90$ sec^{-1}
Detention time, t		$= 600-1800$ sec.
Channel vel. V		$= 0.1-0.3$ m/sec.
Head loss H		$= 0.5$ m
No. of baffles		$= 12-20$ with 180° direction change and well-rounded corners

Velocity
gradient $\quad G = \dfrac{sgH}{u\,T}$

where
$\quad S = $ special gravity
$\quad H = $ headloss per baffle (m)
$\quad = $ Viscosity of water (m^2/sec)
$\quad \quad (0.90 \times 10^{-6}$ m^2/sec at 25°C)
$\quad T = $ detention time per baffle (sec)
$\quad g = $ gravity (9.81 m/sec^2)

There are many examples of older or smaller plants without mechanical equipment where good flocculation has been accomplished simply by gravity flow through baffled channels. These include both under-and-over and around-the-end baffle design. The particular style or design does not seem to be important. What is important is to keep in mind the objective of plug flow and to approach that as closely as possible.

5.4 SEDIMENTATION

a) Horizontal Flow Settling Tanks

Sedimentation or clarification is the removal of suspended matter from water through a process of settling. In the process, particles heavier than water settle to the bottom of an especially designed tank. Further treatment may be needed to remove fine clay and colloidal matter which have a very slow settling rate and are difficult to remove by plain sedimentation. Horizontal flow settling tanks should be used where water contains much suspended matter. The tanks can be used alone, as a pre-treatment step for a slow sand filter, or as an intermediate process between chemical coagulation and rapid filtration.

-48-
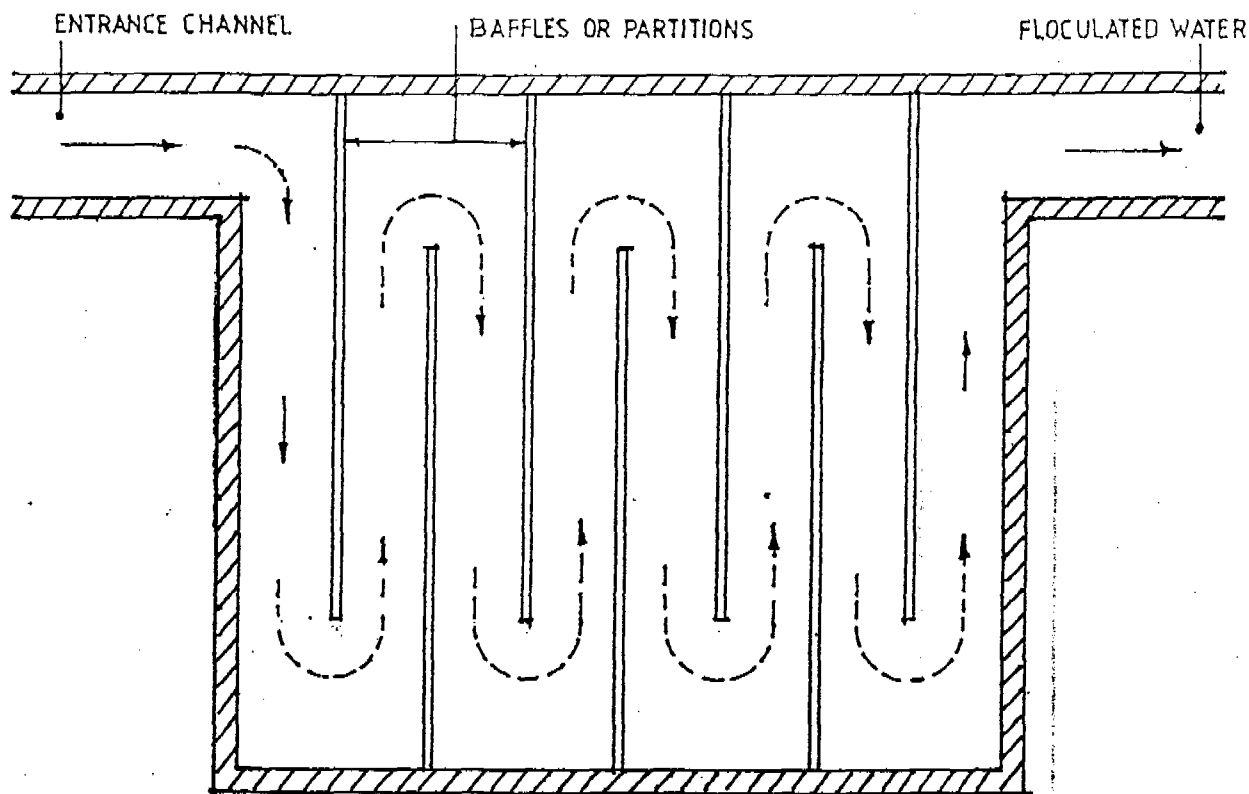
HORIZONTAL-FLOW BAFFLED CHANNEL FLOCCULATOR
PLAN
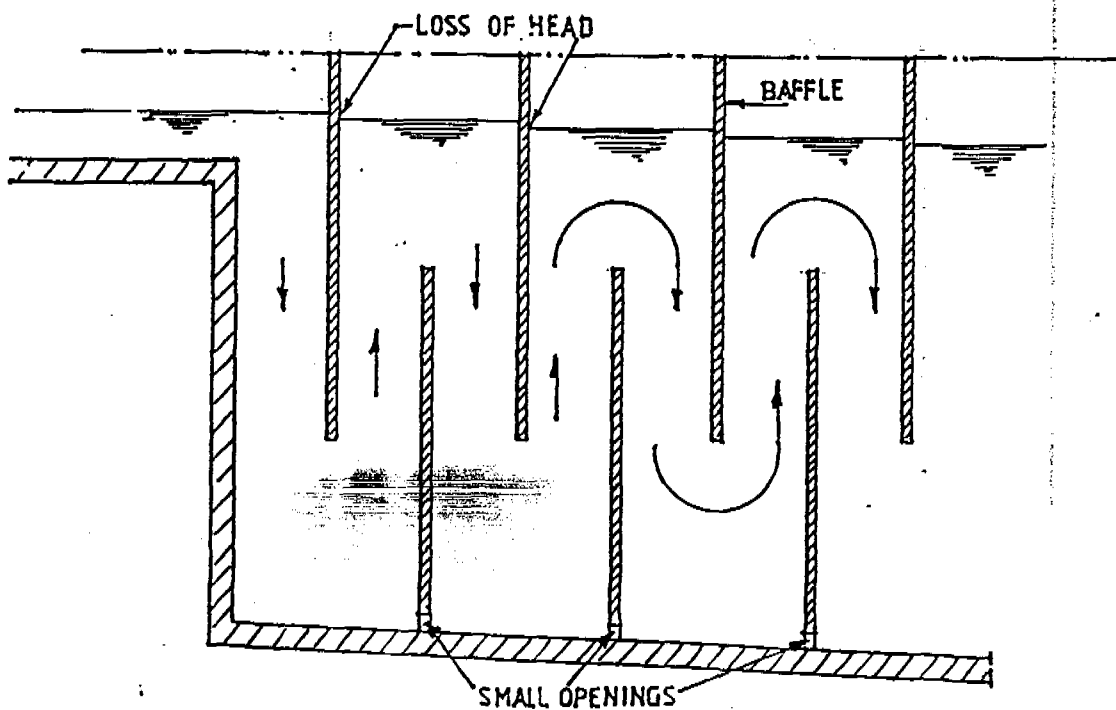
VERTICAL-FLOW BAFFLED CHAMBER FLOCCULATOR
CROSS SECTION

FLOCCULATORS
FIGURE 5.6

The most common design provides for the water flowing horizontally through the tank (Figure 5.7) but there are also designs for vertical or radial flow (Figure 5.8). For small water treatment plants, horizontal-flow, rectangular tanks generally are both simple to construct and adequate.

Sedimentation tanks are designed to reduce the velocity of water entering the tank so that it is retained long enough to permit particles to settle. A sedimentation tank has four sections: the inlet zone, the settling zone, the outlet zone, and the sludge deposit zone. (See Figure 5.7)

o The inlet zone is the area where water enters the tank and is distributed evenly throughout its entire width and depth.

o A baffle or wall separates the inlet zone from the settling zone. The work of the sedimentation tank is done in the settling zone, where the water is held long enough to permit suspended particles to settle. The efficiency of the settling process will be improved if there is no turbulence or cross circulation in the tank.

o In the outlet zone, clarified water is collected from the top layer of the tank. This area is separated from the settling zone by a weir which controls water flow out of the tank and into the outlet zone.

o Finally, the sludge zone collects the material settled from the water. The floor in the sludge zone should have a minimum of 5% slope toward the drain.

Sedimentation tank design is based on the assumption of ideal conditions in the settling zone, such as:

o A uniform flow distribution vertically and horizontally at the inlet and outlet to the settling zone;

o A uniform solids concentration at the inlet to the settling zone, vertically and horizontally.

Consider a particle with settling velocity (Vo) which is just removed (i.e. traverses the full depth (H) of the tank in the detention time (T).

-50-

SEDIMENTATION BASIN

SECTION

PLAN

HORIZONTAL FLOW
SEDIMENTATION TANK
MAIN FEATURES

FIGURE 5.7

-51-

Peripheral Weir

Annular Baffle

Effluent

Influent

Rotating Sludge Scraper

Sludge

Sludge Hopper

RADIAL FLOW TANK

Influent

Effluent

Sludge Bleed

Sludge Blanket

Sludge Drain
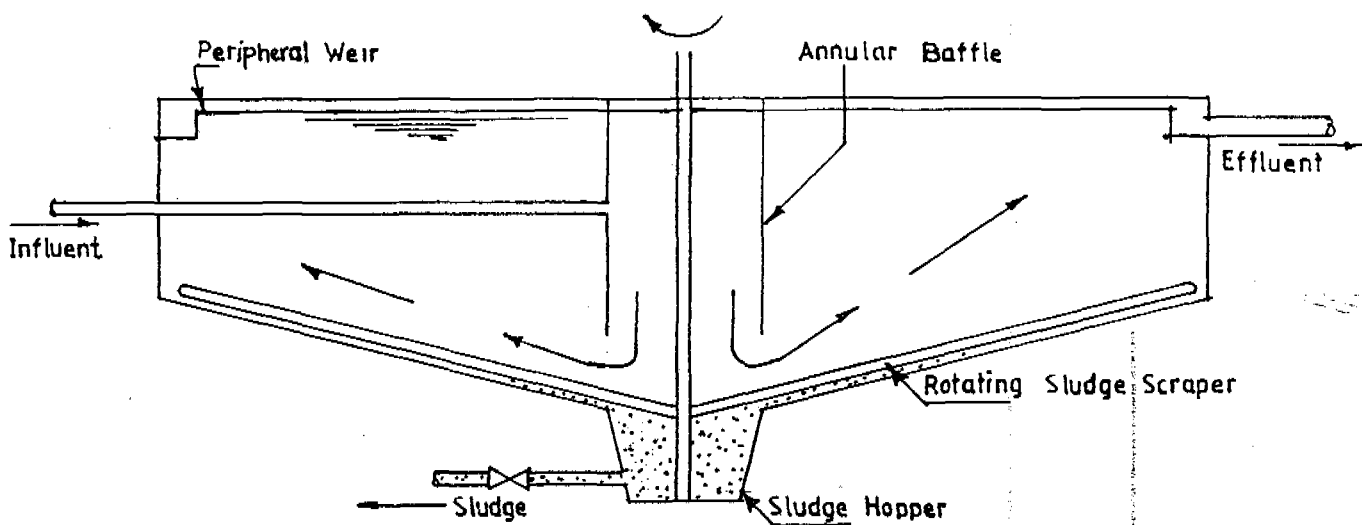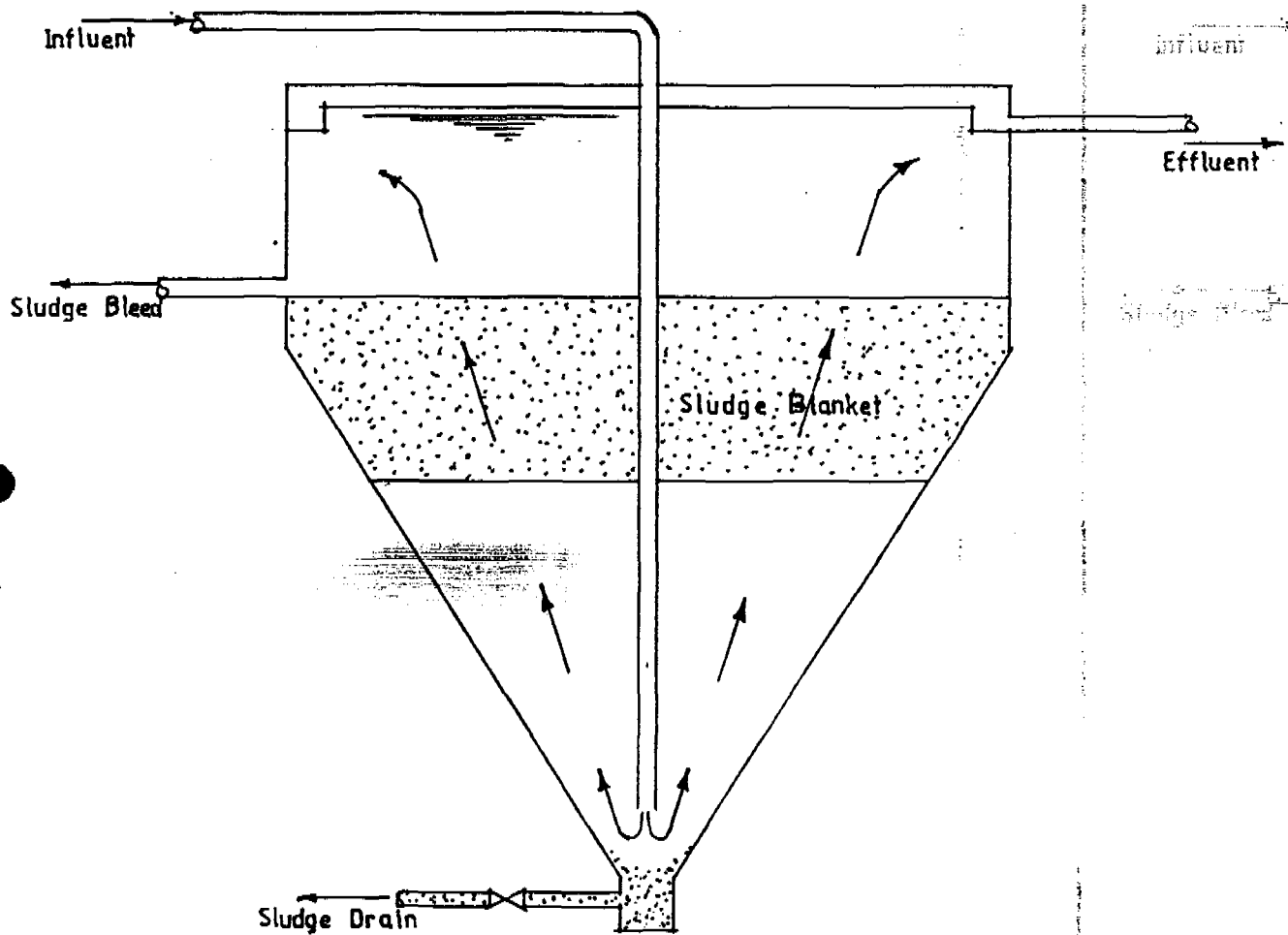
VERTICAL FLOW TANK

RADIAL AND VERTICAL FLOW
SEDIMENTATION TANKS.

FIGURE 5.8

$$Vo = \frac{H}{T} = \text{m/hr where } T = \frac{A.H}{Q} \text{ hr}$$

A = surface area of tank, m^2
Q = flow rate m^3/hr

Then $Vo = \frac{H.Q}{A H} = \frac{Q}{A}$ m/hr which is the surface overflow or loading rate

Note that the tank depth, H, is therefore immaterial. Particles with settling velocity Vs of Vo or greater should all be removed, but when Vs is lower than Vo:

provided H < VsT, they are removed, but if
H > VsT, they are not removed.

Therefore proportion of removed particles is $\frac{Vs}{Vo}$

where Vo is design velocity.

Settling tank design should be based on a settling column analysis of the raw water. Some typical settling velocities are given below:

Material	Partial Diameter (mm)	Settling Velocity (m/hr)
Coarse sand	1.0 0.5	365 194
Fine sand	0.25 0.10	97.5 29.0
Silt	0.5 0.005	10.6 0.14
Fine clay	0.001 0.0001	0.005 (5mm/h) 0.00005 (0.05mm/h)

For plain sedimentation (without pretreatment of chemical coagulation and flocculation), lower surface loading rates should be used.

A typical tank layout is shown in Figure 5.9 and suggested design criteria are as follows:

-53-

GROUND LEVEL

DESLUDGE

SLOPE

SECTION B B

INLET

TO FILTER

G.L.

SECTION A A

B

DESLUDGE

SLOPE

A

V NOTCH

RAW WATER

A

PLAN

B

SEDIMENTATION TANK
TYPICAL DESIGN

FIGURE 5.9

-54-

Surface loading rate (see table below)
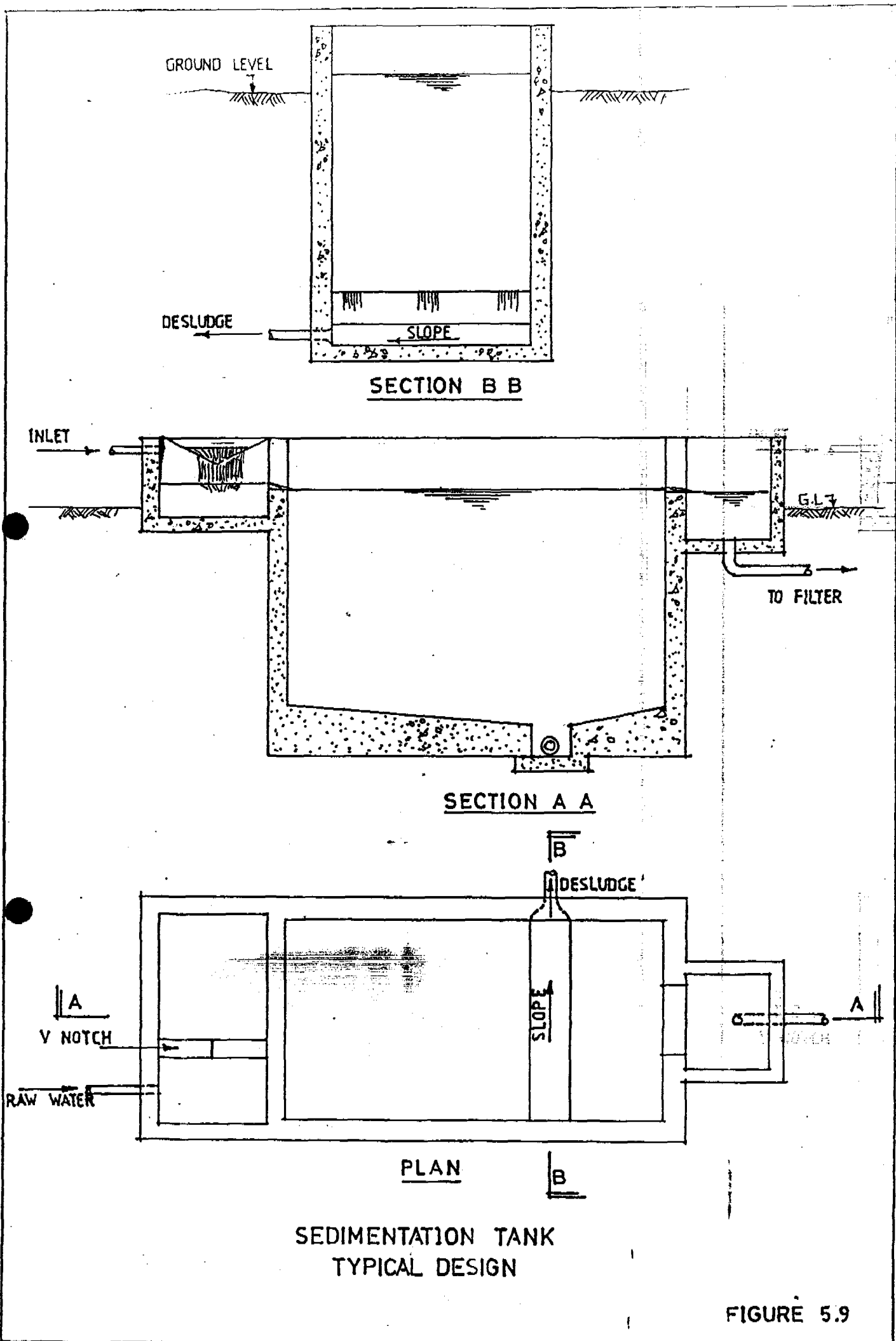Tank depth   2.5-4.0
Length/width ratio   3:1 to 8:1
Detention period   4-6 hrs
Length/depth ratio   20:1
(for basins over 60m long)
Weir overflow rate   < 18 m^3/m.hr
Sludge storage (additional depth) 0.15-0.3m
Diameter (circular tanks)   < 60 m
Average velocity in basin   < 0.02 m/s
Velocity in sludge drains   > 1.4 m/s
Velocity in inlet pipe/channel   0.15-0.4 m/s
     and outlet

Sedimentation after efficient chemical coagulation should remove almost all of the colloidal and suspended impurities in a water. Large floc particles have settling velocities in the range 0.5-3.5 m/hr and for alum floc 1.0 m/hr is commonly used for design purposes.

Recommended surface overflow rates are given below:

T y p e	Q/A, m/day per m		
	Normal conditions	Easy conditions	Very bad conditions
No pretreatment	0.5	1.0	0.1
With chemical coagulation	0.75	1.0	0.4
With coagulant aids	1.13	1.5	0.75

For efficient design most, if not all, of the following factors should be considered.

o   number of tanks;
o   dimensions;
o   velocity of flow;
o   detention time;
o   sludge storage and removal;
o   inlet and outlet arrangements;
o   particle size of solids to be removed; and
o   water temperature.

It is very difficult to take into account all the above factors, particularly since the whole situation can be upset by outside influences such as wind action. In view of this, most designs are based principally on judgement and experience.

Outlet weirs should be designed to maintain suitable settling velocities and to minimise short-circuiting. The rate of flow over an effluent weir should be restricted to 18 m^3/m hr. Vee notch effluent weirs with notches 50 mm deep at 75-150 mm spacing are preferable to plates since the effect of subsidence on the weir would be less acute. Submerged orifices may be used as an alternative to overflow weirs.

For the inlet, distribution of water equally and at uniform velocities requires an inlet arrangement such as baffles with open or submerged ports. Simple inlet and outlet arrangements are shown in Figure 5.10. Sludge removal may be achieved by mechanical sludge collection or by a hopper bottom with perforated pipes through which sludge is removed under hydrostatic pressure. Also rectangular tanks with reverse bottom slope are used with high pressure hoses for manual cleaning. Minimum size of pipe to be used for desludging is 200 mm dia. for non-mechanised units and 100-150 mm dia for mechanised units.

The following criteria should be used for sludge removal:

For manual cleaning   : slope of 1 in 10, hopper
                             bottom 1.2 in 1 - 2 in 1

For mechanical scraping : 1 in 12 slope

Allowable head loss   : 0.50 m

Power required   : 0.15 - 20 $kW/m^3/d$.

Rake speed   : 0.3 m/min

Manual cleaning has the disadvantage that the tank has to be taken out of service, but good plant design normally provides more than one settling tank, even with mechanical equipment. Where the solids load in the raw water is small, a plant can be operated for months between scheduled sludge removal. Small units can be provided with hopper bottoms for even simpler cleaning.

b) Vertical Flow Tanks

Solids contact units are popular in developed countries and combine in a single basin the various clarification operations with flow in an upward vertical direction through a layer (blanket) of flocculated suspended matter. These are often used for combined softening and clarification.

INLET PIPE   OVERFLOW

20mm holes at 60mm c|c
distribute water evenly

INLET

NOTCHED OVERFLOW WEIR

BOLTS
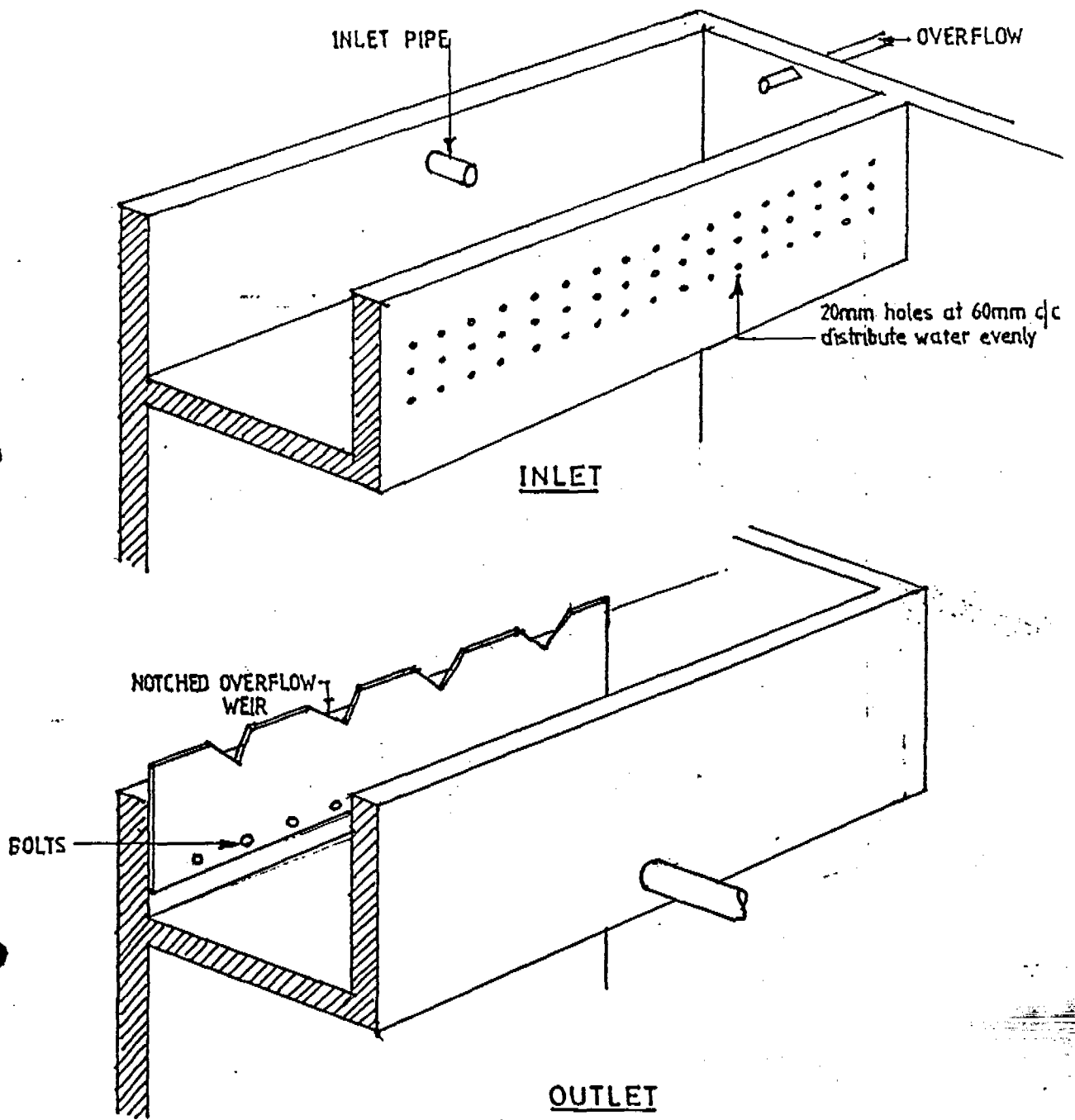
OUTLET

SEDIMENTATION TANK
SIMPLE INLET AND OUTLET ARRANGEMENTS.

FIGURE 5.10

Commonly used criteria are:

Surface loading   = 1.25-2.5 m/hr
Detention period   = 2 hours (not usually used as a
                         criterion)
Weir loading   = < 215 m^3/d/m of weir length.

These clarifiers require competent operation to maintain an optimum sludge blanket layer with suitable controls of sludge withdrawal. The blanket is sensitive to the coagulant dosage and to a number of other factors including the prevailing difference between ambient temperature and the influent temperature.

Experience with this type of clarifier in Sri Lanka has been mixed and often unsatisfactory, and they are not therefore recommended for small water treatment plants.

c) Clarifiers_with_Tube_or_Plate_Settlers

Tube settlers are now being incorporated in the design of many new water and wastewater treatment plants. They are also being used to increase settling capability of existing plants. Much remains to be learned about the design and performance of tube settlers, but experience already indicates they can increase operating rates by two to three times above conventional design criteria under proper conditions.

A recent development in high-rate sedimentation is the plate settler. Plate settlers may be made of metal, plastic, or water-proof plywood. Flow through the plate settlers may be either vertical or horizontal. The design concept is similar to that of tube settlers but is cheaper. The size of structures is kept to a minimum. The basin overflow rates for plate settlers are 5-10 times those of conventional rectangular basins.

Tubes may be circular or square in section, of about 5 cm size and inclined at 60° to the horizontal. Water is passed up through the tubes, and sediment settles out and trickles down, keeping the tubes clean. Tubes are usually prefabricated in packs and, made of PVC they are light and cheap; pipes may also be used. Loading is normally 9 m^3/hr per m^2 (m/hr).

When used for upgrading existing sedimentation tanks, improved surface loading rates of 4-5 times may be obtained, and it is important also to upgrade the sludge removal facilities to deal with the extra volume of sludge. A typical layout of a plate clarifier is shown in Figure 5.11.

RAW WATER

SETTLED
WATER

SLUDGE
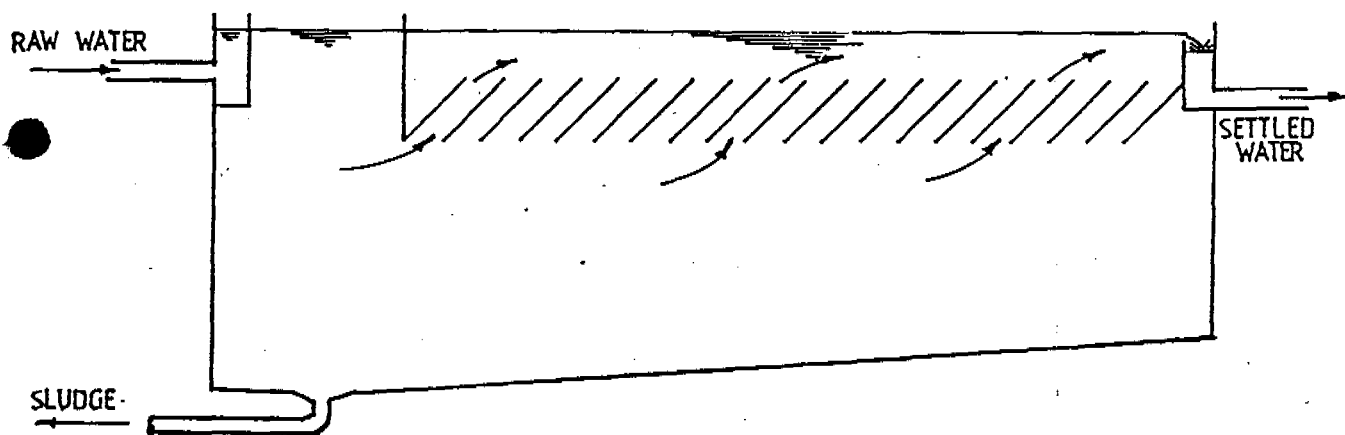
TILTED PLATE CLARIFIER

FIGURE 5.11

5.5 · RAPID FILTRATION

a) Conventional Design

Rapid filtration is recommended only for medium or larger schemes where skilled operators will be available. Filtration can be considered as the final polishing operation in water treatment. If the preceeding units are operating correctly, about 95% of the impurity load will have been removed from the water before it reaches the filters. Although overall purification resulting from the filters is small the process does ensure that the clarity of the treated water is to a high standard .

For filtration the choice is between slow sand filters for (gravity operation) and rapid sand filter (gravity or pressure filters). Rapid filters must be provided with efficient coagulation and sedimentation pretreatment (for water with turbidity in excess of 10-20 units).

A comparison of characteristics of the main types of filters is shown in Table 5.1 and Figure 5.12 shows the main features. A typical design layout is shown in Figure 5.13.

Common criteria for rapid gravity filters are:

At least 2 filters to be provided.
Influent turbidity | 20 units after pretreatment
Unit area | Up to 140 m^2
Filtration rate | 120-360 m^3/m^2d
Head loss · | Up to 2.5 m
Filter run | 1-7 days
Washwater consumption | 1-6% of filtered water
Depth of filter 0.3-0.6m gravel, 0.6-1.0m stratified sand
Gravel graded from 2-50mm over depth of layer.
Sand specification:
    0.4-1.2 mm Effective Size (ES)
    1.5-2.0 mm Uniformity Coefficient (UC)
or passing 16 mesh (1mm)
    retained 30 mesh (0.5mm)

Minimum depth of water when operating - 1m
Maximum inlet velocity - 0.6 m/s
Minimum ratio underdrain area to filter area - 0.003
Backwash velocity - 5-10 mm/s
Minimum backwash time - 5 min.

Wash water tank | Provide for 2 or minimum of 1 filter washes of 10 mins each. Should be capable of being refilled in 60 min.

TABLE 5.1
Characteristics of Sand Filters

CHARACTERISTIC	SLOW FILTER	RAPID FILTER GRAVITY	PRESSURE
Filtration rate	2 m^3/m^2d	120-360 m^3/m^2d	
Size of bed	Large (2000 m^2)	Small (100 m^2)	
Depth of bed	300mm gravel, 1m sand unstratified	500mm gravel, 0.7-1.0m sand stratified	
Effective size of sand	0.35 mm	0.4-1.2 mm	
Uniformity coefficient	2.5	1.7	
Head loss	Up to 1 m	Up to 3 m	
Length of run	20-90 days	1-7 days	
Method of cleaning	Scraping off top layer and washing	Backwash with air or mechanical scour	
Wash water consumption	0.2-0.6% filtered water	1-6% filtered water	
Penetration of suspended solids	Superficial	Deep	
Pretreatment by Coagulation	No	Yes	Yes
Direct on main use	No	No	Yes
Easily extended	No	No	Yes
Covered construction	No	Optional	Yes
Visible operation	Yes	Yes	No
Capital cost	Higher	High	Medium
Operating cost	Low	High	Higher
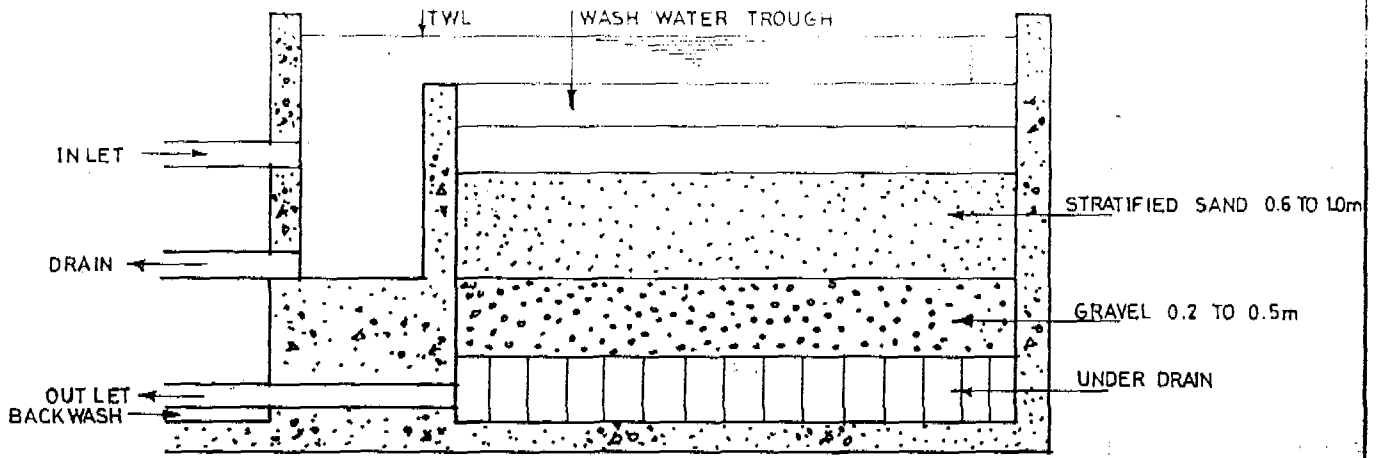
SECTION
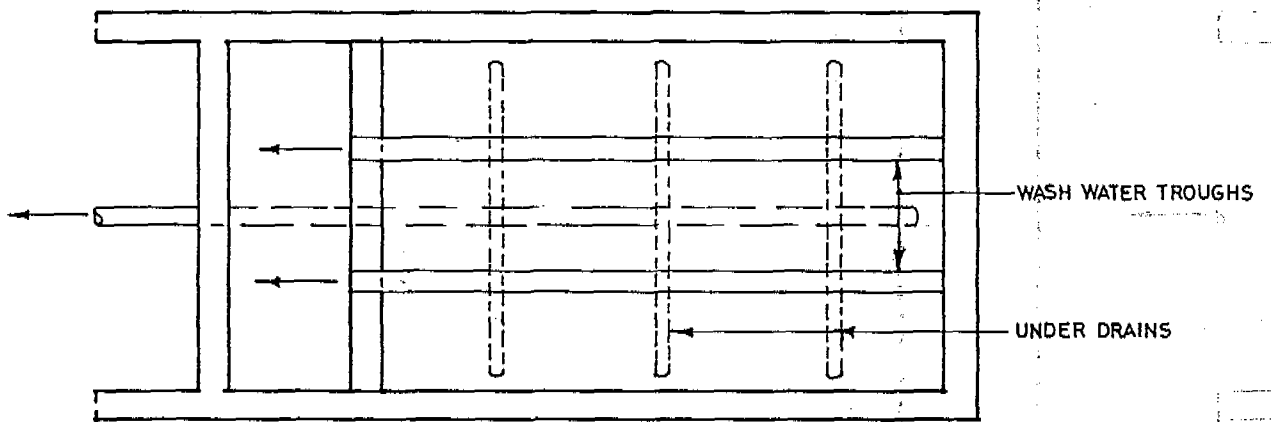
GRAVITY TYPE – PLAN
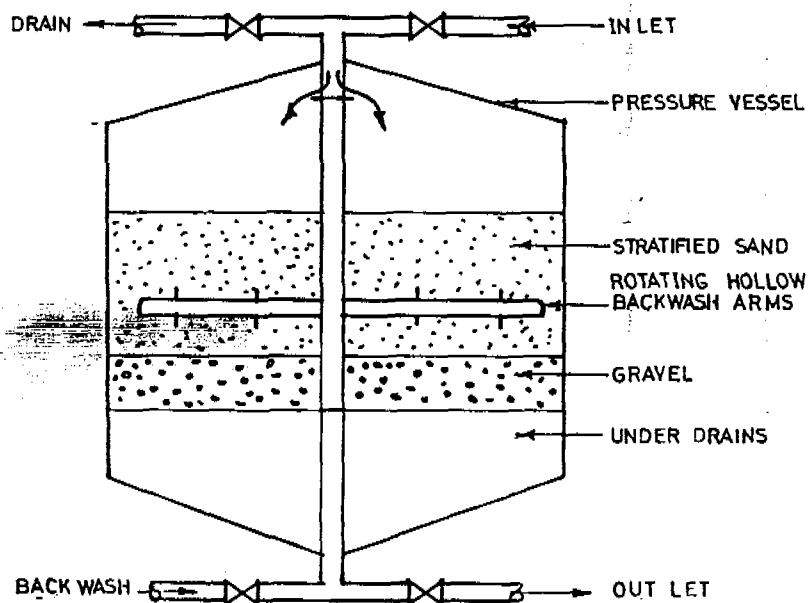
RAPID SAND FILTER
MAIN FEATURES

FIGURE 5.12

SECTION B B

SECTION A A

PLAN
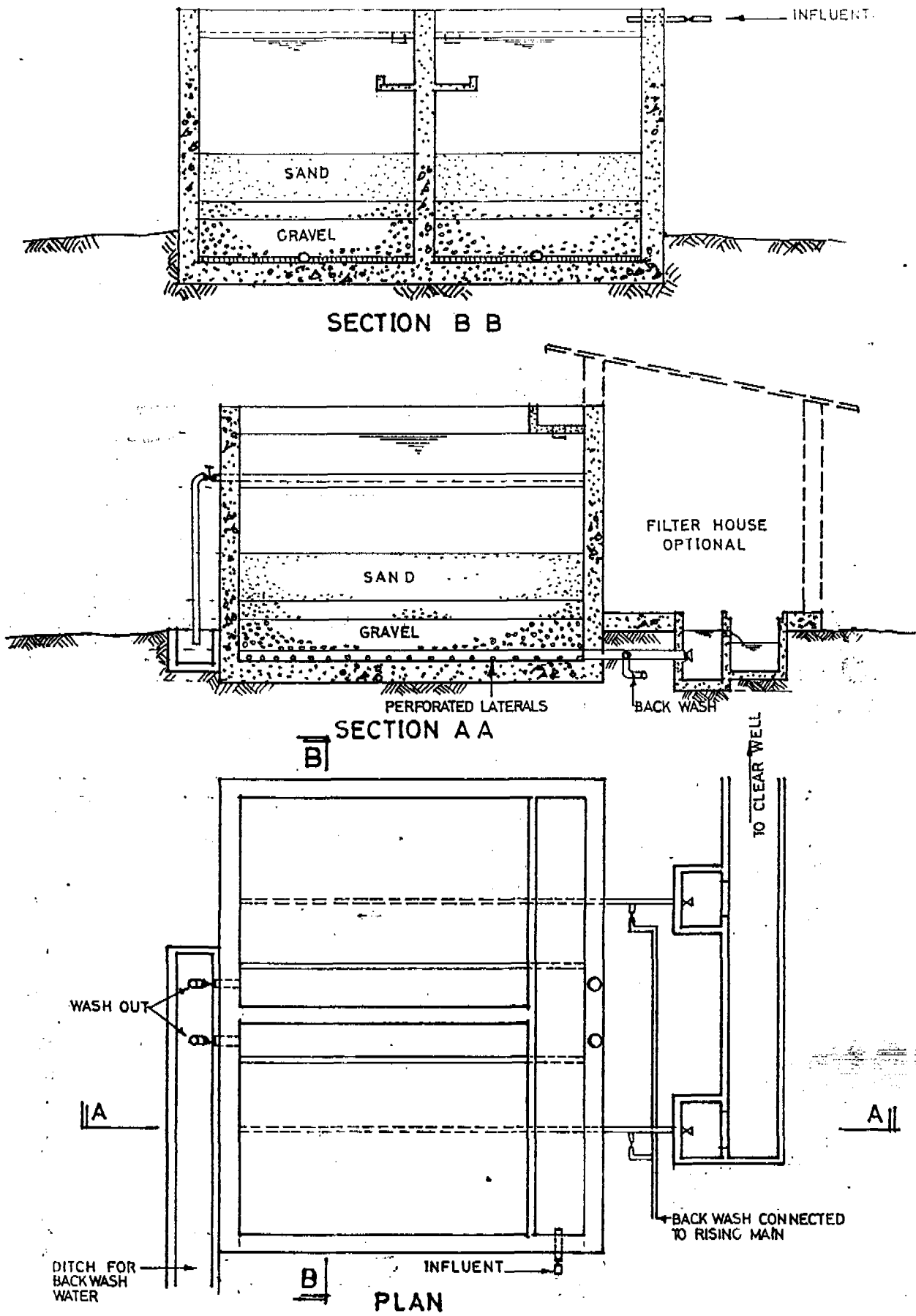
RAPID FILTER
TYPICAL DESIGN

FIGURE 5.13

-63-

Auxiliary scour   —   Mechanical rake in sand 1-10 rev/min. Air scour 0.015-.025 m^3/m^2s for several minutes before backwashing, pressure 20-35 kN/m^2. Surface wash by additional 1-20 mm/s jetted onto surface of sand at 150-450 kN/m^2.

Head loss in wash water system 8-15 m

Wash water troughs— Should be clear of expanded sand with maximum horizontal spacing of 2m. Top of troughs should be level and dimensions such that maximum flow can be carried with 50 mm free board.

Velocity in washwater
    outlet   -2.4-3.6 m/s
Length/width ratio -1.25-1.33
Minimum depth   -2.5 m
Controls   —   sampling taps
               —   loss of head gauge
               —   rate of flow control
               —   rate indication for washwater line
               —   sand expansion gauge.

Underdrains

Should consist of a central manifold with laterals either perforated on their underside or having umbrella type strainers on top (also other types such as porous plate may be used). Typical criteria are as follows:

Minimum dia. of
underdrains   — 20 cm
Dia. of perforations   — 6 to 12mm (staggered at a slight angle to vertical axis of pipe)

Spacing of perforation along laterals:
                          — 7.5 cm for perforation of 6mm ; 20cm for perforation of 12mm

Ratio of total area of perforation to total cross sectional area of laterals: 0.25 for 6 mm size;
                          0.50 for 12 mm size.

Ratio of total area of perforation to the entire filter area: .   — 0.003

-64-

Length/diameter ratio of lateral
$-60:1$

Max. spacing of laterals:	30 cm
Cross sectional area of manifold:	1.5 to 2.0 times the total area of laterals

Velocity of filtered water outlet: 1.0 to 1.8 m/s

b) Multiple and Coarse Media Filters

Until recent years, most rapid sand filters were designed for a rate of 120 $m^3/d/m^2$. Some health authorities were convinced that unnecessary risk was involved in exceeding this rate. Now, rates two or three times as high are becoming quite common. In some cases, successful experience has been demonstrated with rates far higher than that. There are a number of reasons for such a great difference. Most important is perhaps the change in filter media. However, better preparation of the water ahead of filtration, including the use of chemical polymers, is another important reason.

Where the capacity of the piping, channels, and other structures in the plant has made it hydraulically possible, many older plants have been substantially increased in capacity simply by rebuilding the filters to take advantage of new technology. Although this usually means complete rebuilding starting with the underdrain system, it can often be done entirely with local materials and labour and therefore costs much less than building an addition to the plant. One important lesson here is to design new facilities with plenty of extra hydraulic capacity in order to take advantage of such advances in technology.

The conventional rapid filter generally uses sand with an effective size of 0.6 mm and a uniformity coefficient of 1.5-2.0. This results in stratification of the media after backwashing, in which the fine medium remains at the top and the coarse medium at the bottom of the filter bed. This sand arrangement will restrict the effective use of the entire filter bed. To overcome this problem of gradation, two alternative media have been proposed, notably (i) dual-media and multi-media filters, and (ii) coarse size, narrowly graded media filters.

-65-

In multi-media filtration the arrangement of different
sizes of grains is reversed, by use of media of
different sizes and different specific gravity: the
lighter and coarser material at the top and the heavier
and finer material at the bottom. Generally a coarse
medium of low density such as anthracite (specific
gravity = 1.35 to 1.70) over a fine but heavier medium,
like sand (specific gravity = 2.65 to 2.70) is used. On
backwashing in an upward direction, the coarse and
lighter grains remain over the fine and heavier medium.
This makes the penetration and removal of impurities
take place throughout the entire bed.

The size of the media must be carefully selected so that
the water used for fluidizing and re-stratifying the bed
does not cause severe intermixing. Optimum conditions
for minimizing the intermixing exist, depending on the
media type, specific gravity and size involved. As an
alternative to anthracite, polystyrene beads, coconut
shell charcoal, or natural coke may be used. Other
possible materials are crushed coconut husks or pumice.
Heavier materials such as garnet sand, ilmenite or
magnetite may be used as the lower medium.

For small-size rapid filters it is common to use only
two materials in combination; 0.3-0.5 m of sand with an
effective size of 0.4-0.7 mm as the under-layer, topped
by 0.5-0.7 m of anthracite, pumice or crushed coconut
husks with an effective size of 1.0-1.6 mm.

Pilot testing in Sri Lanka using a layer of 50-60 cm
coconut charcoal (specific gravity 1.42) of 0.83-1.4 mm
size over a 25 cm layer of sand (0.5-1.4mm) has shown
good results.

In coarse media filtration, narrowly graded coarse sand
with an effective size of 2mm and placed to a greater
depth can be used instead of sand with smaller effective
size (0.45-0.55) and a large uniformity coefficient
(about 1.5). The filter can be operated at a higher
rate, and the deeper penetration and higher storage
capacities in the narrowly graded coarse medium leads to
longer filter runs. However, due to greater particle
deposition throughout the filter, backwashing
requirements are higher. Generally, air-water
backwashing is necessary in coarse-medium filters and
these requirements are higher than for conventional
filters, and the underdrain system may also require
modification.

When dual media filters are installed in plants it will
be important to provide a high pressure hose line to jet
into the expanded charcoal during backwash. Mud balls
form quickly in the light charcoal media. They must be
broken up and washed out or problems will quickly
develop. A high pressure jet stream directed into the
expanded bed throughout the wash will do the job.

c) Declining Rate Filters

In water treatment plants of a generation ago, high technology was usually represented by automatic rate controllers which were designed to maintain a constant filter rate throughout the run. They were costly, not very reliable, and difficult to maintain properly. This has been another element of great change, since most filter designs now do not use rate controllers; depending on the viewpoint, this is either new or old technology.
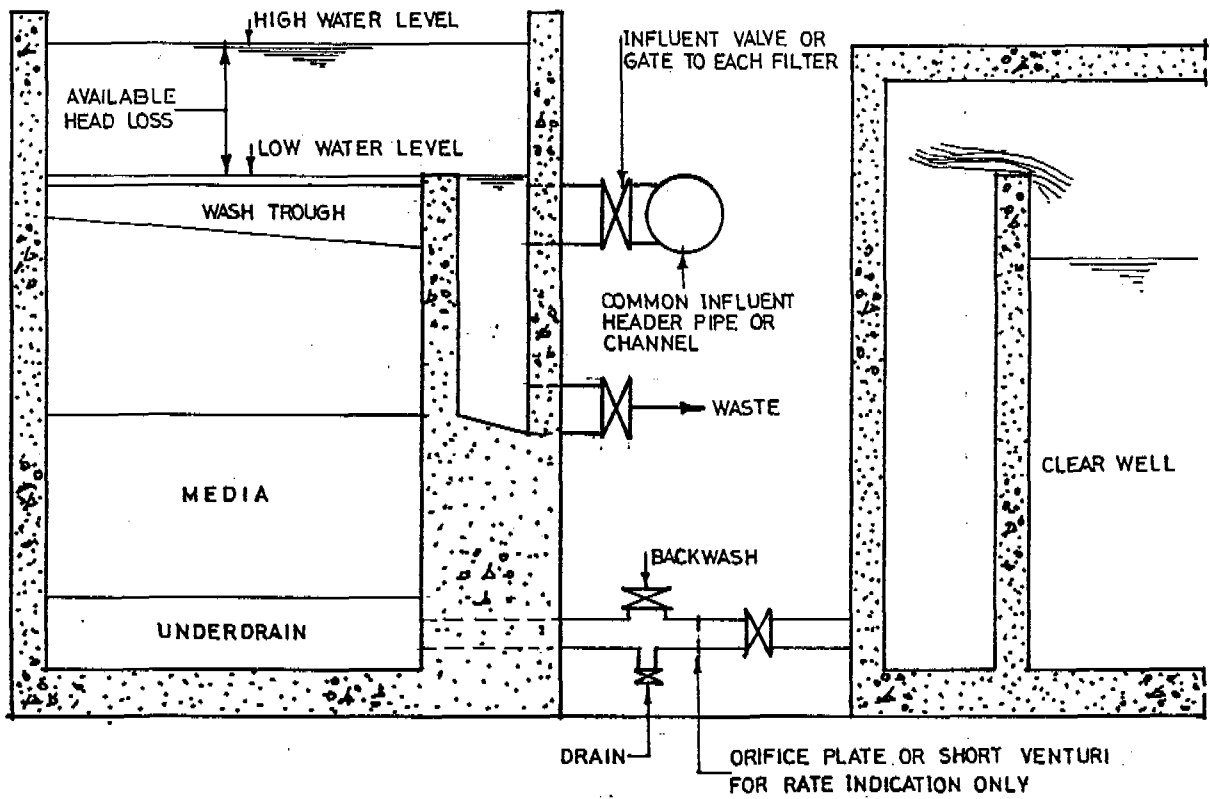
In this method, incoming flow is supplied to all operating filters on a free-flow basis to meet their individual operating rates. There are no effluent controllers. The only control is the effluent overflow level in the clear well. Figure 5.14 shows the general arrangement for declining rate operation.

The method of operation is as follows:

The filter influent enters below the low water level of the filters. The water level is essentially the same in all operating filters at all times. This is achieved by providing a relatively large influent header to serve all the filters and a relatively large influent valve or gate to each individual filter. Thus, head losses along the header or through the influent valve are small and do not restrict the flow to each filter. The header and influent valve will be able to deliver whatever flow each individual filter is taking at the moment.

All of the filters in the bank of filters served by the common influent header get dirty; thus, the flow through the dirtiest tends to decrease the most rapidly. This causes the flow to redistribute itself automatically so the cleaner filters pick up the capacity lost by the dirtier filters. The water level rises slightly in the redistribution of flow to provide the additional head needed by the cleaner filters to pick up the decreased flow of the dirtier filters. The cleanest filter accepts the greatest flow increase in this redistribution. As the water level rises, it partially offsets the decreased flow through the dirtier filters; as a result, the flow rate does not decrease as much or as rapidly as expected.

This method of operation causes a gradually declining rate towards the end of a filter run. The rate increases which do occur always do so on the cleaner filters in the filter bank, where they will have the least effect on filter effluent quality. These rate changes, both upward and downward, occur gradually and smoothly without any automatic control equipment.

-67-

HIGH WATER LEVEL

AVAILABLE
HEAD LOSS

LOW WATER LEVEL

WASH TROUGH

MEDIA

UNDERDRAIN

INFLUENT VALVE OR
GATE TO EACH FILTER

COMMON INFLUENT
HEADER PIPE OR
CHANNEL

WASTE

CLEAR WELL

BACKWASH

DRAIN

ORIFICE PLATE OR SHORT VENTURI
FOR RATE INDICATION ONLY

DECLINING RATE OPERATION

FIGURE 5.14

There are many advantages to this scheme in terms of better filtration and simpler operation and maintenance. However, another very important advantage is that many existing plants can easily be adapted to this process. The needed modification can often be done with local resources. (See also Section 5.5 (g), Upgrading Existing Filters)

d) **Upflow Filtration**

In this process, raw water is fed at the bottom in an upward flow direction and coarse-to-fine media filtration is achieved with a single medium in the direction of filtration, which makes better use of the entire filter bed. The problem with upflow filtration arises when the headloss exceeds the weight of the bed after a certain period of filter run, at which time partial fluidization of the medium occurs. This can be overcome by placing a grid, with a spacing of 100 times the sand grain diameter, just below the top of the sand bed.

Upflow filters have been used successfully in Singapore and Western Europe for some years. They may in some cases be useful as a pretreatment for rapid or slow sand filtration but are suitable only for water with low or medium turbidity (< 80 units), or they will be subject to short filter runs and restricted flow rates, due to high head losses.

e) **Direct Filtration**

Direct filtration is filtration without prior sedimentation. This has been recommended (Refs 2,3) as suitable for many treatment plants in Sri Lanka, since surface water sources are, in general, quite clear for most of the year. It is reported to be suitable for turbidities up to 40 units.

A very light dose of either alum or ferric chloride coagulant is required to destabilize the low turbidity content so that it can be readily trapped on or in the filter media. This would require less use of coagulant, and increased filter loading rates, with an increase in filter washing during times of high turbidity. A contact time of 30 to 60 minutes is desirable prior to filtration if coagulants are used.

In the right conditions a properly designed filter can handle a relatively turbid water by direct filtration. Dual and mixed media are preferred. The main advantages over conventional filtration are:

-69-

- savings in chemical costs;
- reduced sludge handling;
- less land area required;
- reduced construction cost;
- simpler O&M;
- less liable to upsets in coagulation process.

Disadvantages are:

- more frequent backwashing with more water;
- less reliability in effluent quality (bacteriological and turbidity);
- more operator vigilance required to maintain quality.

The suitability of a raw water for direct filtration may be investigated using the filter paper test, see Annex D.
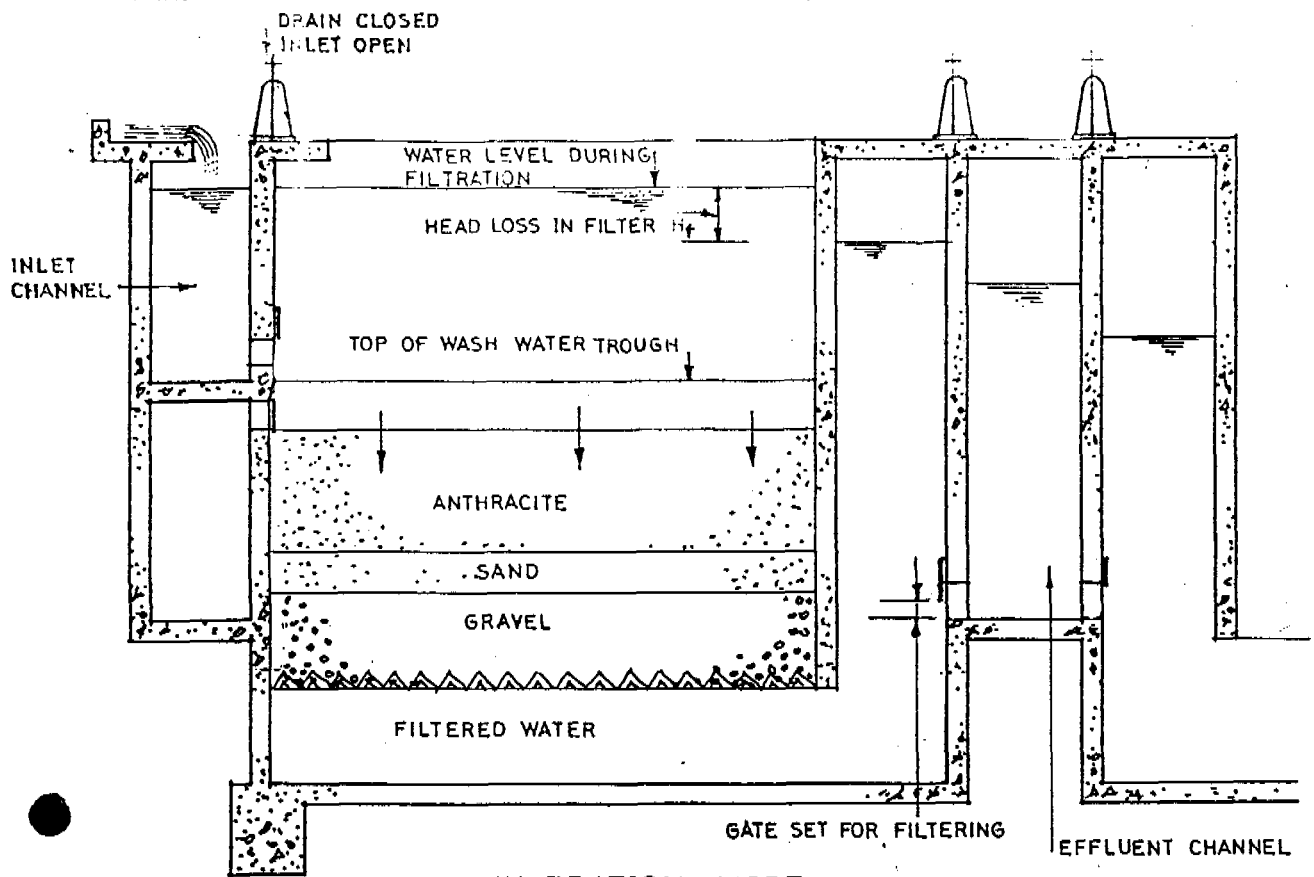
f) Low_Pressure_Backwashing

In this method, a gravity filter may be backwashed by the effluent from the adjoining filter units. To obtain sufficient head, the depth of the filter box must be greater by 1.5 to 2.5 m than for conventional filters, and filter bottoms must be designed for lower headlosses (20-30 cm instead of 1-1.4 m). It requires a minimum of 4 filter units.

Using restricted declining rate mode, the filter units are connected by common inlet and outlet channels. Three gate valves control the backwash process, as shown in Figure 5.15. Typical details of the layout and bottom are given in Figure 5.16.

g) Upgrading_Existing_Filters

When converting existing filters to dual media, the filter box and washwater trough must be carefully examined to be sure that there is enough depth to accommodate the filter media. The bottom gravel support requires roughly 35 to 40cm, the sand 25cm and the charcoal 50 to 60 cm. This makes a layer of from 1.10 to 1.20 m in depth. Another 50 cm is required for bed expansion and a margin of safety to the backwash overflow level, so that at least 1.70 m of filter box is required below the washwater overflow weir level.

The increase in filter loading will be limited by the hydraulic constraints more than the capacity of the filter and filter media. In all existing plants, however, the inlet channels appear to have ample capacity. The filtered water outlet is probably more critical but it is short so that twice the present filter rate may be possible.

-70-

FILTRATION MODE
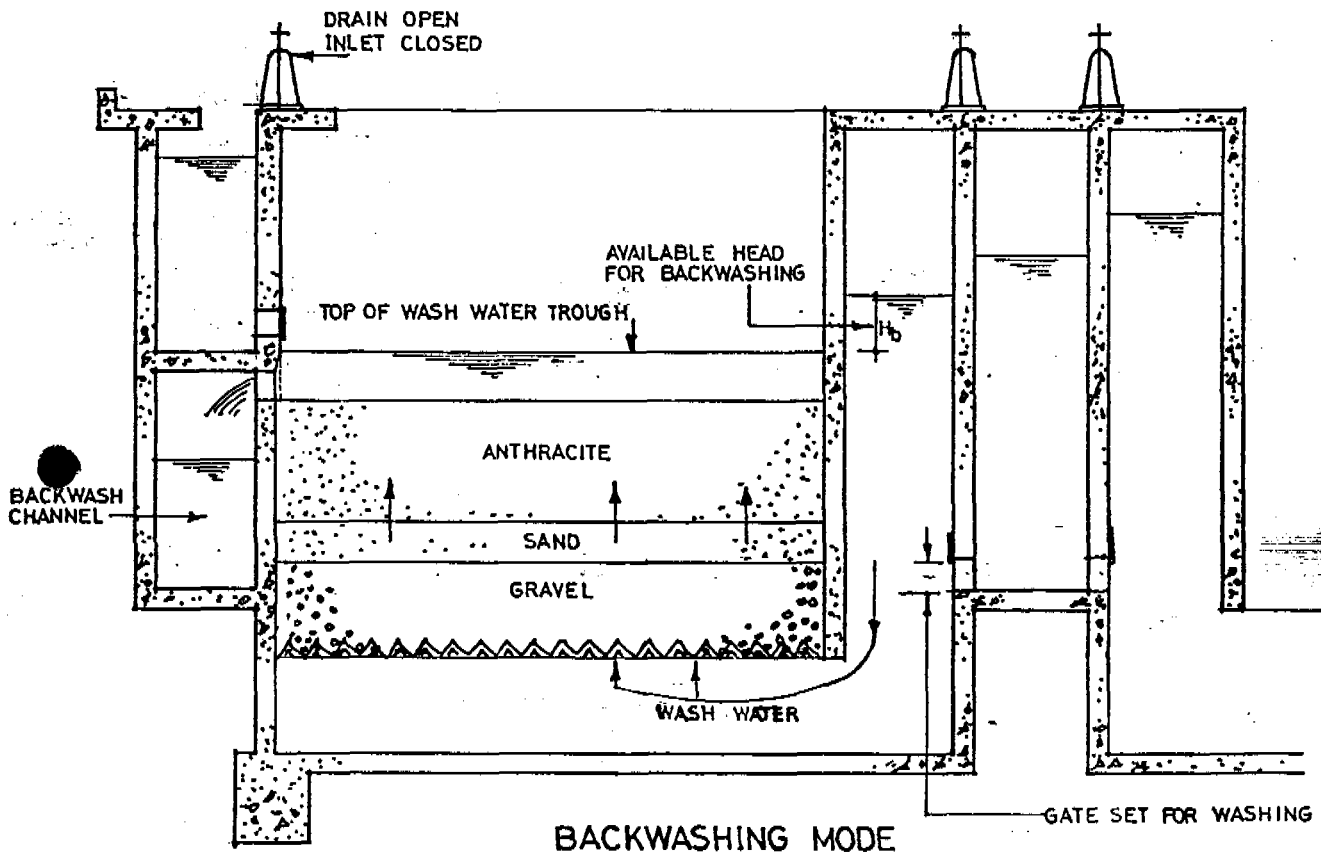
BACKWASHING MODE

LOW PRESSURE BACKWASHING
(INTER FILTER BACKWASHING)

FIGURE 5.15
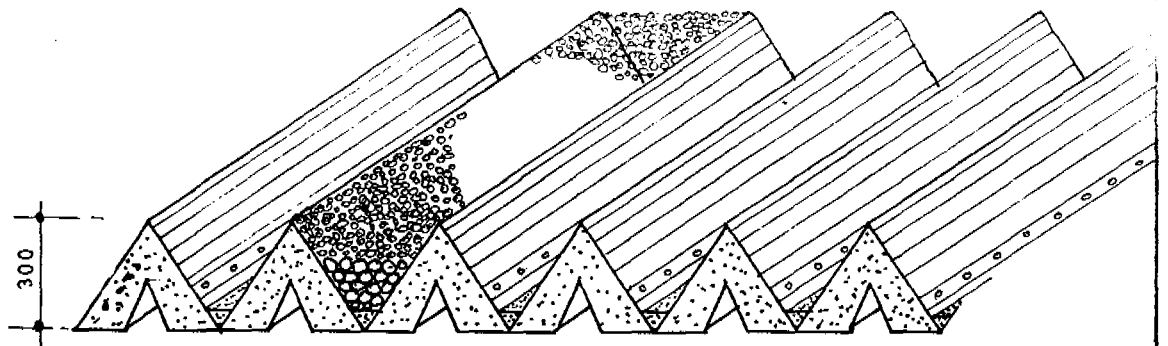
ISOMETRIC VIEW
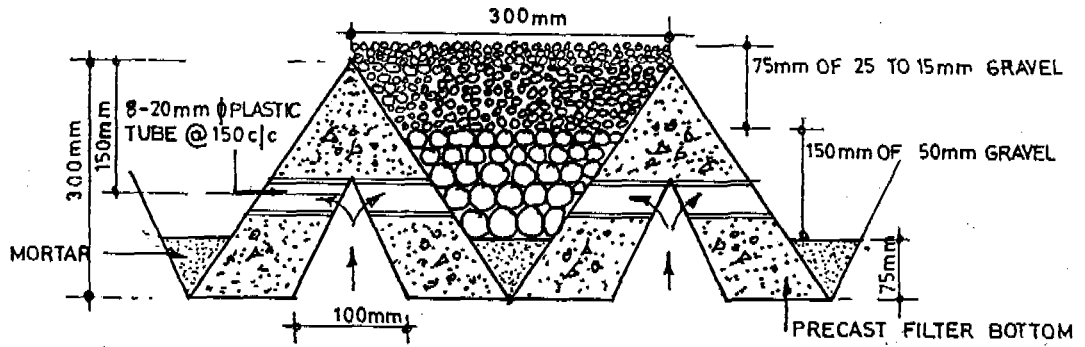
TYPICAL LOW HEAD LOSS FILTER UNDERDRAIN

PLAN OF FILTERS

LOW PRESSURE BACKWASH FILTERS
ARRANGEMENT AND UNDERDRAIN
DETAILS

FIGURE 5.16

Filter rates should be determined for all the filters in the plant. This work can be done at the same time the raw water pumping tests are made. The procedure is quite simple:

1. Fill the filter box to the normal water level with the filter outlet valve closed.
2. Close filter inlet valve.
3. Open filtered water outlet valve.
4. Measure the drop in water level in the filter box by timing a drop of 15 to 20 cm. The timing should be started only when the outlet valve is completely open.
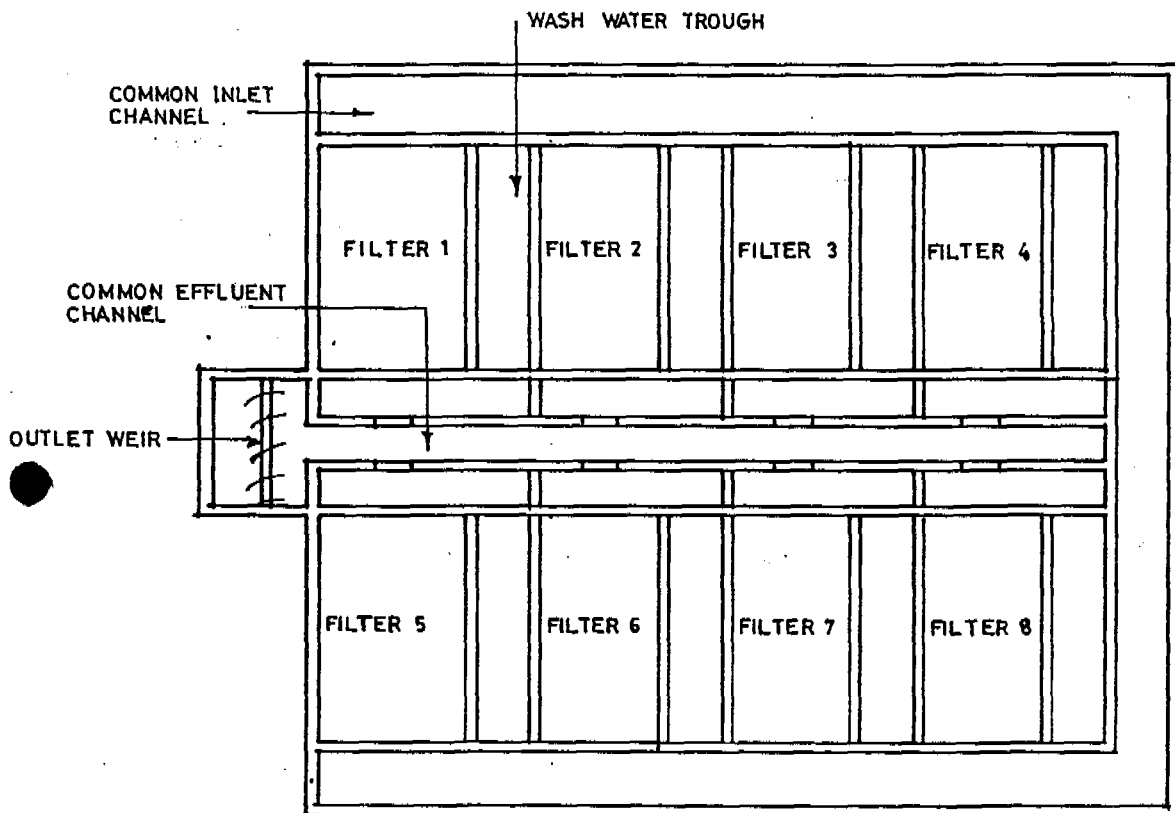
5. Knowing the area of the filter box the volume is easily calculated and the filter rate can be determined.

This information is important as an indicator of the maximum filter rate possible with the existing bed and existing losses in the outlet valves and piping. If the rate is higher than advisable an orifice plate can be installed in the outlet system to introduce a headloss which will reduce the filter rate to anything the designer desires. (See Annex C).

Filter backwash rates should also be determined. This will be just the opposite procedure to the above.

1. Close inlet valve.
2. Filter the water down to a point about 10 cm above the bed.
3. Close filter outlet valve.
4. Start backwash.
5. When wash is on full force measure the rise in the water level before it reaches the bottom of the overflow drains. This distance is sometimes very short so that the available distance may be small.

A good backwash rate is 500 to 600L/min per m^2. The backwash rates in all existing plants are probably all less than 500 L/min per m^2.

Most operators are reluctant to wash filters often. This is understandable because (a) it requires work to open and close several valves; (b) it uses a large amount of water, and (c) it takes a part of the filter capacity out of service. It is necessary to understand the importance of operating filters at a high loading; it is far more economical to increase the filter rate even though the washing frequency goes up. Below is a calculation of production per m^2 of filter bed at 100m/day and 234m/day.

Filter Rate $(m^3/m^2/day)$	Filter Run (Hrs)	Filter Wash Rate $(m^3/m^2/min)$	Wash Waterr Consumed per day (m^3/d)	Filtered Water Lost from Down Time of 30 min/wash (m^3/m^2)	Total Water Lost (m^3/m^2)	Net Water Produced per day $(m^3/day/m^2)$
100	72	0.5	1.67	0.7	2.37	97.63
234	8	0.5	15.00	14.6	29.60	204.40

Even though one filter is washed 9 times as often, the higher rate pays big dividends in production per m^2.

For conversion of filters to direct filtration, testing is required, with a low coagulant dose determined in the laboratory with filter paper (see Annex D). The filter performance should be monitored for turbidity and headloss. If performance is good at the plant rate, cut out one unit and increase the load on the others, continuing until the runs are reduced to less than 12 hours, or the effluent turbidity rises above 4.5 units. This will indicate the increase in filter loading possible without any modifications.

A simple filter head loss gauge is shown in Figure 5.17.

h)   Pressure_Filters

These are based on the same principle as rapid gravity sand filters, but water is passed through the filter under pressure. They usually consist of a prefabricated steel or cast-iron tank placed vertically or horizontally.

Pressure filters are commonly used in situations where it is desired to take advantage of a particular hydraulic gradient and in such case the use of free surface settling tanks is not possible. Coagulants must therefore be added directly to the pressure filter where they build up floc in the filtering medium with consequent rapid choking of the filter and the need for frequent backwashing.

There is economy by avoiding double pumping with this type of filter but they suffer from many disadvantages among which are the difficulty to inspect, clean and replace the media and effective pretreatment by chemicals. They are used generally for industrial needs and swimming pools.

-74-
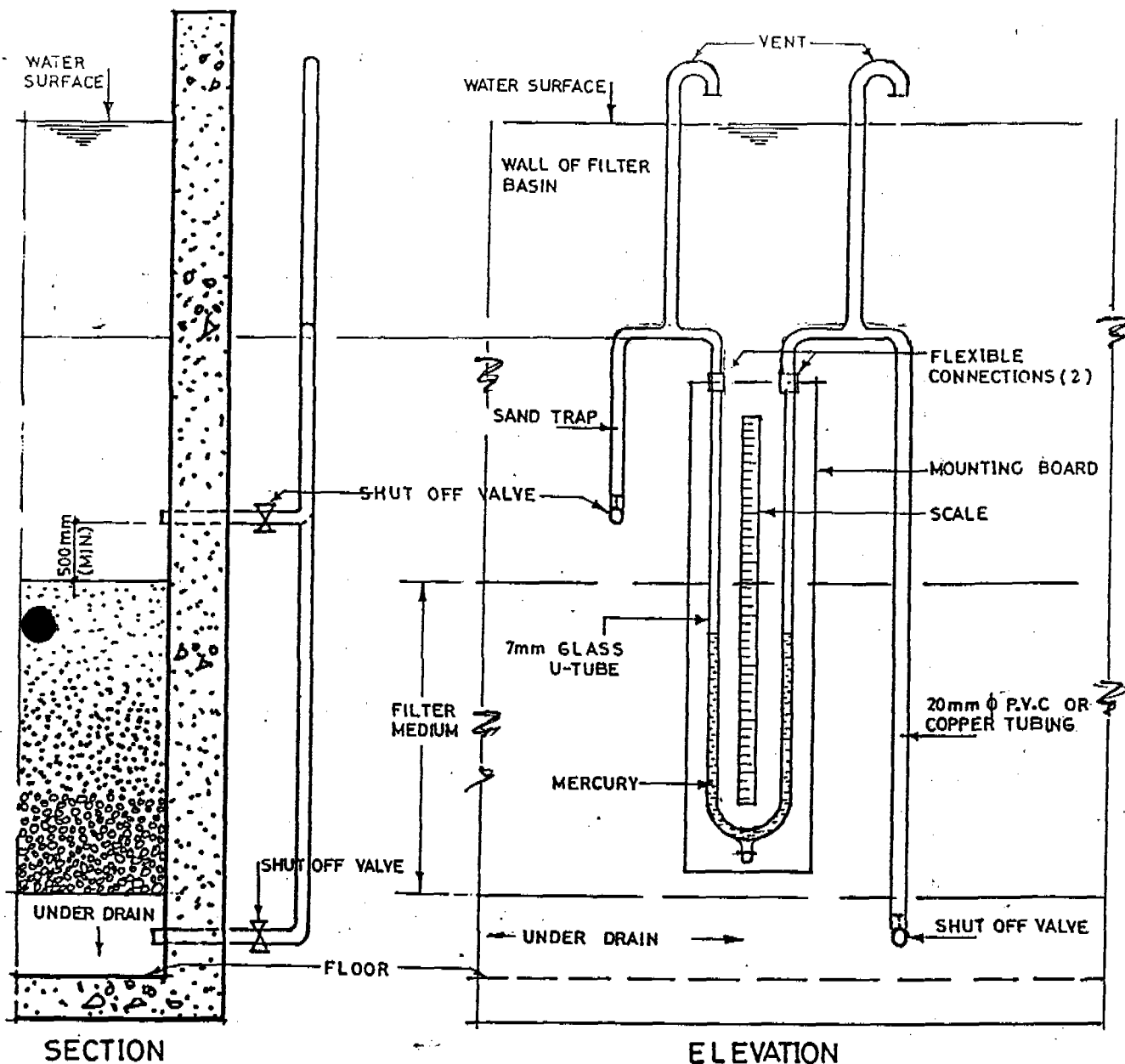
SIMPLE HEADLOSS GAUGE

OPERATION:

The Filter Head Loss is indicated by the displacement of the mercury in the glass U-tube. If there is no head loss, the top of the mercury will be at the same level in both limbs of the tube. As head loss develops, the limb tapped below the filter will rise and the other will fall.

Specific gravity of mercury is 13.6

The sand trap can be flushed by adding water at the vent.

DETAILS:

The scale should slide vertically for adjusting the zero setting and should have divisions of 73.5 mm for each meter of head loss (1000/13.6). The length should be approximately one tenth maximum head loss.

FIGURE 5.17

5.6   SLOW SAND FILTRATION (SSF)

a) Introduction

Slow sand filtration (SSF) has been used as a basic method of treatment in Europe and N. America for many years and is particularly appropriate for application in developing countries by virtue of its simplicity of design and construction and ease of operation and maintenance. Experience with the several existing plants in Sri Lanka, however, has been mixed, and there is a need to pay more attention to raw water quality, pretreatment and operation and maintenance aspects. A programme of monitoring existing SSF plants in Sri Lanka is necessary to provide evaluation and feed-back to assist designers in overcoming the present problems*.

Water is purified in a SSF by slow percolation through a bed of fine sand. Pretreatment is necessary with raw waters having an average turbidity of 25 TU or more, but should be considered also for less turbid raw waters 5 to 25 TU) to improve effluent quality and reduce frequency of cleaning.

SSF is also useful for treating groundwater containing solids in suspension, e.g. ferric and manganic compounds converted by aeration from the soluble state of the salts.

The advantages of SSF may be summarised as follows:

o   high quality of treated water — physical, chemical & bacteriological;

o   ease of construction — local material and skills; little special pipe work or equipment required;

o   low cost and ease of operation — no chemicals or energy; only manual cleaning of filter beds;

o   conservation of water — no wastage; water passed through a filter during the 'ripening' process can be returned to source or diverted to another filter; and

o   ease of sludge disposal — being handled in a dry state there is no pollution of watercourse.

The limitations/disadvantages are:

o   land requirement — SSF requires more land than rapid filters;

---

o   changing water quality, concentration of colloids
     and toxic substances affect biological filtration
     adversely;

o   algae - certain types of algae interfere with the
     working of SSF causing premature clogging.
     Pretreatment or covering of filter beds means
     additional capital cost.

Slow sand filtration is capable of producing a safe
drinking water from a polluted surface source, at
relatively low cost and with O&M by trained operators
from the local community with a minimum requirement of
technical knowledge, or backup. Treatment efficiency
is high in a properly operated plant, with 98% +
removal of bacteria, E-coli and virus reduced by a
factor of 1,000 more, and efficient removal of
parasites, thus providing substantial public health
benefits.

b) Filtration Mechanisms

A combination of different processes act in SSF to
achieve removal of impurities from the raw water.
During several hours of storage above the sand bed,
flocculation and sedimentation of large particles take
place. Daytime exposure to sunlight also has a
bactericidal effect. In the sand bed straining,
adsorption and biochemical and microbial action take
place, with most activity in the top layer of the
filter surface, called the "schmutzdecke."

This surface layer strains out bacteria, algae,
protozoa, rotifers and other micro-organisms which,
through tremendous biological activity, break down the
organic matter in the raw water. As flow continues
downward through the filter bed, this process is
continued by adsorption onto a sticky layer of organic
material around the sand grains and degradable organic
matter, bacteria and viruses are gradually broken into
inorganic compounds such as nitrates, carbon dioxide,
sulphates and phosphates. Most biological activity
takes place near the surface (0.5-2 cm) but the
process continues to a depth of 40-60 cm into the sand
bed. Bed thickness should therefore not be reduced to
less than 70 cm.

Efficiency is restored by scraping off the surface
layer of sand at intervals of from two months to
several months. After cleaning, the filter does not
become fully effective for 1 or 2 days, until the
growth of biological activity develops fully.

-77-

c) Features

The main features of a SSF are shown in Figure 5.18. These are:

o   a head of water above the sand bed to produce the design flow under maximum head-loss conditions;

o   a filter bed of uniform material;

o   an under-drainage system designed to maintain a uniform filtration rate over the whole filter area;

o   regulation and control devices to maintain head of water during operation, drain for cleaning, and refill; and

o   scum removal, flow and head loss measurement and aeration devices.

d) Design

As far as possible, design factors should be based on experience obtained from existing SSF plants in Sri Lanka, using similar types of raw water, and it is hoped that such information will be available from research projects ongoing during 1988.

In the absence of specific design factors, however, the following basic criteria should be followed:

Depth of water above sand 1 - 1.5 m

Freeboard above water level 0.2 - 0.3 m

Height of walls above ground surface > 0.8 m

Depth of media 1.0 - 1.4 m

Grading of sand - effective size 0.15 - 0.35 mm
                 - uniformity coeff. 2 - 3 (preferably < 3)

Filtration rate (V) = 0.1 (average) - 0.2 (maximum) m/h

Gravel filter support 0.3 - 0.5 m depth

Depth of under-drainage sytem 0.3 - 0.5 m

Area (A) per filter bed 10 - 100 m^2

         A = Q/V where Q = flow (m^3/h)
                       V = filtration rate (m/h)

Effluent weir level above sand 30 - 40 mm

-78-
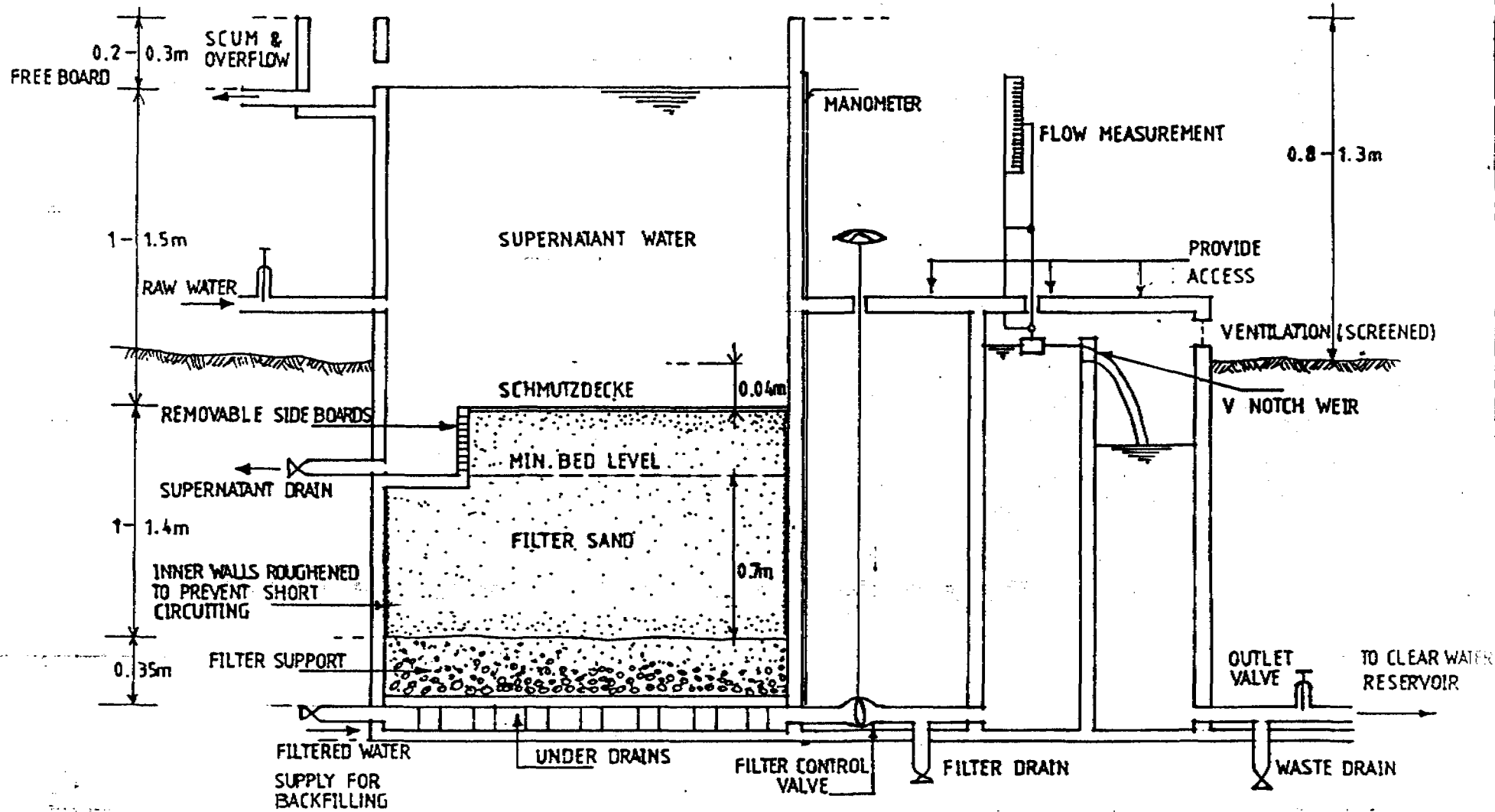
SLOW SAND FILTER – FEATURES

Labels in figure:

- SCUM & OVERFLOW
- 0.2 – 0.3m FREE BOARD
- 1 – 1.5m
- RAW WATER
- REMOVABLE SIDE BOARDS
- SCHMUTZDECKE
- 0.04m
- MIN. BED LEVEL
- SUPERNATANT DRAIN
- SUPERNATANT WATER
- 1 – 1.4m
- FILTER SAND
- 0.7m
- INNER WALLS ROUGHENED TO PREVENT SHORT CIRCUITING
- FILTER SUPPORT
- 0.35m
- FILTERED WATER SUPPLY FOR BACKFILLING
- UNDER DRAINS
- FILTER CONTROL VALVE
- FILTER DRAIN
- WASTE DRAIN
- MANOMETER
- FLOW MEASUREMENT
- 0.8 – 1.3m
- PROVIDE ACCESS
- VENTILATION (SCREENED)
- V NOTCH WEIR
- OUTLET VALVE
- TO CLEAR WATER RESERVOIR

FIGURE 5.18

Provide 2, preferably 3 or 4 filter units (max. rate with 1 filter being cleaned should not exceed 0.2 m/h)

Provide space for additional filter units

To minimise wall length (L), width (W) = $\frac{L}{2} \left(\frac{N+1}{N}\right)$
   where N is number of filter units

The filter media may consist of sand, building sand, or other material such as crushed coral or burnt rice-husk. The media should be clean, rounded and free from clay, loam or organic matter. Details of sand preparation are given in Annex E.

Controls are necessary for the following operations:

o   delivery of raw water;

o   removal of scum from surface;

o   draining supernatant (from above filter);

o   lowering head of water over filter;

o   controlling flow rate;

o   avoiding negative pressure on bed by provision of weir in effluent line;

o   delivery of filtered water to clear well/storage;

o   delivery of filtered or raw water to waste during filter 'ripening' period; and

o   filling of bed from below initially and after cleaning.

Consideration should be given to the following aspects of design:

o   raw water quality and necessity for pre-treatment and/or aeration;

o   necessity for chlorination and clear water storage, pumping and distribution;

o   site location, foundation conditions, space for expansion and pretreatment;

o   availability of source of filter media, and construction materials;

o   office, chlorination room, operator wash room/ toilets and other quarters;

o   fencing and security;

o   provision for cleaning/washing media on a regular basis;

o   high ground water levels producing uplift; and

o   mode of operation - continuous over 24 hours or declining rate over 1 or 2 shifts (8 or 16 hours).

Declining rate filtration is when the operator closes the raw water inlet valve and shuts down the raw water pumps (if any) at the end of his shift, but keeps the filter outlet valve open. The supernatant water will continue to pass through the filter at a declining rate. The longer the period of declining rate filtration, the larger will be the required filter bed area. The required filter bed area (A) may be calculated from the following formula (Ref 14):

$$A = \frac{Q}{0.1a + b}$$

where,

Q =   water demand (m^3/d)

a =   no. of hours per day at normal operation (at rate of 0.1 m/h)

b =   0.5 for 8 hours declining rate filtration

or   b =   0.7 for 16 hours declining rate filtration.

Depending on the method of construction to be adopted, such as:

o   reinforced concrete, mass concrete;

o   stone masonry;

o   brick wall and plaster lining; or

o   masonry on puddled clay embankment,

measures should be taken to prevent short-circuiting down the walls of the filter box, e.g. for a reinforced concrete box, the inner walls should be roughened over the filter bed depth.

-81-

Various types of filter bottoms are possible, having the objectives of supporting the filter media and draining off the filtered water. The openings should be fine enough to prevent escape of the filter media, but not too fine to resist passage of the filtered water and cause too much head loss. Figure 5.10 shows some typical types and sizes for the gravel support and under-drain system.

For systems using pre-fabricated concrete slabs, bricks or porous concrete, it is generally unnecessary to calculate the hydraulic characteristics, since the area of free flow is relatively large (i.e. resistance is small). Detailed calculations for under-drain systems are given in Annex F. For systems using perforated pipes or gravel/broken stones, the following criteria may be used:

Perforated pipes:

- max. velocity in laterals and manifold 0.3 m/s

- spacing of laterals 1.5 m (1 - 2 m)

- size of holes in laterals 3 mm (2 - 4 mm)

- spacing of holes in laterals 0.15 m (.1 - .3 m)

Gravel or broken stones:

- height of layer 0.15 m

- size of gravel 25 - 50 mm

Typical inlet and outlet arrangements are shown in Figure 5.20.

Some aspects of design which are often overlooked are as follows:

o   inappropriate or missing flow rate control devices, filter overloaded or operated at frequent flow rate changes.

o   water level in effluent line lower than top level of sand bed - possibility of negative pressures in bed, air release and additional filter resistance.

o   no provision to fill the sand bed from bottom to top, causing air binding in bed resulting in high filter resistance initially.

o   inappropriate sand size or bed depth, leading to poor effluent quality (coarse sand, small depth) or short filter runs requiring frequent filter cleaning (sand too fine).

-82-

A. 5 TO 10mm

STANDARD BRICKS

CROSS SECTION   LONGITUDINAL SECTION

B. PRECAST CONCRETE SLABS LAID WITH OPEN JOINTS ON PRECAST CONCRETE RIBS.

C. PRECAST CONCRETE BLOCKS WITH HOLES IN THE TOP

D. POROUS CONCRETE POURED IN SITU ON RETRACTABLE STEEL FORMS.

E.

E.

STANDARD CONCRETE TILES ON QUARTERS OF TILES.

TOP VIEW

F. LATERALS (PERFORATED PVC PIPES)

G. GRAVEL 40-100mm

H.
FILTER SAND 0.2-0.3mm

COARSE SAND 1-1.4mm   100mm

GRAVEL 4-5.6mm   100mm   DETAIL OF FILTER SUPPORT

GRAVEL 16-23mm   150mm

ALTERNATIVE UNDERDRAIN AND
FILTER SUPPORT ARRANGEMENTS

FIG. 5.19
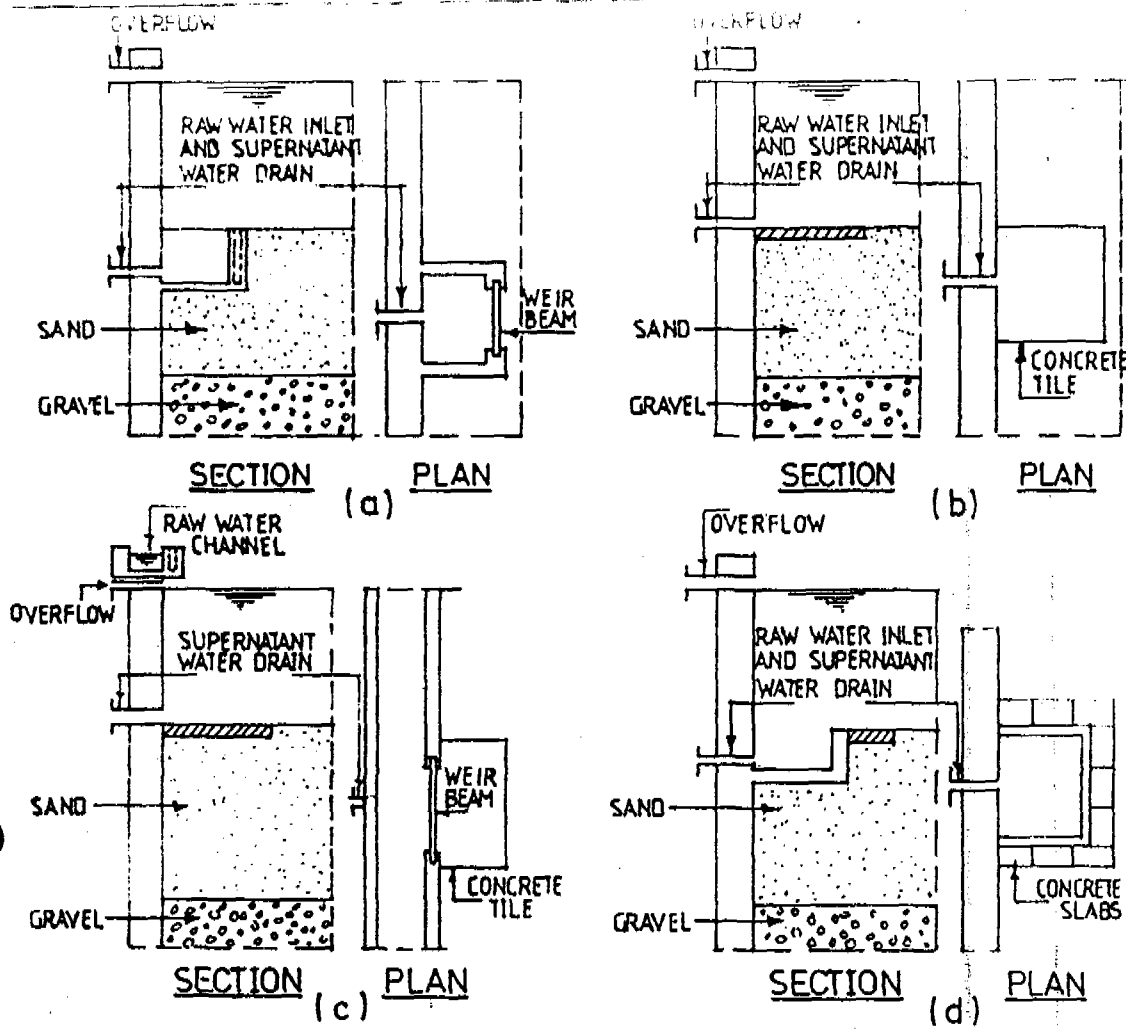
INLET STRUCTURES
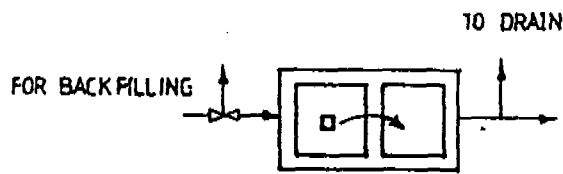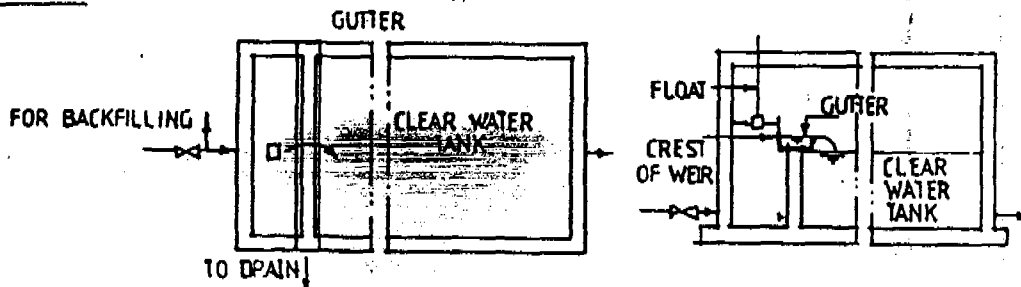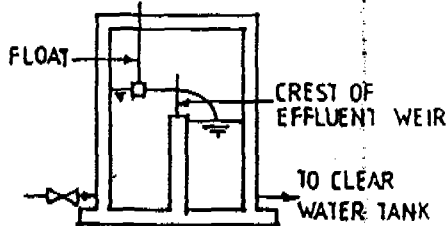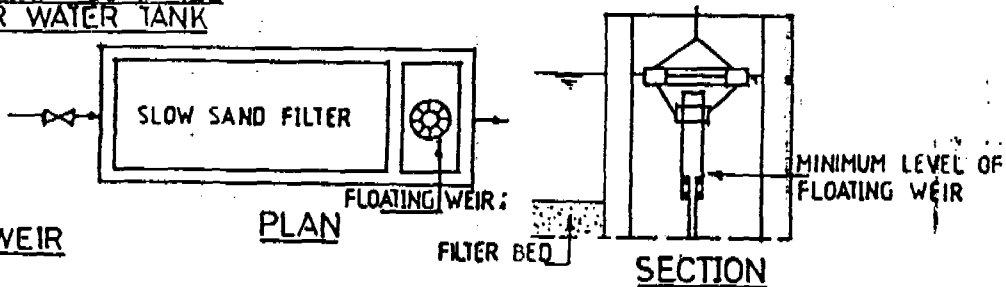
OUTLET STRUCTURES

FIGURE .5.20

e) Pre-treatment Methods

As previously noted, pre-treatment of the raw water is
often required to prevent rapid clogging and frequent
cleaning (i.e. more often than once every 2 - 3
months). Various methods are as follows:

o   Infiltration gallery or trench;

o   Sedimentation tank/lagoon;

o   Horizontal-flow roughing filtration; and

o   Aeration (for standing waters).

The objective should be to reduce the turbidity of the
raw water to an average value of 5 TU or less, and/or
to remove high algae populations.

f) Operation and Maintenance

A SSF may be operated continuously for 24 hours/day or
in declining rate mode for a period of say, 8 or at
most, 16 hours per day. Intermittent operation is NOT
recommended and the water level should never be drawn
down below the media surface, except for cleaning.

Initial charging of the filter bed (and also subsequent
refilling) must be done from bottom to top, preferably
with clean water to drive out air bubbles. When the
water level is far enough above the sand bed to avoid
scouring, raw water may be allowed to enter through the
inlet. When the design water level is reached, the
drain valve is opened and water run to waste, regulated
to about ¼ normal filtration rate, for a period of a
week or two, to allow formation of the " smutzdecke."
During this period, water quality testing is done
periodically, and the flow rate gradually increased to
design rate, until such time as the testing confirms
that the filter is operating satisfactorily. The drain
valve is then closed, and water passed to the clear
water tank.

After a period of operation, as the filter resistance
increases it is necessary to gradually open the
regulating valve to maintain the flow rate, as measured
by the discharge V-notch weir or flowmeter. When the
valve is fully open, and the required flow rate cannot
be maintained, cleaning of the bed is necessary and the
filter must be taken out of service, temporarily. The
raw water inlet valve is closed, and filtration
continued until the water level has dropped nearly to
the sand surface; then the treated water outlet valve
is closed, and the water level drained down to about 20
cm below the sand surface, to permit walking in the
sand.

-85-

Cleaning consists of scraping off, with flat shovels, the top layer of filter surface, about 1.0 - 2.5 cm thick, of dirty sand. The scraped-off material is piled in ridges or heaps, and carted away in wheel barrows/ hand carts wheeled over wooden planks. The dirty sand may be cleaned by washing soon after removal or discarded. The filter is then recharged from bottom to top, and allowed to run to waste for 1 - 2 days while the "smutzdecke" regains its biological activity.

When many such cleanings have been done and the minimum bed thickness (0.7 m) has been reached, the sand bed must be raised to its original thickness by bringing in new sand. However, the top 0.3 - 0.5 m of old sand should be removed, and replaced on top of the new sand, to accelerate the re-ripening process.

When one filter is temporarily out of operation for cleaning or resanding the filtration rate of the other filter units should be increased to take the additional loading, although the maximum rate should not exceed 0.3 m/h.

g) Experience in Sri Lanka

Common operational problems are as follows:

o turbidity and suspended solids concentration in the raw water too high for SSF application. Pre-treatment is necessary in such cases.

o missing maintenance equipment such as shovels, wheel-barrows, tools and sand washing installations. Failing to clean and replace the sand will lead to exhaustion and failure.

o untrained caretakers who do not understand the SSF process and are not motivated to operate the plant properly, leading to inadequate conditioning, clogging, uncontrolled filtration rate, ineffective bacteria removal etc.

o problems due to algae - in minor cases this can be dealt with by pre-treatment.

o insufficient use of comparative physical and bacteriological testing of raw and filtered water to evaluate filter performance.

h) Monitoring/Evaluation

Provision should be made for flow measurement at each filter, and for discharge of filtered water from the whole plant. Regular water quality testing, physical, chemical and bacteriological, of raw water and filtered water, must be carried out to ensure that the filter is operating satisfactorily. Complete records should be kept of flow rates and cleaning/resanding operations on each filter unit, so the long-term evaluation can be carried out.   -86-

a)   Introduction

This type of filter has been developed and promoted in the past few years by Martin Wegelin, a Swiss sanitary engineer, as a pre-treatment to slow sand filtration (SSF), being an alternative to conventional flocculation/sedimentation. One of the problems encountered with SSF is that rapid clogging can occur when raw water turbidity increases substantially for short periods during heavy rains. In other cases, a SSF may clog frequently when the turbidity and suspended solids concentration in the raw water is too high for SSF application. For SSF operation, influent turbidity/SSF should preferably be lower than 10 turbidity units/2-5 mg/L, and in most cases some form of pre-treatment will be required to achieve this. Laboratory and field testing in Switzerland, Tanzania and Thailand has shown that HRF provides a simple pre-treatment solution, appropriate for rural and small urban schemes.

In Sri Lanka, the process is currently being evaluated in the Ukuwela plant in Matale District (the first HRF/ SSF plant in Sri Lanka), designed by Kampsax-Kruger and completed in 1987; design engineers should refer to the monitoring/evaluating report on this plant prepared by Kampsax-Kruger in 1988, in addition to the references cited.

The technology of HRF is still under development and filter efficiency for different raw water characteristics has yet to be established. Experience to date indicates that the filter is effective with raw water turbidities normally between 50 and 200 turbidity units, although indications are that much higher turibidities, up to 1,000 turbidity units, can be effectively handled for short periods, corresponding to short-term high turbidity periods experienced in many small streams in Sri Lanka.

If the bacteriological quality of the raw water is satisfactory, it is feasible for HRF to be used on its own, without SSF.

b)   **Filtration Mechanisms**

The main process is sedimentation, with horizontal flow and gravitational downward drift, causing particles of silt to collect on the upper surfaces of the media. As heaps of particles accumulate, they slip down and drift towards the filter bottom, and over a period of time the filter will fill from bottom to top with separated solids.

c)   Features

The filter box comprises 2 to 4 compartments filled
with gravel media of different sizes, coarsest in the
first compartment where the bulk of solids will be
retained, to finest in the last compartment which acts
as as polisher, removing the last traces of solids.

The raw water enters via an inlet channel across the
filter width and falls over a weir into an inlet
chamber which allows for settlement of coarse solids
and separation of floating material. Water passes into
the first and subsequent filter compartments through
openings in the separation walls. Treated water is
collected in an outlet chamber and discharged over a
weir (for flow measurement) to an outlet channel and
then to the SSF.

The length of the filter box depends on raw water
quality, hydraulic loading and media size. Height
should be limited to about 1.5 m to allow for easy
removal of media for manual cleaning. The width
depends on the required filter capacity. A minimum of
2 HRF units should be provided, to allow for one being
out of service for cleaning, and an adjacent area for
gravel cleaning should be provided.

A layout of a typical HRF is shown in Figure 5.21.

d)   Design

Input requirements:

o   characteristics of the raw water quality in the dry
     and rainy seasons;

          — turbidity;
          — suspended solids or filtrability;
          — E – Coli;

o   daily water demand (present & future);

o   raw water intake site location, elevation, high/low
     flows and and water levels;

o   treatment plant site, elevation, foundation
     conditions;

o   hours of operation of intake, HRF, SSF and distri-
     bution;

o   availability of raw materials – gravel, sand, rock;

o   requirements for pre-sedimentation with or without
     combined raw water balancing tank.

VIEW

PLAN

SECTION A-A

SECTION B-B

W.O = WASHOUT
O.F = OVERFLOW
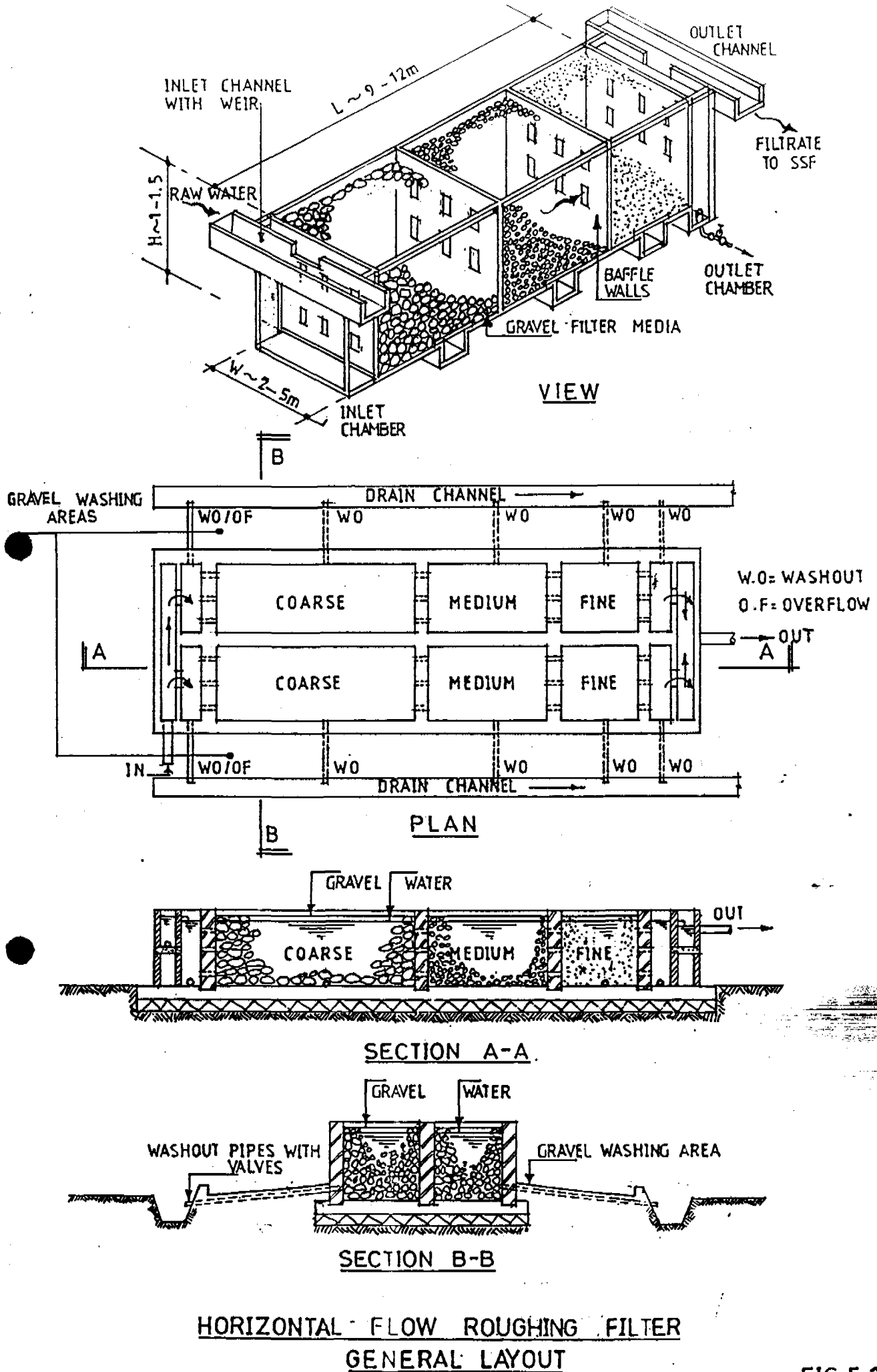
HORIZONTAL FLOW ROUGHING FILTER
GENERAL LAYOUT

FIG. 5.21

-89-

Design criteria:

Filtration rate (m/h) based on X-sectional area of filter);

Filter box: -

                   - length 9-12   m
                   - height 1-1.5 m
                   - width   2-5 m per unit

Media size: -

                   - coarsest 35-40 mm
                   - finest 4 mm;

Length of compartments (see Table 5.2)

Required effluent quality for SSF:

                        10 turbidity units
                        2-5 mg/L SS;

Openings in compartment walls: 10-20% of X- sectional area, evenly distributed.

Other criteria, such as expected efficiency of SS removal, length of filter runs, required maximum head loss, etc. may be estimated by means of a nomogram (Reference 17), if necessary.

To allow for head loss across the filter, which builds up as the length of filter run increases and the filter loads up with silt; the outlet weir level should be 30-40 cm below the top level of filter material (see Figure 5.22).

Filter media may be an inert material such as: . .

        o   gravel;

        o   broken stones/rocks;

        o   broken burnt bricks;

        o   burnt charcoal;   (prior testing at pilot
        o   coconut fibre   plant level is necessary)

Table 5.2

Tentative Design Guidelines for HRF

Maximum suspended solids concentration in presettled water (mg/L)	> 300 high	300 - 100 medium	100 low
Filtration rate m/h	0.5	0.75 - 1	1 - 1.5
Filter length (m) for medium size (mm) 20 mm 15 mm 10 mm 5 mm	3 - 5 2 - 5 2 - 4 1 - 2	3* 2 - 4 2 - 3 1 - 2	3* 2 - 3 2 1
Maximum suspended solids concentration in HRF effluent (mg/L)	5	2 - 3	2

* this gravel size can possibly be omitted.
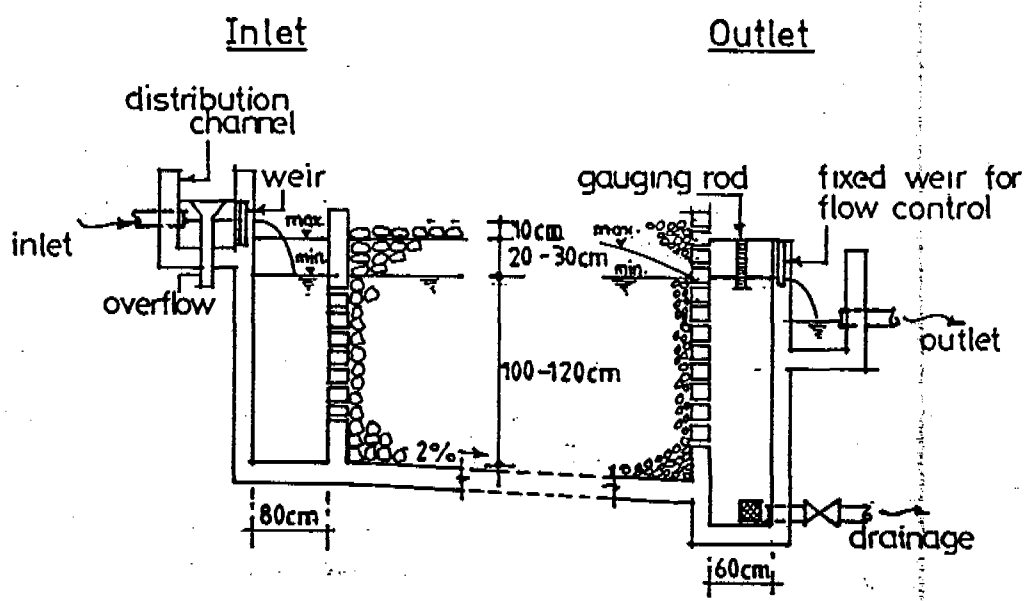
INLET AND OUTLET STRUCTURES OF HRF

FIG.5.22

Typical inlet/outlet arrangements are shown in Figure 5.22 and drainage arrangements in Figure 5.23. Adequate provision of drainage is necessary for filter regeneration (by rapid drawdown at 60-90 m/h, and for emptying for cleaning purposes. Discharge should be to an open drain and arrangements for safe disposal should be provided to avoid erosion and scour. Typical mechanical devices to ensure a constant outflow rate are shown in Figure 5.24.

Owing to the fact that a HRF requires continuous operation, the HRS should also be operated in continuous mode, although the physical nature of the process does not require it. When a SSF is operated, say, for 8 hours overnight in declining rate mode (due, for example, to raw water pumping for 16 h/day in 2 shifts) then the HRF is required to work intermittently. Consideration should be given to providing raw water balancing storage to allow 24-hour plant operation, and therefore smaller HRF and SSF units (see Figure 5.25).

Adequate arrangements should be provided in the design for monitoring/evaluation of flow and water quality (see Section 5.7 (f)).

e)   Operation

Each gravel compartment gradually becomes loaded with deposits, until filter efficiency is exhausted, which, under ideal HRF design should occur in each compartment at the same time.

Occasional lowering of the water table by drainage will increase efficiency and prolong filter runs. During drainage, particle accumulations on the top of the filter media grains tend to collapse and get flushed to the filter bottom. Rapid drainage, if possible, should also flush some accumulated matter from the filter.

Drainage should start at the drain nearest the inlet side, to prevent the bulk of deposited material being drawn into the finer media, thereby clogging it. When the washwater is still turbid after drainage, the same drain should be operated again, after slow filling, until the washwater is relatively clear. Then the next drain from the inlet should be operated, and so on. It is important that refilling, after drainage, is slow so as to avoid stirring up settled material.

A HRF should not be kept dry, unless the media is thoroughly cleaned.

a) LOCATION OF DRAINS
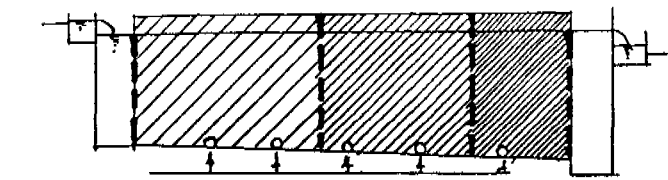
DRAINAGE POINTS SPACING 150-200cm

HOLES 6mmⲫ SPACING 100mm

GRAVEL PACK FOR SMALL AGGREGATES

FLOOR OF FILTER BOX

150mm
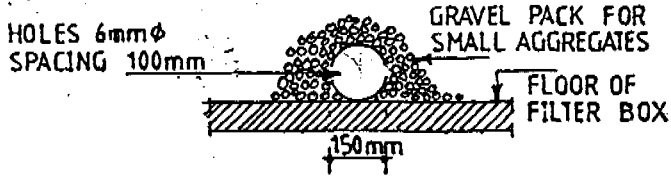
b) PERFORATED PIPE WITH VALVE

FLOOR OF FILTER BOX

300mm

300mm
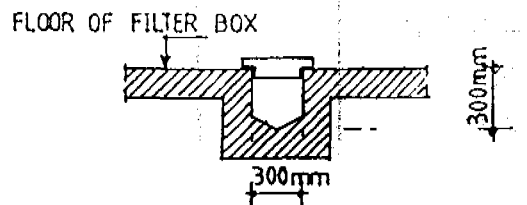
c) SEMI COVERED TROUGH WITH SLIDE GATE

500mm

100mm
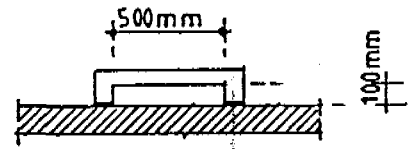
d) PRE FABRICATED CULVERT WITH SLIDE GATE

DRAINAGE SYSTEMS

FIG.5.23

FILTER   OUTLET
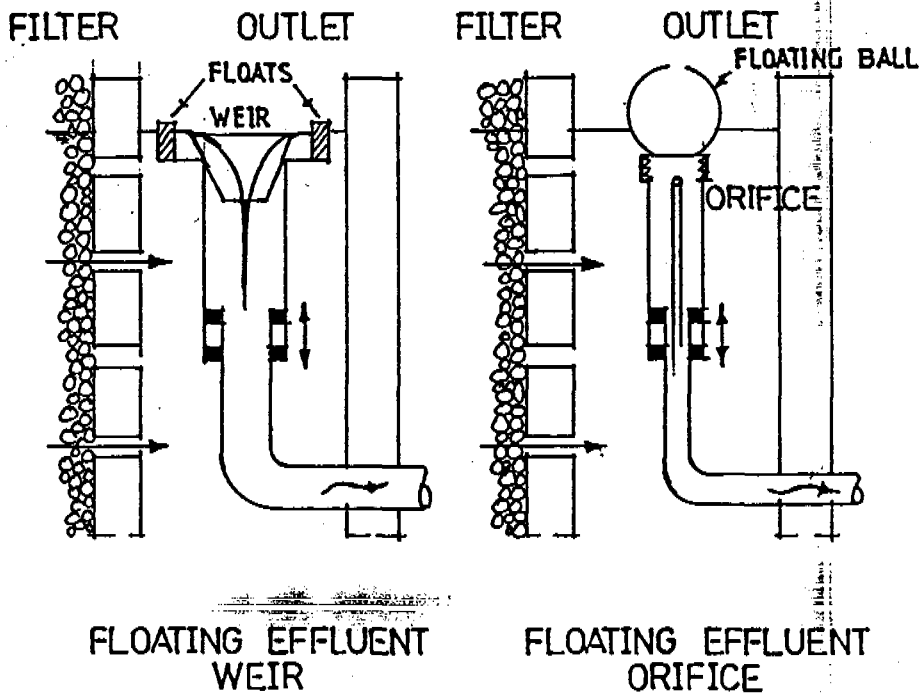
FLOATS
WEIR

FLOATING EFFLUENT WEIR

FILTER   OUTLET

FLOATING BALL

ORIFICE

FLOATING EFFLUENT ORIFICE

MECHANICAL FLOW RATE DEVICES

FIG.5.24

Sed. Tank Res. (raw)	HRF	SSF	Res. (clear)	Total Volume

GRAVITY SCHEME

Continuous operation 24hrs/d

8	42	52	50	152 m³
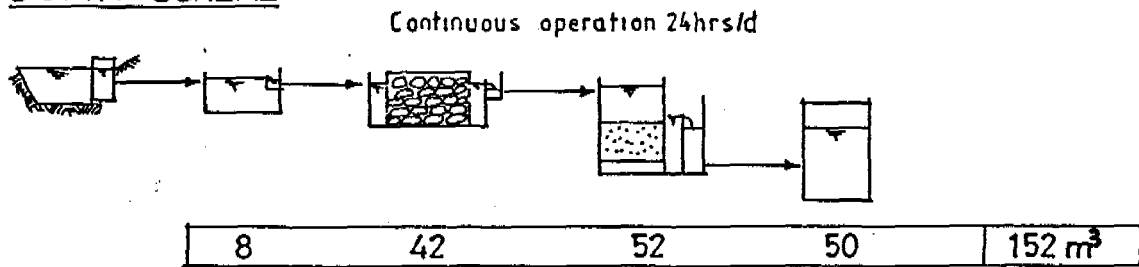
PUMPED SCHEME WITHOUT RAW WATER TANK

intermittent operation 8hrs/d

contin. operation 8hrs. declining " 16hrs.

25	125	109	50	309 m³
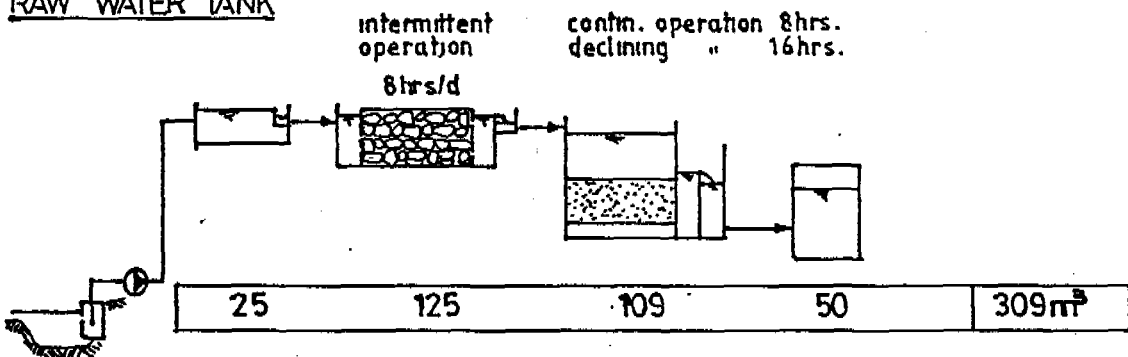
PUMPED SCHEME WITH RAW WATER TANK

continuous operation 24 hrs/day

67	42	52	50	211 m³
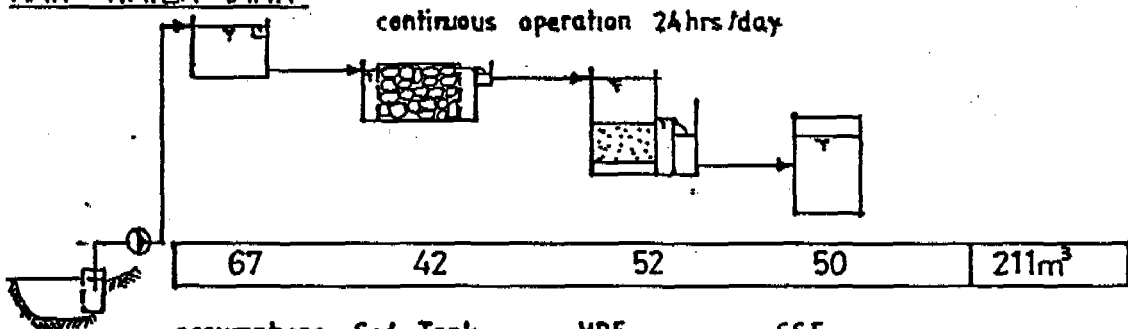
assumptions Sed. Tank $T_d = 2$ hrs.

HRF $V_F = 1$ m/h $L_{tot} = 10$ m

SSF $V_F = 0.2$ m/h $H_{tot} = 2.5$ m

EFFECT OF INTERMITTENT OPERATION ON SIZE OF FILTERS.

FIG.5.25

Filter runs may last 6 months or more, depending on the operating conditions and design, and when the efficiency can no longer be restored by drainage, cleaning must be done. The media must be excavated, washed, and returned to the filter box. This may be done by hand, using locally hired labour, and a media cleaning area should be provided adjacent to the filter. A typical gravel cleaning unit is shown in Figure 5.26. Equipment used in the construction stage, such as wheel-barrows, shovels, etc., should be retained at the plant for media washing purposes during operation.

f) Monitoring/Evaluation

Any new HRFs designed by or for NWSDB should be equipped with adequate flow measuring devices for each filter and a comprehensive program to monitor/evaluate performance over the first year of operation should be carried out.

This should involve the following measurements:

Frequency	:	Ideal	Minimum
Turbidity		Twice/day	Once/day
Suspended Solids	:	Twice/month	Once/month
Faecal Coliforms	:	Twice/month	Once/month
Total Coliforms	:	Twice/month	Once/month

at the following collection points (see Figure 5.27)

1. Inlet	:	Turbidity; SS, FC, TC;
2. Outlet (each filter)	:	Turbidity, FC, TC,
3. Combined outlet	:	Turbidity, SS; and
4. SSF outlet	:	Turbidity, SS, FC, TC.

Arrangements should be made to enable this sampling/monitoring to be done by the plant OIC with RO/RSC Chemist assistance.

washwater   cleaning   drainage   storage
supply   box   channel   place

400

SECTION

15
60
60
15

PLAN

10
20
20
15

1ˢᵗ box for primary cleaning

2ⁿᵈ box for final cleaning

approx. dimensions in cm.

15 60 15 60 15

CROSS SECTION

(a) WASHING

stirring with shovel

drilled steel plates as seives

wooden logs as support

fixed seive installation

wooden frame work

seiving on chains.

swinging seive installation

(b) SEIVING

TYPICAL GRAVEL WASHING AND SEIVING INSTALLATIONS.

FIG.5.26

-97

permanent installation

1/4" pipe, stop cock, flexible tube & clamp.

60

20

provisional installation

flexible tube, resistant to heat and sunlight

60

20
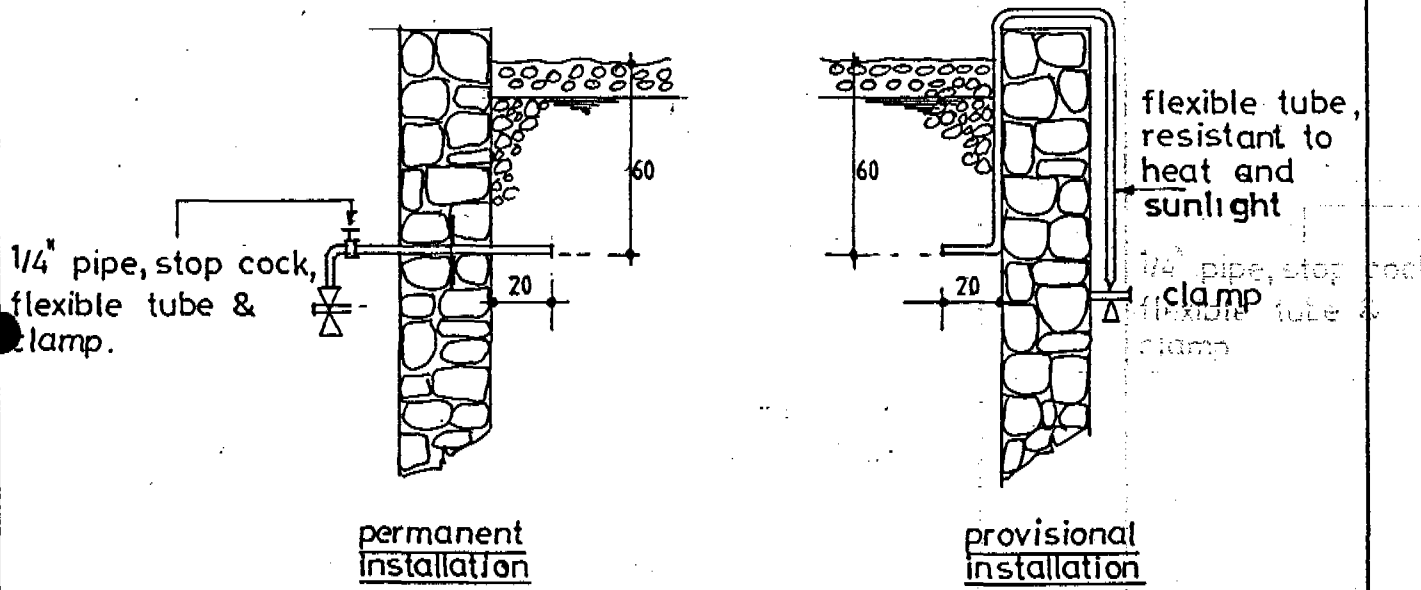
WATER SAMPLING POINTS

FIG.5.27

5.8   DISINFECTION

Chlorine is usually the most satisfactory disinfectant for water supplies, although in situations where cheap electricity is available, ozone treatment may be competitive particularly if colour removal is also required.

Chlorine is a gas at normal temperatures and pressures but is normally supplied as a liquid under pressure of 330-1000 kN/m^2 in cylinders or drums. The rate of withdrawal of chlorine gas is limited by the heat available for evaporation of the liquid.

Solubility of chlorine gas in water varies with temperature:

Temperature °C	Chlorine Solubility
0	14 600 mg/L
20	7 000
33	5 000

The physiological effects of chlorine gas are summarized below:

Concentration by Volume in Air mg/L	Physiological Effect
3.5	Just detectable by odour
30	Produces coughing
40-60	Dangerous for 30 min exposure
1000	Rapidly fatal

Hypochlorite solution may be useful for small installations since it avoids the use of toxic gas. It is available as:

o   High test hypochlorite $Ca(OCl)_2$

o   Bleaching powder $CaOCl_2$

Hypochlorite tablets may be used for emergency disinfection in times of disaster when normal water treatment is not operating.

The main purpose of disinfection by chlorine is to protect health by destroying pathogens in the water and to maintain a minimum chlorine residual in the distribution sytem to provide some additional protection against subsequent contamination. The following minimum free chlorine residuals are recommended at different locations in the water system:

        Clear water/chlorine contact tank   - 0.5 to 0.8 mg/L
        Standposts   - 0.2 mg/L
        Distribution dead-ends   - 0.1 mg/L

Other uses of chlorine are shown in Table 5.3 and include oxidation of iron, manganese and hydrogen sulphide, destruction of taste and odour causing compounds, and control of algae and slime growth at treatment plants.

Chlorine capacity, dosage and contact time: The required capacity of the chlorinator will vary with the chlorine dosage needed for disinfection and the amount of residual to be maintained in the distribution system. Dosage will vary with the chemical and physical characteristics of the water (i.e. temperature, pH, turbidity, and mineral constituents such as iron and manganese). It also varies with the nature and concentration of pathogens in the water and the length of contact time. For design purposes, a chlorine dosage of 2 mg/L may be used for clear water (i.e. turbidity < 10 TU containing little organic and suspended matter, and with iron and manganese concentrations of less than 0.3 mg/L. The total chlorine requirements, recognizing all the above variables, can and should be determined by performing a chlorine demand test as a part of the predesign investigations. A minimum contact time of 20 minutes should be provided either in the pump sump, reservoir and/or in the pipeline before reaching the first consumer.

Mixing: Rapid mixing of chlorine at the point of application is important. Chlorine should be added with a large amount of dilution water through a full channel or pipe diameter diffuser submerged to the maximum depth available.

Chlorine applied prior to a weir will only be driven off by the aeration effect of the hydraulic jump. Chlorine added to the suction side of a pump or the upstream side of a valve can cause severe corrosion problems to brass unless applied a sufficient distance upstream to obtain full dissolution.

Gas Feeders: Chlorination with gas feeders may be by direct-feed pressure injection of gas metered under a constant positive pressure or by solution feed in which the gas is metered through an orifice or rotameter under vacuum by an ejector drawing gas through the feeder into the solution line.

The water supply to operate the injector should have adequate pressure and flow. The chlorinator manufacturer's injection curves should be consulted. Gas chlorinators have practical limitations as to minimum feed rates. Consistent with good accuracy of metering and minimum maintenance, the minimum feed rate is about 0.25 kg/day. However, chlorinators with rotameters can be obtained with as low a capacity as 0.05 kg/day. Normally for chlorinators up to 4.5 kg/day capacity, injectors require water at a minimum pressure of 0.8-1.33 kg/cm^2 above the pressure at the point of application with a minimum flow rate of 9 - 22.5 L/m. Water from the pumping main or from a storage tank can usually be used to operate injectors. Booster pumps for injector supply are required when chlorine solution has to be injected into a pressure main.

Table 5.3
Chlorine Use for Various Treatment
Other than Disinfection

Treatment	Dosage	Contact Time	pH Range	Chlorine Residual Recommended		Amount of Alkalinity Consumed as Ca3
				Type	(mg/L)	
1. AMMONIA REMOVAL						
a. Monochloramine formation	5 times NH$_3$(as mg/L)	> 20 min	> 8.5	Free	0.1	1.22 per mg/L Cl$_2$
b. Dichloramine formation	10 times NH$_3$(as mg/L)	> 20 min	> 4.4-5.0	Free	0.1	
2. TASTE & ODOUR CONTROL	10 times NH$_3$(as mg/L) plus 1 to 5mg/L	> 20 min		Free	1.0	1.22 per mg/L Cl$_2$
a. Phenol destruction	6 to 20 times phenol content	1.5 to 5 hours (stepwise)	7.0	Free	Variable	
3. HYDROGEN SULPHIDE REMOVAL						
a. Oxidation to sulphur	2.2 times H$_2$S content	Instant	5-9 (9 optimum)	Free or combined	0.1	2.6 per mg/L H$_2$S
b. Oxidation to sulphate	8.5 times H$_2$S content	Instant	6.5-9 (9 optimum)	Free or combined		10 per mg/L H$_2$S
4. MANGANESE REMOVAL	1.3 times Mn content	up to 3 hr (variable)	7-10 (10 optimum)	Free	0.5	3.4 per mg/L Mn oxidized
5. IRON REMOVAL	0.64 times Fe content	Instant	4-10 (> 7.7 optimum)	Free or combined	0.1	0.9 per mg/L Fe oxidized
6. ALGAE CONTROL	1-10	Variable		Free	> 0.5	
7. SLIME CONTROL	.1-10	Maintain residual throughout distribution system		Free	0.5	
8. IRON & SULPHUR BACTERIA CONTROL	1-10	-do-		Free	1.0	
9. COAGULANT AID a. Chlorinated Copperas	1 part per 7.8 parts of ferrous sulphate					

The following points should be considered when designing gas chlorination systems:

o   In order to save space and for ease of maintenance, wall mounted chlorinators are recommended. Two units should be installed to provide standby capacity, and to maintain flow during changeover of cylinders or drums.

o   A weighing balance should be provided in the chlorinator room for checking the quantity of gas used.

o   The chlorinator room should be separate with no interconnecting doors to any adjacent room. The floor level should be at or below the other floors of the plant to minimize the effects of gas leaks.

o   Air outlets from the room shall be near the floor and air inlets near the ceiling. Where this is not possible, mechanical ventilation with a capacity of one complete air change per minute should be installed. Fan and light switches should be located outside the room. Doors should open to the exterior of the building. Careful attention should be paid to location of windows and doors to keep cylinders out of the sun.

o   The chlorination room should contain only the connected cylinders. Separate cylinder storage facilities should be provided. All cylinders, connected or stored, should be protected from direct sunlight, rain or other water and be properly restrained to prevent tipping, falling, or any excessive movement, especially the connected cylinders. A guide to cylinder requirements and storage is given in Table 5.4.

o   Chlorine gas and chlorine solutions are highly corrosive and should be conveyed in plastic pipelines.

o   Gas masks must be provided adjacent to the chlorinator room door.

o   A neutralizing chemical for chlorine absorption should be stored at the plant. Table 5.5 shows possible chemicals which could be used with the amount of water required to make a neutralizing solution. A pool for the disposal of leaking cylinders should be located adjacent to the chlorination room.

Hypochlorite Feeders

For small installations a drip feed from a constant head bottle may be satisfactory but for larger supplies, more accurate metering must be provided either by an orifice with a gravity feed tank or by a positive displacement proportioning pump.

Chlorine Cylinder Inventory
(Minimum Supply for 30 days for 2 mg/L
Dosage for 8 to 20 hours daily operation)

Anticipated Daily Chlorine Use (kg/day)	Volume of Water Treated Daily (m^3)	No. of chlorine cylinders			
		30 kg size		67 kg size	
		In Use	Reserve	In Use	Reserve
0.50	250	1	1	1	1
1.00	500	1	1	1	1
2.00	1000	1	1	1	1
4.00	2000	1(2*)	3(2*)	1	1
6.00	3000	1(2*)	5(4*)	1	2
8.00	4000	1(3*)	7(5*)	1(2*)	3(2*)
10.00	5000	1(3*)	9(7*)	1(2*)	4(3*)

(*) For 8 hrs/day operation

Table 5.5

Chemicals and Water Requirements for
Chlorine Absorption in Alkaline Solution

Cylinder Capacity	100% Caustic Soda (kg)	Water (L)	Soda Ash (kg)	Water (L)	Hydrated Lime (kg)	Water (L)
30 kg	40	150	96	265	40	340
67 kg	85	225	205	565	85	720

Although bleaching powder (chloride of lime) normally contains 25-33% available chlorine, for design purposes the lower figure is recommended. Solutions up to 5% may be prepared but a 1% solution is recommended to minimize problems caused by sediment and clogging of the delivery system. A separate mixing tank should be provided with only the supernatant transferred to the feed tank. Feeding equipment, generally of the constant head gravity drip feed type, should be readily dismantleable for cleaning purposes. In general, it can be considered that the practical limit for hypochlorination is up to about 900 L/m of gravity or pumped supply. For greater flows, gas chlorination is appropriate. Sufficient shelf space should be provided to store bleaching powder containers.

A practical guide for disinfection by bleaching powder produced by CEA, is given in Annex G. Table 5.6 gives bleaching powder requirements for disinfection of mains and storage tanks.

Contact Tank

To provide sufficient time for disinfection, a chlorine contact tank (which may be the filter clear well) must be provided to give at least 20 min detention. The tank should be baffled to prevent short circuiting.

Chlorine solution should be added through a diffuser at the inlet preferably in turbulent conditions.

Prechlorination

The purposes are as follows:

o   to reduce taste and odour producing substances;

o   to prevent algal growth in raw water;

o   for oxidation of iron, manganese and hydrogen sulphide; and

o   to aid coagulation when high algal counts exists.

To minimise post chlorination dosage, the total dosage (usually 2 to 5 mg/L depending on the degree of pollution) is applied to leave 0.2 to 0.5 mg/L free residual chlorine in the final delivered water.

For further information, see Reference 19.

Table 5.6

Bleaching Powder Requirements
for Disinfection of Mains and Storage Tanks

(Based on a Dose of 50 mg/L available Cl)

1. Pipes	Amount of Bleaching Powder Required
mm	kg/1000 m
50	0.5
65	0.8
80	1.2
100	1.9
150	4.3
200	7.4
2. Storage Tanks	2.4 kg/10m^3

6.   CHEMICAL FEEDING

The chemicals commonly used in water treatment are for:

o   Coagulation;

o   Disinfection and algal control;

o   pH adjustment; and

o   Special chemical treatment e.g. softening, fluoridation.

The types of feeding devices are:

o   Dry feed; or

o   Solution feed

The strength of solution used for manual feed generally is less than 5% and for mechanical feed it is not greater than 10%.

Alum, usually in lump form, is the most common coagulant. The dosage is in the range of 20-100mg/L. Lime is added when pH and alkalinity are low. The required dosages are determined through jar tests.   The density of alum and lime are 980 kg/m^3 and 670 kg/m^3 respectively.

6.1   CHEMICAL PREPARATION

It is essential to know the strength of the chemical solution in order to know the dosage being applied. All chemical preparation tanks therefore should be carefully measured and volumes calculated.   A measuring stick can be used to give the volume at any depth.

For the preparation of coagulant solution, for example, the exact amount of dry alum cake or ferric chloride and the exact amount of water should be known. In most plants a 10% solution in the day tanks will be satisfactory. Day tanks are those which supply the originally mixed chemical and water. A 10% solution requires 100gm of chemical and 900 gm of water (10% of the total is chemical). The strength then is 100,000 mg of coagulant in each litre of solution, which is a relatively strong solution and very stable. The material is weighed at the desired rate into a solution tank where it is dissolved by the stirring action of propellor type mixing. The alum solution is slightly acidic.

It is important that the dilution water have the lowest pH possible so if lime is being added, this water should be pumped from the filter outlet before the addition of lime. Water from the clear well after addition of lime has a higher pH.

Ferric chloride is a highly acidic compound and must be shipped, stored and dosed in corrosion-resistant equipment.

It is convenient to have each tank supply enough chemical for 24 hours. This makes the operation simpler each day a tank of chemical is prepared.

6.2 CHEMICAL DOSING

Once the chemical solution has been prepared it is necessary to pipe it to the point of application, such as a weir or a Parshall flume.

Dilution of the stock solution from the day tanks should be done as near as possible to the diffuser. The strength of the solution being applied to the raw water should be about 1% or as low as 0.5%. To reduce the 10% solution to 1% requires 100 mL of the stock solution and 900mL of water to make up one litre.

The dosage may be based on the amount of the 10% strength stock solution. If the raw water flow is $10m^3$/min and the required dose is 10mg/L (or 10ppm) it is necessary to apply 100g of chemical to the water each minute. In a 10% strength solution this means one litre of stock solution per minute.

The total solution dosed, however, is much more. If for example, 100mL of stock coagulant is diluted with 900mL of water just before application then the amount applied is 10 L/min at a strength of 1% which is satisfactory for good coagulation.

The system at each plant should be to mix stock solution of 2 to 10% strength and dilute to 1% or less at the diffuser. This will give good coagulation and economize on coagulant.

A frequent problem is that of getting a known amount of stock solution into the raw water. Most existing plants have some kind of dosing pumps which are not usually working properly. It is strongly recommended that if possible the chemicals be applied by gravity through a constant head tank, controlled by a float valve. The system should be reasonably accurate and it should be easy to set a constant dosing rate.

Such dosing systems exist on the market, but it would be preferable for NWSDB to fabricate its own with valves and small tanks purchased specifically for the purpose.

The chemical transport from the day tanks to the point of application should provide a constant dosing flow, preferably by a gravity feed system.

7.   OTHER PROCESSES

7.1   IRON AND MANGANESE CONTROL

Iron and manganese are both undesirable constituents of water, causing the following effects:

o   staining of clothes, plumbing fixtures;

o   causing objectionable metallic tastes;

o   imparting colour and turbidity; and

o   leaving deposits in pipelines.

The problems of iron and manganese are usually confined to ground waters, though they are sometimes present in anerobic surface waters or where industrial wastes are discharged.

Iron and manganese are natural constituents of soil and rocks. Normally present in highly insoluble forms, they are brought into solution when anaerobic conditions prevail and carbon dioxible is present, such as in a deep well or the bottom of a reservoir. Troublesome conditions are likely to occur when the following concentrations are exceeded:

    0.3 mg/L iron
    0.1 mg/L manganese

Iron exists in solution in the ferrous state, usually as ferrous bicarbonate. It can only remain in solution in the absence of oxygen, and generally where the pH is below 6.5. Exposure of the water to air results in the oxidation of soluble ferrous bicarbonate to insoluble ferric hydroxide. Thus, for example, a deep well sample, on exposure to the atmosphere, may become opalescent and discoloured due to oxidation of the soluble salt to insoluble form, and a deposit formed.

Even small traces of iron and manganese can lead to accumulation of deposits in wells, distribution mains and reservoirs, which may be troublesome and objectionable. Under such conditions, particularly where organic matter is present, iron bacteria may develop which will accentuate the problem and lead to complaints of discoloured, turbid and unpalatable water. Sometimes, sulphides are produced, causing an offensive odour.

If iron is not present in the source water, it is usually acquired to some extent from contact with iron pipes or fittings. To establish whether a deep well water, for example, is acquiring iron through the pump, or casing, it is necessary to obtain a series of samples over several hours of pumping at maximum capacity. Should the iron content decrease significantly as pumping proceeds, it is an indication that iron is being acquired.

Manganese occurs in a similar way to iron, and they may both be present at the same time. Deposits and discolouration due to manganese are black rather than rust-coloured as for iron, and the deposits are of higher density and are less easily flushed out. Manganese oxidises less readily than iron and therefore is more difficult to remove.

Problems due to iron and manganese may be minimised by:

o   treatment for removal;

o   protection of iron/steel mains with bituminous linings, or use of non-corrosive materials;

o   avoiding dead end mains;

o   avoiding disturbances in water flow;

o   periodic flushing.

When iron removal treatment is considered, it is necessary to decide what other treatment is required, since this may have a bearing on the method of iron removal. For example:

o   Iron is usually accompanied by considerable free carbon dioxide — it is inadvisable to remove the iron, yet leave the free CO_2 to cause corrosion of pipes and mains.

o   Lime-softening, if required to remove hardness, will also be effective for removal of iron and carbon dioxide.

o   If organic/bacterial purification is necessary, chlorination may be required, which will also assist with removal of iron and manganese.

Methods for iron/manganese removal are as follows:

a)   <u>Aeration</u>

As noted in Section 5.1, aeration assists in dissipation of free carbon dioxide, and oxidation and precipitation of iron. Free CO_2 should be reduced to less than 10 mg/L to remove corrosive properties. Hydrogen sulphide, if present, should also be dissipated. Generally, if only iron is present (with minimal amounts of manganese), simple aeration followed by filtration should be effective. Oxidation is rapid with a pH of over 7.5. pH adjustment may be necessary to increase the removal efficiency.

-109-

Aeration followed by sedimentation and rapid filtration is required when the amount of iron is high or when manganese is present. Sometimes the addition of chemicals is necessary for removal of high carbon dioxide levels, or if larger amounts of soluble components of iron or manganese are present.

Slow sand filters are effective in removing small concentrations of iron, but will clog rapidly in the presence of large amounts.

b) Chemical treatment with lime

This is the most efficient and economical chemical for removal of free carbon dioxide, iron and manganese, particularly after aeration. The dose should result in a final pH of 7.5 to 8.5 after settlement and filtration. The lime must be thoroughly mixed with the water. Aluminium sulphate (alum) may be used to assist flocculation and sedimentation. For removal of manganese, the pH should be higher, above 9.

c) Other Methods

Potassium permanganate is an effective oxidising agent, particularly for manganese which will precipitate when the pH is above 6.5. Application should be followed by filtration by sand, anthracite or a catalytic filter containing iron or manganese oxides. Pilot plant testing is advised.

Prechlorination is useful for removing growths of iron bacteria, and may be used for oxidation of iron and manganese instead of aeration, but aeration is preferred. Prechlori- nation ahead of conventional treatment (coagulation, sedimentation, filtration) at pH values of 6.7 — 8.4 should provide adequate removal of iron.

Contact beds are useful for smaller quantities of water, with pumice as a medium.

7.2   FLUORIDE REMOVAL AND FLUORIDATION

Fluoride in water has been found to be beneficial in the prevention of tooth decay in children, and is commonly added to water supplies in Europe, North America and many other countries. About 1-1.5 mg/L is the optimum dose. Fluoridation is not commonly practised in developing counties since dental decay is not a widespread problem. In concentrations above 3-5 mg/L, however, mottling of teeth occurs, and a reduction in the level of fluorides is desirable.

Fluorides can be removed by various granular insoluble media which remove fluoride as the water percolates through them and which are periodically regenerated by chemical treatment. Suitable media are:

-110-

o   tricalcium phosphate;

o   bone char;

o   Activated   alumina: probably   best   for   large   scale
    removal,   water percolates through a bed of granular media
    of   grain   size 0.3—0.5 mm.   Bed regeneration is necessary
    with weak caustic soda, followed by neutralization.

    Since   these   processes   are   difficult and expensive, the
    best   solution,   where   alternatives   exist,   would   be to
    abandon the source.

7.3   REMOVAL OF COLOURS, TASTES AND ODOURS

These   characteristics   are   often   the cause of a majority of
consumer complaints about water quality.

Colour in water is usually due to:

o   organic matter;

o   biological growths and algae;

o   colloidal iron/manganese;

True   colour   is that imparted to water by soluble matter.   It
is   measured   against standard solutions of a platinum salt in
distilled   water.   Apparent colour is due to suspended matter
such as ferric hydroxide (red or yellow) or algae (green).

Tastes and odours may be caused by:

o   hydrogen sulphide in the water (mostly odour);

o   contact with bituminous surfaces of mains or tanks;

o   contamination by organic matter;

o   contamination by chemicals;

o   growths of algae, etc.

o   treatment   processes,   e.g.   chlorination,   which may
    accentuate   some   tastes   and   odours, particularly in the
    presence of phenols.

Odour   is   measured by threshhold odour number (TON) which
indicates   the   intensity   of   the   odour in water.   It is the
dilution,   with   odour-free water,   required   to   dilute the
odourous   water   to the point where the odour is just
detectable.

Prevention of tastes and odours occuring is the best solution:

o   prevent pollution of source by sewage, trade wastes, etc;

o   control biological growths in reservoirs, tanks and filters, e.g. algae, fungi;

o   careful chlorination, and use of ammonia or dechlorination agents if necessary;

o   cover distribution reservoirs;

o   avoid dead-ends in distribution, flush mains regularly;

o   use material linings and protective paints of good quality.

Methods of removal are as follows:

a)   Aeration

Not particularly effective in removing tastes and odours due to organic matter, biological growths or chlorination since substances causing such problems usually have low volatility. Aeration is effective in reducing free carbon dioxide and hydrogen sulphide.

b)   Activated Carbon

This is a specially processed form of charcoal which may be either in powder or granular form, and is relatively pure carbon which has a highly porous structure with an extensive surface area in relation to its volume - it therefore is effective in removing taste and odour producing substances by adsorption; ordinary charcoal may be used but is not as efficient as activated carbon.

Powdered activated carbon is more effective and efficient for removing tastes and odours than the granular form. It is mixed into a slurry and fed at a constant rate to the water. Contact time should be a minimum of 15 minutes to 1 hour. Removal is usually on the bed of rapid or slow sand filters. In rapid filters, the carbon is washed to waste in the backwashing process; in slow sand filters, the material accumulates on the bed, and should not effect the length of filter run. Maximum efficiency is obtained when the carbon is applied to clarified water, normal doses being in the range of 5-20 mg/L. Efficiency of removal of tastes and odours is much better at lower pH values.

For slow sand filters, a larger initial dose of 40-50 mg/L could be applied to build up an effective concentration throughout the water on the filter, and then reduced to a nominal amount of say 5 mg/L.

Tastes and odours are normally a seasonal phenomenon, and therefore the required dose is likely to be intermittent and variable.

The application point may be at the plant inlet, or after clarification, or both. Sufficient contact time must be allowed prior to coagulation, when the efficiency of adsorption will reduce due to sealing of the carbon surface, as it binds with the coagulant.

Granular activated carbon is usually used for removal of free chlorine, by passing water through a bed of the substance, at a similar flow rate to that of rapid sand filtration - the units are therefore of similar size. In this case the carbon requires periodic regeneration.

Powdered activated carbon requires special care in handling, storage and feeding due to its highly corrosive nature in the presence of water; it is also combustible and should be stored separately from other chemicals, and equipment for handling the dry material should be sealed to confine the dust. See Ref.6 for additional information on this subject.

c) <u>Oxidising agents</u>

These are often effective against tastes and odours. Chlorine is the most common, but has the disadvantage, along with potassium permanganate, that excess oxidant must be removed. Chlorine dioxide is also very effective, being a more powerful oxidant than chlorine, and should only be considered in severe cases of tastes and odours, or when phenolic compounds are a problem.

It should be noted that when using chlorine for this purpose, there is a danger of forming new taste and odour causing compounds which may aggravate the problem. Sufficient oxidant must be used, therefore, to eliminate the original taste and odour causing compounds.

Ozone is probably the most effective taste and odour oxidant available but suffers from high cost and is therefore seldom used.

7.4 ALGAE CONTROL

Algae are a common problem in the surface water sources of Sri Lanka, where they affect the dissolved oxygen balance, and cause clogging of filters and taste and odour problems.

The preferred solution is to control the algae at source, by use of chemical or biological methods to eliminate the algae in the body of water. Copper sulphate is the most commonly used algicide. Reported dosage varies from 0.5 mg/L to 2.0 mg/L, depending on the type of algae concerned, the application being made by towing a sack of copper sulphate crystals behind a boat. It may also be added by solution feed at the entry to a reservoir. Tests should be made to ensure that the proposed dose is not toxic to any fish in the body of water.

Powdered activated carbon has also been used at a dosage of 5-35 kg per 100 m^3. Utilizing the fact that sunlight is essential for growth of algae (as for most other plants), the powdered activated carbon acts as a blackout to reduce the penetration of light into the water. It may be suitable in shallow areas of limited extent.

For spring and borehole supplies algae may usually be prevented by the exclusion of light. In surface waters, this is not usually possible; waters with higher turbidity are less susceptible to algae growths than clear water, due to light penetration.

Unit processes for removal of algae include prechlorination, microstraining, sedimentation and filtration, and the success of these various methods depends on the types and sizes of algae to be removed, and other less well defined variables.

Microstraining with a fabric aperture of 23 micrometres may be suitable if the algae are large, but some are too small to be retained.

Prechlorination with a free chlorine dose of 0.25-2.0 mg/L should be effective in reducing algae. Care must be taken not to apply chlorinated water to slow sand filters - improved initial performance may be followed by clogging due to an impermeable layer induced by the effect of chlorine on algae.

7.5 DESALINATION

Desalination is a very expensive process which should only be considered as a last resort, if no other source is available. Methods are multi-stage flash distillation, reverse osmosis and electrodialysis.

The multistage flash distillation process has remained for sometime the most popular and economic system for the production of large quantities of low salinity water. Preheated water is fed into a series of chambers each of which is held at a slightly lower pressure than the preceding one.

Reverse osmosis works on the principle that certain membranes will permit the passage of fresh water whilst preventing or restricting the transport of salts. Thus if a semi-permeable membrane is used as a barrier between a salt solution and fresh water the solvent (i.e. the water) will flow through the membrane to equalize the salt concentrations on either side. The pressure exerted by dissolved salts is known as the osmotic pressure and the action of osmosis is to balance the osmotic pressures on either side of the membrane. If the salt solution is subjected to a pressure greater than its osmotic pressure, water will pass through the membrane into the other compartment.

Recently, with developments in membrane technology, the process is becoming more viable for small desalination systems. Membranes are reported to have a useful life of 5 years.

In the process of electrodialysis, in which electrical potentials are used to transport ions through selectively permeable membranes, it is possible to reduce dissolved solids in water quite effectively. Many plants using this electrodialysis principle are in use in various parts of the world with capacities up to 0.12 m^3/s. The process is suitable for industrial applications, such as for soft drinks.

For several up-to-date articles on desalination, see Ref.8 pages 159-189.

7.6   SOFTENING

In certain cases treatment may be required for the removal of hardness. Since hardness is not a health hazard, this form of treatment may only be necessary in Sri Lanka for specialised industrial uses where scaling may be a problem (i.e. in high pressure boilers). Hardness may be removed either by chemical precipitation or by ion exchange. Generally, ion exchange is the more suitable method, being simpler in operation and maintenance.

Certain natural materials, such as zeolites (complex sodium alumino-silicates) and greensands have the property of exchanging one ion in their structure for another ion in solution. Thus natural zeolites will exchange their sodium ions with calcium and magnesium ions in water, thus softening the water completely. The finished water is high in sodium, but unless it was originally a very hard water, the sodium level is not harmful.

The ion exchange material can be regenerated with a strong solution of sodium ions, such as common salt solution.

Since ion exchange treatment produces effluent with zero hardness it is usually sufficient for domestic supply purposes to treat a portion of the water only which can then be blended with unsoftened water to produce a final hardness of 20–50 mg/L.

To allow production of satisfactory softened water by sodium cycle treatment the water should have a composition such that after treatment the water does not contain more than 900 mg/L sodium. To prevent damage to the ion exchange material the raw water should not contain more than 1.5 mg/L iron and manganese, nor more than 5 units turbidity unless suitable pretreatment can be provided.

Ion exchange beds may be gravity or pressure type and are operated in much the same way as a rapid filter.

Common design criteria are:

Rate of flow	120–480 m^3/m^2d
Depth of bed of resin	At least 0.75 m
Depth of supporting gravel	At least 0.3 m (graded)
Backwash rate	350–600 m^3/m^2d
Underdrain area	0.18% of bed area
Freeboard above bed	50% of bed depth
Length of run	Depends on hardness of raw water.

Further details of design and operation of ion exchange plants are given in Reference 6.

8. CORROSION CONTROL

8.1 EFFECTS OF CORROSION

Corrosion of metals due to the presence of aggressive waters and soils is a continuing cause of breakdowns, problems and expense in water schemes. Affecting pipelines, plant and equipment, it can be responsible for the following effects:

o   widespread deterioration of mains due to aggressive water and/or soils, causing weakening and subsequent pipe bursts;

o   localised deterioration of pipes resulting in leakage;

o   complete or partial blockage of pipes by corrosion products;

o   dissolution of metals causing possible toxicity, or other water quality problems;

o   discolouration of water due to corrosion products;

o   failure of equipment due to weakening or corrosion products.

To understand these effects and to design suitable protective measures, some understanding of the processes of corrosion is necessary.

8.2 PROCESSES OF CORROSION

Corrosion of iron is an oxidation process requiring the presence of water and oxygen. Two reactions take place as corrosion proceeds, the first being reduction of dissolved oxygen (the cathodic reaction). The second involves electrons being removed from the metal surface and replaced by the anodic reaction which results in the oxidation of metallic iron to ferrous ions. The two reactions proceed together and are often termed a corrosion cell.

The overall rate of reaction is sensitive to the rate of transfer of oxygen to the metal surface in the cathodic region. In the absence of oxygen, corrosion is normally inhibited completely, but in the presence of oxygen corrosion products limit the rate of reaction by reducing the rate of transfer of oxygen to the metal surface. In non-aggressive soils a completely unprotected pipe would suffer from general corrosion which would start at a high rate but would eventually decline. The rate of internal corrosion would be expected to be high as the flowing water maintains a high concentration of oxygen. Fortunately the corrosion products inhibit the transfer of oxygen to the pipe surface and so tend to reduce the rate of dilapidation.

-117-

Other factors could also help to inhibit corrosion. For
example, during the corrosion process local increases in pH
can cause precipitation of calcium carbonate from water
saturated with calcium carbonate; under suitable conditions
this will form a scale on the pipe surface and inhibit oxygen
transfer. However, serious problems usually occur when
localised corrosion proceeds under special conditions. More
often than not such problems are usually associated with
damage to the oxide film which is naturally produced on the
pipe wall at the manufacturing stage. Damage at a particular
point leads to the establishment of a localised corrosion
cell and is accompanied by a concentrated attack on the small
anodic site, which leads to a localised corrosion pit.

Corrosion cells can also be established under a variety of
special conditions involving galvanic action, differential
cells, stress corrosion, microbial corrosion and stray
electric currents.

Galvanic or bimetallic corrosion arises from the contact of
metals with very different electrochemical potentials. The
electropositive metal is cathodic to the electronegative
metal and promotes its corrosion. For example, graphite is
cathodic to cast iron and is present as inclusions in cast
iron pipe. Galvanic corrosion promotes the dissolution of
the iron leaving the graphite as the only structural
material. Table 8.1 shows the electrochemical series for
metals in sea water. The more noble metals and alloys more
resistant to corrosion are at the top of the list.
Generally, if two metals widely separated on this list are
used in contact with one another the less noble may suffer
accelerated corrosion, and reduce the rate of corrosion of
the more noble.

Different permeabilities of soils in trenches or where debris
is present inside pipelines can cause differences in oxygen
concentration. Localised differences in dissolved oxygen
near the metal surface can then establish corrosion cells and
lead to localised corrosion.

Anodic reaction can arise in the presence of sulphate
reducing bacteria. This type of corrosion is common in
water-logged soils where oxygen diffusion is poor. Black
ferrous sulphide is precipitated and is an indication of
microbial corrosion. Similar reactions can occur on the
internal surfaces of pipes but require anaerobic conditions
for sulphate reducing bacteria to survive. This is unlikely
under reasonable water quality and flow conditions but can
arise in certain circumstances. Bacterial slimes or other
growths on the interior surface can also cause these
conditions.

Induced direct currents can cause external pipe corrosion at
the point where the current leaves the pipes. These should
only cause concern in special limited circumstances but can
be aggravated when metal service pipes are used to earth
domestic electrical appliances.

-118-

Table C.1

Electrochemical Series for Common Metals and Alloys in Sea Water

More
Resistant
to
Corrosion ↑
 Platinum
Gold
(Graphite)
Silver
Austenitic stainless steel (passive)
Ferritic stainless steel (passive)
Nickel (passive)
Silver solder
Monel
Copper-nickel alloys
Bronzes
Copper
Brasses
Nickel (active)
Tin
Lead
Lead-tin solders
Austenitic stainless steel (active)
Ferritic stainless steel (active)
Cast iron
Steel
Cadmium

Less
Resistant
to
Corrosion ↓
 Aluminium
Zinc
Magnesium alloys
Magnesium

The more noble metals and alloys, more resistant to corrosion, are at the head of the list. In general, if two metals widely separated in this list are used in contact with one another the less noble may suffer accelerated corrosion and reduce the rate of corrosion of the more noble.

8.3 ASSESSING CORROSIVITY OF WATER

The main characteristics which affect the corrosive tendency of water are pH, hardness and alkalinity. Chlorides, by increasing conductivity, facilitate the corrosion process; dissolved oxygen is necessary for the cathodic process. Chlorination, practised normally, should not contribute to corrosion. Manganese, in excess of 0.03 mg/L can stimulate corrosion of copper pipes in hot water systems.

Waters with pH below 7 will dissolve most metals to an appreciable extent, and are generally unlikely to deposit protective scales of calcium carbonate on the metal. The ability of water to dissolve or to deposit calcium carbonate is used as a guide to the water's corrosive tendency and is measured by the Langelier Saturation Index (L)

$$L = pH - pHs$$

Where pHs is the pH of the water at the same temperature when saturated with $CaCO_3$.

pHs may be obtained from the formula:

$$pHs = (9.3+A+B) - (C+D),$$

using the values in Table 8.2

A negative Langelier Index indicates that the water will tend to dissolve calcium carbonate and a positive Index indicates that it will tend to deposit calcium carbonate.

8.4 ASSESSING CORROSIVITY OF SOIL

The resistivity of a soil is dependent on the dissolved chemicals, type of soil, moisture content, etc. and therefore is an excellent indicator of the amount of corrosion to be expected in any given situation. Thus metals buried in high resistivity soils will suffer relatively little corrosion as compared to metals in low resistivity soils where the elctrochemical action may progress at fast rates.

In addition to the corrosivity at any one location as indicated by the soil resistivity, much corrosion is due to change in soil along the pipe line. The greater and more abrupt the change, the greater the corrosivity in the low resistance soil immediately adjacent to the high resistivity soil.

A relatively simple soil resistivity survey over the route of any pipeline will indicate all areas likely to be corrosive in accordance with the following guide:

Table 6.2

Values for Determination of Langelier Index

Total Solids (mg/L)	A	Calcium Hardness (mg/L CaCO₃)	C	Methyl Orange Alkalinity (mg/L CaCO₃)	D
50– 300	0.1	10– 11	0.6	10– 11	1.0
400–1000	0.2	12– 13	0.7	12– 13	1.1
		14– 17	0.8	14– 17	1.2
		18– 22	0.9	18– 22	1.3
		23– 27	1.0	23– 27	1.4
Temperature °C	B	28– 34	1.1	28– 35	1.5
		35– 43	1.2	36– 44	1.6
		44– 55	1.3	45– 55	1.7
0– 1	2.6	56– 69	1.4	56– 69	1.8
2– 6	2.5	70– 87	1.5	70– 88	1.9
7– 9	2.4	88– 110	1.6	89– 110	2.0
10–13	2.3	111– 138	1.7	111– 139	2.1
14–17	2.2	139– 174	1.8	140– 176	2.2
18–21	2.1	175– 220	1.9	177– 220	2.3
22–27	2.0	230– 270	2.0	230– 270	2.4
28–31	1.9	280– 340	2.1	280– 350	2.5
32–37	1.8	350– 430	2.2	360– 440	2.6
38–43	1.7	440– 550	2.3	450– 550	2.7
44–50	1.6	560– 690	2.4	560– 690	2.8
51–56	1.5	700– 870	2.5	700– 880	2.9
57–63	1.4	880–1000	2.6	890–1000	3.0
64–71	1.3				
72–81	1.2				

(1) Obtain values of A,B,C and D from above table
(2) pHs = (9.3 + A + B) − (C + D).
(3) Saturation index = pH − pHs

-121-

o   Values below 2000 ohm-cm are usually corrosive except
     when they lie between areas of lower value, or when
     they comprise the highest values in a long section of
     line.   Values below 1000 are sure to be corrosive;
     below 500 severe and rapid corrosion is almost
     inevitable.

o   Values above 12,000 ohm-cm are almost never corrosive,
     except occasionally when a short section lies between
     much higher resistivity soil, but even then the
     corrosion is usually mild.

o   Values between 2000 and 12,000 ohm-cm must be assessed
     for corrosivity by comparison with adjacent sections;
     relatively low areas will tend to be corrosive as will
     areas of rapid and large change in soil resistivity;
     again, the areas of low resistivity will be the
     corrosive areas.

Further details on assessing soil conditions are given in
Reference 9.

8.5   PREVENTION OF CORROSION

Where tests and observations indicate that corrosion is or
may be a problem, steps must be taken to reduce or eliminate
its effects.   Corrosion control systems commonly used for
pipelines and steel structures are coatings, cathodic
protection, polyethylene sleeving, and use of corrosion -
resistant materials.   At the treatment stage a corrosion
inhibitor, such as sodium silicate, could be applied.

Coatings must be properly applied to dry, corrosion free
metal, and must be free from pinholes or damage to be
effective.   They should bond with the metal and give
insulation against electrical current.   Common materials
available are:

o   Cement mortar - suitable for internal lining of ductile,
     cast iron and galvanised steel pipe;

o   Epoxy - highly resistant, durable and suitable for
     internal and external use;

o   Polyethylene sleeving - required external protection for
     ductile pipe in the US;

o   Zinc coating (galvanising) - commonly applied as
     protection for steel and ductile pipe at manufacturing
     stage.

o   Bitumen and coal tar — commonly used for internal and external coatings, protective paints and sheathing: care must be taken to ensure that internal use will not result in objectionable tastes;

o   Wrapping tapes — these are available in a variety of materials and forms for factory or site application.

Care must be taken to ensure that any damage to protective coatings or linings during transport or installation is thoroughly repaired to the specified standard. Particular attention should be paid to pipe joints.

Steel pipes tend to be of large diameter and are usually of strategic importance and bitumen sheathed. It is essential for the sheathing to be thick and continuous in order to afford adequate protection but where soils are corrosive an added safeguard is introduced in the form of cathodic protection which would continue to give shielding in the event of the sheathing becoming damaged. This will inevitably set up a galvanic cell with the bare metal forming an anode at which localised pitting would develop.

"Long-line" corrosion cells can also occur where sections of the pipeline run through aggressive low-resistivity soils which form anodic areas relative to parts of the pipeline passing through soils of higher resistivity.

There are two methods of applying cathodic protection, namely by installing sacrificial anodes or by introducing an impressed current to the pipeline. The sacrificial anode is made of a metal with greater negative potential than that of the pipeline. It is submerged in electrolyte and connected externally to the pipelines to form a galvanic cell under which the protective or 'sacrificial" anode corrodes in preference to the cathodic pipeline. Specially formulated alloys of magnesium, zinc or aluminium are used for making sacrificial anodes.

In the case of the impressed current system, the protective current is not derived from the potential difference between dissimilar metals. Instead, protective anodes are energised by DC power, usually fed from rectified AC power supply, and current flows from these anodes though the electrolyte to the pipeline and thence through an external connection to the negative terminal of the DC power source.

Deciding which type of protective system to adopt depends on many factors. Size and complexity of structure, nature of the electrolyte and the environment, availability of power supply, proximity of "foreign" structures which may affect or be affected by the flow of current etc. all figure in the decision. A detailed technical evaluation should be carried out.

Further detailed information on cathodic protection is given in Reference 9.

Corrosivity, as measured by negative values of Langelier Index, is common in groundwaters in Sri Lanka and it may be necessary in some cases to stabilise these waters by the adjustment of pH and alkalinity to achieve calcium carbonate equilibrium. Treatment could comprise aeration to remove excess CO_2 and exposure to a bed of calcium carbonate. Normally, for minor schemes, such treatment would not be cost-effective, but pilot studies could be initiated in special cases.

9. PLANT DESIGN AND LAYOUTS

9.1 GENERAL CONSIDERATIONS

Water treatment plants should in general be low in capital cost and easy and economical to operate. Maximum use of gravity and hydraulic energy in the treatment process and use of labour instead of equipment in plant operation result in the most effective and economical treatment plant.

Fundamental design principles to be adopted are:

o   maximise use of gravity (e.g. for chemical feed);

o   conserve head loss throughout the plant and system;

o   utilise staged construction in 5-10 year stages to minimise capital costs;

o   allow for losses through plant, e.g. raw water pump capacity should be 5-10% greater than treated water output of plant.

o   adopt design innovations such as declining-rate filtration control, higher filter rates, direct filtration, inter-filter backwashing, high-rate settling in plates and tubes;

o   define required treated water quality, and design for specific characteristics of available raw water after thorough sampling and testing programme. :

o   where the design is for an extension to an existing works the need to maintain the operational status of the works during construction should be addressed.

o   maximise use of hydraulic means for mixing, flocculation and filter rate controls;

o   minimise use of equipment such as pumps, motors, and chemical feed equipment.

o   provide adequate facilities for parallel operation (i.e. duplication of tanks) and standby (including emergency generators).

o   maximise use of hydraulic means for mixing, flocculation and filter rate controls;

o   provide adequate facilities for chemical handling, storage, mixing, including day tanks, flushing/ dilution water connections and dosing.   Provide storage for 6 months chemical requirements.

o   provide 2 hours storage in clear well.

An example of typical design criteria for a full treatment works is given in Table 9.1

9.2   SITE LAYOUT

Careful selection of site and plant layout can make a big difference in capital costs and ease of O&M. Take into consideration:

o   elevation;
o   flood levels;
o   area required for future extensions;
o   cost of clearing, earthworks, embankments;
o   site drainage and wastewater disposal;
o   foundation conditions;
o   security of site;
o   access and roads, particularly access to intake site which should preferably be close by to allow ease of monitoring and supervision.

In particular:

o   utilise variations in site topography to match the required hydraulic profile of plant;

o   site chemical building high enough so that coagulants and lime can be fed by gravity from day tanks on upper level, as close as possible to points of application;

o   use ramps or elevators to raise chemicals to required storage level;

o   avoid excessive differences in head between units, e.g. locate clear water tank alongside filters rather than below them; head loss through the plant should not exceed 2 m, without aeration. Aeration, if necessary, will add 1.5 - 2.0 m.

o   minimise head losses in pipes and channels, e.g. avoid 90° channel turns;

o   carry out adequate soil investigations for foundation design, e.g. trial pits, boreholes.

o   administration facilities should be provided as required for offices/laboratory, reception area, scheme level stores, staff room, toilet and a small workshop. A control room may be required for larger works. Provide adequate lighting and ventilation.

Some typical plant profiles and flow schematics are given in Figs. 9.1 - 9.7.

TABLE 9.1

TYPICAL DESIGN CRITERIA FOR A
46 ML/d WATER TREATMENT WORKS

Item	Criteria
Intake Capacity	
Design flor for treatment	555 L/s (48000 m^3/d)
Grit Removal	
Number of channels	2
Width (each)	3.5 m
Length	22.0 m
River level	3.0 m above MSL
Velocity (horizontal @555 L/s)	0.16 m/s
Velocity (setting @ 555 L/s)	7.2 mm/s
Detention time (@555 L/s)	2.3 min
Travelling Screens*	
Number	2
Width	2.0 m
River Level	3.0 m
Velocity	0.45 m/s
Plant Capacity	
Design flow, raw water in	555 L/s (4.7% allowance for loss through works)
Design flow, treated water out	530 L/s (46000 m^3/d)
Wet Well	
Number	1
Volume	69.5 m^3
River level	3.0 m above MSL
Detention time	2.09 min
Raw Water Pumps	
Number	3 (1 standby)
Type	Split case centrifugal
Capacity (each)	278 L/s at 20 m TDH
Speed	1460 rpm
Power	93 kW
Raw Water Metering	
Number	1
Size	500 mm
Type	Venturi tube*
Capacity	220 to 800 L/s

Item	Criteria
Chemical Mixing Basin	
Number	1
Detention time	20 s
Volume	11.2 m^3
Mixer	15 kW
G	1000 s^{-1}
Flocculation Basins	
Number	2
Width (each)	3.962 m
Length (each)	81.65 m
Volume (each)	333.6 m^3
Detention time	23 min
Settling Basins	
Number	2
Width (each)	13.9 m
Length (each)	45.0 m
Side water depth (each)	2.438 m
Up flow rate	1.6 m/h
Detention time	1.5 h
Weir overflow rate	2.89 L/s/m
Weir length (each basin)	96 m
Filters	
Number	8
Type	Gravity sand downflow
Flow (each)	69.3 L/s
Filtration rate	7.3 m/h
Surface area (each)	34.4 m^2
Width (each)	4.3 m
Length (each)	8.0 m
Headloss	1.8 to 2.5 m
Backwash rate	35 to 50 m/h
Surface wash rate	5 to 10 m/h
Treated Water Reservoir	
Number	1
Capacity	4600 m^3
Width	25 m
Length	40 m
Side water depth	4.6 m
Detention time	2.41 hr

Item	Criteria
Treated Water Pumps	
Number	3 (1 standby)
Type	Vertical turbine
Capacity (each)	265 L/s at 146 m TDH
Speed	1460 rpm
Power	634 kW
Chemical Systems:	
Chlorine	
Type	900 kg cylinders
Dosage range	1-5 mg/L prechlorintion
	1-2 mg/L postchlorination
Maximum feed rate	225 kg/d prechlorination
	90 kg/d postchlorination
Chlorinators (1 duty,	1-225 " prechlorination
(1 standby)	1- 90 " prechlorination
Chlorine storage (average use)	90 d
Alum	
Type	Granular in bags
Dosage range	10-60 mg/L
Day tanks	2
Volume (each)	12.96 m^3
Area	4.32 m^2
Side water depth	3 m
Mixers per tank	1 vertical paddle
Feed pumps (positive displacement duplex)*	2 (1 standby) prelime
Feed pumps (positive displacement simplex*)	2 (1 standby) postlime
Capacity (5% solution)	1135 L/h prelime
	1135 L/h postlime
Lime storage (average use)	90 d

* Not recommended, included as example only.
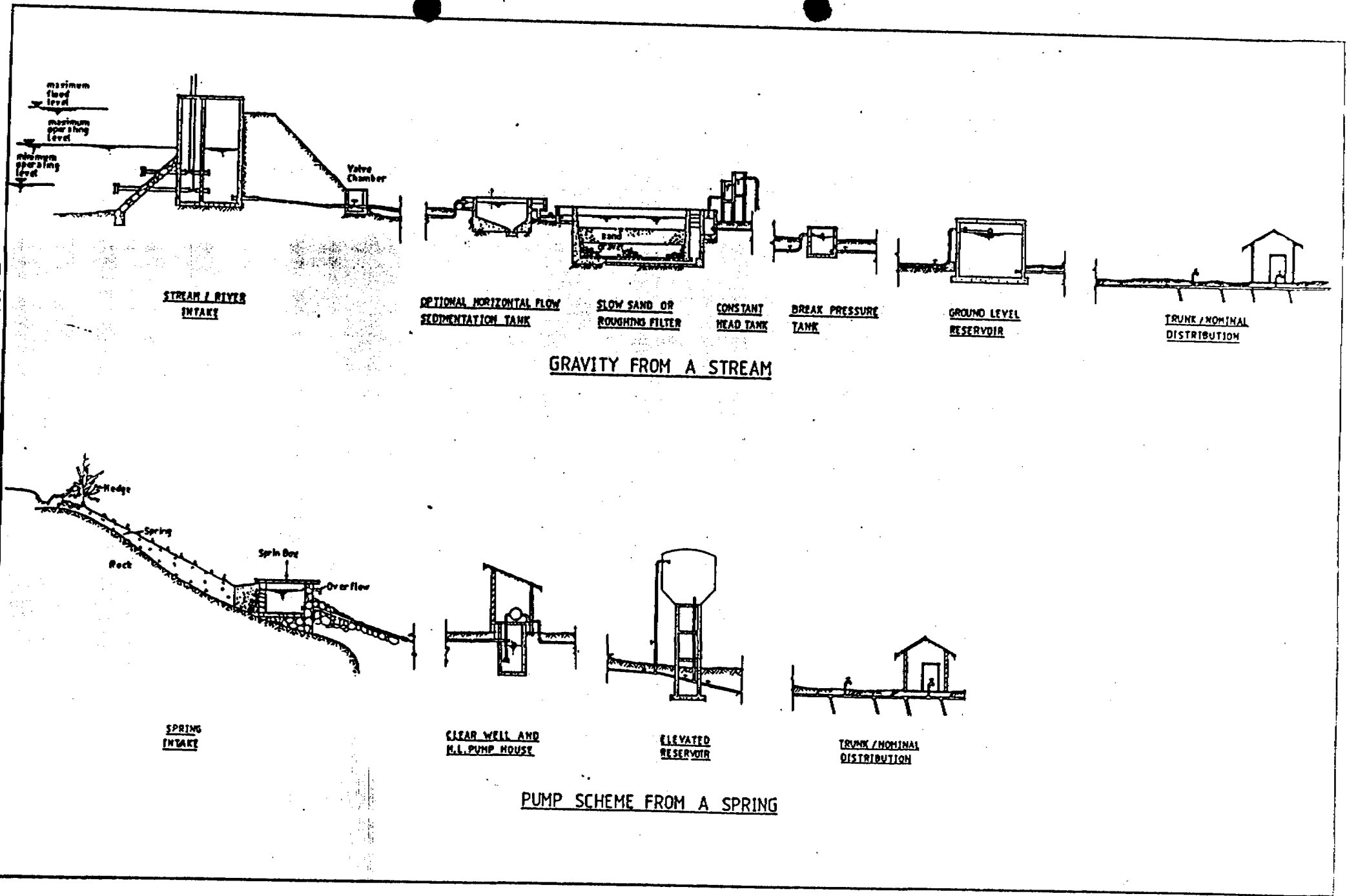
EXAMPLES OF FLOW DIAGRAM

Source: Kampsax-Kruger, Planning of Rural Water Supply and Sanitation Programme in Matale & Polonnaruwa Districts – Consolidated Report for DANIDA, December 1988.
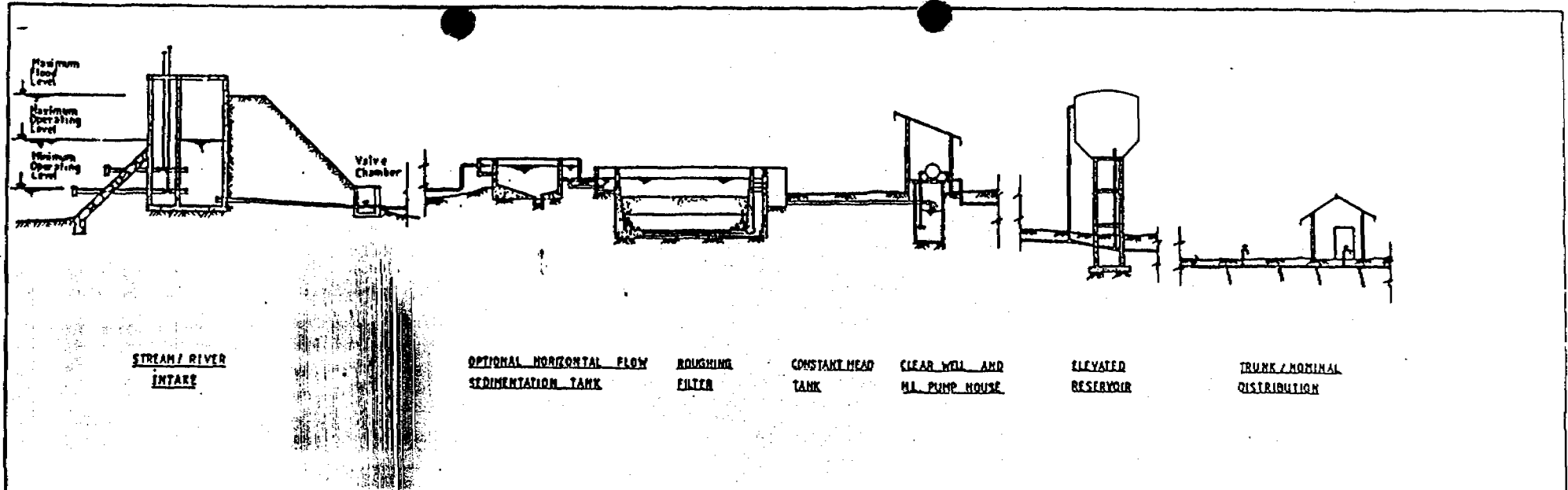
FIGURE 9.1

maximum flood level
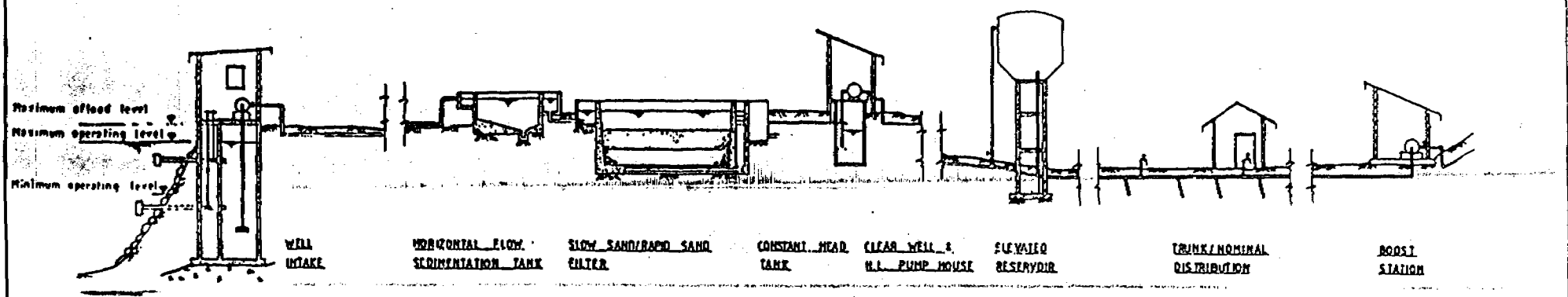
minimum operating level.

Valve Chamber

INTAKE
LAKE POND
BARRAGE

OPTIONAL HORIZONTAL
FLOW SEDIMENTATION
TANK.

SLOW SAND, OR
ROUGHING
FILTER

CONSTANT
HEAD
TANK

BREAK
PRESSURE
TANK

GROUND
LEVEL
RESERVOIR

TRUNK/
NOMINAL
DISTRIBUTION

GRAVITY FROM A STREAM ABOVE HUMAN SETTLEMENT

Hedge

Spring

Rock

Spring Box

Over flow.

SPRING
INTAKE

BREAK PRESSURE
TANK.

GROUND LEVEL
RESERVOIR

TRUNK/NOMINAL
DISTRIBUTION

GRAVITY FROM A SPRING

EXAMPLE OF FLOW DIAGRAM

-131-

FIGURE 9.2

GRAVITY FROM A STREAM

maximum flood level
maximum operating level
minimum operating level

Valve Chamber

STREAM / RIVER INTAKE

OPTIONAL HORIZONTAL FLOW SEDIMENTATION TANK

SLOW SAND OR ROUGHING FILTER

CONSTANT HEAD TANK

BREAK PRESSURE TANK

GROUND LEVEL RESERVOIR

TRUNK / NOMINAL DISTRIBUTION

PUMP SCHEME FROM A SPRING

Hedge

Spring

Rock

Spring Box

Overflow

SPRING INTAKE

CLEAR WELL AND H.L. PUMP HOUSE

ELEVATED RESERVOIR

TRUNK / NOMINAL DISTRIBUTION

EXAMPLE OF FLOW DIAGRAM

FIGURE 9.3

-132-

PUMP SCHEME FROM A STREAM ABOVE HUMAN SETTLEMENT

STREAM / RIVER INTAKE | OPTIONAL HORIZONTAL FLOW SEDIMENTATION TANK | ROUGHING FILTER | CONSTANT HEAD TANK | CLEAR WELL AND H.L. PUMP HOUSE | ELEVATED RESERVOIR | TRUNK / NOMINAL DISTRIBUTION

PUMP SCHEME FROM A SURFACE WATER SOURCE

WELL INTAKE | HORIZONTAL FLOW SEDIMENTATION TANK | SLOW SAND/RAPID SAND FILTER | CONSTANT HEAD TANK | CLEAR WELL & H.L. PUMP HOUSE | ELEVATED RESERVOIR | TRUNK/NOMINAL DISTRIBUTION | BOOST STATION

EXAMPLE OF FLOW DIAGRAM

FIGURE 9.4

Maximum Flood Level

Normal Flow Level

Minimum Flow Level

PONTOON INTAKE AND
L.L PUMP HOUSE

(OPTIONAL) HORIZONTAL FLOW
SEDIMENTATION TANK

SLOW SAND RAPID SAND
OR ROUGHING FILTER.

CONSTANT
HEAD TANK

CLEAR WELL AND
H.L. PUMP HOUSE

ELEVATED
RESERVOIR

TRUNK / NOMINAL
DISTRIBUTION

(OPTIONAL) BOOSTE
STATION

PUMP SCHEME FROM A SURFACE WATER SOURCE

CASCADE
AERATOR

CHEMICAL
STABILISATION POND

CLEAR WELL AND
H.L PUMP HOUSE

ELEVATED
RESERVOIR

TRUNK / NOMINAL
DISTRIBUTION

PUMP SCHEME FROM BORE HOLE

BORE HOLE
WELL

RIVER INTAKE STRUCTURE
SCREENS & PUMP STATION

WATER TREATMENT WORKS

TYPICAL SCHEMATIC FLOW DIAGRAM

FIGURE 9.5

TYPICAL HYDRAULIC PROFILE

FIGURE · 9.6·

HYDRAULIC PROFILE
(INPUT Q = 28 M/S)

PROCESS FLOW DIAGRAM

WATER TREATMENT WORKS
HYDRAULIC PROFILE &
PROCESS FLOW DIAGRAM

FIGURE 97

-136-

9.3 RAW WATER INTAKE

Address the following points when siting and designing the raw water intake;

o   normal, high and low water levels; calculate wet well volume at low water level.

o   intrusion of salinity in tidal reaches;

o   protection of raw water pumps from abrasion - provision of grit channels, gates and screens;

o   provision for cleaning intake channel;

o   access for maintenance and removal of intake pumps and motors, including overhead crane if necessary.

o   allow for minimum of 2 duty pumps and 1 standby, and room for expansion if required.

o   in dry wells, provide a sump and pump for drainage.

9.4 FLOW MEASUREMENT

There are many examples in existing plants of non-functioning differential pressure type flow meters, such as Dall or Venturi tubes. In some cases breakdown is due to damaged electronics, probably caused by voltage fluctuations and surges, and in other cases is simply due to lack of maintenance.

For this reason it is recommended that Parshall flumes or weirs be used in place of Dall/Venturi tubes on raw water lines from rivers. Parshall flumes or weirs have the advantage that they are self-cleaning, need minimal maintenance and are sufficiently accurate for Board requirements. They also provide ideal locations for efficient coagulant mixing.

Where use of differential pressure meters cannot be avoided, the electrical interface should be eliminated and a direct pressure recording instrument installed, preferably including a counter read-out of total flow.

On existing plants where the meters are not working or suspect, flow is often incorrectly assessed by using the pumping rate of the installed pumps from the nameplate details. Incorrect assessment of raw water flow obviously leads to a lack of control of dosing rates of chemicals. Measurement of the pumping rate in plants may be made using the following procedure:

1. Stop raw water pumping.

2. Continue filtration until clarifier level is down to a point where it is easy to measure the rise in water level in the basin.

3. Close the outlet valve from clarifier to filters.

4. Close valves on all filters. At this point the plant is completely out of service with the clarifier at a relatively low level, say 1.0 to 1.5 m below normal.

5. Start one raw water pump.

6. With a stop watch record the time required to raise the level 0.5 or 1.0m. Carefully mark these levels prior to starting the pump.

7. If there are two or more pumps run a test for each.

8. Calulate the pumping rate based on the time and known volume.

9. If two or more pumps are operated together, run a test for each combination of pumps used.

9.5 INSTRUMENTATION AND CONTROL

Instrumentation required for control of the process is as follows:

o   Measurement of rate of flow and total flow of water.

o   Measurement of filter head loss.

o   Measurement of chlorine dosage.

Details are given below:

a) Parshall flume or similar device with float operated flow rate indicator and totaliser, but not a chart recorder, for measuring the inflow. Units should be cubic metres per hour, and cubic metres.

b) Propeller meters with flow rate indicators and totaliser for measuring wash water and for measuring outflow, either before or after the clear water reservoir as (a):

c) Staff gauges for measuring the water level in each filter. Units should be metres.

d) Manometer tubes tapped through the filter wall for measuring the head loss in each filter. Units should be metres.

e)   Chlorinators, either Wallace & Tiernan or Fisher Porter, for post chlorination and also pre-chlorination if applicable in the process, with duplicate chlorinators in each instance, for standby. Units should be kg/h.

See also Design Manual 5, Mechanical, Electrical and Instrumentation Aspects of Design, Section 4, Instrumentation.

9.6   FACILITY OF OPERATION

Consideration must be given to making the operator's work easier by properly designing all necessary access to all parts of the plant which require it, such as chemical application points, flocculation, settling basin overflow, filters during washing, recorders and gauges. Provide walkways, handrails, ladders and manholes.

If operators encounter difficulties of access the inevitable result will be poor plant operation and maintenance.

As a check, have experienced O&M staff review the design drawings with this in mind, prior to finalization.

In addition, the design and specification should provide for a plant which will be straightforward to construct and erect, with robust equipment, simple to operate, requiring a minimum of skilled workers for operation and maintenance.

9.7   LABORATORY

A small laboratory will normally be required at the treatment works to monitor the treatment process. The physical and chemical determinations performed in the laboratory will present evidence whether:

o   the water has been properly pre-conditioned for each operation of the treatment process;

o   each unit process is functioning as it should;

o   the finished product is clear, free from objectionable taste and odour, chemically stable, and bacteriologically safe for human consumption.

The design of the laboratory should be based upon the quality assurance testing programme recommended for the treatment works and will include at least the following determinations:

Daily pumping   — hourly, daily rates. Hours of operation.

Raw water turbidity	—	Daily if the raw water is stable, every 3 hours if the turbidity is changing.
Settled water turbidity	—	3 times daily at the same hours, 8,16, 24 hours for example.
Filtered water turbidity	—	Each filter 3 times per day, (same as above).
pH — raw water	—	Once per day if raw water is stable, every 3 hours if raw water is changing.
pH — finished water	—	After post lime addition 3 times each day.
Colour	—	Where colour is a problem — raw water once per day when stable or 3 times per day when colour is changing. Finished water once per day.
Alkalinity	—	Twice per month when raw water is stable. Each day when raw water is changing.
Iron & Manganese	—	Where known to be low — raw water content once per month. Where it is a problem — once per week when water is stable — raw and finished water. Once per day when changing — raw and finished water.
Coagulant dose	—	Rate of application in mg/L Total coagulant consumed per day. Record rate changes.
Lime dose	—	Rate of application in mg/L. Total used per day.
Water stability	—	There is no direct test except the Langelier Index which is a good indicator. Once per week.
Hardness	—	Once per month.

The laboratory should be designed with the various functions allocated to different sections of the room. Bacteriological examination (which includes glassware washing, media preparation, sterilisation, innoculation and incubation), general chemistry, and special analyses will be performed in their specified areas. A fume hood should be provided for work involving obnoxious fumes. Distillation apparatus may also be required.

-140-

The furnishings should provide storage and bench space for instruments, equipment, and work area. Utilities necessary are hot and cold water, compressed air, vacuum, propane, and electrical outlets. The laboratory requirements should be checked with the O&M Department and process control section early in the design stage.

9.8   LESSONS FROM EXISTING PLANTS

In planning and designing a water treatment plant, it can be advantageous to be aware of the various operating problems which are currently occurring at existing plants. The following is a summary of comments made by Wagner, UNDP advisor, who surveyed a number of plants in 1983: (Ref.2)

o   The raw water during most of the year in most of the localities is low in both turbidity and colour.

o   Basic information on water quality is generally not available due to lack of laboratory equipment and supplies. Trained personnel are needed (Note that this aspect is being improved under the USAID Project)

o   Although most plants were producing a satisfactory water, plant control is precarious and undoubtedly there are times when the quality standards are not being met.

o   All but two plants visited have vertical flow, contact sludge blanket type clarifiers. All the operators reported relatively frequent problems of floc clouds rising to the surface and flowing to the filters; relatively rapid filter clogging results.

o   Most of the vertical flow clarifiers were loaded to the maximum or overloaded both for flocculation and settling. When flows and turbidities were relatively constant these basins performed satisfactorily.

o   Filters in all plants were underloaded with two exceptions where they were operating in the conventional range of filtration rate; all will take a much higher load: the constraints will be hydraulic. Low dosage, relatively high rate direct filtration is possible in most plants during a good part of the year. (This was confirmed by pilot filter testing).

o   The locally produced charcoal with a specific gravity of 1.42 proved to be a good filter media. Many of the plant filters can be changed to dual media without great structural complications. When required, therefore, filter rates can normally be substantially increased.

-141-

o   More pilot filter work needs to be done to refine the
     media specifications both in particle size and depth.
     Pilot filters should be constructed to operate at all
     problem plants and at other selected plants.

o   Mediocre to very poor coagulant dosing and dispersion
     (rapid mixing) was encountered in all but one plant.
     Because of the unusual feature of all plants having
     aeration it was relatively easy to correct this
     problem (by constructing a weir in the channel from
     aerator to clarifier, and installing a coagulant
     diffuser at this point).

o   Future designs should incorporate maximum use of
     hydraulic means for mixing, flocculation and filter
     rate control. Also the flocculation and settling
     basins should be horizontal flow to provide good
     process control.

On the basis of his surveys, Wagner made the following
specific recommendations relating to plant design:

o   Equip all plants with basic laboratory instruments and
     supplies which will make possible the collection of
     basic data on raw and treated water quality as well as
     the treatment process.

o   Provide day tanks for the preparation of stock
     solutions of coagulants and gravity systems for
     dosing;

o   Provide hydraulic mixing of the coagulant solutions
     with the raw water at weirs or flumes. These provide
     excellent mixing;

o   Provide hydraulic flocculation in well designed
     baffled basins. These include easily cleaned units
     and a design based on experience with the raw water to
     be treated. These basins are separate from settling
     to provide independent control of the flocculation
     process;

o   Provide horizontal flow settling basins with a
     properly designed entrance baffle to assure plug flow.
     The settled water take-off system should cover a large
     portion at the outlet end of the basin to reduce
     upflow currents to a minimum;

o   Utilize dual or single media filters designed with
     declining rate control and a box depth to accommodate
     a relatively deep media bed. The hydraulic design
     would allow for high rate filtration without future
     piping changes. It should have a backwash capacity to
     assure complete washing in 5 to 8 minutes. Install
     simple filter headloss gauges.

o   Utilize inter-filter backwashing.

o   Utilize direct filtration wherever possible.

9.9   SPECIFICATIONS

The specification for a water treatment works may be
either:

a)   A detailed specification closely defining the process
      and equipment which the Board believes will produce
      the required performance.

b)   A performance specification which leaves the choice of
      process and equipment to the tenderer.

It is recommended that type (a) be used on normal NWSDB
schemes.   However, for larger schemes or in particular
circumstances type (b) may be preferred.

Annex H lists information which needs to be included in a
detailed specification, whereas Reference 11 goes
thoroughly into the subject of performance specifications.

Various aspects of commissioning need to be addressed at
this stage:

o   Testing of equipment.

o   Responsibility for plant operations and decisions
     during commissioning — usually the contractor will be
     responsible but this should be stated and it should
     also be specified that Board operators will be closely
     involved.

o   Responsibility for supply and payment of chemicals
     during commissioning.

o   Supply of water for testing purposes.

o   Simulation of alarm conditions to test full
     operational efficiency and familiarize staff with
     procedures.

o   Provision for training, manuals, and the nature of
     acceptance trials.

See also Reference 11 and Procedure Manual P3, Commis-
sioning Guidelines.

9.10 POST - PROJECT APPRAISAL

Once the works is in operation, its performance should be monitored, to ensure that it is operating in the most efficient manner, according to the original design. If defects in design go unreported, they will continue to be built into future works and the quality of works will not improve. It is recommended that a post project appraisal report be prepared by DGM (O&M) after the scheme has been in operation for 1 year, for providing feed back to Planning, Design and Construction.

REFERENCES

(Mostly in NWSDB Library)

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19. AWWA - Water Chlorination Principles and Practices, Manual M20, 1973.

-145-

A N N E X E S

A.   Design Example – Cascade Aerator

B.   Design Example – Hydraulic Flocculator

C.   Design Example – Orifices Plate for Metering/Control of
      Rapid Gravity Filter with Declining Rate
      Flow.

D.   Filter Paper Test

E.   Preparation of Sand Filters

F.   Hydraulics of Under-Drain Systems

G.   Practical Guide for Dosage of Bleaching Powder in
      Disinfection of Public Drinking Water Supplies

H.   Information to be included in the Tender Specifications
      for a Water Treatment Works.

DESIGN EXAMPLE – CASCADE AERATOR

Design for removal of iron from groundwater pumped at rate of 4000 m^3/d. (2.78 m^3/min).

Assume cascade area   = 2.0 m^2/m^3/min of flow
Area required   = 2 x 2.78 = 5.56 m^2

Circular design

Diam. of lower step

$$d_1 \quad = \sqrt{\frac{(5.56 \times 4)}{\pi}} = 2.7 \text{ m}$$

and say d_2 = 1.9 m

d_3 = 1.1 m
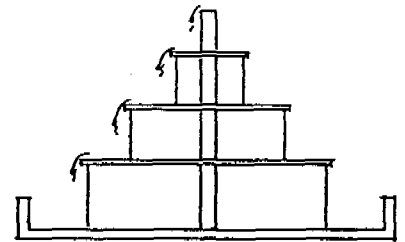
(Step width should not be less than 400 mm to make sure that water does jump the step).
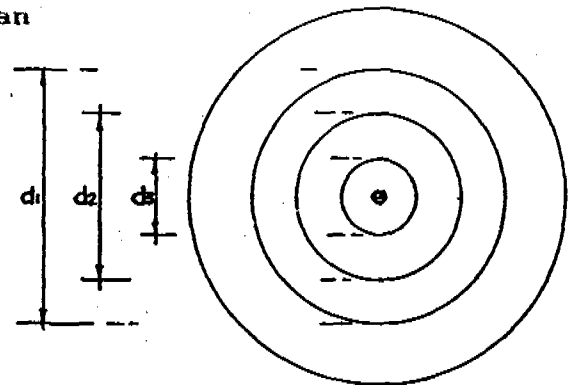
Provide 3 concrete steps, each of height 50 cm. Total height of drop, including inlet pipe   = 2.0 m

Length of lower step, = π D   = 8.48 m
     Overflow rate = $\dfrac{2.78}{60 \times 8.48}$ = 0.005, therefore OK

     (Recommended maximum overflow
     rate 0.1 m^3/m.min)

For inlet pipe, assume vel   = 1.5 m/s.
     Required area = Q/V   = $\dfrac{2.78}{60 \times 1.5}$ = 0.0309 m^2

                  therefore d = 200 mm.

Rise of jet h $= \dfrac{v^2}{2g}$ $= \dfrac{1.5^2}{2.(9.81)} = 0.115$ m

A-1

DESIGN EXAMPLE - HYDRAULIC FLOCCULATOR

Flow = 28 L/sec

Total basin length = 16.8 m   Water temperature = 25 °C

Depth of water in basin = 1.4m   Viscosity of
$\qquad$ of water $\mu = 0.90 \times 10^{-6}$ m^2/sec

Basin width = 0.84 m

Assume 0.15m minimum baffle spacing at basin entrance.

Velocity between baffles, $V = \dfrac{Q}{A} = \dfrac{0.028}{(0.15)(0.84)}$ = 0.222 m/sec

Headloss around baffle, $H = 3.3 \dfrac{V^2}{2g} = \dfrac{3.3(0.222)^2}{2 \times 9.81}$ = 0.0083 m

Detention time around baffle, $T = \dfrac{L^*, \text{length of travel}}{V, \text{velocity}} = \dfrac{2.8}{0.222}$ = 12.6 sec

Velocity gradient, $G = \sqrt{\dfrac{sgH}{\mu T}} = \sqrt{\dfrac{1(9.81)(.0083)}{0.90 \times 10^{-6} \times 12.6}}$ = 84.7 sec^{-1} (OK)

Assume 0.46m maximum baffle spacing at basin exit

Therefore, $V = \dfrac{.028}{(0.46)(0.84)}$ = 0.0725 m/sec

$T = \dfrac{2.8}{.0725}$ = 38.6 sec

$H = \dfrac{(3.3)(0.0725)^2}{2(9.81)}$ = 0.0009 m

$G = \sqrt{\dfrac{1(9.81)(.0009)}{(0.90 \times 10^{-6})(38.6)}}$ = 15.9 sec^{-1} (OK)

Total detention time in flocculator basin   = $\dfrac{\text{Basin volume}}{\text{flow rate}}$

Total $T = \dfrac{(1.4)(0.84)(16.8)}{0.028}$ = 706 sec = 11.8 min   (A bit short)

----

*$L = \dfrac{(D-W)}{2} + W + \dfrac{(D-W)}{2}$ Where D = depth of water
$\qquad\qquad$ W = baffle spacing

$= 2D$

rangement of Baffles and Total Headloss Calculation

sume baffle spacing to vary from 0.15 to 0.45 m in 0.075 m
crements leaving the last 1.2 m as a transition section into the
dimentation basin (see Figure B.1). Baffles should be removable in
der to be able to adjust the spacing for best flocculation. Wooden
ffle boards 12mm thick about 100 to 150 mm wide could be installed
to grooves made in the concrete wall. After final adjustment, the
se of the baffles could be filleted with grout to prevent solid
osits (a gap should be left in the grouting for drainage
poses). Basins should be designed to drain easily.

mary of Headloss and G Values

mber & Spacing of Baffles (m)	Velocity in Baffle (m/sec)	Headloss per Baffle (m)	Total Headloss (m)	$G*(sec^{-1})$
) @ 0.15	0.222	.0083	.158	84.7
: @ 0.23	0.145	.0035	.049	47.0
3 @ 0.30	0.111	.0021	.017	31.3
) @ 0.38	0.088	.0013	.012	21.5
5 @ ●46	0.073	.0009	.005	16.0

Total headloss   = 0.241 m

'elocity gradient, G at the start of each section.
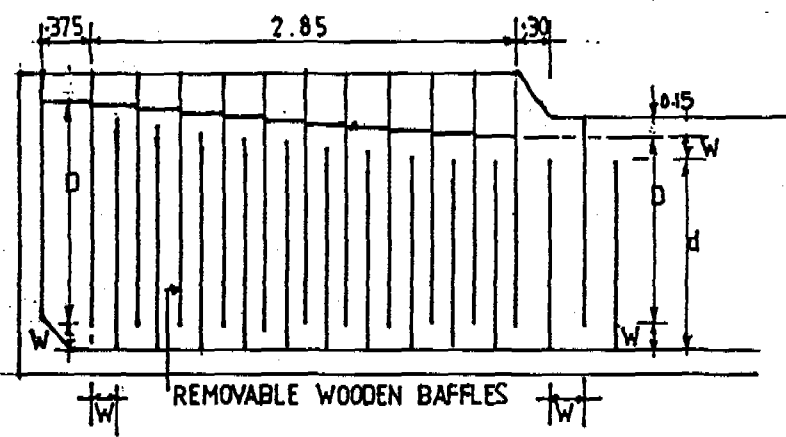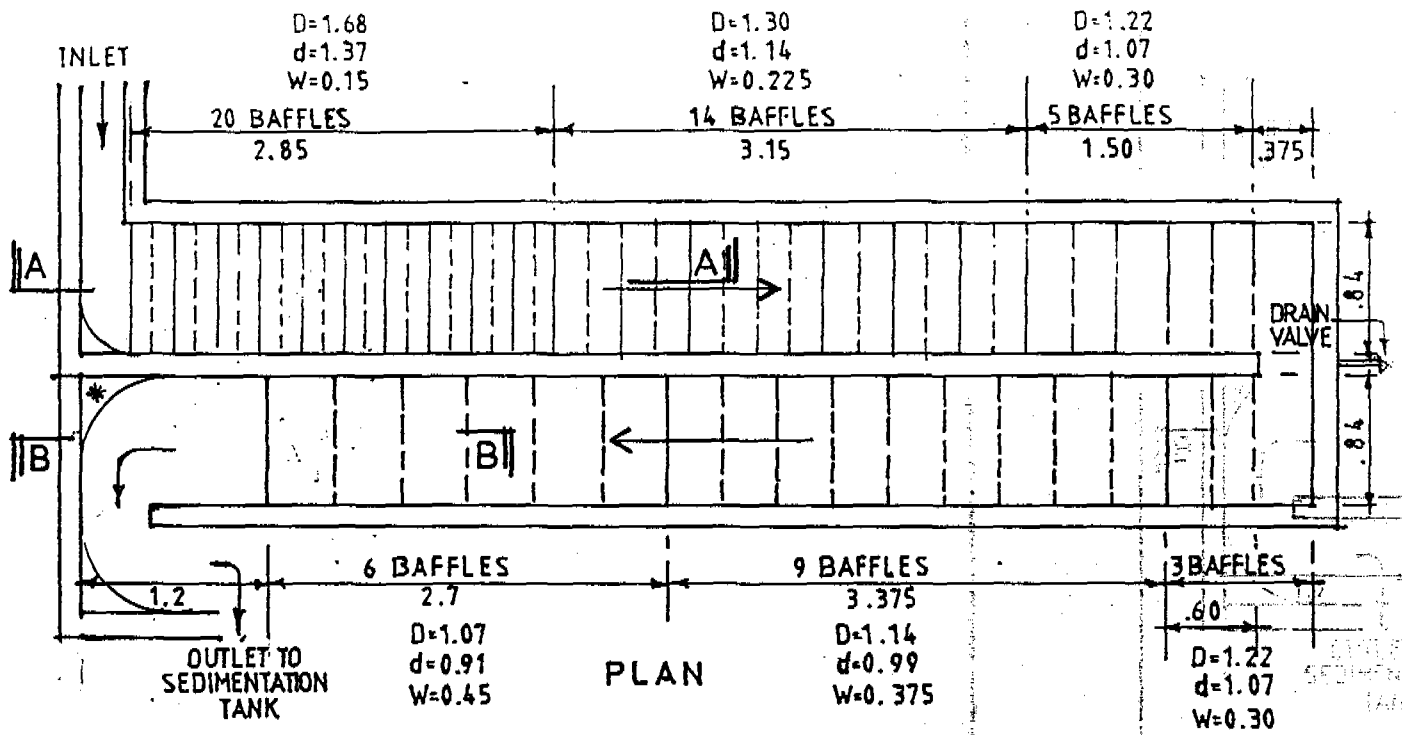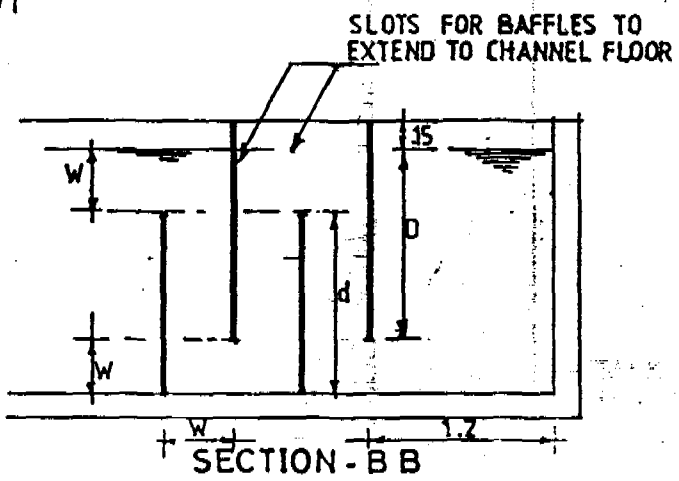
HYDRAULIC FLOCCULATOR
(CONCEPTUAL DESIGN)

FIGURE B.1

DESIGN EXAMPLE – ORIFICE PLATE FOR METERING/CONTROL OF
RAPID GRAVITY FILTER WITH DECLINING RATE FLOW

An orifice plate control is to be designed for a rapid sand filter with the following information:

o   Sand media: depth = 0.6m; E.S. = 0.55 mm; U.C. = 1.5 (see Fig.C.1)

o   Gravel underdrain section: 100 mm layer   25-50 mm gravel
                                 50 mm layer   12-25 mm gravel
                                 50 mm layer   6-12 mm gravel
                                100 mm layer   3- 6 mm gravel
                                 50 mm layer   6-12 mm gravel
                                 50 mm layer   12-25 mm gravel
                                125 mm layer   25-50 mm gravel

o   50 mm P.V.C. laterals with 10 No.12 mm holes/per lateral.

o   Average filtration rate = 100 m³/m²d

o   Maximum filtration rate required = 150 m³/m²d.

o   Area of each filter = 26.8 m²

Calculate Headloss During Filtration

Headloss equation for sand or gravel layers:

$$h/l = \frac{k}{g} \, \nu \, V \, \frac{(1-E)^2}{E^3} \left(\frac{6}{\psi}\right)^2 \sum_{i=1}^{n} \frac{X_i}{(d)} \quad \text{(Kozeny Equation)}$$

Where k = 5

ψ = 0.75 (shape factor for angular material)
l = depth of sand media or gravel layer
E = porosity of sand media or gravel layer = 0.40
ν = viscosity @ 20°C = 1.009 centistoke
V = filtration rate @ 150% of design = 1.74 mm/sec.

C-1

Sand and Gravel Sieve Analysis Calculations

Sand

d (Passing) (mm)	d (Retained) (mm)	Geometric Mean Size $d_i = \sqrt{d_1 d_2}$	Fraction Between Sieves X_i	$\dfrac{X_i}{d_i^2}$
0.420	0.292	0.35	0.01	0.082
0.595	0.420	0.50	0.16	0.650
0.707	0.595	0.65	0.20	0.473
0.841	0.707	0.77	0.25	0.422
1.000	0.841	0.92	0.21	0.248
1.410	1.000	1.19	0.16	0.113
1.680	1.410	1.54	0.01	0.004
			$= \overline{1.00}$	$= \overline{1.982}$

Gravel

3.18	2.00	2.52	1	0.157
6.35	3.18	4.49	1	0.500
12.70	6.35	8.98	1	0.012
25.40	12.70	17.96	1	0.003
50.80	25.40	35.92	1	0.008

1. **Headloss in Sand Media** (depth $l = 0.6$m)

$$h/l = \frac{(5)(1.009)}{9810} \ (1.74) \ \frac{(1-0.40)^2}{(0.40)^3} \left(\frac{6}{0.75}\right)^2 (1.982)$$

$$= \ 0.638$$

$$h \ = \ (0.638) \ 0.6 = 0.383 \ \text{m}$$

2. **Headloss in Gravel Underdrain**

$$h/l = \frac{(5)(1.009)}{9810} \ (1.74) \ \frac{(1-0.40)^2}{(0.40)} \left(\frac{6}{0.75}\right)^2 \sum_{i=1}^{n} \frac{X_i}{(d_i)^2}$$

$$= (0.322) \ \sum_{i=1}^{n} \frac{X_i}{(d_i)^2}$$

For 3-6 mm gravel layer 100 mm thick

$$h = 0.1 \ (0.322)(0.50) = 0.0161 \ m$$

For 6-12 mm gravel layer 100 mm thick

$$h = 0.1 \ (0.322)(0.012) = 0.0004 \ m$$

For other layers headloss is obviously negligible

Total headloss in sand and gravel layer = 0.383 + 0.017 = <u>0.40 m</u>

3. <u>Headloss in laterals</u>

No. of holes per lateral   = 10 x area of hole = 1.131 cm^2

No. of laterals per filter = 64

Total No. of holes   = 640

No. of holes per m^2 = $\dfrac{640}{26.8 \ m^2}$ = 23.9 holes per m^2

q/hole   = $\dfrac{1.736}{23.9}$ = 0.0726 L/s per hole

Entrance loss @ hole = h   = $(q/CA)^2 \ \dfrac{1}{2g}$ (Orifice equation)

$$h = \left(\frac{72.6 \times 10}{0.6(1.131) \times 10} \right)^2 \ \frac{1}{2\,(9.81)} = \underline{.058 \ m}$$

<u>Losses in laterals</u>

q/lateral = 0.0726 x 10   = 0.726 = L/s

A, area of 50mm lateral   = 0.00196 m^2

V, velocity in lateral   = $\dfrac{0.726 \times 10^{-3}}{.00196}$ = 0.37 m/s

Headloss in lateral   = 1/3 total headloss over lateral = 1/3 SL

For 50 mm PVC pipe, C   = 140, V = 0.37 m/s, S = 3.7/1000
                            L = 1.83 m

headloss in lateral   = 1/3(0.0037)(1.83)   = <u>.0023 m</u>
                                                     (negligible)

4. Headloss in Manifold

Entrance loss @ tee @ manifold (see sketch) = $0.6 \dfrac{V^2}{2g}$

Exit loss into manifold = $1.0 \dfrac{V^2}{2g}$

Total loss = $1.8 \dfrac{V^2}{2g}$ say $\dfrac{2V^2}{2g}$

$V = \dfrac{2q}{A}$ $= \dfrac{2(0.726 \times 10^{-3})}{.00196}$

$= 0.74$ m/s @ Tee
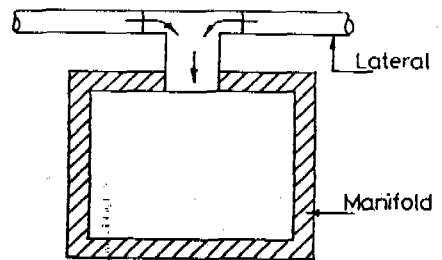
Manifold entrance & exit losses = $\dfrac{2V^2}{2g}$ = $\dfrac{(0.74)^2}{9.81}$ = $\underline{0.056m}$

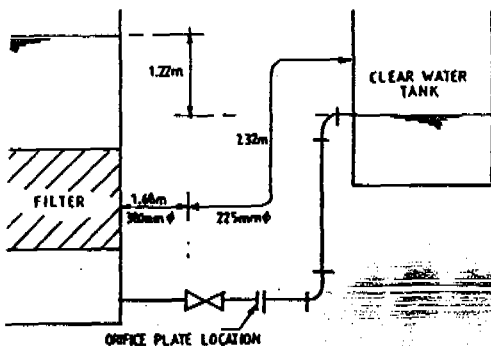Headloss along manifold channel is negligible since it is designed to carry backwash flow.

Total headloss in laterals & manifold = 0.058 + 0.056 = $\underline{0.114m}$

5. Effluent Pipe Losses

$Q = 150$ m^3/m^{2}d = 1.736 L/s x 26.8 = 46.53 L/s

Velocity in 225 mm dia pipe = 1.17 m/s $\dfrac{V^2}{2g}$ = 0.0698 m

Velocity in 380 mm dia pipe = 0.41 m/s " = 0.0086 m

Headloss thru pipe fittings (see sketch)

380 mm dia entrance = (0.5)(0.0086) = .0043
380 mm run of tee's(2) = (2)(0.6)(0.0086)= .0103
380 mm to 225mm reducer =(0.25)(0.0698) = .0175
225 mm gate valve = (0.20)(0.0698) = .0140
225 mm medium elbows (2)=(2)(0.3)(0.0698)= .0419
                                    Total   0.088m

Piping Schematic

Headloss in piping (for 225mm dia pipe only; 380 mm dia losses negligible)

@ C = 110, $h_F = \dfrac{8.4}{1000}$ (7.32 m) = 0.061 m

Total piping headloss = 0.088 + 0.061 = 0.149 m
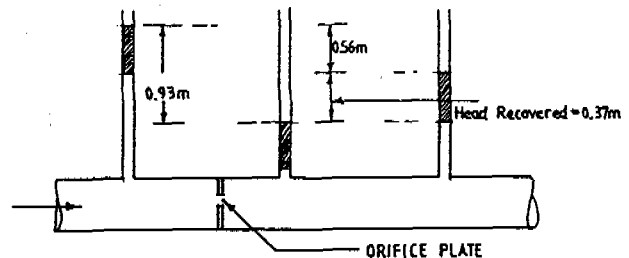   =======

C-4

6. <u>Orifice Plate Meter Losses</u>

Assume low water level in filter is set at 1.22 m
above outlet of pipe at clear water tank (design setting)

Required orifice meter loss = 1.22 m − losses in filter and
outlet piping
= 1.22 − (0.40 + .114) − 0.149

= 0.557 m say <u>0.56 m</u>

Overall orifice plate loss 60% of meter headloss (see "Applied
Hydraulics", H. Addison P.643)

Total meter loss = $\dfrac{0.56}{0.6}$ = 0.93m

ORIFICE PLATE

Area of orifice meter required, A = $\dfrac{Q}{C\sqrt{2gh}}$ = $\dfrac{46.53 \times 10^{-3}}{0.6\sqrt{2(9.81)(0.93)}}$

Where Q = 46.53 L/s, h = 0.93,   C = 0.6

A = 181.5 cm²

Orifice diameter, D = $\sqrt{\dfrac{(4)(181.5)}{\pi}}$ = 152 mm dia.

d orifice/d pipe < 0.8
.152/225 = 0.68 < 0.8 therefore OK

Orifice plate thickness = $\dfrac{d\ pipe}{50}$ = $\dfrac{225}{50}$ = 4.5 mm

See Fig.C.2 for orifice plate details

7.   Calculate Losses under Minimum Flow Condition

Min. flow = 50 m^3/m^2d = 15.51 L/s

Loss in filter bed & gravel, h ∞ Q = $\dfrac{15.51}{46.53}$ x 0.40 = 0.133m

loss in lateral & manifold, h ∞ Q² = $\left(\dfrac{15.51}{46.53}\right)^2$ x .114 = 0.013m

Total loss, filter & underdrain   = 0.146 m

Headloss in 225 mm piping (neglect losses in 380 mm dia.pipe):

V = 0.39 m/s $V^2/2g$ = 0.0078 m

Losses in fittings, h = k $\dfrac{V^2}{2g}$ = 1.05 (.0078) = .0082

Losses in pipe   = $\dfrac{1.12}{1000}$ (7.32) = .0082 m

Losses in fittings & pipe   = .0082 +.0082   = 0.0164

Orifice plate loss
    @ 15.51 L/s (Q/CA $\sqrt{2g}$)² = $\left(\dfrac{15.51 \times 10^{-3}}{(0.6)(0.01815)(4.43)}\right)^2$ = 0.103 m

Overall headloss over orifice plate = 60% (0.103)   = 0.062

Total losses in filter,
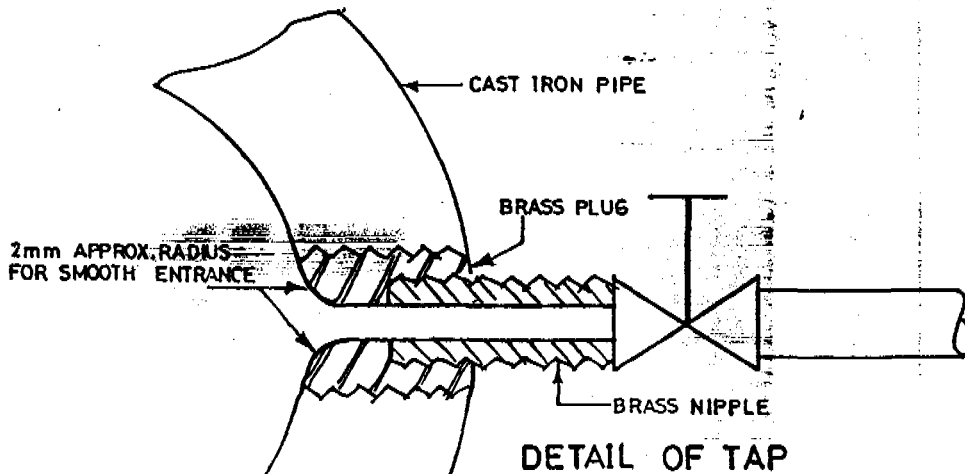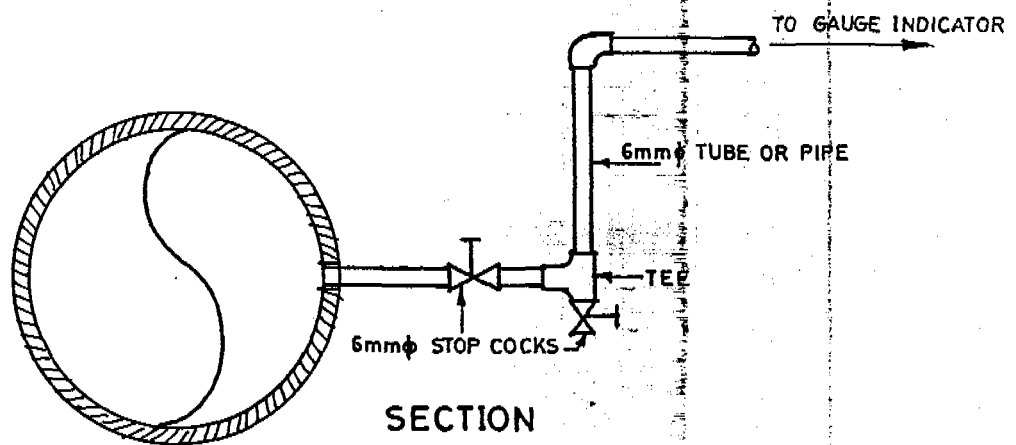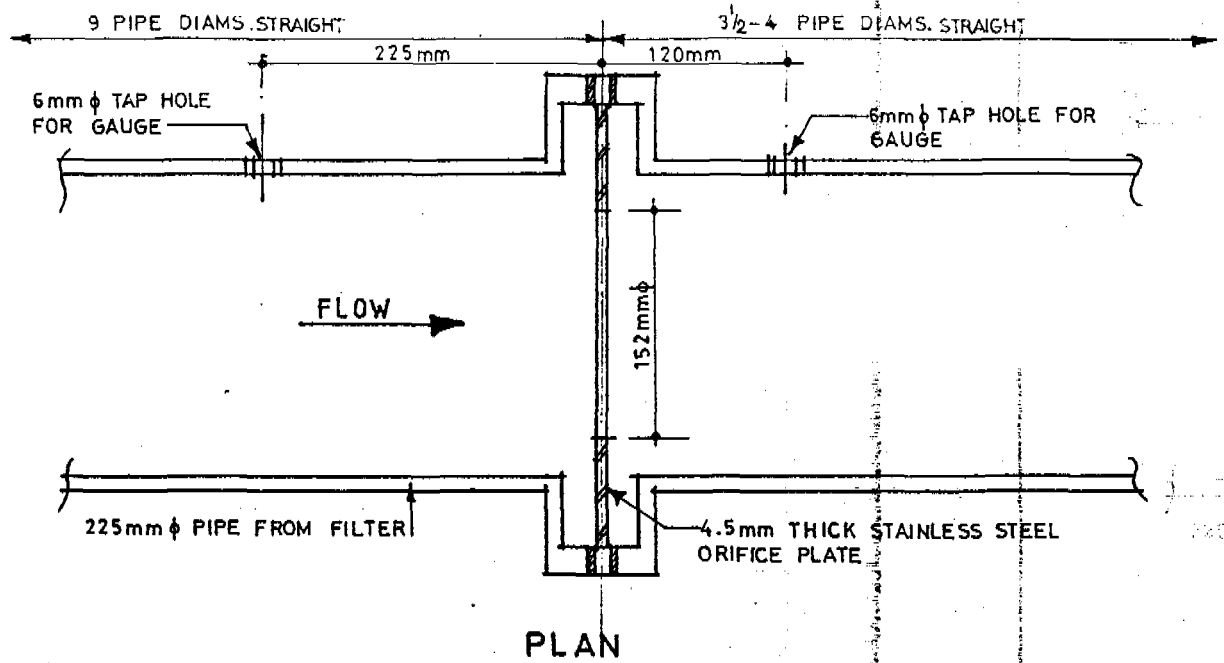   piping & orifice = 0.146 + 0.016 +0.062   = 0.224 m

Fig.C.3 is a plot of the above calculated data. It is seen that without orifice plate control, the filtration rate when the filter is clean (at minimum headloss in the media) could reach 370 m^3/m^2d.   By inducing additional headloss, the orifice plate reduces the maximum filtration rate to any desired lower rate.
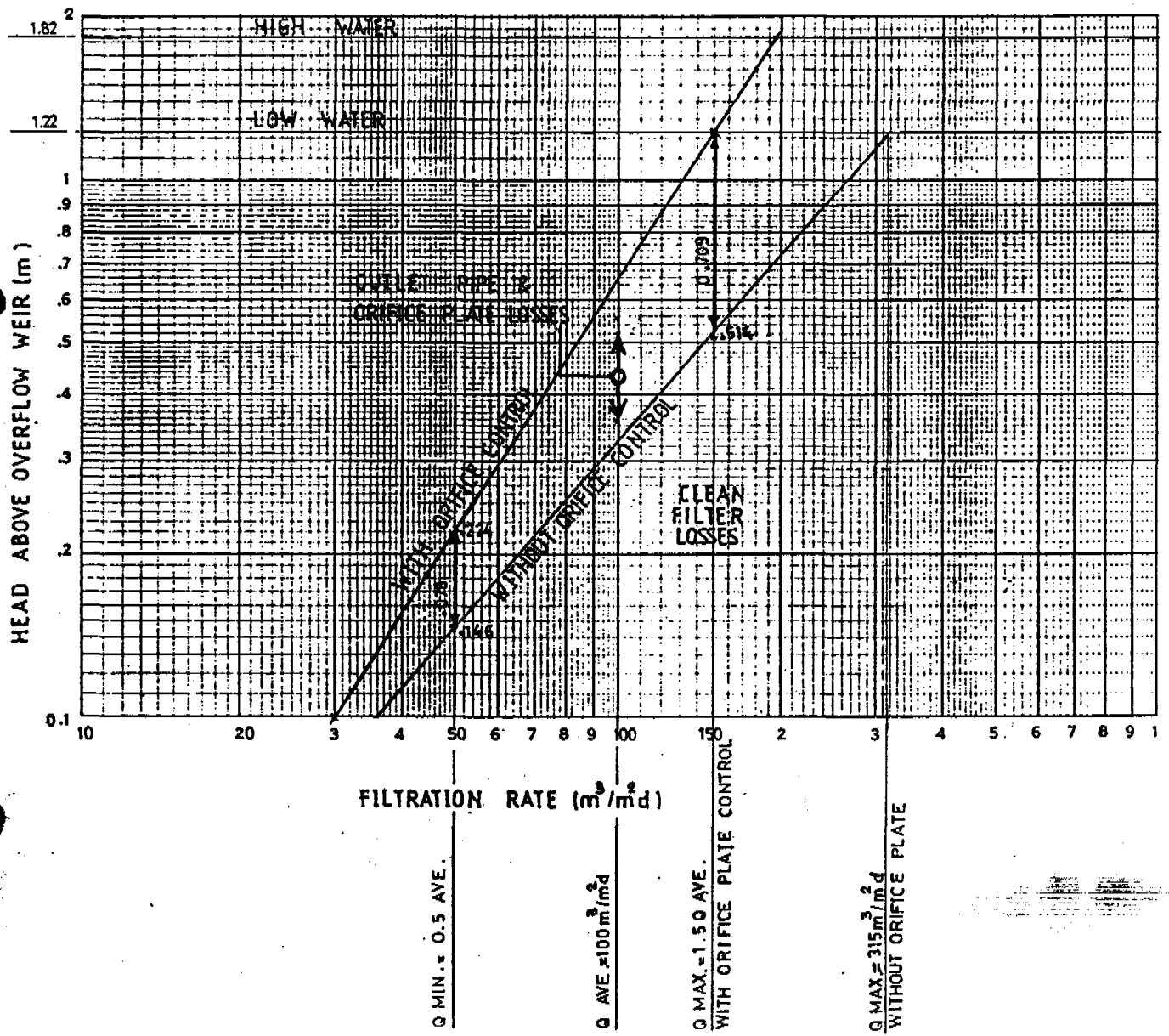
PER CENT PASSING

GRAIN SIZE (mm)

d_{60} = 0.83mm

d_{10} = 0.55mm (E.S)

UC = 1.5

SPECIFIED FILTER SAND

C-7

SPECIFIED FILTER SAND

FIGURE C1

9 PIPE DIAMS. STRAIGHT   3½-4 PIPE DIAMS. STRAIGHT

225mm   120mm

6mm φ TAP HOLE
FOR GAUGE

6mm φ TAP HOLE FOR
GAUGE

FLOW →

152mm φ

225mm φ PIPE FROM FILTER

4.5mm THICK STAINLESS STEEL
ORIFICE PLATE

PLAN

TO GAUGE INDICATOR →

6mm φ TUBE OR PIPE

TEE

6mm φ STOP COCKS

SECTION

CAST IRON PIPE

BRASS PLUG

2mm APPROX. RADIUS
FOR SMOOTH ENTRANCE

BRASS NIPPLE

DETAIL OF TAP
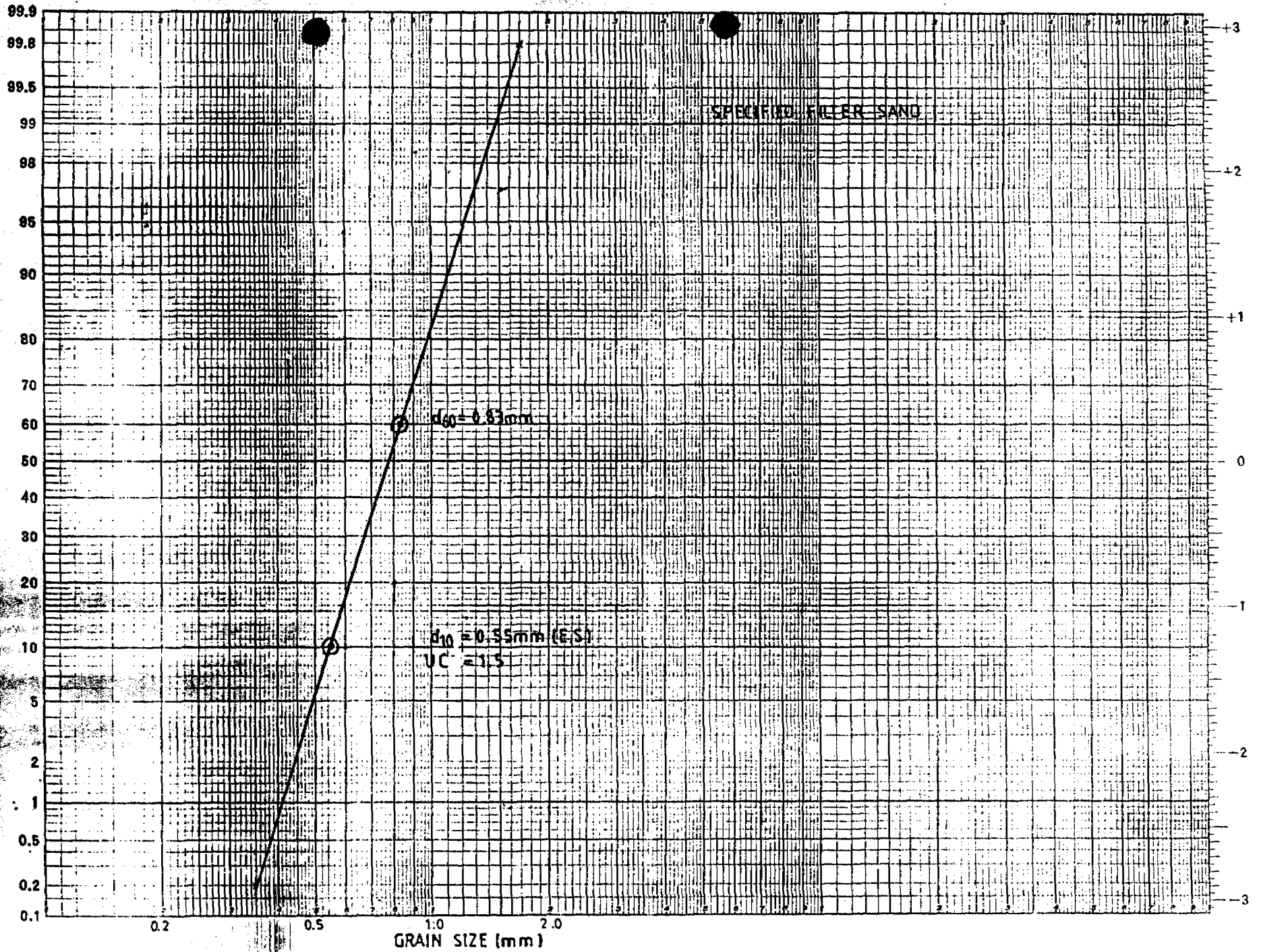
Orifice Meter Details.
Figure. C2

FILTRATION RATE VS HEAD

Figure. C3

FILTER PAPER TEST

The filter paper test is to determine the filterability of raw water treated with various doses of metallic coagulant or cationic polymer. This is quite different from the standard jar test to determine clarification.

The filterability depends on the characteristics of the turbidity in the raw water and the coagulant added. In Sri Lanka the coagulants which may be used are locally produced ferric chloride and imported aluminium sulphate. Polymers, while very effective, are also imported and are not very practical for this country because of their high cost.

The first step in this test is to filter the raw water with no coagulant added. This gives a good indication of the size of the turbidity particles. If the turbidity is substantially reduced with filter paper only, then the water is a good candidate for direct filtration. The paper used is Whatman No.40. The results obtained from this paper are comparable to that of a plant filter.

The second step is to dose a series of one litre jars with various dosages of coagulant beginning with very low dosages and working up to higher ones. Mix these with the jar test mixing machine. Add the dose with the machine at maximum rotation. Good, quick coagulant dispersion is essential. Apply the coagulant at a dilution of 1mg/L. For example in a four-jar series the first trial might be 0.5, 1.0, 1.5 and 2.0mg/L respectively in the jars. After 20 or 30 seconds of maximum rotation it is reduced to about 40 rpm for 2 or 3 more minutes. A sample at each jar then is filtered through the No.40 paper and turbidity read.

An example of a test on Ambalantota water is shown in Fig. D.1. The raw water had a turbidity of 25 NTU but when filtered without coagulant the turbidity was reduced to 5. This indicated that the water was an excellent candidate for direct filtration. Testing was continued with polymer, $FeCl_3$ and $Al_2(SO_4)_3$. The results are clear: the polymer is most effective but $FeCl_3$ is also very good. Alum was the least effective.

Fig. D.2 shows the results of a test at Matara. The results are quite different than those at Ambalantota. In this case $FeCl_3$ was not very effective while 5 to 6 mg/L of alum gave a satisfactory result. Again the polymer was most effective. Here alum is the preferred coagulant.

It is important that the coagulant dose be relatively low for direct filtration. High dosages of alum or iron coagulant produce large amounts of floc which quickly clog the filter. If more than 10 to 12mg/L is required to obtain destabilisation then it is doubtful that direct filtration is possible.

Once the filter paper test is completed and the results analysed, the next step is to further test with a pilot filter. This would be those cases where there is a good possibility of success with direct filtration. Galle, Matara, Ambalantota, Kandy, Kegalle and Kurunegala are all good candidates.

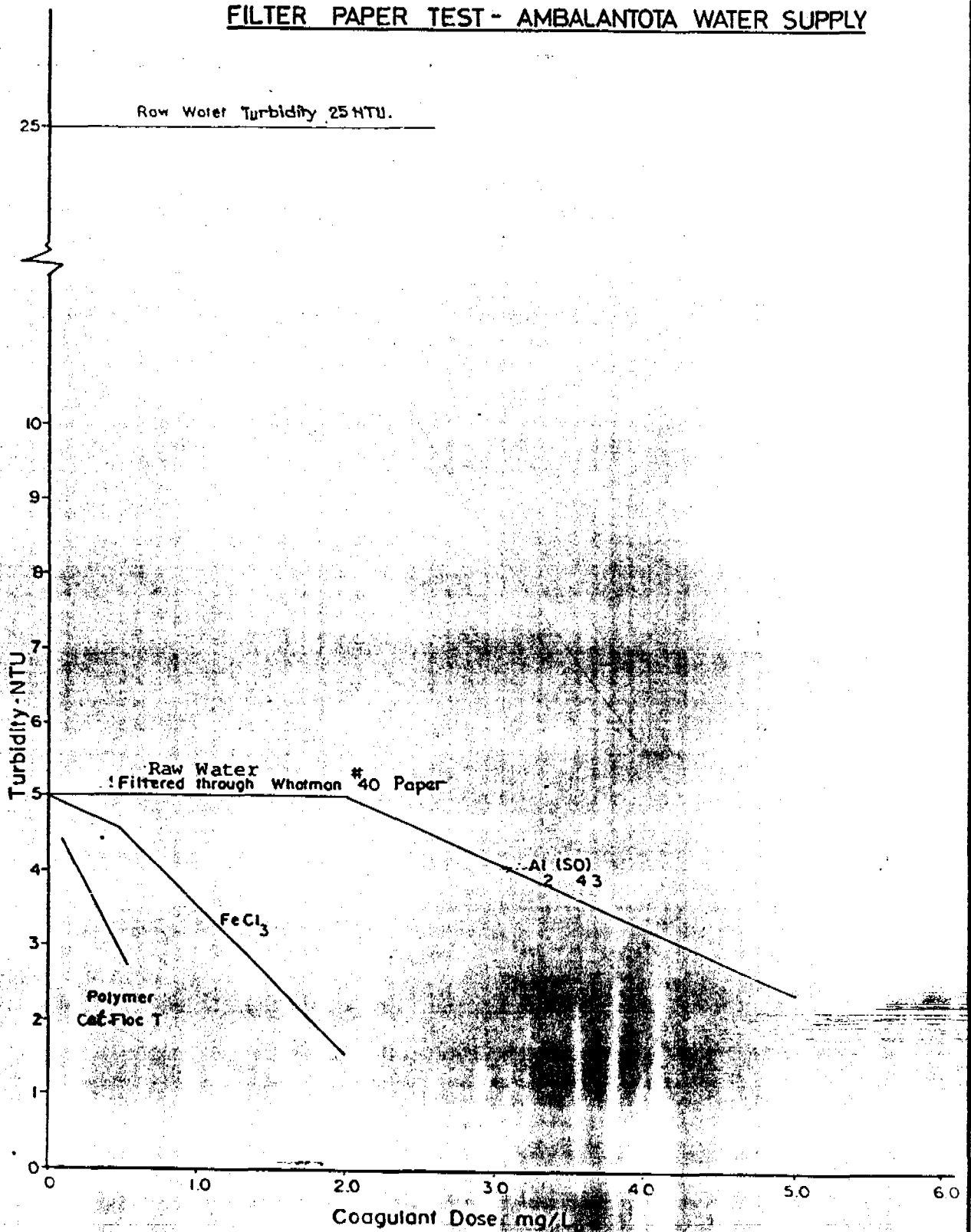# FILTER PAPER TEST - AMBALANTOTA WATER SUPPLY

Raw Water Turbidity 25 NTU.

Raw Water
Filtered through Whatman #40 Paper

$Al_2(SO_4)_3$

$FeCl_3$

Polymer
Cat-Floc T

Turbidity-NTU

Coagulant Dose mg/L

Figure D.1
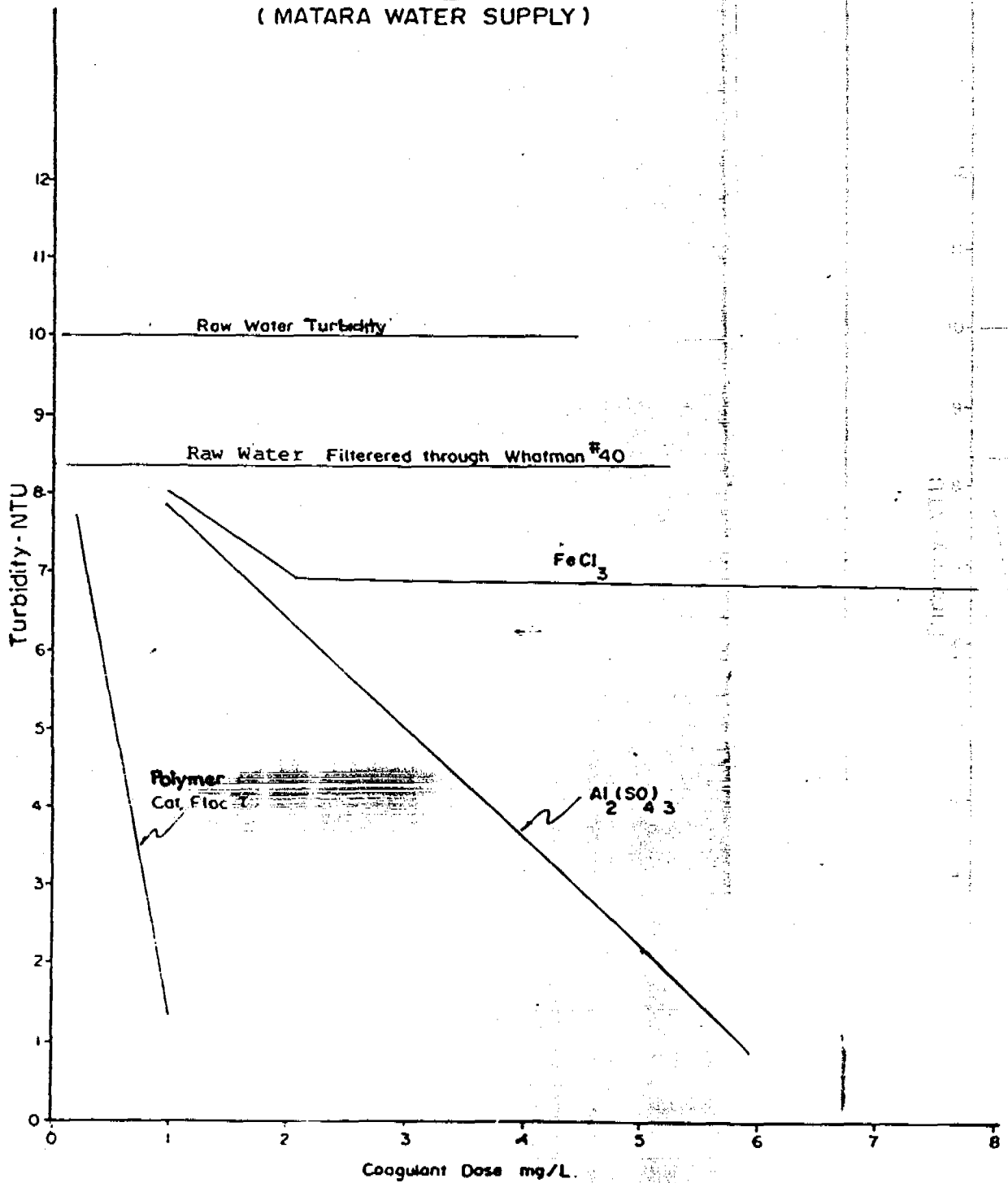
FILTER PAPER TEST
(MATARA WATER SUPPLY)

Figure D.2

PREPARATION OF SAND FOR FILTERS

1. **Basic Principles:**

   Natural sands and crushed materials normally used as filter media differ in size and size distribution, in shape, shape variation and in density and chemical composition. Properties of flow through filtering materials and resulting purification are related to the parameters of grain size and size distribution.

2. **Grain Size and Size Distribution:**

   Grain size is commonly determined by sieve separation of a representative sample through calibrated sieves. The sample is shaken through a vertical stack of successively finer sieves, the weights of sand held between sieves are added in sequence and cumulative weights are obtained. After conversion into percentage by weight, a summation curve can be plotted. Plotting of the values on logarithmic probability paper traces an almost straight line (Fig. E.1, E.2). By inter- polation, the geometric mean size and variance may then be determined to define the sample. Commonly used parameters are the effective size (ES = d_{10}) and uniformity coefficient (UC = d_{60}/d_{10}).

3. **Preparation of Sand:**

   Natural sand may be too coarse or too fine for filtration. If filter sand is specified in ES and UC and a sieve analysis of the stock sand has been made, the "too coarse" and "too fine" portions to be wasted from the stock can be determined.

   The percentage of usable sand, $P_u = 2 (P_{60} - P_{10})$ where P_{60} and P_{10} are the percentages corresponding to d_{60} and d_{10}.

   The percentage of sand below which the stock sand is too fine to use is :

   $$P_f = P_{10} - 0.1 P_u$$
   $$= P_{10} - 0.2 (P_{60} - P_{10})$$
   $$= 1.2 P_{10} - 0.2 P_{60}$$

   The percentage of sand above which the stock sand is too coarse to use is :

   $$P_c = P_{60} + 0.4 P_u$$
   $$= P_{60} + 0.4 \times 2 (P_{60} - P_{10})$$
   $$= 1.8 P_{60} - 0.8 P_{10}$$

From the cumulative size frequency curve, the sizes of stock sand corresponding to P_f and P_c are determined (d_f and d_c). The sizes below d_f and above d_c will have to be separated out from the stock sand to bring it to the desired specification.

The percentage of sand which is too coarse is screened out and wasted. In practice, it seldom pays to remove or waste more than 10% of fine sand or more than 50% of coarse sand (nor to use sand coarser than 2mm). The percentage of sand which is too fine can be washed out through a grit or sand washer (similar to an upflow settling tank). An example is given below:

Filter Sand Desired (mm)		Stock Sand Available (mm)	
ES (d_{10}) =	0.30	0.28)	
d_{60} =	0.75	0.95)	Fig. E.1
UC (d_{60}/d_{10}) =	2.50	3.40)	

% of usable sand P_u = $2(P_{60} - P_{10})$

= $2(48 - 12)$

= 72%

% of too fine sand P_f
(below which should
be wasted, d_f)

= $1.2\,P_{10} - 0.2\,P_{60}$

= $1.2 \times 12 - 0.2 \times 48$

= 4.8 % (d_f = 0.20 mm)

% of too coarse sand P_c =
(size above which
should be wasted d_c)

$1.8\,P_{60} - 0.8\,P_{10}$

= $1.8 \times 48 - 0.8 \times 12$

= 76.8% (d_c = 1.40 mm)

The following formula (by Hazen) for 75% removal of particles by washing relates the washing rate to particle size removed:

$$D \text{ (mm)} = 0.318 \cdot f \cdot \frac{S}{A}$$

where,   f is 3.0 for ordinary boxes.

S = overflow rate (spill) in L/min.

A = box area in m^2

or

$$D(\text{mm}) = 0.0065 \cdot f \cdot \frac{S}{A}$$ where   S in gal/min

A in ft^2

4. Specifications for Filter Sand

Characteristics

- avoid too great a uniformity of small size.
- sand to be hardened resistant quartz or quartzite, free of clay, fine or soft particles.
- ingition loss not greater than 0.7% by weight.
- soluble portion in HCL not greater than 5% by weight.
- silica content not less than 99.8%
- specific gravity - 2.55 to 2.65

As a thumb rule, the limits of size of filter sand may be assumed as:
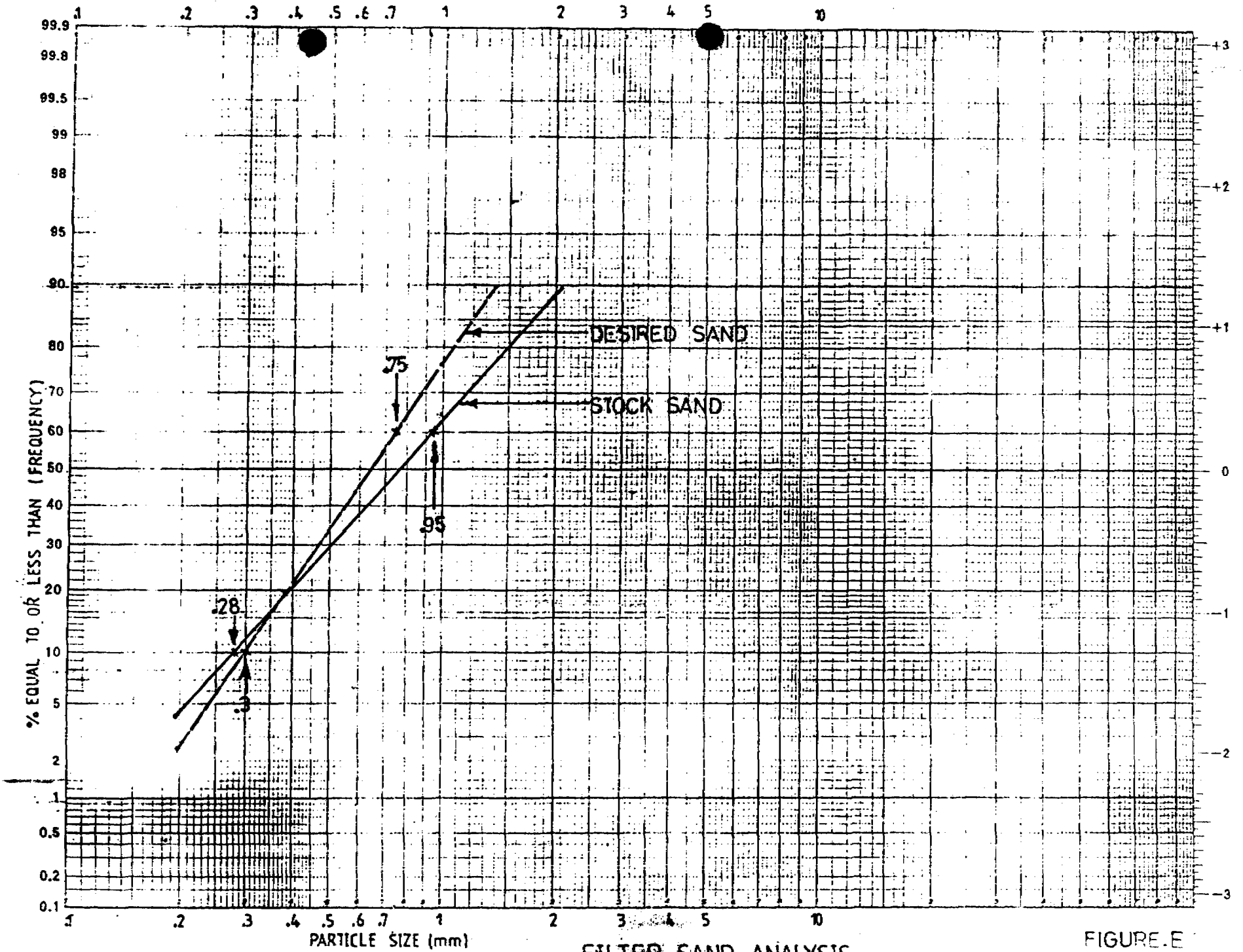
Rapid Sand Filter :

$$ES = 0.35 - 0.5 \text{ mm}$$
$$UC = 1.3 - 1.7$$

Standard British Sieve numbers and sizes between

16 (1.003 mm) and 52 (0.295 mm)

Slow Sand Filter :

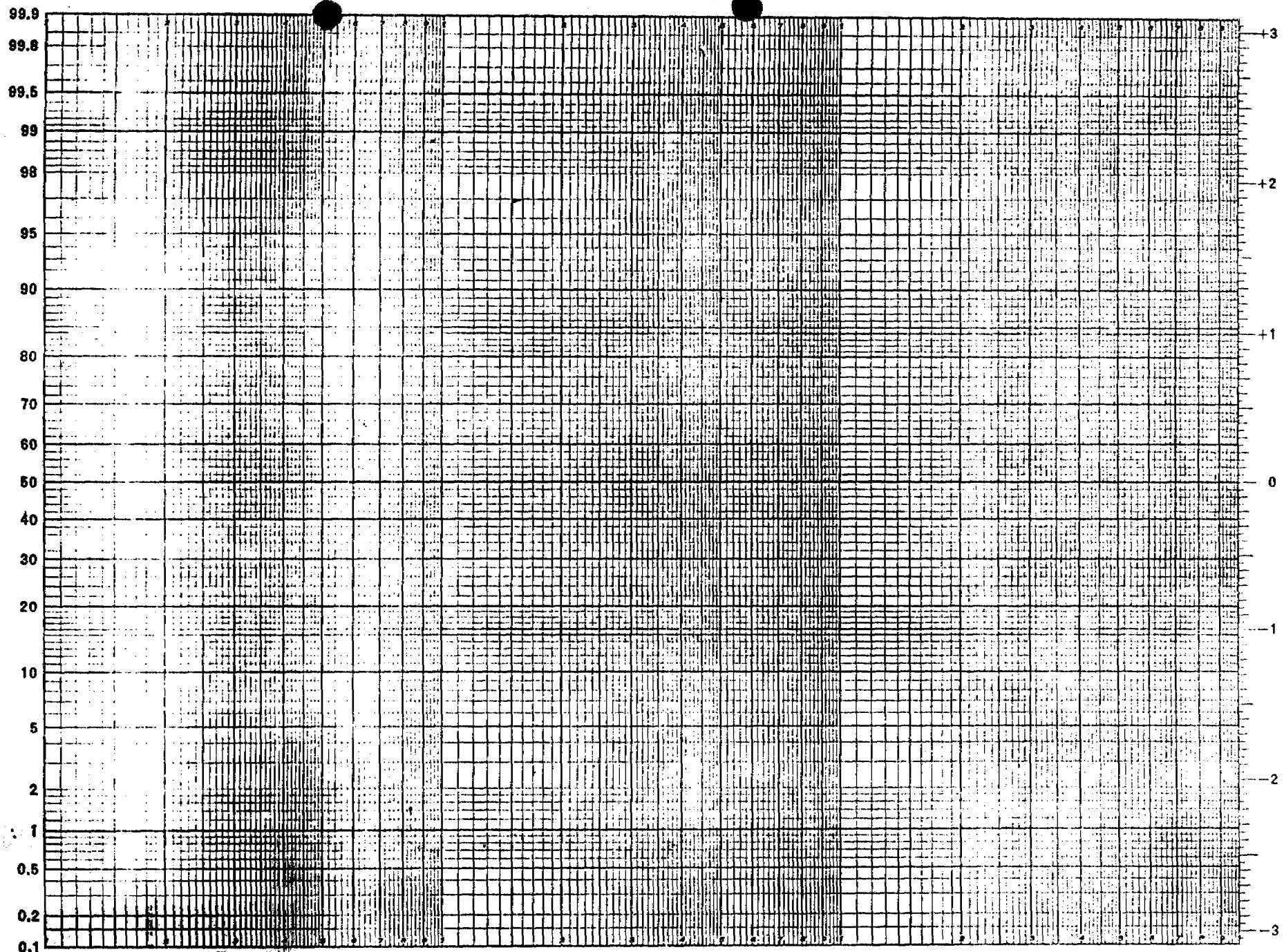|	Preferred	Possible
ES =	0.25 - 0.35 mm	(.15 - 0.35 mm)
UC =	2 - 3	(2 - 5)

Standard British Sieve numbers and sizes between

14 (1.204 mm) and 100 (0.152 mm)

FILTER SAND ANALYSIS

FIGURE.E

E-4

SAMPLE LOG PROBABILITY PAPER

E-5

FIGURE

The under-drain system of gravel layers, laterals and manifold should be designed to ensure even filtration over the entire filter bed. This is achieved by keeping the total resistance in the under-drain system, H_u to about 25% of the resistance through a clean filter bed, H_b at its minimum depth of 0.7 m during the design filtration rate V (filtration rate with one filter out of operation).

$$H_b = \frac{Vd}{k}, \text{ where } k = \text{permeability coefficient (m/h)}$$
$$d = \text{min. depth of filter sand(0.7m)} \quad \left(\begin{array}{l}\text{Darcy's}\\\text{Law}\end{array}\right.$$

The above requirement is that:

$$H_u < 0.25\ H_b$$

The following table shows the allowable under-drain resistances for sands having different values of ES & UC.

ES	0.25	0.25	0.35	0.35
UC	2	3	2	3
k* (m/h)	3	4.3	5.8	8.5
H_b(mm)(@ max. V = 0.2 m/h)	40.0	28.0	20.7	14.1

(k* values @ 20°C)

In other words, the coarser the media and the more non-uniform its size, the larger the laterals and manifold and the smaller the lateral spacing has to be to ensure uniform filtration over the entire bed.

The total under-drain resistance, H_u = resistance in gravel layers, H_g
+ resistance in laterals, H_l
+ resistance in manifold, H_m

a) **Resistance in gravel layers (typical)**

Thickness of layer (mm)	Passing (mm)	Retained (mm)	k (m/h)	Resistance H_g (mm) @ 0.2 m/h	(mm) @0.5 m/h
100	1	1.4	30	0.7	1.7
100	4	5.6	250	negligible	0.2
100	16	34.0	7,700	"	negligible
200	25	50.0	18,800	"	"
		Total		0.7	1.9
		Say		1.0 mm	2.0 mm

b) **Resistance in laterals H_l**

Pipe friction loss = $1/3\ l_1 \cdot S_1$

Where: S_1 = slope of hydraulic gradient of laterals
= $10.2\ Q^2 n^2/d^{5.33}$ (See Table F.1 for $10.2\ n^2/d^{5.33}$ factor)

l_1 = length of lateral (m)

d = diameter of lateral (m)

Q = flow in lateral (m³/s)

n = roughness coefficient. (0.01 for PVC)

Velocity head $= 2 (V_1^2/2g) = V_1^2/g$

where V_1 = velocity in lateral (m/s)

$g = 9.81$ m/s^2

Total resistance in lateral, $H_1 = 1/3\ L_1 . S_1 + V_1^2/g$

c) Resistance in manifold, Hm

Manifold conduit friction loss $= 1/3\ L_m . S_m$

Where S_m = slope of hydraulic gradient of manifold

$= V_m^2 (n /R^{2/3})^2$ (see Table F.2 for $(n /R^{2/3})^2$ factor)

V_m = velocity manifold (m/s)

L_m = length of manifold (m)

R = hydraulic radius of manifold

= area of manifold X-section / wetted perimeter

n = roughness coefficient. (n = 0.013 for concrete)

Velocity head $= 2 (V_m^2/2g) = V_m^2/g$

Total resistance in manifold $H_m = 1/3\ L_m\ S_m + V_m^2/g$

For a preliminary layout of laterals and manifold system, if L = length of filter bed, use lateral length $L_1 = 1/4$ L and manifold length L_m = L, and check under-drain resistance as shown above.

Example

Assume : Q = 336 m³/d = 14 m³/h = . 0039 m³/s

V = 0.1 m/h (normal) 0.2 m/h (maximum)

Filter media E S = 0.25 mm U C = 2

Calculate: length, width and underdrain lateral and manifold sizes

Solution: Total area of filter $= \dfrac{Q}{V} = \dfrac{14}{0.1} = 140$ m²

No. of filters, N = 2

Area of each filter, A = 70 m²

if L = length of filter

$W = \dfrac{L}{2}\ \dfrac{N+1}{N} = \dfrac{L}{2}\ \dfrac{3}{2} = 3/4$ L

A = L. 3/4L

70 = 3/4L²

L = 10m ∴ W = 3/4 (10) = 7m

F.2

a) Check lateral size:

Size of lateral used	= 50 mm (PVC Pipe)
Length of lateral	= 1/2 W = 3.5 m
Width of lateral drainage	= 1.5 m (spacing)
Total no. of laterals	$= \dfrac{2\ L}{1.5} = \dfrac{2\ (10)}{1.5} = 13.3$ say 14

$$Q_l = \frac{Q}{\text{no. of laterals}} = \frac{0.0039}{14} = 0.0003 \text{ m}^3/\text{s}$$

$$V_l = \frac{Q_l}{\text{Area of 50 mm lateral}} = \frac{0.0003}{0.002} = 0.14 \text{ m/s}$$

$$V_l^2/g = 0.002 \text{ m} = 2 \text{ mm (vel. head loss)}$$
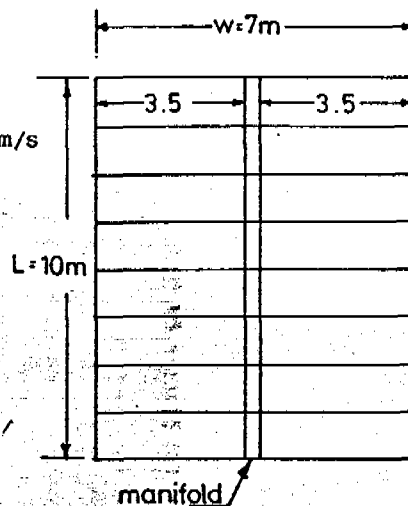
$$S_l = 10.2\ Q_l^2\ n^2/d^{5.33}$$

Using Table F.1

$$S_l = (0.0003)^2\ (8772) = 0.0008$$

$$H_l = (1/3)\ (3.5)\ (0.0008) + 0.002 \text{ m}$$
$$= 0.0009 + 0.002$$
$$= 0.0029 \text{ say 3 mm}$$

b) Check manifold size:

Size of manifold used = 150 x 150 mm

Length = 10 m

$$V_m = \frac{Q/\text{filter}}{\text{Area of manifold}} = \frac{0.0039}{0.0225} = 0.17 \text{ m/s}$$

$$V_m^2/g = 0.0029 = 3 \text{ mm}$$

$$S_m = V_m^2\ (n/R^{2/3})^2$$

Using Table F.2

$$S_m = (0.17)^2\ (0.013) = 0.0004$$
$$H_m = (1/3)\ (10)\ (0.0004) + 0.0029$$
$$= 0.0029$$
$$= 4.2 \text{ mm}$$

c) Total allowable resistance for E S = 0.25 mm and U C = 2

$$H_b = 40 \text{ mm}$$

$$H_u = H_g + H_l + H_m$$
$$= 0 + 0 + 4.2$$

$$H_u = 4.2 \text{ mm} \longleftarrow 25\% \text{ of 40 mm, therefore OK}$$

Hence, 50 mm dia. laterals @ 1.5 m spacing with manifold at centre is good.

F.3

(1) Chec: with longer laterals & side manifold

$Q_l = \dfrac{.0039}{7} = 0.0006$ m³/s

$V_l = 0.28$ m/s (0.3 m/s is OK)

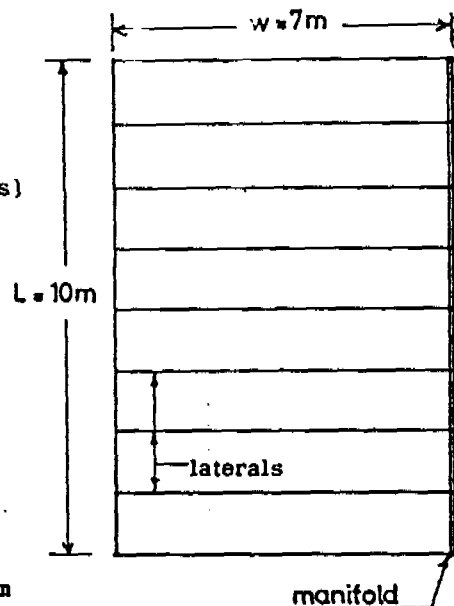$V_l{}^2/g = 0.008$ m $= 8$ mm (vel. head loss)

$S_l = (0.0006)^2$ (8772) $= 0.0032$

$H_l = (1/3)$ (7) (0.0032) $+$ 0.008

$= 0.0075 + 0.008$

$= 0.0155 = 15.5$ mm

$\therefore H_u = 1.0 + 15.5 + 4.2$ mm

$= 19.7$ say 20 mm $>$ 25% of 40 mm

Therefore, 50 mm lateral 7 m long is not good @ 1.5 m spacing.

Calculation Tables

Table - F.1

Lateral Diameter, d (mm)	Cross Sectional Area (m²)	$10.2n^2/d^{5.33}$	Value of Qmax* (m³/s)
50	0.002	8,772	0.0006
80	0.005	716	0.0015
100	0.008	218	0.0024

* Based on velocity in lateral = 0.3 m/s (n = 0.01 for PVC)

Table - F.2

Channel Size (height & width) (mm x mm)	Cross Sectional Area (m²)	Hydraulic Radius (R)	$R^{2/3}$	$\left(\dfrac{n}{R^{2/3}}\right)^2$	Qmax* (m³/s)
150 x 150	0.0225	0.038	0.113	0.013	0.007
150 x 200	0.0300	0.043	0.123	0.011	0.009
150 x 250	0.0375	0.047	0.130	0.010	0.011
150 x 300	0.0450	0.050	0.136	0.009	0.014
200 x 200	0.0400	0.050	0.136	0.009	0.012
200 x 250	0.0500	0.056	0.146	0.008	0.015
200 x 300	0.0600	0.060	0.153	0.007	0.018
250 x 250	0.0625	0.063	0.158	0.007	0.019
250 x 300	0.0750	0.068	0.167	0.006	0.023
300 x 300	0.0900	0.075	0.178	0.005	0.027
300 x 350	0.1050	0.081	0.187	0.005	0.032
300 x 400	0.1200	0.086	0.195	0.004	0.036

* Based on velocity in manifold = 0.3 m/s (n = 0.013 for Concrete)

PRACTICAL GUIDE FOR DOSAGE OF BLEACHING POWDER IN
DISINFECTION OF PUBLIC DRINKING WATER SUPPLIES

1. QUANTITY OF BLEACHING POWDER NECESSARY

The Table below gives the Unit Dosage Rate and Unit
Quantities of Water so that the user can work out multiples
of these quantities to meet individual needs.

A minimum of 0.3 to 1.0 ppm*Residual Chlorine after a contact
period of 30 minutes of more, should be the basic criterion an
individual water supply scheme should aim at. If a contact
period of 30 minutes is not physically feasible under existing
situations, a higher rate of dosage may be desirable to
safeguard those concerned. Consumers should also be educated
to be able to accept a higher chlorine content in their water
and to use it for consumption only after a sufficient contact
period.

Unit Quantity (Flow Rate)	Bleaching Powder** required to make a dosage of 2 ppm	To make a 1% Standard Strength Solution, dissolve in
1000 gallons	0.08 lb or 36 to 40 gms. (about 2 heaped tablespoonsfull).	0.2 gallon or 0.9 litre of water

* 1 ppm = one part per million or 1 lb Cl₂ per million lb of water
** Quantities calculated on the basis of 25% available
chlorine.

Note :

(i) Always use fresh bleaching powder. Preferably within
a week from the date the sealed drum is opened. The
chlorine content in bleaching powder (strength)
drops very rapidly with time. (Fresh bleaching powder
contains 25 to 37% chlorine content)

(ii) Quantity of bleaching powder and/or dosage rate of
solution will need to be increased according to its

life and strength, controlled by the test results of
residual chlorine after 30 minute contact period.

(iii) If there is no equipment to measure the residual
chlorine readily available, the dosage rate should
be such that there is a slight but distinctly
detectable smell of chlorine in the water after the
30 minute contact period.

Equivalents & Conversions

1 heaped tablespoonfull of bleaching powder weighs about 25 gms.
1 heaped cupfull (standard S.L. teacup) weighs about 160 gms.
If a coconut shell scoop is used, standardize its capacity.
1 pound weight   = 453.6 grams
1 gallon(Imp.)   = 4.546 litres and weighs 10 lbs or 4.546 Kg.
1 cubic metre   = 1000 litres
1000 gallons(Imp.) = 4.546 cubic metres

2. HOW BLEACHING POWDER IS ADDED TO WATER SUPPLIES

(i) Drip-feeders are usually used to ensure a regulated
dose of chlorine (bleaching powder solution) into
water supplies.

(ii) Usually 1-2 % solution of available chlorine is
used for drip-feeding purposes. Some of the NWS & DB
schemes use up to 5% strength, depending on the size
of the scheme.

(iii) Estimate the daily requirement of bleaching powder at
about 2 ppm dosage rate or more to begin with and
dissolve this quantity in water to get a concentrated
solution of about 1% in a convenient container.
(For every 0.08 lb. or 40 gm. or 2 heaped tablespoons-
full of bleaching powder, use one fifth of a gallon of
water. See Table.) Make sure that the bleaching
powder is properly dissolved in water without leaving
any lumps of the powder. The initial requirement of
chlorine for a system that has not been using chlorine
may reach up to 10 times the normal dose but the daily

dosage requirement will fall to normal levels gradually.

(iv) Allow this solution to stand for about one hour to let the white lime particles settle down. Decant the slightly milky solution gently into another container or it may be used directly in the drip feeder. If the solution is used after less than an hour of settling further settlement of fine powders can accumulate in the feeder in the form of sludge. To prevent clogging of the feeder, frequent cleaning is advisable.

(v) The choice of the type of feeder will depend on the availability, suitability and convenience of operation for each system (see the examples attached)

(vi) The point of application of the chlorine solution should be such that it will have a chance of getting throughly mixed into the water supply and also have a contact time of 30 minutes before it reaches the first consumer. If such a situation is not possible, the consumers concerned should be warned to consume water only after a storage of 30 minutes or more and they should be ready to accept the use of water with higher doses of chlorine.

(vii) The amount of the 1% chlorine solution used should be adjusted to such a rate that there is at least 0.3 ppm residual chlorine at the end of the distribution system. Make sure that there is a slight but distinctly noticeable smell of chlorine at that point if there is no means of measuring the ppm content.

(viii) At the initial stages it may not be possible to achieve this situation even with 10 times increased dosage rates. But this situation will improve if the maximum tolerable rate of dosage is kept up.

(ix) Constant monitoring of effective chlorination by checking the presence of chlorine at the end as well as in all parts of the distribution system is important, because the quality of the water received at the source changes with weather and season. Regular monitoring will also provide information on possible sources of contamination. Adjust the dosage to meet the chlorine demand.

Warning

Handle bleaching powder and the 1% chlorine solution with great care.

It irritates sensitive areas of skin. Use rubber gloves where
necessary. It is highly corrosive to metals. It will bleach
coloured clothing white. Rinse throughly with water if affected.

Store bleaching powder in a cool and dry place.

TABLE SHOWING BLEACHING POWDER AND WATER REQUIREMENTS TO MAKE 1% CHLORINE
SOLUTIONS AT A DOSAGE RATE OF 2 ppm FOR DIFFERENT RATES OF WATER SUPPLY
IN GALLONS PER UNIT TIME (per hour or per day)

| Rate of supply (gallons)	Amount of bleaching Powder required		Amt. of water in gallons to make 1 % chlorine solution
	lb	or gms.	
1000	0.08	40	0.2
1500	0.12	60	0.3
2000	0.16	80	0.4
2500	0.20	100	0.5
3000	0.24	120	0.6
3500	0.28	140	0.7
4000	0.32	160	0.8
5000	0.40	200	1.0
6000	0.48	240	1.2
7000	0.56	280	1.4
8000	0.64	320	1.6
9000	0.72	360	1.8
10000	0.80	400	2.0

Issued by
Permanent Inter-Agency Committee for the Continuous
Monitoring and Review of Water Quality in Public
Water Supply Schemes

CENTRAL ENVIRONMENTAL AUTHORITY

MINISTRY OF LOCAL GOVERNMENT, HOUSING AND CONSTRUCTION

A. Floating bowl chlorinator to feed chlorine solution at a constant rate

Details of Floating Bowl
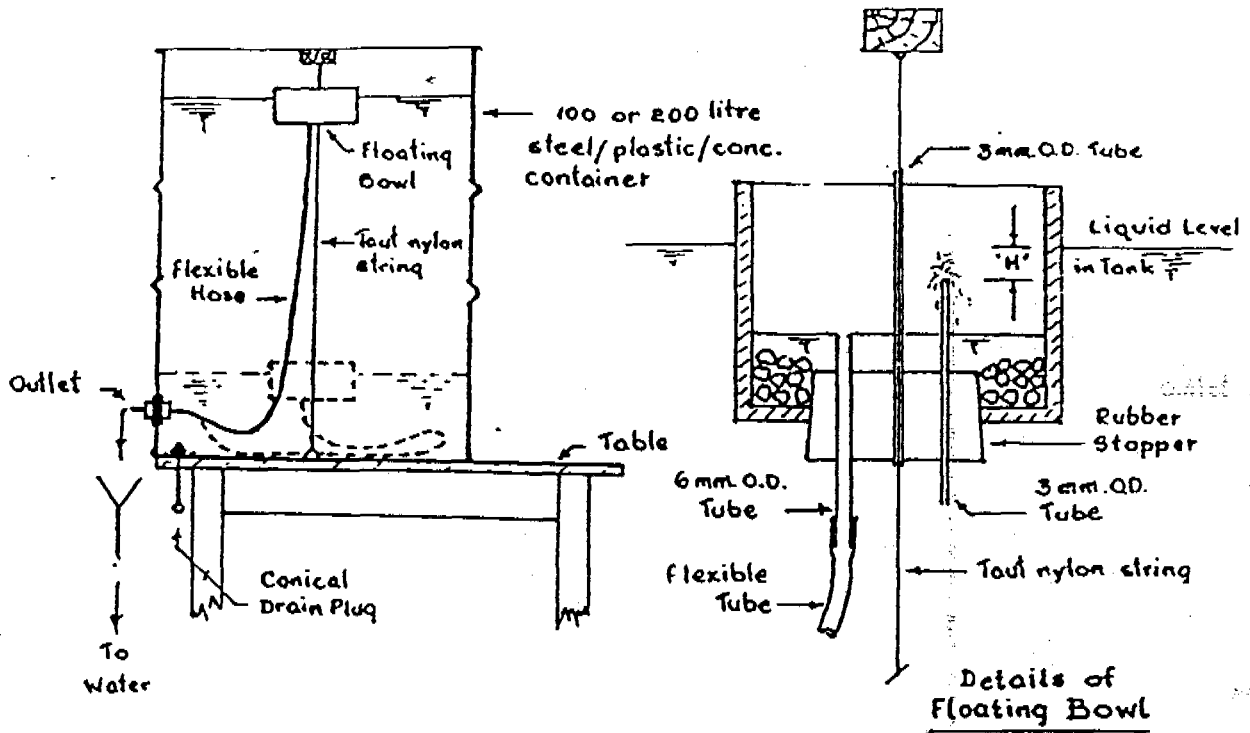
The construction of this simple type of chlorinator is shown above. The "floating bowl" can be turned out of any suitable non-corrodible material. Satisfactory results can be obtained with

1) a large wide-mouthed plastic bottle with the bottom removed,

2) a six inch-diameter PVC pipe with one end cemented or welded, or

3) any available bowl with a hole cut for a rubber bung.

The central 3 mm-O.D. tube with a nylon string passing through it serves to position the bowl in the centre of the tank and to avoid the risk of its tipping over. Placing stones in the bowl helps to keep it upright.

While glass tubing, 3 mm and 6 mm outer diameter (O.D.), can be obtained from dealers in medical and laboratory supplies, ball-point pen casings could also serve the purpose.

The two tubes on either side of the central guide-tube can be moved by sliding to a desired height above the rubber bung. The rate of

drip- ;ed is controlled by th: difference in height " i" between the
outle: cf the 3mm - OD.tube and the level of the liquid in the tank.
The outlet of this tube may be narrowed down, if necessary, to
reduce the rate of flow of the solution to convenient levels.

B. Simple arrangement for feeding chlorine solution
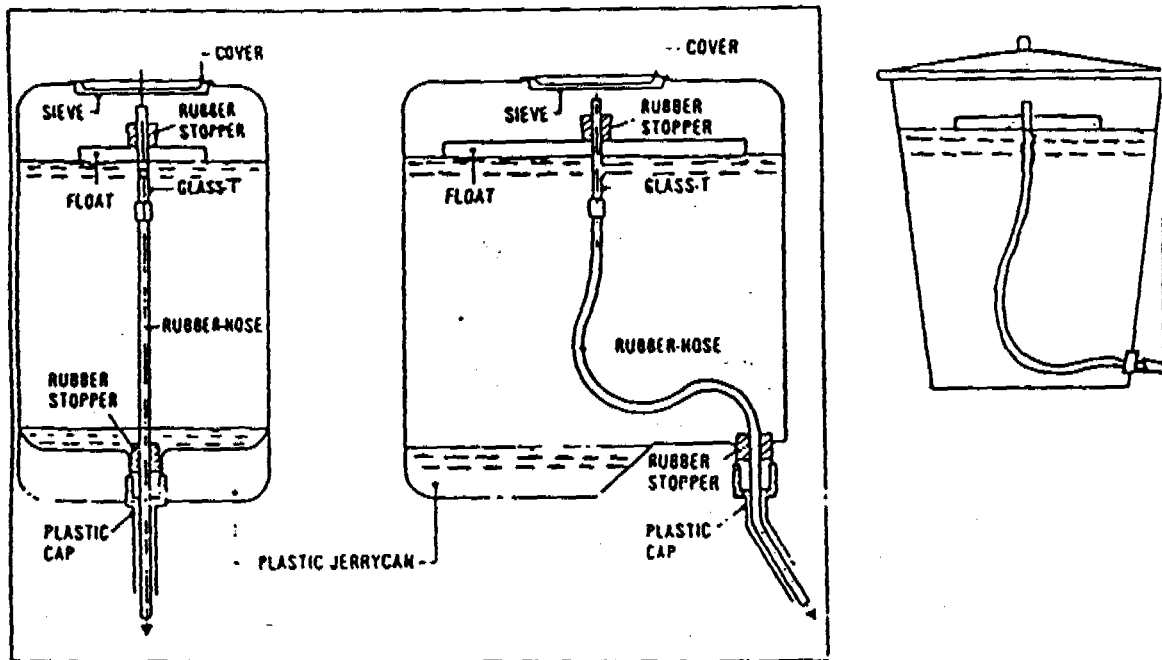
The above arrangement is a simple and useful method for
dispensing chlorine solution for water supplies. However,
the addition of a pinch-cock arrangement or a drip-control
device of a type similar to those used in medical transfusions
is necessary to control the rate of flow of the solution.

Construction

Any suitable container, can or bucket made of plastic or non-
corrodible material is satisfactory. Other components should
also be of non-corrodible material. While the tube inside the
container needs to be very flexible, the outlet tube may be
of a sturdier material. Styrofoam, certain packing materials
or even certain types of soft wood can be used for the float
(certain materials may, however, decompose in a short time).

INFORMATION TO BE INCLUDED IN THE TENDER SPECIFICATIONS
FOR WATER TREATMENT WORKS

(Adapted from Manual on Water Supply and Treatment,
Ministry of Works and Housing, New Delhi, 1977)

General

The principal requirement must be a spacious and convenient layout. The structure should represent a pleasing appearance with aesthetic features forming a balance between function and form. The interiors of the structures shall be eye appealing and in keeping with the objectives of the plant viz., production of pure and wholesome water.

While the mode of design and construction could be a matter of individual choice, it should ensure that all materials, construction and fabrication details for different units including doors and relevant windows conform to the specifications and codes of practice wherever available and in their absence, to the established standards.

Adequate provision shall be made in the civil engineering works for laboratory, office buildings, administration area, sanitary facilities, water supply, etc. The area requirement of these ancillary facilities shall be stipulated. Roadways with adequate lighting shall be provided. Adequate ladders or steps and handrails shall be provided where required for easy access to each unit of the treament plant and wherever necessary, walkways shall be provided. Interconnecting facilities shall be provided to enable the operator to move freely for maintenance and operation of the plant.

All water retaining structures shall be designed in conformity with BS 5337, 1976 or BS 8007, 1987 while the other structures shall be designed according to CP 110, 1972 or BS 8110, 1985.

The tender specifications should include all process requirements and equipment.

A.   Process Requirements

    1.   The following data shall be furnished to the tenderers:

        a)   Raw water analyses comprising monthly average figures preferably for a full year period covering seasonal variations in respect of at least the following. If the full year data is not available, the worst seasonal values may be given:

            o   pH
            o   Turbidity
            o   Total alkalinity
            o   Total hardness
            o   Chlorides
            o   Coliform organisms (MPN)

b) Any other additional data, if the water is known to contain constituents or contaminants which are required to be removed.

- o Phenols
- o Tastes and odours
- o Colour
- o Carbon dioxide
- o Algal content
- o Iron
- o Manganese
- o Hardness (Carbonate and non-carbonate along with magnesium content of water)
- o Fluoride content, and
- o Chlorine demand.

c) Hydraulic data such as the relevant raw water inlet and filtrate outlet levels.

2. The following requirements shall be furnished:

a) The flow requirements of all plant in terms of the net output expected of the plant for a given period of time, say 23 ½ hours a day (allowing for washing of the filters, etc. and also overload capacity.)

b) The quality of the treated water in terms of pH, turbidity, coliform organisms (MPN) and E. coli; and where needed iron, manganese, hardness (carbonate and non-carbonate along with magnesium content of water) fluoride content and colour.

c) Design parameters for various treatment units such as chemical dosing, rapid mixing, slow mixing, sedimentation, filtration and chlorination as well as special processes like aeration, microstraining, iron and manganese removal, fluoride removal, taste and odour control as per specific local reqirements and in accordance with the details furnished in the Manual.

d) A suggested layout of a Water Treatment Works including the following details, to the extent possible.

- o Unit sizes and location structures;

- o Schematic flow diagram showing flow through various units;

- o Piping arrangement including bypasses, showing the material and size of pipes as well as direction of flow;

- o Hydraulic profile of the units showing the flow of water.

- o Contour map of the area including provision for future expansion.

- o Approach roads and water supply facilities for construction purposes.

o   Other information about the site such as proneness to flooding, groundwater table fluctuations, type and nature of soils met up to maximum anticipated depths, soil characteristics like bearing capacity and corrosivity, intensity and duration of rainfall and total annual rainfall, locations of areas for disposal of excavated spoils and of borrow pits if required for filling purposes.

e) The contract should establish where guarantees apply and clearly define their requirements. Performance guarantees must be demonstrated by a test run of specified length or over an agreed period of operation.

B   Mechanical Equipment

1. The following data may be given while inviting tenders for pumping plant:

a) Number of units required to work in parallel.

b) Nature of liquid to be pumped:

    o   Fresh or salt water
    o   Temperature of liquid
    o   Specific gravity
    o   Amount of suspended matter present.

c) Required capacity as well as minimum and maximum amount of liquid the pump must deliver.

d) Suction conditions:

    o   Suction lift or suction head
    o   Constant or variable suction conditions.

e). Discharge conditions:

    o   Maximum/minimum discharge pressures against which pump has to delivery liquid.

    o   Static head description: constant or variable.

    o   Friction head description and how estimated.

f) Type of service: continuous or intermittent.

g) Pump installation: horizontal or vertical position (if vertical type of pit, wet or dry).

h) Power available to drive the pump.

i) Space, weight or transportation limitations.

j) Location of installation

k) Special requirements with respect to pump design, construction or performance.

2. The following requirements may be indicated:

a) The equipment as well as the components shall conform to the relevant standards and in their absence, to any other accepted international or national standards.

b) Any special duty conditions such as temperature, humidity, corrosive atmosphere should be specified.

c) Submerged structural parts except hot rolled sections shall not be less than 6 mm thick under normal atmospheres and 8 mm in aggressive atmospheres.

d) Prime movers and allied components such as electrical motors, starters, switches, reduction gear, drive mechanisms, bearings, plummer blocks, etc., shall be of approved make.

e) All rotating machinery, particularly gears, shall be designed with adequate safety margins and service factors.

f) An itemwise price list of spare part shall be furnished by the tenderer. At least two years' requirement of fast moving spares should be supplied along with the equipment.

g) The supplier of special equipment like softeners, recording gauges, rate controllers, chlorinators, proportioning chemical feeders, meters, etc., shall furnish the services of a competent representative for a specified number of days during a specified period to instruct the plant operating personnel in the maintenance and care of the equipment and to conduct tests and make recommendations for producing the most efficient results.

h) Equipment selection with respect to specifications, spare units, spare parts and servicing can affect maintenance, operating and investment costs. It is the Board's responsibility to incorporate into the contract all requirements and limitations which affect cost. Equipment performance is usually guaranteed by the manufacturer.

The contractor shall furnish bonds covering items of work like mechanical equipment, piping etc. for a specified period as a guarantee of satisfactory operation and correction of any defects in the work, material or equipment furnished by them.

On special equipment extended guarantees, maintenance over a period of time and supervision of or complete installation may be provided by the manufacturer. On most large equipment, the manufacturer provides field service with respect to installation.

H-4

1) All water-submerged parts, rotating mechanical parts, and steel pipes under water shall be adequately protected after surface preparation. Oil, grease, dirt, soil and all surface contaminants from structural and fabricated steel parts shall be removed by cleaning with solvent, vapour, alkali emulsion or steam. Loose rust or paint, weld patter, etc. shall be removed by hand chipping, scraping, sanding, wire brushing and grinding, the bare finished shafting, finished flanges and other mechanical surfaces protected by grease line or rust-protection measures. Structural mechanism supports and superstructures, walkways, handrails, fabricated shafts, etc., shall be protected with at least one coat of primer and two coats of paint.