MINISTRY OF LOCAL GOVERNMENT, HOUSING AND CONSTRUCTION

# NATIONAL WATER SUPPLY AND DRAINAGE BOARD

SRI LANKA



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DESIGN MANUAL D7

WASTEWATER TREATMENT

DRAFT 1

JANUARY, 1989

## WATER SUPPLY AND SANITATION SECTOR PROJECT

(USAID SRI LANKA PROJECT 383-0088)

# DESIGN MANUAL D7

# WASTEWATER TREATMENT

DRAFT 1

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## ABBREVIATIONS

| | |
|---|---|
| BOD | Biochemical oxygen demand (5 d at 20 °C) |
| C | Centigrade |
| CEA | Central Environmental Authority |
| COD | Chemical oxygen demand |
| cm | centimetre |
| cu | cubic |
| d | day |
| F | Fahrenheit |
| g | gram |
| h | hour |
| ha | hectare |
| kg | kilogram |
| l | litre |
| MJ | Mega Joule |
| MLSS | Mixed liquor suspended solids |
| MPN | Most probable number |
| m | metre |
| mg | milligram |
| min | minute |
| ml | millilitre |
| mm | millimetre |
| p | person |
| ppm | part per million |
| rev | revolution |
| SS | Suspended solids |
| s | second |
| sq | square |
| TSS | Total suspended solids |
| uc | microcurie |
| W | Watt |
| y | year |

# SECTION 1

## INTRODUCTION AND SCOPE

This Wastewater Treatment Design Manual is intended to provide NWSDB Engineers with sufficient background knowledge and process design criteria to enable preliminary designs to be prepared for urban wastewater infrastructure projects. It should provide enough information to enable alternative wastewater management strategies to be formulated and evaluated and for the infrastructure to be sized, and therefore costed at least for preliminary estimating purposes. The manual does not deal with aspects of detail engineering design, construction or O&M.

Although the NWSDB has the responsibility for providing urban sewerage systems in Sri Lanka, such systems are limited at present and are likely to remain so because of economic constraints. Until such time as the demand for sophisticated waterborne sewerage systems increases, the NWSDB should retain only a small cadre of personnel who have sufficient knowledge in the field to enable them to prepare feasibility studies and tentative cost estimates. Recourse should be made to consulting firms for detail design assistance until such time as the work load justifies the creation of a comprehensive design team within the NWSDB. Responsibility for the O&M of major urban wastewater collection, treatment and disposal facilities should remain vested in the local authorities.

The manual deals specifically with wastewater from urban areas where properties are provided with piped water supply connections and cistern-flush toilets. It does not deal with low cost on-site technologies suitable for peri-urban, squatter upgrading or rural area projects.

The main topics covered are:

- o Discharge standards
- o Wastewater characteristics
- o Gravity sewers and sewage pumping
- o Treatment systems
- o Ocean outfalls
- o Septic tanks
- o Industrial wastes

The current Sri Lankan standards relating to wastewater discharges and water quality are included in Appendix B.

The manual stresses the need for local field surveys to be carried out wherever possible because of the dearth of information available on wastewater characteristics in Sri Lanka.

This manual was prepared under the USAID Water Supply and Sanitation Sector Project.

SECTION 2

# WASTEWATER DISCHARGE CONTROL REQUIREMENTS

The discharge of untreated wastewater to the environment can result in unacceptable pollution levels which render the environment unsuitable for designated uses. This is recognised in Sri Lanka and the CEA has been established specifically to control environmental quality (Ref.Government of Sri Lanka (1980)). In the context of urban wastewater discharges the two main concerns are adverse public health impacts and environmental degredation generally either of water bodies or land.

## 2.1 Public Health Impacts

It is recognised that inadequate water supplies and defective excreta disposal systems are, among other factors, responsible for a wide range of diseases. In the urban environment where households are provided with piped water connections and cistern flush toilets, the level of water use in the house is generally well in excess of the tentative threshold level of 50 l/d.p below which water-related diseases become manifest. The incidence of water-related disease in such areas is likely to be related to poor hygiene practices, food contamination, and improper use of water and excreta disposal systems, rather than with the levels of provision of water supply and sanitation facilities.

Nevertheless, the discharge of raw domestic wastewater to water bodies or to land does constitute a public health risk. The types and numbers of pathogens in the wastewater are related to the state of health of the contributing community, hence the logical concern to ensure that hospital wastes are adequately treated before discharge. The risk to a person coming in contact with sewage contaminated water, or with sludge on land for example, depends on such factors as the number of pathogens present, the dose required to cause infection, and the person's general health status and level of immunity.

It is not possible to generalise on public health risks because of the multiplicity of factors involved. As a result, general guidelines are used in an attempt to reduce the perceived risks, and in the case of pathogens the coliform bacteria level is usually adopted as a surrogate indicator for pathogens.

Recommended coliform levels for marine waters are discussed in Section 8. The Sri Lankan standards incorporate a coliform level of 5000/ 100 ml (95 percentile) for an inland water source to be used for water supply purposes (see Table B1) regardless of the type of water treatment process to be used. Recent studies in the USA have suggested that faecal coliform levels in treated wastewater discharged to parks should not exceed 500/100 ml, regardless of the recreation activity taking place. Schwebach and others (1988). This is a considerable relaxation on earlier standards and perhaps reflects the fact that necessary infective dose levels of most pathogens are far higher than the levels a person in contact with treated wastewater or sludge would normally encounter.

### 2.1.2 Reduction of Pathogens during Wastewater Treatment

Typical reductions in pathogens afforded by various wastewater treatment processes are shown in Table 2.1. The reductions should be viewed as general guidelines, considerable variations may exist in specific circumstances. A significant proportion of the pathogens present in the sewage are concentrated in the sludge. Traditional sludge treatment processes such as digestion and drying will result in considerable reductions but will not necessarily eliminate pathogens completely. High temperature processes to 60°C such as composting or heat treatment are probably more effective. Relative efficiencies of sludge treatment processes are shown in Table 2.2.

## 2.2 Discharge Standards

In order to control environmental quality it is necessary to control wastewater discharges through the use of standards. The logical concept is to base treatment levels on the desired quality of the environmental sector receiving the wastewater (inland waters, marine waters, land). This concept implies that designated environmental quality criteria should be specified for different environmental uses. It is not the purpose of this Design Manual to discuss the wide range of environmental quality criteria, but reference must be made to the existing Sri Lankan standards, and these are given in Appendix B.

The concept of relating wastewater discharges to environmental use is embodied in the Sri Lankan standards, with tolerance limits already established for public water supply sources (Table B1), and for discharge of water to public sewers and for irrigation (Tables B4 and B5). Tolerance limits for wastewater discharges to inland waters and marine waters are also laid down (Tables B2 and B3), although the limits for discharge to inland waters relate only to industrial wastes and the limits are not strictly related to receiving water use. However, pollution control is still in its infancy in Sri Lanka and there is little justification for legislating compre- hensive standards for all possible environmental use alternatives when monitoring and corrective action resources are minimal.

There are as yet no standards for domestic sewage discharges to inland waters. Until such time as domestic sewage standards are promulgated it would seem logical to base treatment levels for such wastes on the BOD and SS levels of 30 and 50 mg/l respectively stipulated in Table B2 for discharge of industrial wastes to inland waters.

The standards in Table B2 are specified as maximum (not to be exceeded) values. Studies on variations in domestic sewage effluent quality elsewhere have shown that for 95 percentile limits (not to be exceeded in more than 5% of samples), the associated mean values are typically as follows:

| | BOD (mg/l) | SS (mg/l) |
|---|---|---|
| 95 percentile | 30 | 50 |
| Mean | 16 | 30 |

TABLE 2.1

EFFECT OF VARIOUS TREATMENT SYSTEMS ON

PATHOGEN LEVELS

| Treatment System | Reduction from Level in Raw Sewage (%) | | |
|---|---|---|---|
| | Bacteria | Viruses | Ova[a] |
| A Primary Settling | 30 | <50 | 20–80 |
| B Stabilisation Lagoons[b] | 90–98 | <50–98 | 70–98 |
| C Activated Sludge/ Biological Filtration | 90–98 | 90–98 | 70–98 |
| D Stabilisation Lagoons + Maturation Lagoons | 99 | 95–98 | 70–99 |
| E Disinfection + C or D | >99 | 99 | >95 |
| F Sand Filtration + B or C | >99 | 99 | >99 |
| G Disinfection + F | 100 | >>99[c] | >99 |
| H Chemical Coagulation + F | 100 | >>99[c] | >>99 |

a Ova are concentrated in the sludge, removal rates refer to liquid stream only.

b Treatment efficiency depends on the number and type of lagoons.

c Some evidence suggests that ozone and chlorine dioxide are more effective than chlorine for killing viruses. Four log reductions may be achieved with these treatment systems.

TABLE 2.2

RELATIVE EFFICIENCIES OF SLUDGE TREATMENT PROCESSES

IN REDUCING PATHOGEN LEVELS

| Treatment Process | Relative reduction | | |
|---|---|---|---|
| | Poor | Moderate | Good |
| Raw sludge storage | *Ascaris* ova<br>*Taenia* ova | Viruses<br>Bacteria | |
| Digestion | *Ascaris* ova<br>*Taenia* ova | Hookworm ova | Viruses<br>Bacteria<br>*Entamoeba* crysts |
| Composting | | | Viruses<br>Bacteria<br>Fungi<br>Helminth ova |
| Lime treatment | *Ascaris* ova | | Bacteria |
| Heat treatment | | | Viruses<br>Bacteria<br>Helminth ova |
| Irradiation | *Ascaris* ova | | Bacteria<br>Viruses |

Ref. Carrington (1978)

Larger variations would probably occur for stabilization lagoon systems, particularly for SS. It is recommended, therefore, that the Sri Lankan standards in Table B2 be interpreted as 95 percentiles and the sewage treatment processes described in Section 6 be designed to produce an effluent quality of BOD 20 mg/l and SS 30 mg/l.

SECTION 3

# WASTEWATER CHARACTERISTICS

In order to design any wastewater collection and treatment system it is necessary to define present and projected future wastewater characteristics. Survey data for Sri Lankan wastewaters are virtually non-existent, with the exception of some specific industrial waste surveys. For urban areas where monitoring data are not available it is recommended that field surveys be carried out in selected areas to determine current loads. The surveys should be carried out over 24-hour periods in order to reflect diurnal variations in waste flows and to identify infiltration rates in the sewerage system. The survey areas should be selected on the basis of representative housing types, covering the predominant socio-economic levels, including if possible examples of housing developments likely to be typical in the future.

The main components of the wastewater for the purpose of treatment system design and discharge to the environment are flow, BOD, SS, nitrogen compounds, posphorous and pathogens. In the absence of specific and comprehensive Sri Lankan field survey data on wastewater characteristics, it would be inadvisable to speculate on the full spectrum of wastewater components. However, estimates of domestic sewage flow and BOD loads are presented since these two parameters constitute the fundamental design inputs for virtually all categories of treatment process.

## 3.1 Existing Wastewater Characteristics

In the absence of comprehensive survey data estimates of sewage flows can be made from water use records. The proportion of the water use which is returned as sewage varies primarily in accordance with the socio-economic characteristics of the household. In affluent areas in the USA, for example, the return ratio may be as low as 50% because a substantial proportion of the water is used for garden irrigation. In Asia, return ratios typically fall within the range 70 to 90%. An average return ratio of 80% is probably realistic for urban areas in Sri Lanka unless field survey data indicate otherwise.

The data in Table 3.1 give typical water use characteristics for metered connections in Greater Colombo and the regions. The data exclude standposts since the major water use of standposts is in situ for bathing and laundry with the wastewater being discharged to the storm drainage system.

On the basis of an average urban area house occupancy of 6.0 and a return ratio of 80%, domestic sewage flows can be estimated at about 140 l/d. p in Greater Colombo. In regional urban areas the use of a return ratio of 80% may not give a realistic picture since the main reasons for lower regional water use compared to Colombo are that socio-economic levels tend to be lower in the regions and alternative water sources such as wells are readily available. As a result the regional sewage flow is likely to be much closer to the metered water use than in Colombo. For regional urban areas, therefore, a domestic sewage flow of say 100 l/d. p would probably be more realistic pending field survey verification.

TABLE 3.1

TYPICAL METERED WATER USE DATA

| Category | Water Use - $m^3$/month connection | |
|---|---|---|
| | Greater Colombo | Kandy Region |
| Domestic | 30.6 | 19.5 |
| Commercial | 106 | 25.2 |
| Industrial | 579 | - |
| Government | 606 | - |
| Tourist Hotel | 1157 | 17 |
| Institutional | 210 | - |
| School | - | 43.4 |
| Religious | - | 43.4 |
| Port | 2705 | - |

NOTES. a) Colombo data for June-August 1988

b) Kandy data for July 1988 for 22 schemes.

The data in Table 3.1 also give an indication of water use in non-domestic categories. The overall metered water use in Greater Colombo is 48 $m^3$/month connection, but 91% of the water use is accounted for by the domestic, commercial and Government categories. For designing specific drainage area sewers allowances should be made for such non-domestic uses. Typical allowances are as follows but they should be verified by field checks.

| Land Use | Factor |
|---|---|
| Schools | 0.1 persons/pupil |
| Hospitals | 3 persons/bed |
| Office | 3 persons/100 $m^2$ |
| Commercial | 3 persons/100 $m^2$ |
| Hotels | 4.4 persons/room |

In order to determine BOD loads, field surveys are mandatory. In Sri Lanka the per capita BOD load is likely to be in the region of 35 to 50 g/d depending on the socio-economic level of the area. Faecal matter will account for 25 to 30 g of BOD/d.p, with vegetarians tending to have higher BOD levels than meat eaters on account of the higher weight and moisture content of vegetarian faeces. The proportion of the BOD load above the faecal matter base of 25 to 30 g is accounted for entirely by sullage loads.

## 3.2 Projected Wastewater Characteristics

Projections of future per capita domestic sewage loads need to take into account a number of factors such as increased availability of water supply and increased affluence resulting in a more widespread use of such appliances as washing machines. Based on the existing field survey data for a range of specific socio-economic levels covering the predominant housing categories, the likely future housing category projections can then form the basis for estimating future waste loads. In this way there is more likelihood that future load projections will reflect planned development rather than relying on a general average load factor for the whole urban area.

There is often a tendency to assume that per capita loads in less developed countries will eventually attain the same levels as those experienced in the USA and some European countries. This may not necessarily be the case. It is important to realise that the future BOD and flow increase ~~will be attributable~~ to sullage and not to excreta. It is highly probable that per capita excreta BOD loads in Sri Lanka are in fact higher than those in the USA and other more developed countries because of the relatively low meat consumption in Sri Lanka and the consequent higher proportion of vegetarians and neo-vegetarians in the contributing population.

It is, of course, a matter of conjecture as to whether or not the per capita BOD loads will ever attain the 60 to 80 g/d range recorded in Europe and the USA. If garbage grinders are prohibited, as they are in many urban areas in Europe, the BOD load should never exceed 60 g/d.p. Even in the more affluent higher socio-economic levels of Southeast Asia, such as Singapore, domestic sewage characteristics are today only around 180 l/d.p and 51 g BOD/d.p.

Previous projections of sewage flows for the Colombo area were based on an average annual increase of about 0.9% over the period 1978 to 2000 (Engineering Science, 1981). This is not unrealistic, it would imply a year 2000 domestic sewage flow of about 160 l/d.p assuming an occupancy of 6.0, or about 175 l/d.p if average occupancy rates fell to 5.5. Sewage flow increase rates of 2 to 3% per year were common in the more developed countries during the 1960s and 1970s as a direct result of increasing affluence and the more widespread provision of water using appliances. Even in Singapore the domestic sewage flow and BOD loads increased at annual rates of 2.0 and 1.3% respectively over the ten-year period to 1983.

It is considered that unless field survey data indicate otherwise, an annual sewage flow increase of 1% be adopted for major urban areas in Sri Lanka where water supply restrictions are not a constraining factor. A target BOD load of 50 g/d.should be adequate at least up to the year 2000.

## SECTION 4

## GRAVITY SEWERS

### 4.1 Hydraulic Criteria

Wherever technically and economically possible gravity sewers should be designed to maintain minimum scouring velocities at average daily flows anticipated in the design year in order to prevent accumulation of settleable materials and to retard and, if possible, prevent corrosion resulting from the formation of hydrogen sulphide. In order to ensure that slimes and deposits are removed from the pipe walls and invert at least once during the day, peak flow velocities (in the design year) should be no less than 0.8 m/s since slime growth occurs rapidly at velocities below 0.76 m/s. Where both sewage temperatures and BOD are high, a higher velocity should be aimed for. Under such circumstances a velocity of 1.0 m/s would probably safeguard the system from sulphide- generated corrosion.

Ideally, flow velocities under present peak flow conditions should be at least 0.7 m/s. In practice, the range of flows, from present day to the design year may be too large to accommodate these velocity criteria, particularly where both BOD and temperature are high. In such cases flushing and additional maintenance of the sewers may be required during the initial years of operation and in some cases it will inevitably be necessary to use pipe materials which are able to withstand acid attack.

With respect to erosion of pipe materials, velocities of 6 m/s or higher may be satisfactory but it is recommended that as a general guideline a maximum velocity of 4 m/s be adopted.

The major factor affecting the roughness coefficient in sewers, apart from the pipe material and method of jointing, is the slime which builds up as the pipe as exposed to sewage. The velocity is related to the roughness coefficient by the Manning formula:

$$V = (1/n)\ R^{0.67} S^{0.50}$$

Where V = velocity (m/s)
R = hydraulic radius (m)
S = hydraulic gradient (m/m)
n = Manning's roughness coefficient

The following Manning 'n' roughness values are recommended for sewer pipes flowing full for a velocity of about 0.75 m/s:

| | |
|---|---|
| Spun concrete | n = 0.014 |
| Asbestos cement | 0.014 |
| Clay | 0.013 |
| uPVC | 0.013 |
| Vertically cast concrete | 0.014 |

Flow velocities in part-full pipes are generally lower than those predicted using the same roughness value for the pipe flowing full. For example, the n value for a sewer flowing half full is about 1.22 times greater than the n value for the sewer flowing full. The largest difference typically occurs at a flow depth of about 0.25 diameters, when the n value increases to about 1.3 times the pipe full value. The effect of a higher roughness value in early years of operation caused by slime accumulations would be to reduce the effective carrying capacity of the sewer. For example, if the n value was 0.015 rather than 0.014, the pipe full capacity would typically be reduced by about 8 percent. Since higher roughness values are more likely to arise in early years when flows are low, a slight error in the n value adopted for design should not be significant.

The data in Table 4.1 indicate desired minimum and maximum gradients calculated on the basis of peak flow velocities being in the range of 0.8 to 4.0 m/s for a pipe roughness (Manning n) of 0.014. In certain circumstances factors such as rates of development, topography, ground conditions, economics and the need to prevent hydrogen sulphide emission may necessitate different velocities being adopted. In flat areas, the control of sulphides by providing adequate velocities may result in excessive excavation costs. An economic comparison should be made between the cost of additional excavation, possible additional pumping costs and the use of traditional less expensive pipe materials and the cost of more expensive pipe materials which are inert to sulphide attack.

## 4.2 Peak Factors and Infiltration

In order to determine the required capacity of interceptor sewers it is necessary to determine the peak to average flow ratio. Peak factors traditionally exclude infiltration and are calculated as follows:

(average domestic flow x peaking factor) +
(average industrial flow x peaking factor) +
infiltration.

Typical peak factors for domestic flow excluding infiltration range from about 2.5 for a population of 100 000 through 3.3 for 10 000 people to greater than 4.0 for a population smaller than 2000.

In the Greater Colombo area measured peak factors in the old sewerage network are heavily influenced by the high infiltration rates caused by the use of cement mortar joints. It is recommended that the peak factor used for domestic sewage flows be as given below. This relationship has been well-proven elsewhere and is similar to the formulae used in other urban areas in Asia.

Peak Factor = $4.7\ P^{-0.11}$

Where P = Population (in thousands)

TABLE 4.1

RECOMMENDED GRAVITY SEWER DESIGN CRITERIA

| Factor | Unit | Value |
|---|---|---|
| Hydraulic Design | | |
| Minimum velocity | m/s | 0.8 |
| Maximum velocity | m/s | 4.0 |

Minimum/Maximum gradient for sewer flowing full (m/100m)
(For Manning 'n' 0.014).

| Diameter (mm) | Maximum | Minimum |
|---|---|---|
| 100 | 44.0 | 1.76 |
| 150 | 25.0 | 1.00 |
| 225 | 14.6 | 0.58 |
| 300 | 9.9 | 0.40 |
| 375 | 7.4 | 0.29 |
| 450 | 5.8 | 0.23 |
| 525 | 4.7 | 0.19 |
| 600 | 3.9 | 0.16 |
| 750 | 2.9 | 0.12 |
| 900 | 2.3 | 0.09 |
| 1050 | 1.9 | 0.07 |
| 1200 | 1.6 | 0.06 |
| 1350 | 1.3 | 0.05 |
| 1500 | 1.2 | 0.05 |

| Factor | Unit | Value |
|---|---|---|
| Structural Design | | |
| Minimum depth to top of sewer | m | 1.0 |
| Minimum depth to top of sewer in roads | m | 1.2 |
| Minimum diameter of collection sewer | mm | 200 |
| Minimum diameter of property connection | mm | 100 |
| Maximum manhole spacing: | | |
| Sewers 200 to 600 mm in diameter | m | 100 |
| Sewers greater than 600 mm in diameter | m | 150 to 200 |
| Minimum manhole diameter: | | |
| Sewers 900 mm or less in diameter | mm | 1200 |
| Sewers 1000 to 1200 mm in diameter | mm | 1500 |
| Sewer invert drop at 90 degree bends | mm | 30 |
| Inverted siphons: | | |
| Minimum number of pipes | No. | 2 |
| Design velocity | m/s | 1.2 |

The formula gives the following results:

| Equivalent Contributing Population | Peak Factor |
|---|---|
| 1000 | 4.7 |
| 1000 to 10 000 | 4.7 to 3.6 |
| 10 000 to 50 000 | 3.6 to 3.1 |
| 50 000 to 100 000 | 3.1 to 2.8 |

Peak factors for industrial flows should be limited to 2.0 times average flow as discussed in Section 7.

Infiltration rates are often expressed as a proportion of dry weather sewage flow. However, since infiltration is a function of the quality of construction of the sewerage system, it is recommended that infiltration be based on a sewered area or pipe length basis.

For new housing developments comprising medium/high-cost housing it should be possible to limit infiltration to 6 $m^3$/d.ha. This will necessitate a high standard of sewer and manhole construction. In particular, strict pipe testing before backfill will be necessary and careful supervision of pipe bedding, trench backfilling and the construction of property connections must be carried out. In addition, no brick manholes should be used, all pipes must be with flexible joints and manhole covers should be locked to prevent them from being stolen.

For connecting existing sewered developments to trunk sewers, an infiltaration allowance of 18 $m^3$/d.ha should be used to reflect the general poor quality of sewer, particularly lateral and manhole construction, and the use of right-joint pipes.

## 4.3 Sulphide Problem

Sulphides occur in sewage in both soluble and insoluble form. Each person normally discharges 1 to 1.5 g S/d, of which about 70% arises from sulphate in urine. Sulphates are present in most water supplies and may be as high as 80 g S/l in some areas. Industrial wastes from tanneries, textiles, and chemical operations often contain sulphides in high concentration.

In the absence of oxygen, sulphate reducing bacteria in the slime layer on the sewer wall reduce sulphates to sulphides. The outer layer of slime is usually aerobic but between the outer layer and the sewer wall an anaerobic zone exists which may be up to 3 mm thick and it is in this zone that sulphate reduction occurs. The sulphides diffuse out of the anaerobic zone and are either oxidised to thiosulphates in the aerobic zone or pass into the sewage stream as dissolved sulphides. Depending upon the pH and other factors the sulphide escapes to the sewer atmosphere as hydrogen sulphide gas ($H_2S$). When $H_2S$ gas escapes from solution it may be oxidised on the pipe wall above the sewage flow by the bacteria, *Thiobacillus*, to produce sulphuric acid which causes corrosion.

There appears to be evidence of sulphide corrosion problems in the existing Colombo sewerage network, despite the dilution afforded by infiltration/inflow. Most of the sewage arriving at the pump stations is septic.

It is recommended that at the preliminary design stage sulphide generation calculations be performed for all sewer sections in order to check the likelihood of sulphide production. PVC liners can be used for concrete pipes, particularly those of 900 mm diameter and above in the downstream sections of the main trunk sewers. In addition, imported VC pipes or imported PVC lined collar-jointed pipes may have to be used for some of the smaller diameter sewers in critical areas with sacificial concrete being specified for intermediate diameters.

## 4.4 Property Connections

Property connections should be of an adequate diameter to reduce the problem of blockage. Because service connections receive only intermittent flow they invariably experience intermittent formations of stoppages during normal operations and these are removed by wave action rather than by the maintenance of a minimum flow velocity. A typical construction gradient would be 1.67 m/100 m. A high quality of workmanship to eliminate backfalls and protruding joints is essential.

## 4.5 Structural Considerations

A summary of recommended structural criteria regarding the design of gravity sewers is included in Table A. The major items for gravity sewers are as follows:

- o Manholes are required wherever the sewer diameter or gradient changes, at junctions, and at regular intervals for straight run sewer lines.

- o Drop manholes should generally not be considered because of hydrogen sulphide release. If the change in invert elevation is high and a drop manhole is necessary, corrosion can be prevented by the use of glass reinforced plastic linings and epoxy mortar benchings. The drop manhole should be provided with a means to clean out materials which lodge in the drop pipe.

- o A terminal manhole should be provided at the upstream end of each sewer.

- o Service connections to sewers should discharge at an angle of approximately 45° to the sewer in the direction of flow. On large trunk sewers in developed areas, consideration should be given to construction of small diameter, parallel sewers to which service connections are made. The small diameter sewers should discharge to the trunk sewer at manholes.

- o Reticulation sewers at head manholes should be sufficiently deep so as to make future extensions and connections possible. A minimum depth of 1.5 m would be reasonable.

- o Maximum depth to invert in urban areas and for open cut construction should preferably be less than 6 m to minimise construction problems in poor ground conditions and to reduce surface disruption and costs.

- o Inverted siphons should be avoided if possible but if feasible alternatives are not available then they should be of dual pipe and encased with reinforced concrete to prevent damage and flotation. Adequate provision should be made for cleaning the siphon and sharp bends should be avoided with the rising leg of the siphon being limited to approximately 15 % slope.

- o Pipe bedding should be granular or concrete depending upon availability of materials and cost. For flexible jointed pipes a concrete bed must be broken at each pipe joint. By using a suitable bedding the strengh of a pipe relative to applied loads can be increased by a "bedding" factor. However, it is absolutely essential that the pipe laying specification be followed since bedding, jointing and backfill are integral components of the pipe structure. This is particularly critical for flexible wall pipes.

- o Minimum diameters of reticulation sewers should not be less than 200 mm. Pipe to pipe connections should not be made for diameters exceeding 200 mm.

- o As far as is possible, routes of sewers and force mains should be selected so as to cause minimum disruption of traffic. Tunnels (auger boring, pipe jacking) should be considered for sensitive sewer routes if feasible alternatives are not available.

## 4.6 Materials

The following factors are among the more significant that should be evaluated before selection of materials for sewerage facilities:

- o Life expectancy
- o Previous local experience
- o Resistance to internal and external corrosion
- o Resistance to abrasion
- o Roughness coefficient and its effect on flow characteristics
- o Ease of handling and installation

- Structural strength both in place and during handling
- Cost of supply, transport and installation
- Local availability

The following materials are normally used for sewers:

- Vitrified Clay (VC)
- Asbestos Cement (AC)
- Reinforced Concrete (RC)
- Steel
- Cast Iron
- High density polyethelene (HDPE)
- Unplasticised polyvinylchloride ( PVC)
- Glass reinforced plastic (GRP)

Vitrified clay pipe with flexible rubber joints offers superior resistance to corrosion and no lining is required. It does require careful handling during transport and laying as it is easily damaged. Clay pipes manufactured in Sri Lanka are unglazed, have a low strength and do not have a flexible watertight joint.

Asbestos cement pipe can be used for both gravity sewer and pressure pipe applications. Pressure pipe is manufactured in various classes suitable for certain working pressure ranges, whereas gravity sewer pipe is manufactured to suit various loading conditions and required crushing strengths. Only autoclaved asbestos cement pipes should be considered for sewers. Pipe joints consist of a collar with two rubber rings. A bitumen lining can be applied to the pipe. Although the light weight of AC pipe is an advantage in construction of sewers and pressure mains the pipe requires careful handling and offers poor resistance to sulphide corrosion even when lined, and should only be used if there is no likelihood of corrosion.

Reinforced concrete pipe is manufactured with various degrees of corrosion protection. Several wall linings are available including high alumina cement mortar, polyvinylchloride sheets, and increased wall thickness as sacrificial concrete. PVC linings can be applied to pipes of 750 mm diameter and above. Several types of pipe joints are available, including the spigot and socket type with rubber rings. Reinforced concrete pipe is manufactured in classifications related to pipe strength and the maximum allowable load to which the pipe can be subjected. Sri Lankan manufactured concrete pipes do not incorporate watertight flexible joints.

Steel pipe is manufactured with linings of bitumen or sulphate resistant cement mortar. Spigot and socket joints, flanges, or mechanical joints are commonly used on small diameter pipelines of up to 750 mm and welded joints are used on larger diameter lines. The best use of steel pipes is for inverted siphons, pump stations, and in large diameter pressure mains.

Cast iron pipe is manufactured with either flanged or spigot and socket joints. Because of its relatively high cost and weight, the application of cast iron pipe is usually limited to internal piping at pump stations and above-grade pressure mains.

High density polyethylene and unplasticised polyvinychloride pipes are becoming more popular particularly for the smaller diameter sewers. Since both pipes are flexible the design of the pipe/trench system is much more critical than for rigid pipe materials. Glass reinforced plastic (GRP) is an alternative, but also flexible, pipe material which is suitable for sewers.

Flexible pipes rely on the side support of the earth backfill around the pipe for stability more than do non-flexible pipes. Consequently, in an urban environment, where the side supports can be removed during construction of underground services in the future, pipe failures could be more frequent. Also, flexible wall pipes may not be suitable in poor ground conditions.

Since at the present time the number of experienced sewerage contractors in Sri Lanka is limited and since there is virtually no local experience with large diameter flexible pipes it is recommended that only imported vitrified clay and reinforced concrete pipes be used for all sewers which are the direct responsibility of local authorities. This will also ensure homogeneity of lateral connections and future extensions. Flexible pipes could be used for property connections within the property boundary. All pipes, regardless of material, should have flexible joints. Recommended sewer pipe materials are summarised in Table 4.2.

Manholes should be constructed generally of precast sections with a concrete surround. Manholes in locations where sulphide corrosion is expected can be protected with epoxy mortar benching and the walls can be either PVC lined or coated with epoxy. If the latter protection is used it will require checking and repairing during routine maintenance. Brick manholes should not be used because of the risk of high infiltration rates. It is also recommended that step irons are not used and that sewer maintenance gangs ~~be provided~~ with aluminium ladders for access. All manhole covers should be locked to prevent them being stolen.

## 4.7 Sewer Construction Techniques

There are two basic sewer construction techniques. These are open trench construction and tunnelling which is considered to be any construction method which results in the placement or construction of an underground conduit without continuous disturbance of the ground surface.

TABLE 4.2

RECOMMENDED SEWER PIPE MATERIALS

(TECHNICAL ASPECTS)

| Item | Up to 300 mm | 375 to 875 mm | 900 mm and above |
|---|---|---|---|
| Material | Vitrified clay[a] | Reinforced[b] concrete | Reinforced concrete |
| Lining | None | RC: Sacrificial concrete | Polyvinylchloride sheet |
| Joints | Spigot and socket with rubber ring | Spigot and socket with rubber ring | Spigot and socket with rubber ring |
| Standard of Manufacture | BS 65 and 540 | Concrete: BS 5991 | AS 1342 |

a. VC can be used up to 600 mm diameter if economic

b. RC pipe with sacrificial lining is the preferred material. If corrosion is a major problem VC or PVC-lined concrete are preferable alternatives.

#### 4.7.1 Open Trench Construction

The main recent improvements in techniques for open trench construction are in the location of existing services and the improvements in excavating and shoring systems for open trenches. Location of existing services is important in the urban environment to prevent disruption to critical supply during construction and to minimise utility costs. Increasing use of non-metallic pipes makes disturbance location of such pipes difficult, but ground probing radar is being developed as well as other methods that involve inserting a transmitter (sonde) into the pipeline.

Most of the recent techniques in trench excavation have been developed primarily for small services such as gas and electricity, requiring narrow shallow trenches, and are not really applicable to sewer construction. Various manufacturers have developed patented shoring systems for trench construction to speed construction and increase safety for construction personnel.

#### 4.7.2 Tunnel Construction

Tunnel construction methods applicable to sewer construction are:

- Auger boring methods
- Pipe jacking
- Mining methods

Auger Boring. Auger boring methods involve forcing a line of pipes through the ground in a hole formed by an auger driven from a shaft. The main limitations in the use of the system are adverse ground conditions, flatness of grade and obstructions. Significant differences in hardness of the ground often result in deflection of a bore whilst boring across the interface, hence this should be avoided if possible. Preferably, lines should be designed with slopes of 1 in 80 or steeper so that practical variations in boring may result in slopes of 1 in 100 or at the worst 1 in 200. Obstructions including underground services may limit the placement of the shafts and may also require complete exposure prior to boring to ensure safe conditions. Auger boring is capable of providing full face holes of about 200 to 300 mm diameter and up to about 900 mm diameter with reaming. Maximum distances which have been bored between shafts is 100 m, the distances being constrained by spoil transportation and auger friction.

Pipe Jacking. Pipe jacking is a method of forcing a line of pipes through the ground while excavation is carried out at the face with a shield. The excavated material in all cases is removed as a slurry.

The method of excavation for conventional pipe jacking is by hand but some pipe jacking has been carried out using partial or full face machines. Pipe jacking systems have been used with all types of shields and machines, and jacking is now an alternative lining system for a tunnel. Pipe jacks have been used in conjunction with most forms of ground stabilisation including compressed air, chemical consolidation and dewatering. Excavated material is removed by skip, conveyors or pneumatic transmission except in the case of slurry machines where the excavated material is removed in the slurry.

Shafts for pipe jacking can be of any shape or size necessary to accommodate the permanent works and temporary jacking set-up. The conventional form of pipe jacking can be carried out within shafts of up to 3 m diameter or sections 3 m by 2 m. A base heading may be necessary at the bottom of the shaft to accommodate the jacking equipment.

In the UK tolerances on pipe jacks are generally specified as plus or minus 75 mm in line and 50 mm in level, although much closer tolerances are normally achieved. As the limiting minimum diameter for conventional pipe jacking is about 900 mm, the effective diameter of the finished system can be reduced if required by either installing a smaller pipe or by reshaping the invert. A relatively recent development in pipe jacking is the fullface remote control machine. These are manufactured by a number of companies. In Japan such machines are in use for a range of diameters of 450 mm and larger but recent developments in Germany have resulted in a machine capable of laying pipes 150 to 750 mm diameter. This latter machine jacks a large diameter (about 850 mm) pipe and a smaller sleeve pipe is inserted enabling one machine to lay a variety of pipe diameters. This is an advantage over conventional fullface machines which are generally used for a specific diameter.

The use of pipe jacking in urban areas is constrained by the need for long jacking pits. These range from 7 to 9 m in length for conventional fullface machines to 5 m for the German remote control machine, although German manufacturers are now attempting to limit the pit to 4 m long by 3 m wide.

In Germany, tolerances for fullface machine pipe jacking are generally limited to less than 30 mm in height and 100 mm in width from the specified axis. Machines are designed generally to drive lengths of 100 m of pipe.

All pipe jacking systems involve the purchase of expensive specialised equipment, and it is unlikely that any single local contractor will purchase the equipment without a specific contract on which it can be used.

<u>Mining Methods</u>. Mining methods are generally applied to conduits with finished diameters of 1.8 m or larger which are constructed using shields, fullface boring machines or by open face mining or heading.

With a shield it is necessary to install a primary lining of sufficient strength to support the surrounding earth and to provide a backstop for the jacks which advance the shield. Boring machines generally have cutters mounted on a rotating head which advances into the heading. Machines may be braced against the walls of the excavation or the primary lining.

Open face mining is manual or machine assisted excavation with timber or steel supports for the excavation.

In all tunnelling systems a secondary lining is generally installed for sewers and is formed from precast or concrete guniting or pipes with the outer voids being grouted. Shafts are required for construction and their number and location depend upon the need for compressed air, the size of tunnel and depth below ground. Shafts do not need to be located directly over the tunnel, they can be located in an adjacent area with access to the works through a short connecting tunnel.

# SECTION 5

## PUMP STATIONS AND PRESSURE MAINS

Pump stations are required wherever the topography is too flat to permit gravity sewer construction and whenever it is necessary to transport flows against the natural grade for delivery of wastewater to a treatment plant or adjacent drainage area. Pump stations include both low-lift stations for pumping into a gravity system and pump stations delivering into pressure mains. Table 5.1 summarises the recommended design criteria for pump stations and pressure mains. Typical designs for different types of pump stations are shown in Figures 5.1 to 5.4. Other significant principles for the design of wastewater pumping facilities are described below:

- Pump stations should be either of the screw or centrifugal pump type. Maximum lifts for single-stage screw pump stations should be up to 10 m but the maximum lift depends on screw diameter. Where centrifugal pumps are necessary, submersible units can be used for a total station capacity (at peak flow) up to about 200 l/s. For larger stations wet-well/dry-well systems should be used.

- Pump station structures should be designed for the anticipated design year flow and initial pumping capacity should provide for flows expected no less than 10 years in the future.

- All fittings should be flanged and pipework should be of flanged ductile iron.

- To avoid potential flooding problems, pumps in dry well/wet well configurations should either be driven by vertical drive shafts with the motors located above flood levels or be submersible motor, close-coupled pumps.

- The number of pumps should be such that the wet well volume does not become excessively large and the number of pump starts does not exceed the manufacturers' recommended maximum. Variable speed pumps should generally be avoided although a more detailed analysis for individual pump stations should be carried out at detail design stage.

- Pump operation should be automatically controlled using a wet-well level sensing system which sequences pump operation with the rise and fall of the water surface.

- Maximum water level in the wet well should, when possible, be set no higher than the sewer invert.

- Pump stations should have sufficient capacity to deliver the expected peak flow when one of the largest pump units is out of operation.

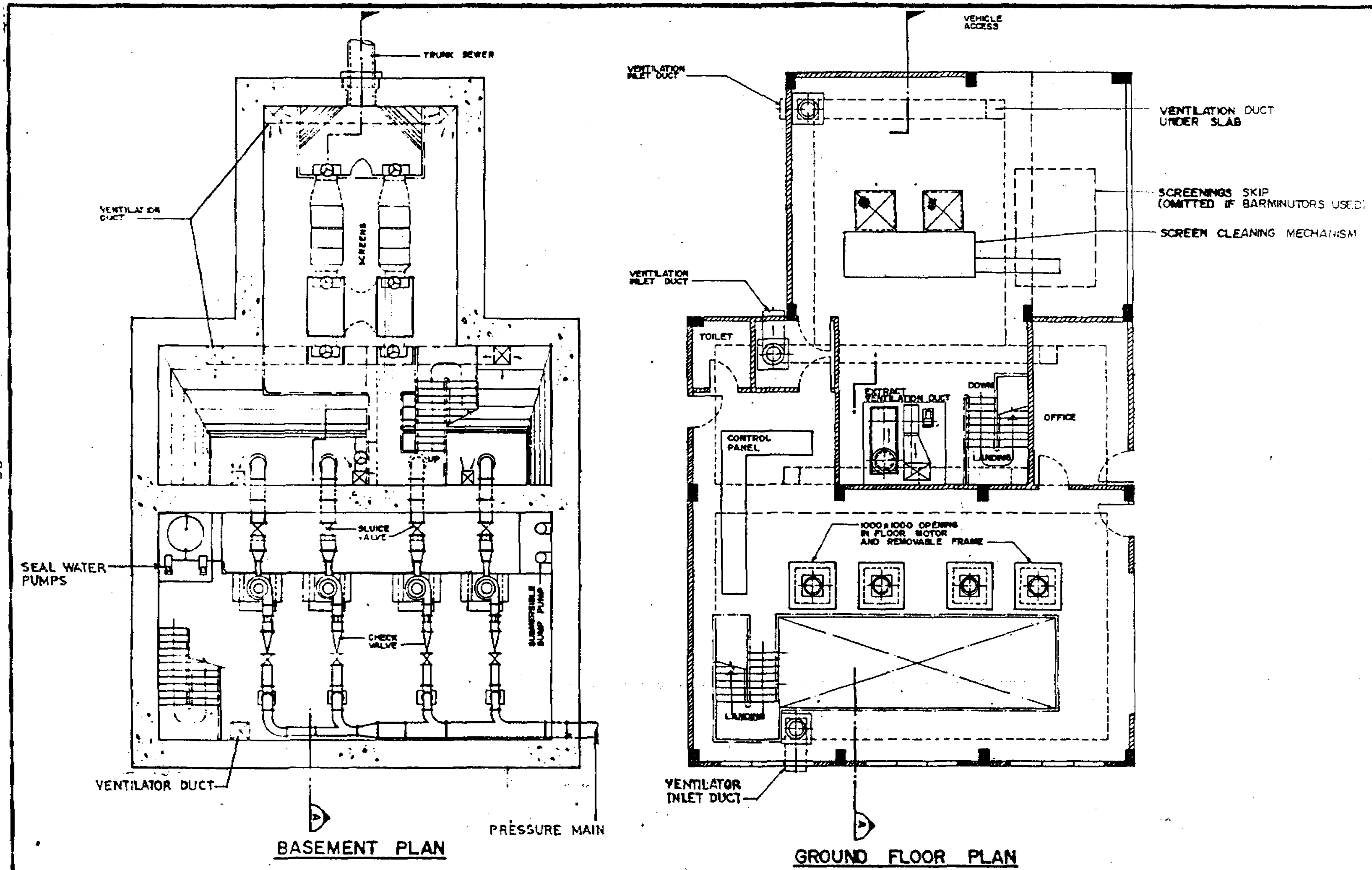
TABLE 5.1

RECOMMENDED DESIGN CRITERIA FOR PRESSURE MAINS

AND PUMP STATIONS

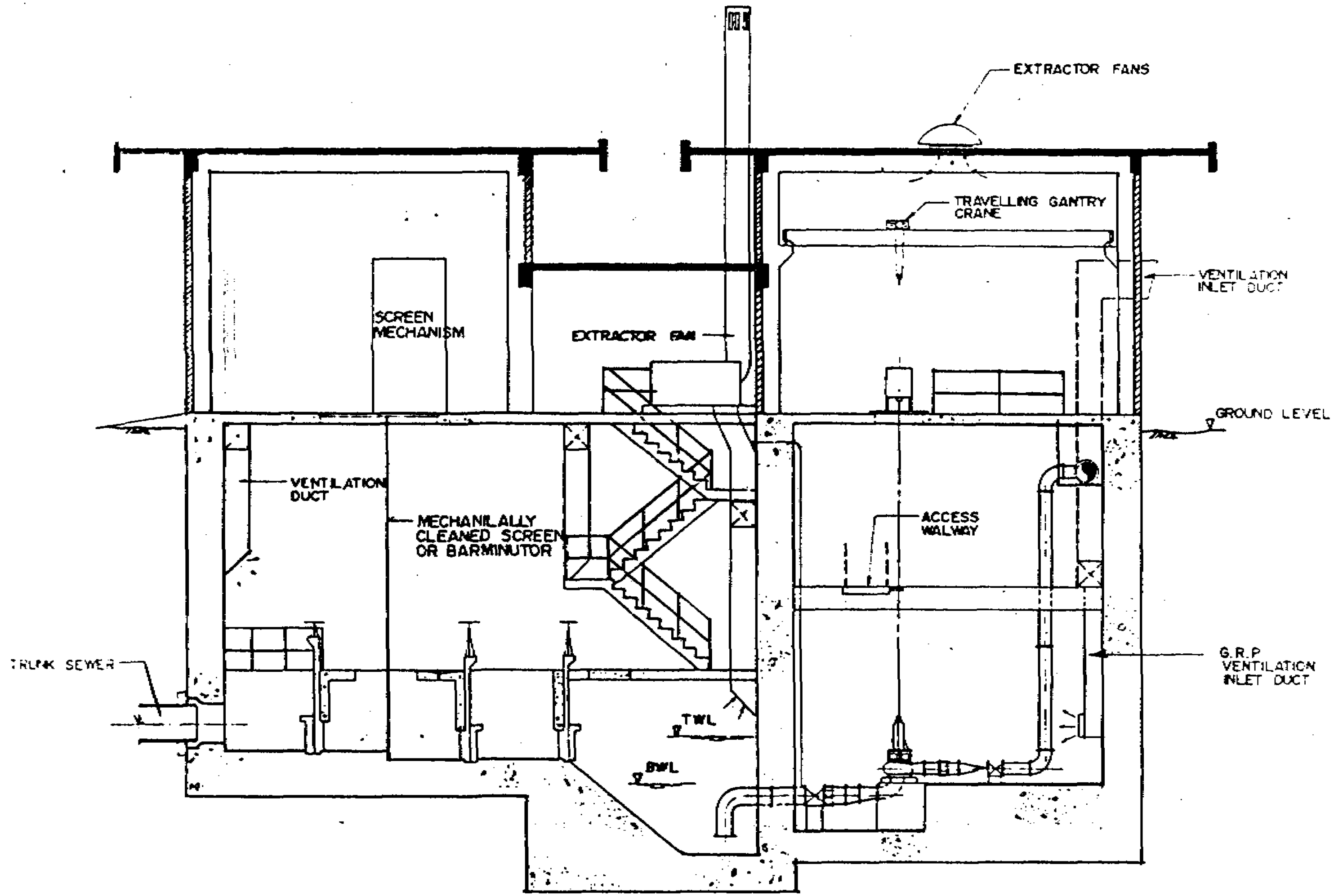
| Factor | Units | Value |
|---|---|---|
| Pressure Mains | | |
| Flow velocity: | | |
| Minimum at present peak flow | m/s | 0.75 |
| Maximum at ultimate peak flow | m/s | 4.0 |
| Economic design velocities:[a] | | |
| Maximum at peak flow in initial phase | m/s | 1.5 |
| Maximum at ultimate peak flow | m/s | 3.0 |
| Friction coefficient (Hazen-Williams) "C" | | |
| Asbestos cement pipe | - | 120 |
| Cast iron or ductile iron pipe | - | 110 |
| Steel pipe | - | 110 |
| Minimum depth to top of pipe | m | 0.9 |
| Pump Stations | | |
| Maximum velocity in discharge piping | m/s | 2.4 |
| Minimum number of pumps at pump stations (including standby) | No. | 2 |
| Minimum pump cycle time | min | 10 |

a These are typical values, economic design velocities should be developed for local conditions and power costs.




TYPICAL PUMP STATION PLANS

FIGURE 5.1

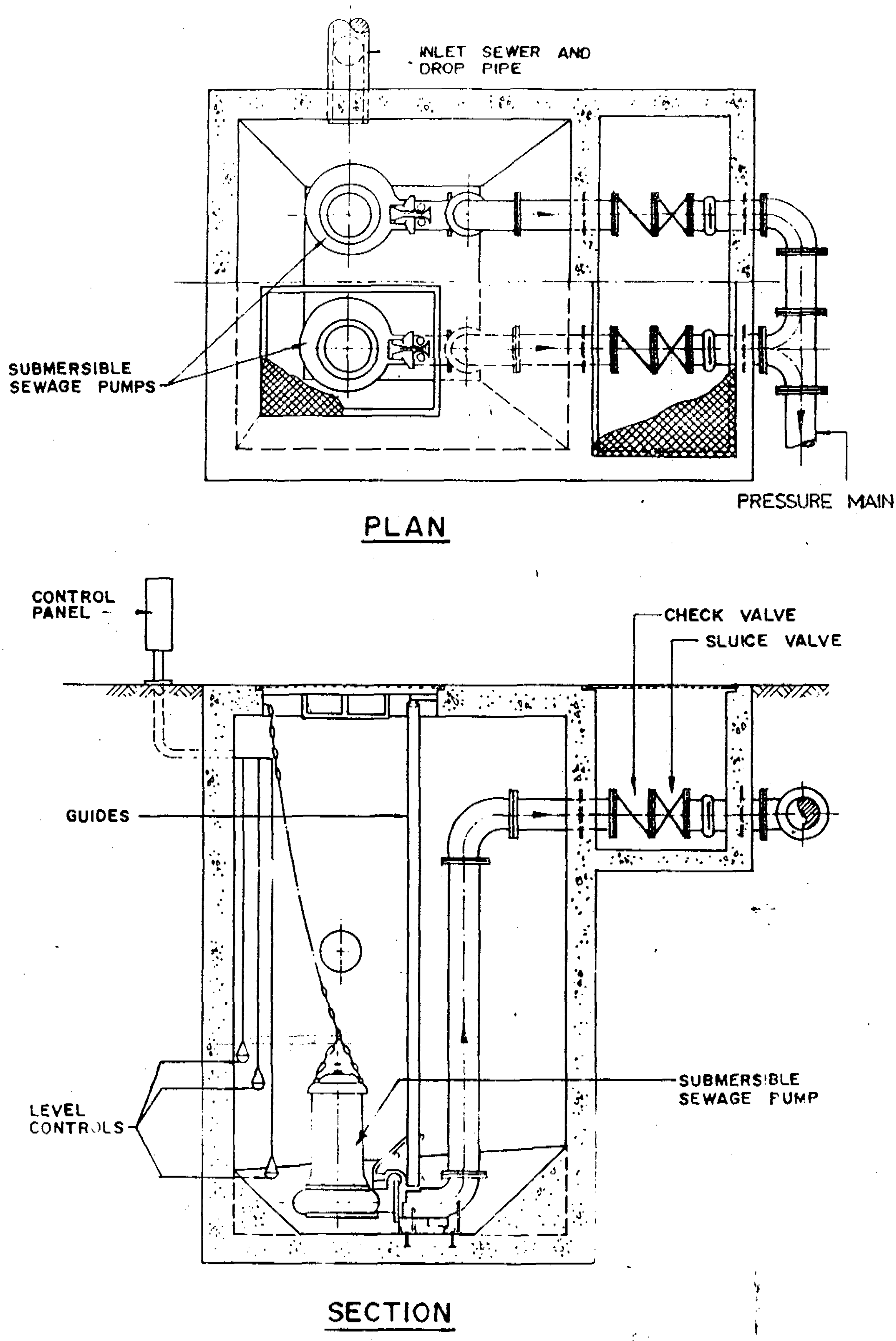



SECTION A - A

# TYPICAL PUMP STATION SECTION

(NOT TO SCALE)

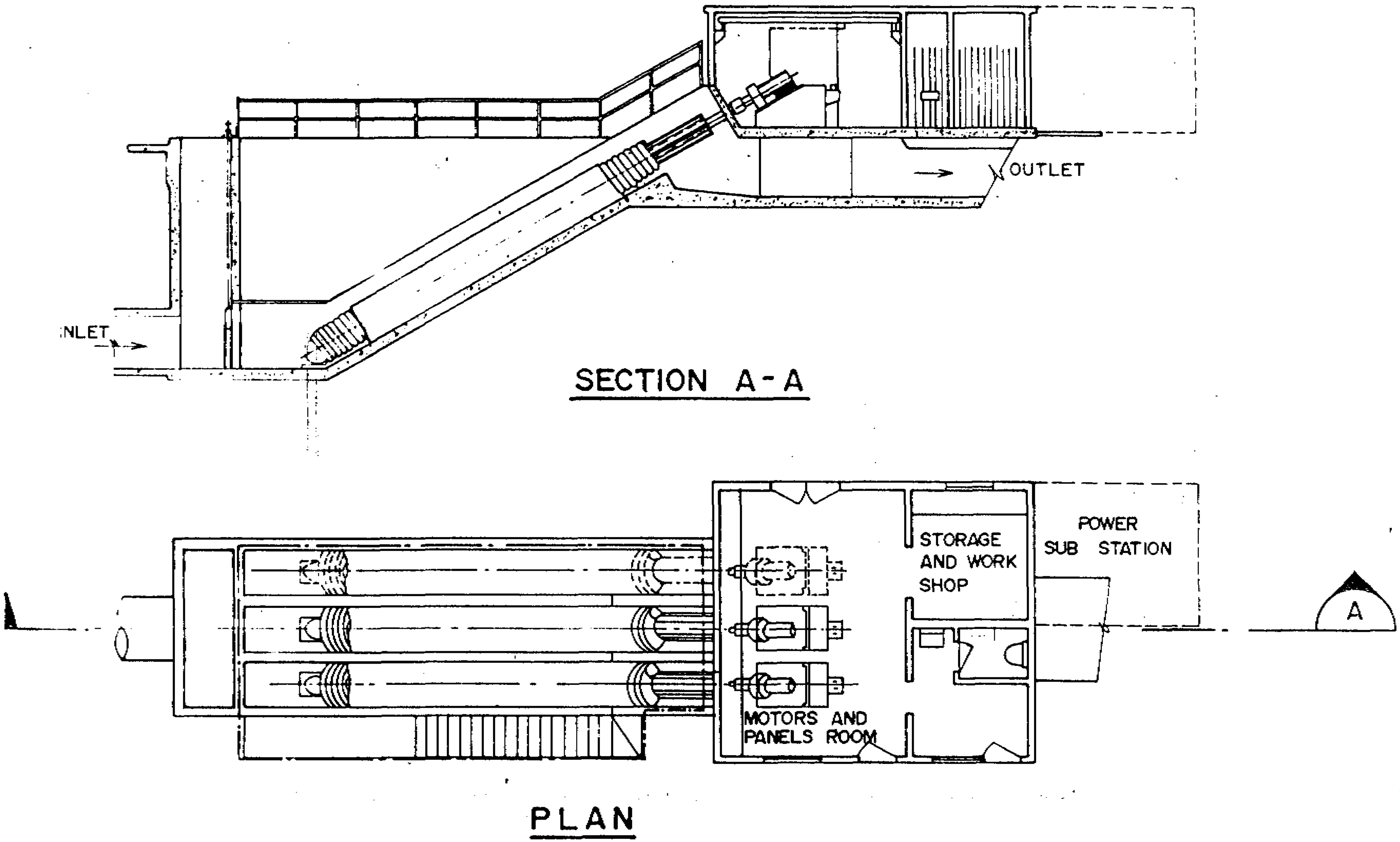
FIGURE 5.2




TYPICAL SUBMERSIBLE PUMP STATION

(NOT TO SCALE)

FIGURE 5.3




TYPICAL SCREW PUMP STATION

(NOT TO SCALE)

FIGURE 5.4

- o Centrifugal pumps should have a speed no higher than 970 rev/min and should be suitably designed to avoid plugging or jamming by coarse solids and pumps should be preceeded by trash racks to eliminate large solid materials. Size of trash rack openings will depend on pump size but will generally be between 50 to 150 mm.

- o Wet well hoppers should be built with slopes no flatter than 2 horizontal to 1 vertical and preferably steeper.

- o Wet well areas should be adequately ventilated for safety of operation and odour control. Dry well areas should have ventilation to provide a safe and appropriate working environment.

- o Major pump stations should be provided with electricity from two independent sources (looped supply) and be given priority restoration by the power authority when outages do occur. When security of electrical power supply cannot be assured, the use of standby generators or engine drives as well as in-system storage and by-pass should be considered. By-passed discharges under emergency conditions should be directed to a suitable point of disposal.

- o Staged construction of pressure mains and installation of pump station equipment should be considered in situations where the ultimate flows greatly exceed present flows. Where recommended design velocities cannot be achieved with a single pipeline, staged construction should be evaluated.

- o Surge relief should be provided as required at each pump station.

- o Ideally, pressure mains should be built with a rising gradient. Air release valves should be provided at high points and washouts at low points.

Submersible pumps should be fixed speed and a standby unit should be provided. The pumps should be able to allow 100 mm objects to pass without clogging. Pump speeds should be no higher than 1500 rev/min and preferably lower than 970 rev/min.

## SECTION 6

## WASTEWATER TREATMENT SYSTEMS

The bases for selection and comparison of alternative wastewater treatment systems include cost, ease of operation and maintenance, adaptability, land requirement and ease of expansion or modification.

Treatment systems for organic wastes such as sewage can be categorised as fixed or suspended growth systems. The following discussion covers the range of available processes but emphasis is given to stabilisation pond processes which are highly appropriate for climatic conditions in most of Sri Lanka and are the least onerous in terms of operation and maintenance and sludge disposal.

### 6.1 Preliminary Treatment

All treatment systems are preceeded by screening and flow measurement. In addition activated sludge and biological filtration systems will be preceeded by grit removal facilities. Typical design criteria for screening and grit removal are as follows:

| | | |
|---|---|---|
| Screens | Limiting velocity | 1 m/s |
| | Opening | 20 mm |
| Aerated Grit Chamber | Retention at peak flow | 3 min |
| | Depth: width ratio | 1: 2 |

### 6.2 Fixed Growth Systems

There are basically two types of fixed-growth systems: the biological (trickling) filter and the rotating biological contactor (RBC). In the former, the sewage is distributed such that a thin film of liquid passes over a fixed medium. Air, passing through voids in the medium, diffuses through the liquid to the biomass film. The RBC rotates thin plastic discs to which the microorganisms are attached, through a tank containing the sewage.

#### 6.2.1 Biological Filters

The biological filter consists of a bed of medium such as rock, slag or plastic through which settled sewage trickles. Sewage is applied to the medium through a travelling distributor. Biological growths become attached to, and form a slime layer over, the medium.

An underdrain system collects the treated wastewater and any detached biological solids and the effluent is then further treated. The underdrain system is also important as a porous structure through which air can circulate to keep the bed aerobic.

The systems considered require preliminary treatment, primary and secondary settlement and sludge treatment. Recommended design criteria are set out in Table 6.1 and a typical flow diagram is shown in Figure 6.1. Filters incorporating a plastic medium are generally up to 8 m deep while filters incorporating a mineral medium are generally about 2 m deep. The specific surface areas of different media vary from about 69 $m^2/m^3$ for crushed gravel to about 240 $m^2/m^3$ for plastics. Because plastic medium has a higher cost per unit volume compared to mineral medium, plastic medium filters are often used for treatment of high strength industrial wastes or for partial treatment of strong domestic sewage where advantage can be taken of their properties of high voidage and specific surface area. However, they are also widely used for treating normal strength domestic sewage where land is not available for mineral medium filters.

### 6.2.2 Rotating Biological Contactors

The rotating biological contactor takes the form of a series of closely spaced circular discs mounted on a shaft. The assembly is submerged, almost to shaft level, in a trough through which wastewater passes. The shaft is rotated slowly in the direction of the wastewater flow.

Biological growths become attached to the surfaces of the discs and eventually form a slime layer (biomass) over the disc. The rotation effects oxygen transfer, keeps the biomass in an aerobic condition and also causes excess biomass to slough from the discs into the mixed liquor and out of the process basin. This sloughing maintains a uniformly thick biomass and prevents clogging of the discs. Some RBC manufacturers add air to the trough to cause rotation of the discs and to increase the dissolved oxygen content of the mixed liquor.

The systems considered require preliminary treatment, primary and secondary settlement and sludge treatment. Recommended design criteria are set out in Table 6.1 and a typical flow diagram is shown in Figure 6.1.

Plant sizes, in population equivalents, exist from 300 and smaller to 400 000 with the most common applications in the 5000 to 20 000 range. For packaged plants a balancing tank is generally required where the peak to average flow is more than 2.5 to 1.

## 6.3 Suspended Growth Systems

In high-rate growth systems, oxygen for the biological process must be supplied by mechanical means. This is the primary difference between high-rate suspended growth systems and comparable fixed growth systems. In low-rate suspended growth systems (stabilisation ponds), oxygen is supplied by natural surface re-aeration from wind and by photosynthesis of algae.

TABLE 6.1

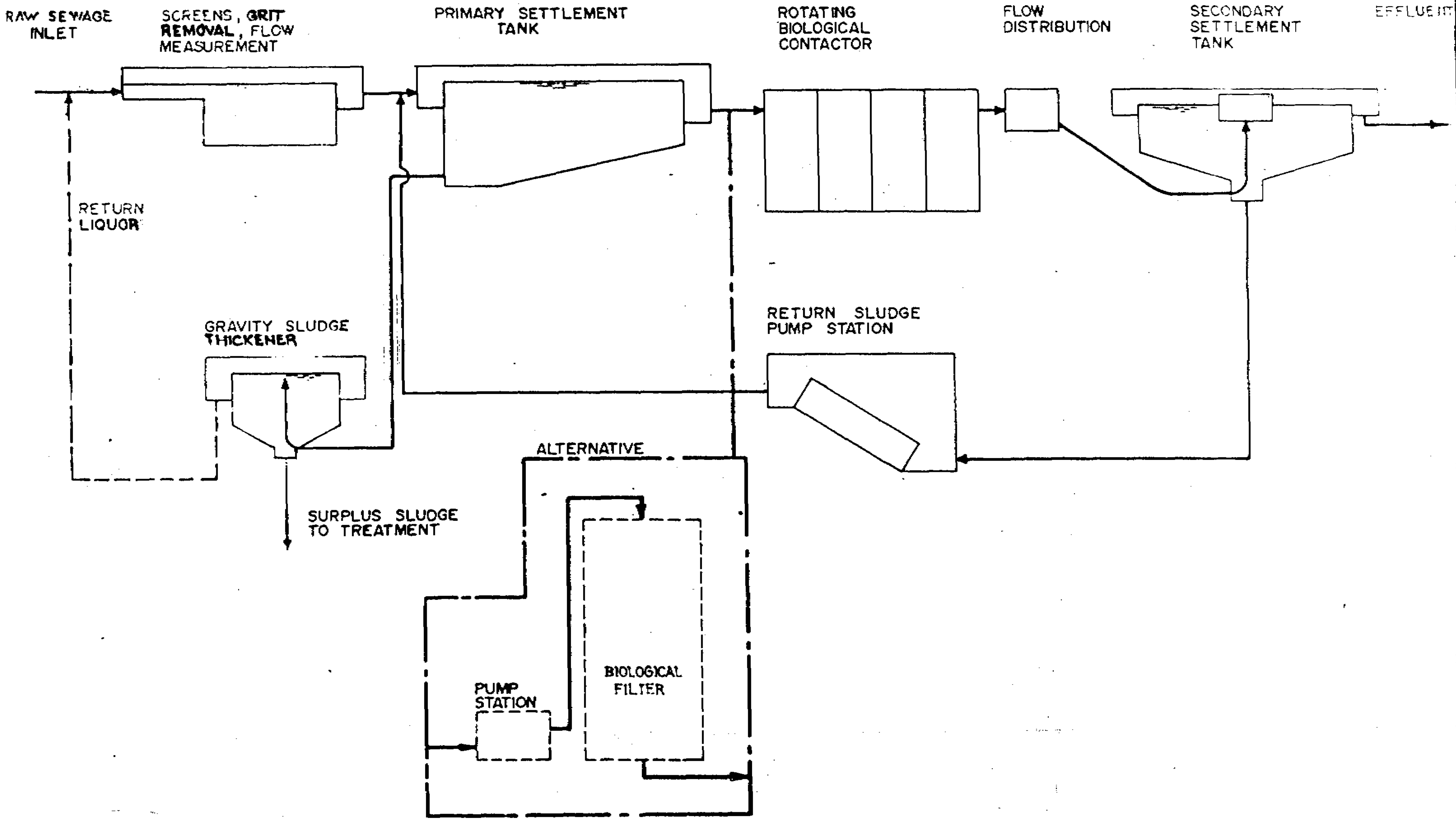
DESIGN CRITERIA FOR FIXED GROWTH SYSTEMS

| Process | Parameter | Design Criteria |
|---|---|---|
| Primary Settlement | Overflow rate at peak flow | 60 $m^3/m^2.d$ |
| | Retention at peak flow | 1 h |
| | Weir loading at peak flow | 500 $m^3/m.d$ |
| Rotating Biological Contactor | BOD specific loading | 15 to 20 $g/d.\ m^2$ |
| | Tank volume | 5 $l/d.\ m^2$ |
| | Maximum disc dia. | 3600 mm |
| | Maximum peripheral velocity | 0.3 m/s |
| Biological Filter (mineral medium) | Organic loading | 0.1 kg $BOD/d.\ m^3$ |
| | Hydraulic loading | 1 $m^3/m^2.d$ |
| Biological Filter (plastic medium) | Organic loading | 1 to 5 $m^3$ |
| | Hydraulic loading | 36 to 72 $m^3/m^2.d$ |
| Secondary Settlement Tank | Overflow rate at peak flow | 32 $m^3/m^2.d$ |
| | Retention at peak flow | 1.5 h |
| | Weir loading | 200 $m^3/m.d$ |

Filter depths typically 1.8 m for mineral medium and 6 to 8 m for plastic medium.

For nitrification using RBCs the hydraulic loading rate on the nitrifying discs should be limited to about 0.05 $m^3/m^2.d$ medium surface.

For nitrification with mineral and plastic medium filters provide a second filter with a maximum hydraulic load of 0.75 $m^3/m^2.d$ and 30 $m^3/m^2.d$ respectively.




FLOW DIAGRAM FOR BIOLOGICAL FILTER TREATMENT SYSTEMS

FIGURE 6.1

### 6.3.1 Activated Sludge Systems

Conventional Activated Sludge. Prior to aeration preliminary treatment is effected by screens and grit removal after which the sewage is settled in primary settlement tanks to remove gross solids. The sludge removed is then further treated before ultimate disposal. Effluent from the primary settlement tanks then passes to the aeration tanks.

Retention in the aeration tanks is between 6 and 8 hours and the sludge age is normally between 4 and 6 days. Recommended design criteria are set out in Table 6.2 and a typical flow diagram is shown in Figure 6.2.

Oxygen Enriched Activated Sludge. This process is similar to the conventional activated sludge process but the aeration tanks are usually covered, and high purity oxygen is fed into the tanks. Mixing is generally by mechanical aerators, although an alternative system uses fine bubble diffusers to inject oxygen into the mixed liquor in open tanks. The mixed liquor suspended solids are higher than in conventional activated sludge and the retention time in the tank is shorter.

Oxygen is generated using either cryogenic or PSA (molecular sieve) techniques depending upon oxygen output requirements.

The system requires primary and secondary settlement and sludge treatment. Recommended design criteria are set out in Table 6.2 and a typical flow diagram is shown in Figure 6.2.

There are more than 100 oxygen enriched plants in operation around the world, but they are sophisticated and require experienced operations personnel and are not recommended for domestic sewage treatment in Sri Lanka at the present time.

Deep Shaft Process. This process is a high intensity oxygen transfer system which exploits the high hydrostatic pressure, long gas-liquid contact time, small bubble size and high but uniform turbulence within a deep shaft typically 60 m deep. The system is a proprietary system.

Sewage is circulated through a downcomer to the bottom of the shaft and returns through a concentric outer or adjacent riser shaft. Air is introduced part way down the downcomer. Mixed liquor overflowing from the shaft contains micro-bubbles and must be passed through a bubbler stripper before settlement. New process designs by the manufacturers now reduce the volume of the shaft and replace the bubble stripper by an aeration tank followed by conventional settlement tanks. Some systems use flotation thickeners for separation of the suspended biomass. Return activated sludge is recirculated to the shaft.

The process requires primary and secondary settlemernt and sludge treatment. Recommended design criteria are set out in Table 6.2 and a typical flow diagram is shown in Figure 6.3.

TABLE 6.2

DESIGN CRITERIA FOR ACTIVATED SLUDGE TREATMENT

| Process | Parameter | Design Criteria |
|---|---|---|
| Primary Settlement Tank | Overflow rate | 60 $m^3/m^2.d$ at peak flow |
| | Retention | 1 h at peak flow |
| | Weir loading | 500 $m^3/m.d$ at peak flow |
| Conventional Aeration Tanks | Organic loading | 0.4 kg BOD/kg MLSS |
| | Mixed liquor suspended solids | 2000 mg/l |
| | Oxygen requirement | 1.0 kg $O_2$/kg BOD removed[a] |
| | Retention period | 6 h[b] |
| Secondary Settlement Tanks | Overflow rate | 100 kg/d.$m^2$ equivalent to 40 $m^3/m^2.d$ 2500 mg/l |
| | Retention | 1.5 h at peak flow |
| | Weir loading | 200 $m^3/m.d$ peak flow |
| Oxygen system Aeration Tanks | Organic loading | 0.6 kg BOD/kg MLSS |
| | Mixed liquor suspended solids | 5000 mg/l |
| | Oxygen requirement | 1 kg $O_2$/kg BOD removed |
| | Retention period | 2 h |
| Secondary Settlement Tanks | Overflow rate | 40 $m^3/m^2.d$ |
| | Retention | 1.5 h at peak flow |
| | Weir loading | 200 $m^3/m.d$ peak flow |
| Deep Shaft System | Organic loading | 1 kg BOD/kg MLSS |
| | Mixed liquor suspended solids | 4000-6000 mg/l |
| | Oxygen requirement | 1 kg $O_2$/kg BOD applied |
| | Depth | Typically 60 m |
| | Hydraulic loading rate (secondary settlement tank) | 24 $m^3/m^2.d$ peak flow |

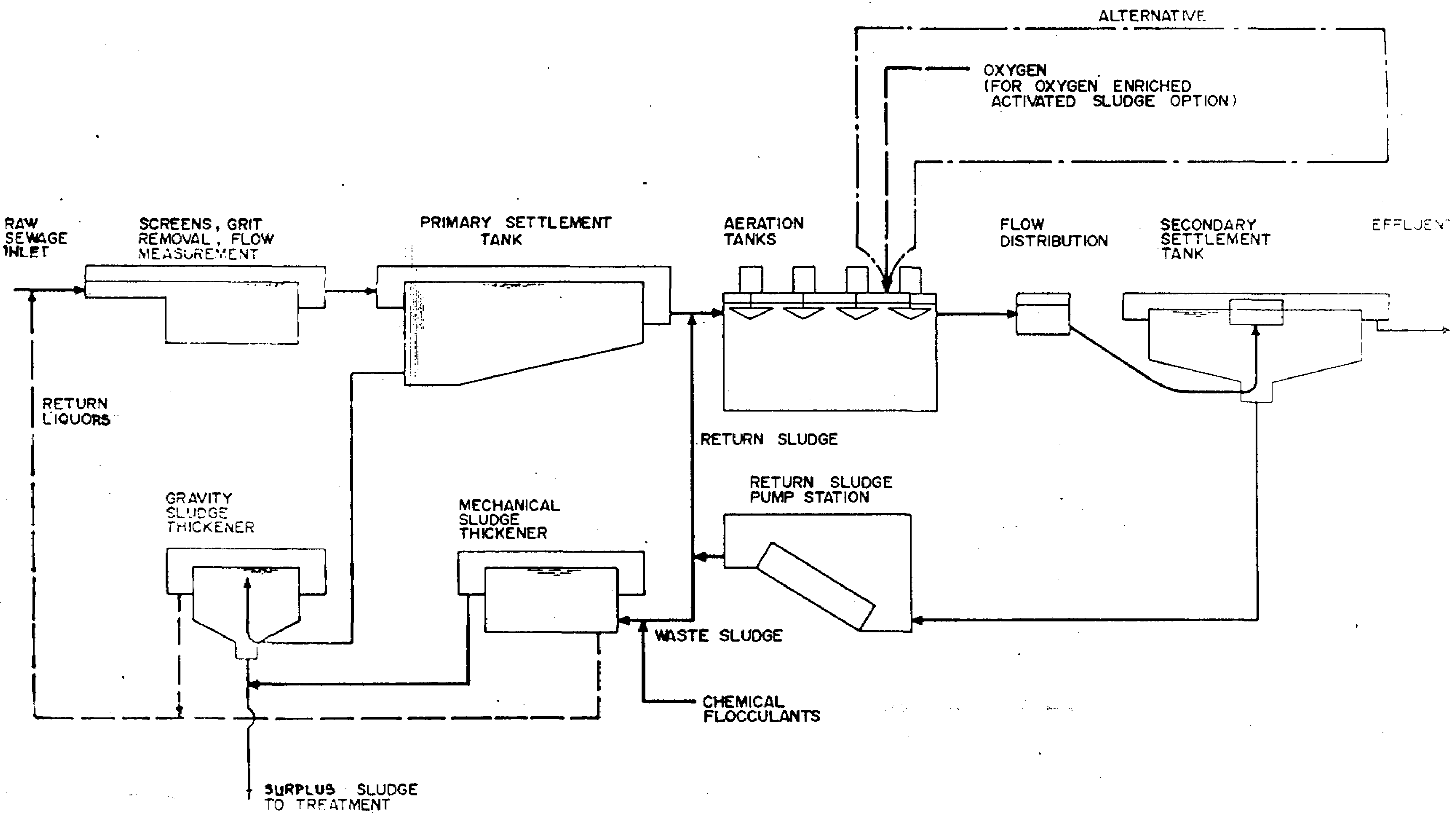
TABLE 6.2

DESIGN CRITERIA FOR ACTIVATED SLUDGE TREATMENT
(continued)

| Process | Parameter | Design Criteria |
|---|---|---|
| Oxidation Ditch | Minimum Retention Period | 1 d |
| | Liquid Depth | Up to 4 m |
| | Organic Loading Rate | 0.15 kg BOD/d. kg MLSSS |
| | Mixed Liquor suspended solids | 3000 to 6000 mg/l |
| | Aeration Requirement | 2 kg $O_2$/kg BOD applied |
| | Sludge Age | 20 - 40 d |
| | Hydraulic Loading Rate (secondary settlement tank) | 22 $m^3/m^2.d$ at peak flow |

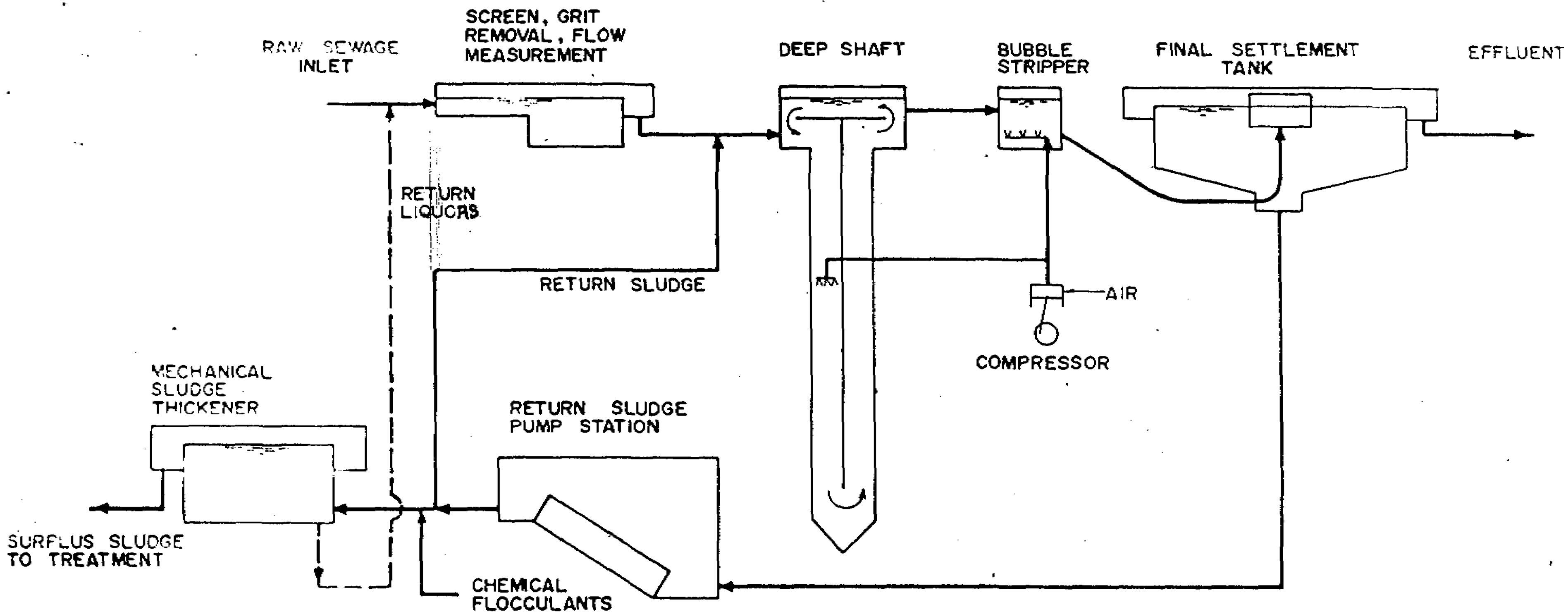
a. For nitrification additional oxygen requirement of 4.25 kg/kg ammonia nitrogen.

b. 8 h for nitrification.




FLOW DIAGRAM ACTIVATED SLUDGE TREATMENT SYSTEMS

FIGURE 6.2




FLOW DIAGRAM DEEP SHAFT TREATMENT SYSTEM

FIGURE 6.3

There are only twenty-seven full scale Deep Shafts in operation or under construction throughout the world treating wastes from population equivalents of 2000 to 400 000. Of these seventeen are in Japan four in North America and six in Europe. Ten of the plants treat domestic sewage and the remaining seventeen are for industrial effluents only. The process is sophisticated and requires experienced operations personnel, it is not recommended for domestic sewage in Sri Lanka at the present time.

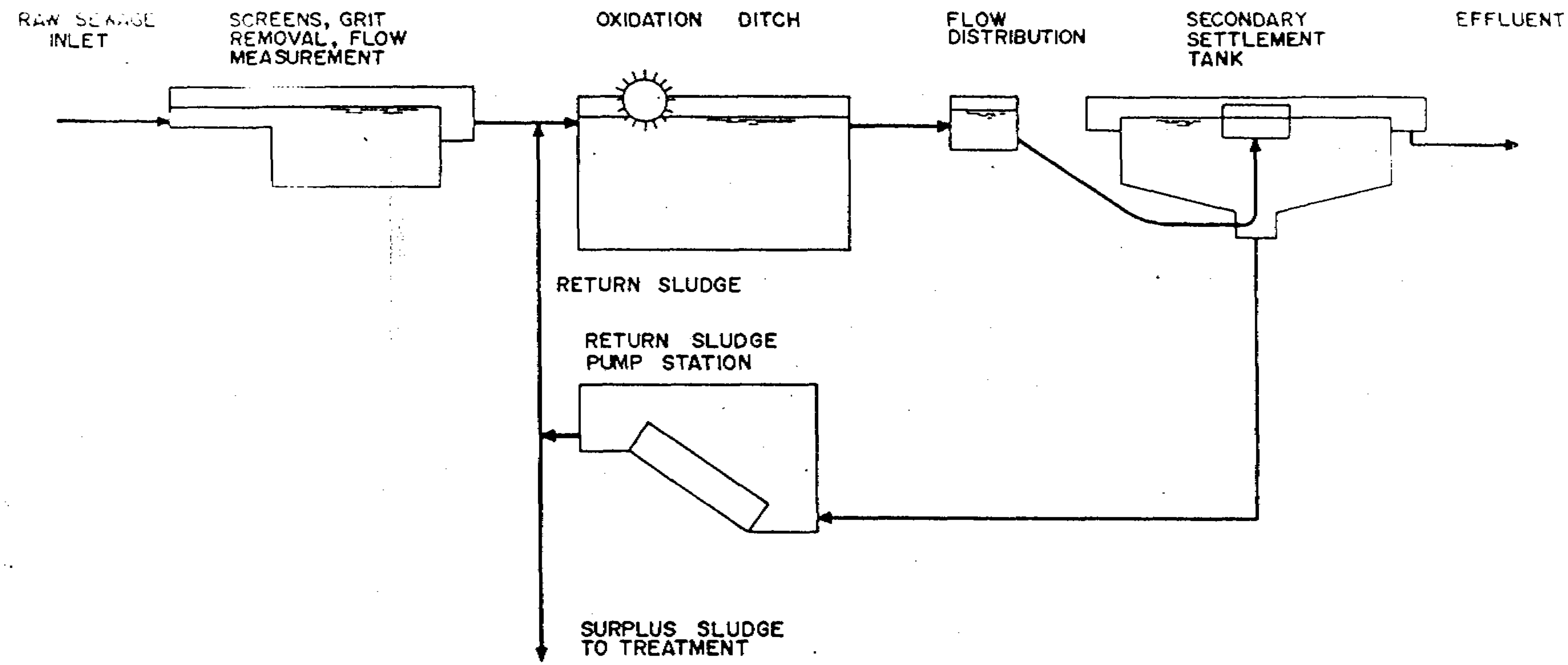
Oxidation Ditch. The oxidation ditch is a form of the extended aeration activated sludge process employing a high concentration of mixed liquor suspended solids. The aeration ditch is in the form of an oval "racetrack" which results in significant land savings compared to pond systems. The system requires a settling tank to remove activated sludge solids from the final effluent, and a pump to return the solids back to the ditch. Excess solids must be removed as sludge on a regular basins. Recommended design criteria are set out in Table 6.2 and a typical flow diagram is shown in Figure 6.4. The oxidation ditch is widely used in Asia and is probably the most straightforward of the alternative activated sludge processes.

### 6.3.2 Lagoon Systems

Aerated Lagoons. Two basic alternative aerated lagoon systems are considered, the completely mixed aerated lagoon system (CMAL) and the partially mixed or facultative aerated lagoon system (FAL). The CMAL process requires more energy than the FAL process, but if used in series with the FAL, an economic system may be effected. Aerated lagoons are more sophisticated than stabilisation ponds, but require less area to achieve similar reductions in BOD.

Design criteria for aerated lagoons are set out in Table 6.3. Criteria are given for the FAL process, a two-stage system comprising a facultative aerated lagoon followed by a facultative pond with 2 days retention. This series has the lowest power requirement. The facultative pond serves as a settling lagoon for the solids carried over from the aerated lagoon. Sludge will have to be removed from the second pond at intervals of a few years. For the CMAL system three lagoons in series are recommended but, because the lagoon depths are greater than with the FAL system, the land requirements of the CMAL system are less.

Stabilisation Ponds. These can be provided in a variety of combinations, covering anaerobic, facultative and maturation pond systems. Anaerobic ponds are generally most efficient with relatively concentrated wastes and are often used where land availability is a serious constraint. However, because of the greater risk of odour associated with anaerobic ponds their buffer zones are generally larger than for facultative ponds, and if located close to residential developments, the extra land required for the buffer may counteract the saving achieved in pond area.




FLOW DIAGRAM OXIDATION DITCH TREATMENT SYSTEM

FIGURE 6.4

TABLE 6.3

DESIGN CRITERIA FOR AERATED LAGOONS

| Parameter | Design Criteria |
|---|---|
| Facultative Aerated Lagoon System (FAL) | |
| Aerated Lagoon: | |
| Minimum detention period | 2.5 d |
| Maximum liquid depth | 4 m |
| Minimum mixing power | 5 $W/m^3$ lagoon volume |
| Oxygen requirement | 0.8 to 1.2 kg consumed/kg BOD removed |
| Approximate BOD removal | 60% to 70% |
| Dissolved oxygen concentration | 2 mg/l |
| Embankment protective lining (inner slopes) | Cemented rip-rap, 0.3 m thick extending from top to bottom of embankment slope |
| Minimum freeboard | 1.0 m |
| Facultative Lagoon (solids collection): | |
| Minimum detention period | 2.0 d |
| Liquid depth | 2.0 m |
| Complete Mix Aerated Lagoon System (CMAL) | |
| Aerated Lagoon: | |
| Minimum detention period | 1.4 d |
| Maximum liquid depth | 5 m |
| Power requirement | 8 $W/m^3$ lagoon volume |
| Primary Aerated Facultative Lagoon: | |
| Minimum detention period | 1.8 d |
| Maximum liquid depth | 4 m |
| Power requirement | 2 $W/m^3$ lagoon volume |
| Secondary Aerated Facultative Lagoon (solids collection): | |
| Minimum detention period | 1.0 d |
| Maximum liquid depth | 4 m |
| Power requirement | 0.8 $W/m^3$ lagoon volume |
| Overall BOD removal | 60–70% |

In facultative ponds, the upper layers are maintained in an aerobic state by algal photosynthesis and to a lesser extent by surface aeration, while the lower layers are anaerobic as a result of the digestion of settled organic solids. Maturation ponds are essentially polishing ponds, they are aerobic and are usually provided to reduce pathogen levels in the final effluent.

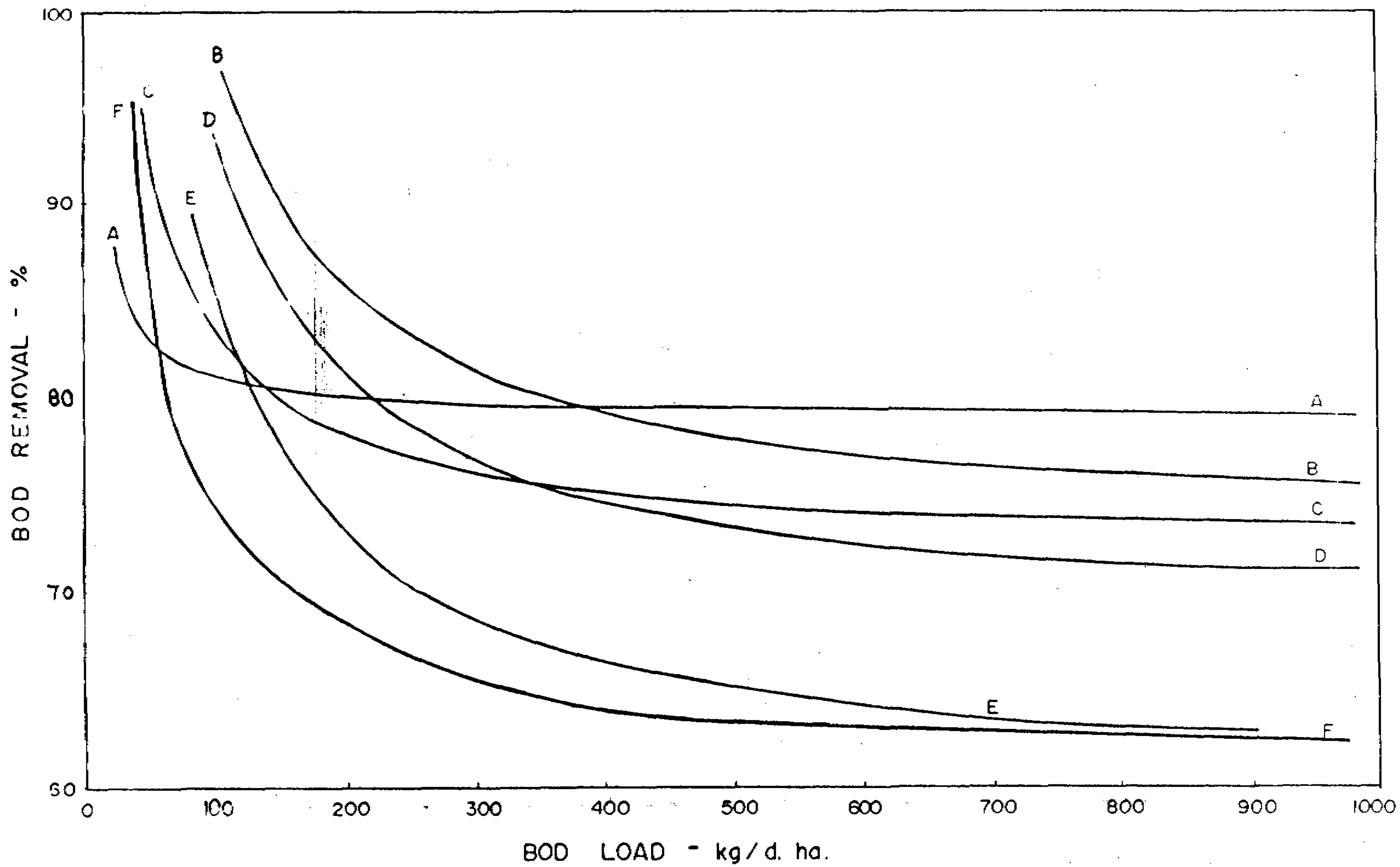
A variety of design criteria has been developed in different countries for various pond categories. Most are based on empirical equations and care must be taken when using criteria developed in other locations. Direct experience in Sri Lanka is lacking, although climatic conditions are generally ideal for this process.

BOD removal rates in anaerobic ponds are related, among other factors, to temperature and retention time. For the typical BOD loading range of 0.1 to 0.4 $kg/m^3.d$ BOD removal varies from 50% at one day retention to 70% at five days retention for temperatures in excess of 20°C. Loading rates outside this range may give rise to odour problems. For temperatures from 15 to 20°C BOD reductions would be expected to be about 15% less. For an average monthly minimum temperature of 22.5°C, a BOD removal rate of 60% could be expected at a loading rate of 0.25 $kg/m^3$ d. Average annual temperatures in Sri Lanka are generally in the range 20 to 28°C, with the exception of Nuwara Eliya which has an annual average temperature of about 16°C. For pond systems, monthly average minimum temperatures should be used.

Pond depths should be greater than 2.5 m if possible to ensure anaerobic conditions prevail, a depth of 4 m is a typical optimum provision to minimise land area requirements. An optimum hydraulic retention time is usually specified as two days, with one day being an absolute minimum.

For design purposes a sludge accumulation rate (excluding grit) of 0.04 $m^3/y.p$ should be adopted and at least two ponds should be constructed in parallel to allow for desludging to be carried out on a regular basis, preferably when the pond is half full of sludge. Typical desludging frequencies for anaerobic ponds are 3 to 5 years.

There are a number of criteria in use for the design of facultative pond systems, some of the more comprehensive reviews of the different methods have been presented by Mara (1976), Gloyna (1971), Mc Garry and Pescod (1970), Environmental Protection Agency (1971) and Bradley (1983). The various design criteria for primary ponds are presented in Figure 6.5. The equations relating to the BOD reduction curves shown in Figure 6.5 are as follows, where BOD (R) and BOD (L) relate to BOD removed and BOD load respectively, both expressed in kg/d. ha.




COMPARISON OF DESIGN EQUATIONS FOR BOD REMOVAL IN PRIMARY FACULTATIVE PONDS

FIGURE 6.5

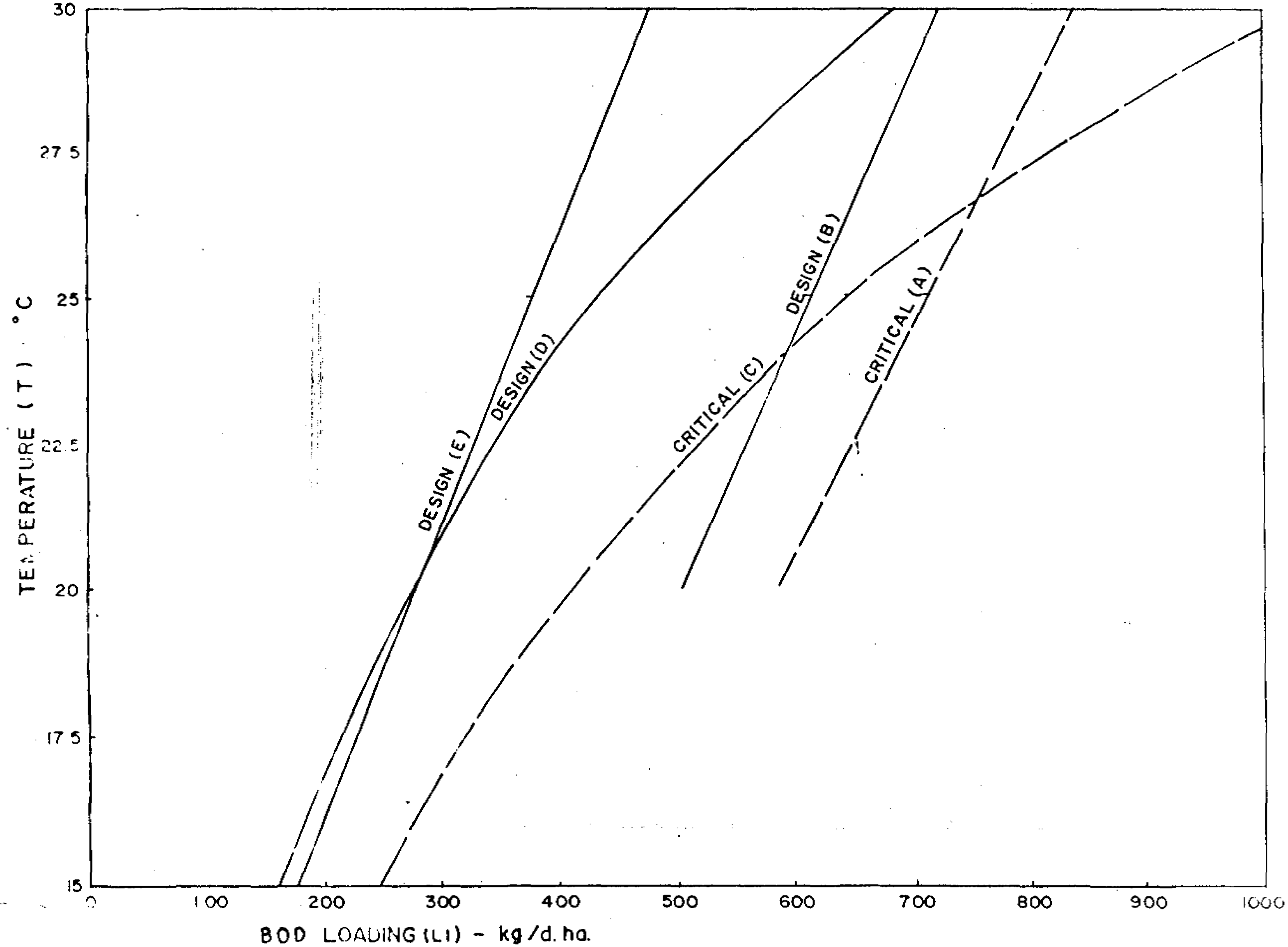
| Curve | Equation | Country | Reference |
| --- | --- | --- | --- |
| A | BOD (R) = 0.79 BOD (L) + 2 | Brazil<br>India | Mara and Silva (1979)<br>Siddiqui (1973) |
| B | BOD (R) = 0.73 BOD (L) + 25 | Nigeria | Arthur (1981) |
| C | BOD (R) = 0.725 BOD (L) + 10.75 | Worldwide | Mc Garry and Pescod (1970) |
| D | BOD (R) = 0.68 BOD (L) + 26 | India | Arceivala (1970) |
| E | BOD (R) = 0.60 BOD (L) + 25 | USA | Hermann and Gloyna (1958) |
| F | BOD (R) = 0.61 BOD (L) + 13 | Malaysia | Bradley (1983) |

All the curves show the same trend with a range of removal efficiencies varying from about 64 to 79 % for loading rates in excess of 500 kg/d.ha. Below loading rates of 250 kg/d. ha all the design curves exhibit rapidly increasing BOD removal efficiencies.

Although all the design equations shown in Figure 6.5 supposedly relate to primary facultative ponds there is no doubt that some of the pond systems represented by the equations are not strictly facultative, particularly at the higher loading rates. The design curve based on the USA data (curve E) is applicable to those ponds which always contain a surface aerobic layer although surface aerobic layers are unlikely to be continually present at loading rates above say 400 kd/d.ha. In some instances ponds may be facultative during the day but become anaerobic at night; such ponds are neither strictly facultative nor strictly anaerobic. There is also a possibility that the operating efficiency of a facultative pond may be higher during the initial years of operation because equilibrium conditions have not been reached in the bottom layers, hence design equations based on relatively new pond systems may predict BOD removal efficiencies which are optimistic for long-term operation.

In order to avoid odours being generated from facultative ponds, the odiferous end products of the anaerobic digestion taking place in the lower layers must be oxidised or ionised before they are allowed to escape to the atmosphere. Algal photosynthesis in the top layers provides oxygen and an alkaline environment for such oxidation and ionisation to occur. Since photosynthesis does not occur in the absence of solar radiation, most odour problems are associated with night time or early morning conditions when the aerobic zone has become depleted. Increasing the organic load results in an increased oxygen demand and the problem is accentuated.

Critical loading rates, when a pond remains anaerobic all the time, are related to mean monthly ambient air temperature as shown in Figure 6.6. The design curves shown in Figure 6.6 are an attempt to introduce a factor of safety and would permit some periods of anaerobiosis during the night. The equations for the curves in Figure 6.6 are as follows:




RECOMMENDED MAXIMUM BOD LOADS TO PRIMARY PONDS AS A FUNCTION OF TEMPERATURE

FIGURE 6.6

| Curve | Equation | Reference |
|---|---|---|
| Critical A | BOD (L) = 25.6T + 75 | Arthur (1981) |
| Design B | BOD (L) = 22T + 65 | Arthur (1981) |
| Critical C | BOD (L) = 11.2 $(1.054)^T$ | Mc Garry and Pescod (1970) |
| Design D | BOD (L) = 7.5 $(1.054)^T$ | Mara (1976) |
| Design E | BOD (L) = 20T − 120 | Mara (1976) |

Temperature in °F for equations C and D, in °C for others

It is apparent from Figure 6.6 that for a mean minimum temperature of 25°C (Colombo area), recommended design loads would be from about 375 to 625 kg/d. ha, allowing for some hours of anaerobiosis, with a critical maximum loading being in the range 650 to 720 kg/d.ha. For locations in the Nuwara Eliya area design and critical loadings would be of the order of 125 and 200 kg/d.ha.

For single facultative ponds or primary facultative ponds in a series of ponds it is recommended that if odour problems are to be minimised the pond should be designed on the basis of the mean minimum monthly temperature with the average organic loading not exceeding say 250 kg/d.ha in the Colombo area. If occasional odours can be tolerated, these will usually occur just before dawn, the average loading can be increaed to say 500 kg/d. ha. Odours could be expected to occur for about a 4-hour period every day at this loading, during the coldest, most cloudy months.

It is recommended that for both loading rates, an average BOD removal of 75% be assumed. This is similar to that suggested by Arthur (1983) in the most recent World Bank guidelines for pond design in warm climates, where a "safe" design load of 390 kg/d. ha is proposed for a temperature of 22.5°C. Safe in this context refers to complete failure, some period of odour release would still be expected.

Primary pond volumes should be based on a depth of 1.5 m with a minimum hydraulic retention time of 5 days. The water depths should not be less than 1.0 m to prevent vegetation growth on the pond bottom. The first part of the pond should be deepened (say over one quarter of the length) to allow for sludge accumulation. An additional depth of up to 2.5 m could be provided. Floating material in the feed sewage can best be trapped by a baffled entrance.

Sludge accumulation should be based on 0.04 $m^3$/y.p. For large ponds, sludge removal can be achieved by pumping from a pontoon, for small ponds draining and allowing to dry by evaporation is probably more practicable. Desludging frequencies are typically once every 15 to 20 years for primary facultative ponds based on sludge being removed when it occupies up to one half of the pond volume. Secondary facultative ponds should not need desludging. In order to permit sewage to be treated during desludging two primary ponds should be provided in parallel and sized so that if one pond is decommissioned for desludging the other pond will not be loaded to such an extent that the critical loading rate is reached. In order to minimise costs only one train of ponds need be provided if they are temporary or to be expanded within say ten years.

None of the design equations are suitable for secondary facultative ponds in a series because BOD removal rate constants in secondary ponds are considerably less than those in primary ponds. On the basis of an average recommended design removal rate in primary ponds of 75%, it is suggested that the secondary pond removal rate be taken as 40% to give an overall BOD removal in a two-series system of about 85%. Recommended loading rates for secondary ponds in climatic conditions similar to Colombo should be taken as 175 kg/d. ha for low-load systems and 250 kg/d. ha for high-rate systems.

Design depths of secondary ponds can be taken as 1.5 m, surface dimensions may be the same as for the primary ponds but if BOD removal is to be optimised then the secondary pond should have a smaller area than the primary pond, subject to the recommended loading rates not being exceeded and the hydraulic retention time not being less than 3 to 5 days in order to allow for reasonable bacterial removal.

Maturation ponds are provided to reduce pathogen levels, the bulk of the BOD removal having been achieved in the preceeding ponds. As a result, maturation ponds should not be taken into account in calculating overall BOD reduction.

The faecal coliform removal in a series of ponds is expressed by the following:

$$FC\ (E) = \frac{FC(I)}{(1 + kb\ t_1)\ (1 + Kb\ t_2)\ \ldots}$$

where
- $FC(E)$ = faecal coli. in the effluent
- $FC(I)$ = faecal coli. in the sewage feed
- $Kb$ = bacterial removal rate constant
- $t_1, t_2$ = hydraulic retention time in pond 1, pond 2

The bacterial removal rate constant is related to temperature as follows:

$$Kb(T) = 2.6\ (1.19)^{T-20}$$

where $T$ = design temperature (°C)

Depths of maturation ponds are typically 1.0 to 1.5 m with an absolute minimum detention time of 3 days, preferably 5 days. Design criteria for stabilisation pond systems are summarised in Table 6.4.

TABLE 6.4

DESIGN CRITERIA FOR STABILISATION PONDS

| Parameter | Design Criteria |
|---|---|
| Anaerobic | |
| Loading rate | 0.25 kg BOD/d. $m^3$ |
| Hydraulic retention | 2 d (absolute min. 1 d) |
| Water depth | 2.5 - 4.0 m |
| Sludge accumulation | 0.04 $m^3$/y. p |
| BOD removal | 60% (at 22.5°C) |
| First Stage Facultative | |
| Preferred surface loading rate | 250 kg BOD/d. ha |
| Maximum surface loading rate[a] | 500 kg BOD/d. ha |
| Minimum detention period | 5 d |
| Maximum liquid depth | 1.5 m |
| Minimum freeboard | 0.5 m |
| Embankment protective lining (inner slope) | Cemented rip-rap, 0.3 m thick extending from top of embankment to minimum of 0.5 m below liquid surface |
| Embankment protective lining (outer slope) | Turfing and/or rip-rap if subjected to external water wave action |
| Liquid seal lining | Clay or polyethylene sheets |
| Sewage inlet and outlet structures | Multiple |
| Expected BOD removal | 75% |
| Sludge accumulation rate | 0.04 $m^3$/y.p |
| Second Stage Facultative | |
| Preferred surface loading rate | 175 kg BOD/d. ha |
| Maximum surface loading rate | 250 kg BOD/d. ha |
| Minimum detention period | 5 d |
| Expected BOD removal | 40% |
| Other physical parameters | As for first stage |

TABLE 6.4

DESIGN CRITERIA FOR STABILISATION PONDS
(continued)

| Parameter | Design Criteria |
|---|---|
| Maturation Pond | |
| Minimum detention period | 3 d (preferably 5 d) |
| Liquid depth | 1.0 - 1.5 m |
| Other physical parameters | As for facultative ponds |
| Overall Treatment Efficiency[b] | |
| BOD removal[c] | 85% |
| Faecal coliform removal[d] | 300/100 ml in final effluent |

a Based on average minimum temperature 25°C. At this loading rate there is a risk of occasional odour for a few hours in the early morning.

b Two-stage facultative ponds of same surface area and one maturation pond.

c Ignores any BOD removal in maturation pond.

d Two facultative and one maturation pond in series, based on 25°C, raw sewage faecal coliform 10 million/10 ml.

## 6.4 Tertiary Treatment

If it is necessary to achieve an effluent BOD lower than 20 mg/l a tertiary treatment process should be employed. The most effective unit is a rapid gravity sand filter comprising sand of 0.8 to 1.7 mm nominal diameter at a depth of up to 1.5 m. Design maximum flow should be limited to 165 $m^3/m^2.d$ and an operating head of 3 to 4 m is necessary. Backwashing using sand filter effluent should be carried out daily at a rate of 10 $l/s.m^2$, the cleaning efficiency can be improved if air scour is added, a typical air flow rate being 20 $l/s.m^2$. BOD and SS reductions of 70 and 80% respectively can be attained but there is usually a reduction in DO of about 4 mg/l as a result of biological activity in the sand bed.

## 6.5 Proprietary Treatment Systems

A package sewage treatment plant is usually defined as a sewage treatment plant which is fabricated at the factory and is taken to the site as a complete unit ready for use. Many proprietary systems are available in package plant form for lower flows and as an equipment package for inclusion in concrete tanks for larger flows. Many systems are variations of well-established processes such as extended aeration and RBCs, while some rely on specific patented items or process. Some of the processes mentioned in the previous sections are proprietary systems i.e. Deep Shaft and the various oxygen-enriched activated sludge systems e.g. Unox, Oasis, Megox. The two most common proprietary systems are based on the extended aeration and RBC process. Extended aeration systems are available from a number of manufacturers in package treatment plants handling wastes from populations up to about 4000.

The system is similar to the oxidation ditch system which necessitates a relatively low organic loading and long aeration time. Sewage after screening is fed into the aeration tank which is followed by a settlement tank from where sludge is returned to the aeration tank. Surplus sludge is diverted for further treatment. Various manufacturers provide different configurations e.g., separate aeration and settlement tanks (Simon Hartley) and concentric aeration and settlement tanks (Inka). Numerous plants are in operation around the world. The contact stabilisation process is a variation of the extended aeration process and has been proven to be an effective treatment system in many countries. Proprietary contact stabilisation plants are available to treat wastes from populations up to 10 000 or more.

The system concepts for package RBC units are the same as described in Section 6.3.1 but various manufacturers have developed package plants with different settling compartment and sludge handling configurations. Systems are available up to aroud 1100 $m^3/d$ capacity as package plants. Sludge requires further treatment. The systems are relatively simple to maintain with the only moving parts being the shaft and in some systems, an integral sludge scraper.

6.6 Sludge Disposal

All treatment systems produce sludge which must be disposed of in a manner which does not give rise to nuisance or public health problems. The quantity of sludge depends on the treatment system; typical values range from about 4 $m^3/d$ for 1000 persons for an oxidation ditch process to less than 0.1 $m^3/d$ for 1000 persons with oxidation ponds. Screenings, in addition, would amount to about 0.01 $m^3/d$ per 1000 persons and grit quantities to about 0.05 $m^3/d$ per 1000 persons.

Typical sludge production rates would be as follows for treatment systems designed to produce a final effluent BOD of 20 mg/l (30 mg/l for oxidation ponds). The sludge quantities include primary and excess biological sludge:

| Process | Sludge (kg/d.p) |
|---|---|
| Primary Settlement | 0.04 |
| Activates Sludge: | |
| Conventional | 0.06 |
| Oxygen-enriched | 0.06 |
| Deep Shaft | 0.07 |
| Oxidation Ditch | 0.02 |
| Bilological Filters, RBCs | 0.05 |
| Oxidation Pond | 0.005 |

For disposal of sludge the following factors form the basis of all designs:

- There should be no hazard to the health of the public and workers at the site of disposal.
- The deposited sludge should not cause nuisance from insects or rodents, neither should there be nuisance from odours.
- The deposited sludge should not cause ground or surface water pollution.
- After tipping the deposited sludge must remain firm and compact.
- The sludge can be disposed of either in a liquid form or for landfill in dewatered form.

Digestion will destroy the readily putrescible organic matter in the primary sludge together with a large number, but not all, of the pathogenic micro-organisms. As a result, digested sludge should not cause nuisance from insects, rodents or odours and it will constitute a greatly reduced health hazard. Sludge from lagoon systems is essentially digested when it is removed.

The basic methods available for processing the sludge prior to landfilling are air drying with or without prior mechanical dewatering, mechanical dewatering alone, or incineration. For a given sludge, the texture and dryness of the dewatered sludge are dependent upon the particular dewatering process used and the type of conditioning, if any, adopted, i.e. each of the mechanical methods of dewatering sludges do not produce sludge cakes of the same dryness, and certain types of conditioning are not suitable to some dewatering processes. In addition, some conditioners, e.g. lime, will destroy a large number of the micro-organisms in the sludge, so reducting further the public health hazard. The use of polymers in dewatering requires that the sludge be previously digested to reduce the problems of odours and decomposition at the ultimate disposal site. If disposal of liquid sludge to land is feasible, digestion would not be necessary from aerated lagoons or from activated sludge processes with long sludge ages, i.e. oxidation ditches.

Recommended design criteria are set out in Table 6.5 and typical flow diagrams are shown in Figure 6.7.

Digestion of sludge can be in unheated tanks which under conditions similar to the Colombo area should ensure mesophilic digestion. Gas production should be adequate for power generation of about 0.12 $kWh/m^3$ of sewage treated based on a gas production of 1.0 $m^3$ for each kilogram of volatile matter destroyed, and an energy value of 22 $MJ/m^3$ of gas.

## 6.7 Miscellaneous Design Criteria

### 6.7.1. Odour Control

Preliminary facilities should be grouped together in a compact arrangement to facilitate covering and removal and treatment of foul gases if shown to be necessary. Mechanically vented air should be treated in scrubbing towers using sodium hypochlorite or similar chemicals or passed through odour absorption media such as activated carbon.

The remaining sewage treatments units should not be covered nor mechanically ventilated with the exception of sludge draw-off chambers at primary settlement tanks.

For odour removal using activated carbon filters a typical $H_2S$ design input would be 15 mg/l for inlet works and raw sewage pump stations and 50 mg/l for raw sludge sumps. Three carbon regenerations can usually be tolerated, at intervals of about four months, before needing to replace with new carbon.

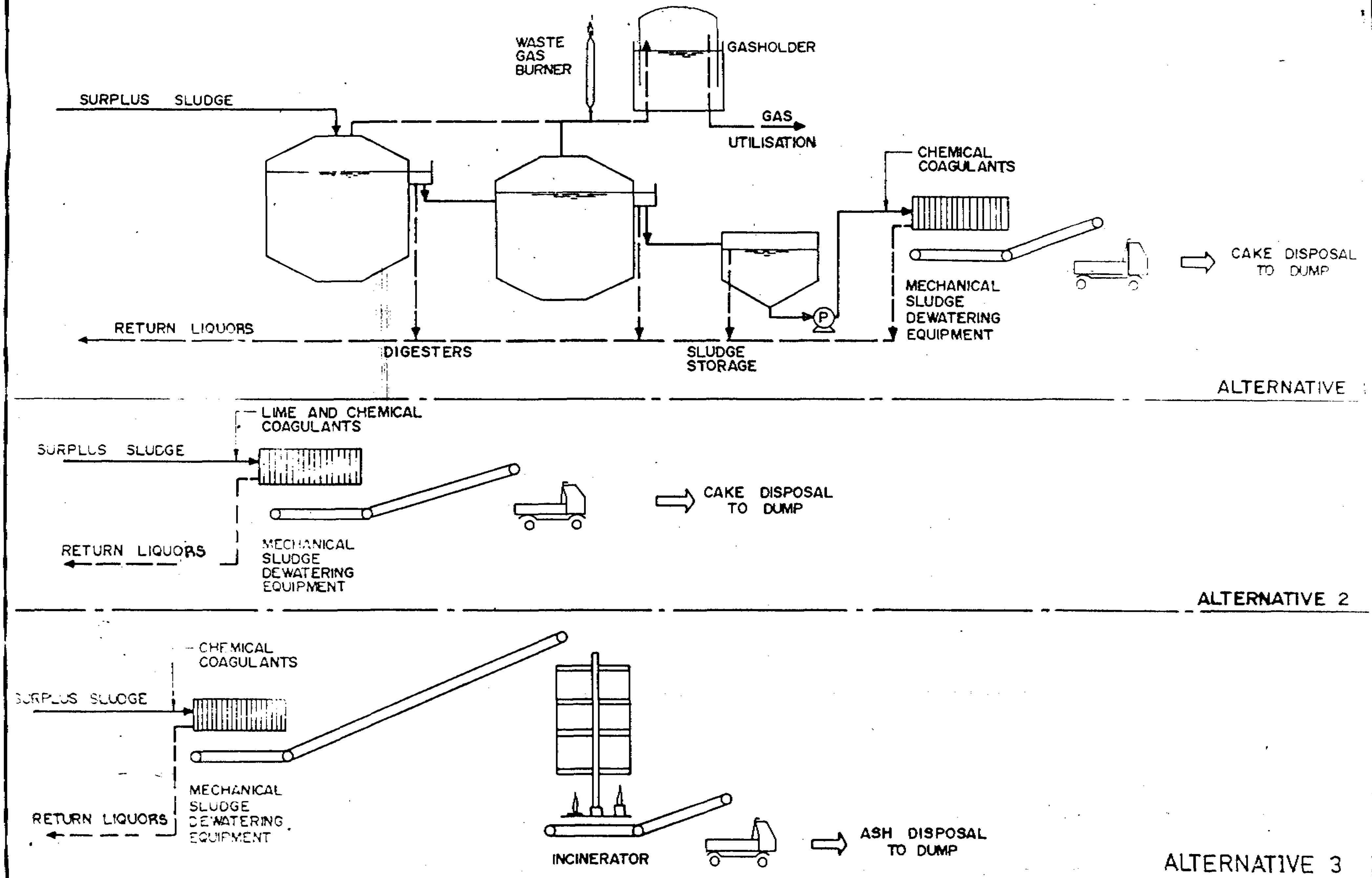
Sludge treatment facilities should have mechanical ventilation where raw sludge is the medium being handled but otherwise ventilation should only be provided as required to provide an appropriate working environment inside buildings.

TABLE 6.5

DESIGN CRITERIA FOR SLUDGE TREATMENT

| Parameters | Design Criteria |
|---|---|
| Gravity Thickening (Primary and Fixed Growth Sludges) | |
| Input concentration | 3% |
| Output concentration | 5% |
| Retention | 24 h |
| Flotation Thickening (Secondary Suspended Growth Sludges) | |
| Input concentration | 5000 mg/l |
| Output concentration | 4% |
| Solids loading | 10 kg/h. $m^2$ |
| Digestion | |
| Retention | 30 d |
| Temperature | Ambient |
| Plate Pressing | |
| Chemical | 2% polymers |
| Plate size | 1200 x 1200 x 25 mm |
| Cake solids | 35% |
| Solids loading | 4000 $kg/m^3$ |
| Belt Pressing | |
| Chemicals | 2% polymers |
| Belt width | 0.5 to 1.5 m |
| Cake solids | 25% |
| Solids loading | 200 kg/h. m |
| Sludge Incineration | |
| Loading | 30 kg/h.$m^2$ hearth area |
| Drying Beds | |
| Sludge depth | 300 mm |
| Feed sludge solids (digested) | 6% |
| Sludge cake solids | 40% |
| Drying time[a] | 4-8 weeks |

a Depends on season. Advisable to allow grass to grow on sludge to aid evapo-transpiration of rainfall.




TYPICAL SLUDGE TREATMENT SYSTEMS

FIGURE 6.7

6.7.2 Buffer Zones

The provision of buffer zones round treatment systems in order to provide visual screening and an area for dilution of obnoxious odours is standard practice in many countries. This concept is already established in Sri Lanka for different classifications of industry (CEA, 1988). The guidelines for buffer zones in other countries vary over a wide range depending on the type of sewage treatment process and the sensitivity of the population residing nearby. Recommended buffer zones should be based on an evaluation of land values and imputed aesthetic and other benefits for specific locations. However, as a general guideline it is recommended that wherever possible regional treatment systems should be located in industrial areas which have compatible land use. If this is not possible a **minimum** buffer zone should be provided which would include a beautification zone and would be, say, 30 m in residential areas for regional activated sludge or fixed-growth systems which generally exhibit lower odour risks than pond systems. For pond systems the buffer zone should be greater because of increased odour risk. Factors influencing buffer zone selection are discussed in Bradley and others (1987).

## SECTION 7

## INDUSTRIAL WASTES

Industrial waste in urban areas can be dealt with either by treatment on site or, for wastes which are compatible with the domestic sewage treatment processes, by treatment in admixture with domestic sewage in a regional plant after pretreatment, if necessary, at the factory. The choice between regional or on-site treatment for compatible (treatable) substances such as BOD and organic SS is invariably a matter of economics.

### 7.1 Discharge to the Sewerage System

The advantages of discharging compatible wastes to a regional sewerage system can be summarised as follows:

- o Variations in individual industrial pollution loads are often much greater than in large regional treatment works with the result that considerable expertise would be required at industrial waste treatment plants to maintain purification efficiency.

- o Some industrial wastes may be difficult to treat in isolation without, for example, buffering or nutrient addition. Mixing with large volumes of sewage provides such additions at no extra cost.

- o There is a shortage of skilled personnel to operate a large number of individual industrial waste treatment plants.

- o Sludge removal can be carried out more efficiently from a regional sewage treatment works than from a large number of industrial waste treatment plants.

- o The risk of water pollution through plant malfunction or inadequate effluent monitoring increases with the number of treatment plants.

- o Few of the factories which were established before the necessity to control pollution discharges have sufficient land on which to construct a treatment facility.

- o Treatment in a regional facility is usually more cost-effective and the industrialist will not be involved in short term, large capital expenditures.

The degree of pretreatment depends on the type of regional sewage treatment facility to which the industrial wastes are being discharged, the nature of the sewerage system and the characteristics of the industrial waste. There are certain basic criteria which are common to all waste regulations, these are the prohibition of discharges of explosive or inflammable substances such as solvents or petrol, the avoidance of shock hydraulic or organic loads, maintaining temperature below 40°C, maintaining the pH between 5 and 9 and careful control to within stipulated limits of discharges of oil and grease, unsettleable solids, corrosion-inducing substances, and other incompatible substances such as metals and toxic chemicals. It is not difficult to comply with these restrictions. The pH range may be relaxed if the volume of effluent is small in relation to the volume of waste in the receiving sewer so that immediate neutralisation occurs within the sewer. Industrial waste discharges of BOD and organic SS to the regional system are perfectly acceptable provided that adequate treatment capacity is available in the regional plant.

Sri Lankan tolerance limits for industial discharges to the sewerage system are set out in Table B.4. These are interim standards and include additional parameters for cases where the regional sewage treatment plant effluent is used for irrigation. The standards stipulate a maximum temperature of 45 °C, this must be considered as an absolute maximum, a temperature of 40°C would ensure less risk of adverse effects on sewerage structure or treatment process organisms. The standard limit of BOD 200 mg/l could be relaxed if adequate treatment plant capacity is available.

There is usually little justification for forcing industries to pretreat their wastes for BOD and SS provided that the maximum BOD and SS concentrations are of the order of 500 mg/l and that the suspended matter can be shown not to give rise to problems of sludge in the sewerage system. If factories are eventually to be charged for pollution loads, higher concentrations of BOD and SS may be acceptable. The important aspect is to ensure that the organic load represented by the industrial BOD and SS has been accounted for in the design of the treatment system.

Pretreatment. There are a wide variety of unit processes available for pretreatment; the selection of the appropriate process should be made on a case-by-case basis after first studying the possibilities of water conservation and waste reduction by adopting improved housekeeping practices.

For compatible pollutants such as BOD and SS the following unit processes are typical:

1. Coarse solids separation
2. Grit removal
3. Equalisation
4. Neutralisation
5. Dissolved air flotation
6. Sedimentation
7. Biological treatment
   - activated sludge
   - ~~lagoon systems~~
   - biological filtration

8. Physical
   - chemical treatment
   - chemical coagulation
   - filtration
   - activated carbon adsorption

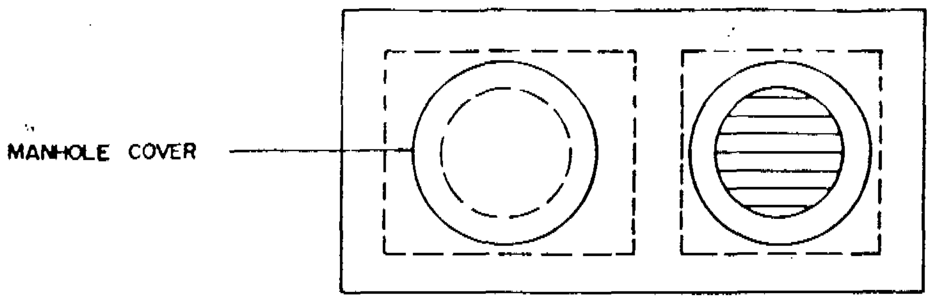
Most pretreatment systems utilise unit processes 1,2 and 3 as a basic minimum requirement. Equalisation is important in order to reduce the fluctuations in flow and pollution load in the sewerage system and at the treatment works. In terms of flow the aim is often to restrict the maximum daily to average daily waste flow ratio to less than 1.5, or 2.0 if the industrial waste flows are small in comparison to domestic sewage flows. Flow balancing can be achieved by constructing holding lagoons and allowing the waste to discharge to the sewers at a controlled rate. An often more convenient and less costly solution is to stagger the times of individual waste discharges. It may be necessary to keep the holding lagoons aerated or continually stirred to prevent the onset of anaerobic conditions and the settling out of solid matter. It is suggested that a maximum to average flow ratio of 2.0 be adopted as a general guideline. The
such as metals and toxic chemicals. It is not difficult to comply with these restrictions. The pH range may be relaxed if the volume of effluent is small in relation to the volume of waste in the receiving sewer so that immediate neutralisation occurs within the sewer. Industrial waste discharges of BOD and organic SS to the regional system are perfectly acceptable provided that adequate treatment capacity is available in the rall waste discharges with a factory sewer exceeding this dimension. Pretreatment for removal of solids can be achieved by more sophisticated screens for the large factories, including mechanically raked bar screens or drum screens. Settling tanks may be required in some instances, particularly for flour mills where very high concentrations of suspended solids can be present in the waste.

The discharge of oil, grease, fats, waxes and similar substances can give rise to serious problems of blockage in the sewers, blinding of screens and pumps and reduction in oxygen transfer efficiency in biological treatment processes. Oils and greases can also cause serious foaming problems in aeration systems.

It is recommended that a general guideline be adopted that there should be no visible oil or grease in any waste discharge and that the maximum concentration should not exceed 100 mg/l as ether extractable material. This concentration is higher than the interim limit of 30 mg/l stipulated in the Sri Lankan standard (see Table B4).

The installation of oil and grease traps on all waste lines from industrial establishments using these substances should be mandatory. It is essential that the traps be located in an easily accessible position and regularly cleaned. Dimensions of a trap suitable for small industries such as car repair shops, canteens, etc. are shown in Figure 7.1.

For larger industries with significant discharges of oil and grease such as edible oil factories, slaughterhouses, petroleum storage depots, etc., it is necessary to provide more sophisticated pretreatment systems. Typical examples are gravity separators with oil skimmers, dissolved air flotation units, with or without the addition of protein precipitants, and oil filters using proprietary filter medium. In some cases chemical treatment by acid cracking may be necessary to break oil emulsions. In many of these large scale cases the recovered oil and grease can be reused or processed to make lower grade products thereby recovering the cost of the pretreatment unit.

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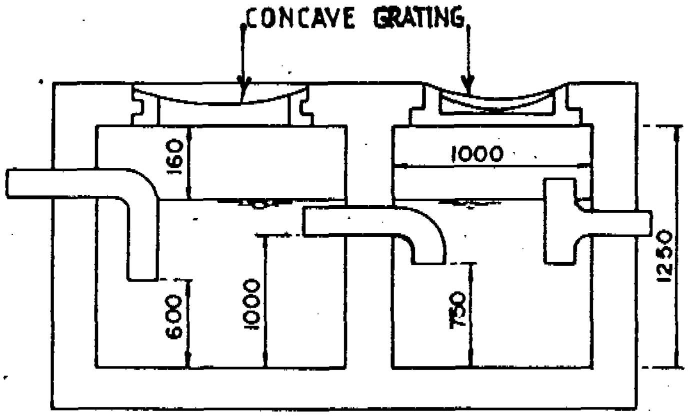



SECTION

ALL DIMENSIONS IN mm
SIZE FOR 5 TO 10 MINUTES RETENTION
NOT TO SCALE

GREASE TRAP ON WASTE DRAIN LINES
FROM SMALL FACTORIES AND CATERING ESTABLISHMENTS

FIGURE 7.1

Dissolved air flotation systems are common in the food industry, particularly when valuable by-products can be reclaimed. Although biological treatment systems may be capable of achieving complete treatment rather than pretreatment, in some cases they are used for pretreatment in order to reduce BOD loading to a combined treatment facility. Plastic medium roughing biological filters, high rate activated sludge systems (low sludge age), anaerobic and aerated lagoons are typical examples of biological unit processes used for pretreatment. Depending upon space and economics, physical-chemical processes may be used instead of biological systems.

For incompatible pollutants such as metals and toxic chemicals the following unit processes are typical:

1. Coarse solid separation
2. Grit removal
3. Equalisation
4. Neutralisation
5. Dissolved air flotation
6. Sedimentation
7. Filtration
8. Chemical coagulation/precipitation
9. Activated carbon adsorption
10. Chemical conversion

Substances which interfere with, or inhibit, the efficiency of operation of the biological treatment systems, and contaminate the sludge and final effluent so as to endanger the environment are referred to as incompatible substances. Typical examples are heavy metals, organics, cyanides and asbestos. Where the final disposal of the waste is to land for agricultural purposes the risk of accumulation of such substances in the soil should be recognised.

Sri Lankan standard for certain incompatible substances are given in Table B4. It is suggested that cadmium also be included, with a maximum limit of 1.0 mg/l, and a further stipulation that the total heavy metal content in each discharge does not exceed 10 mg/l unless the industrial waste volume represents a very small porportion of the total flow in the sewer.

The selection of the unit process depends on the type of incompatible pollutant and the required discharge standard. Processes 1 to 4 are as important for incompatible substances as they are for compatible substances, since shock loads must be avoided. Inorganic dissolved metals are commonly removed by chemical precipation. Chemical conversion is often necessary to convert the incompatible pollutant to a less harmful form. Typical examples are the reduction of hexavalent chromium to the trivalent form and oxidation of cyanide to nitrogen gas. Dissolved organic substances may be removed by activated carbon adsorption.

In all industries detailed surveys should be carried out before installing pretreatment facilities in order to investigate the possibilities of improved housekeeping to reduce pollution loads and associated costs. It is an established fact that in most countries industries usually have little concern for waste control where water supplies and raw materials are readily available, and where there is no history of environmental pollution control in the immediate area. When discharge limitations are imposed on industry there is an incentive for the industrialist to study ways of reducing water consumption and thereby reducing waste flows. Segregation of more contaminated waste streams, installation of cooling towers, use of counter-current washing systems, and recycling of less contaminated wastes for process use are typical methods whereby pollution loads can be reduced.

Very often it is impossible to recover potentially valuable materials from the waste, typical examples are grease and oil, blood from slaughterhouses, and grains from flour mills. Good housekeeping within the factory can be improved by reducing and collecting spillages, regulating water use by suitable valves, etc.

In the larger industries it is usually worthwhile commissioning a water use survey by persons experienced in this field. The cost of the survey can often be recovered many times over by the savings resulting from reduced water consumption and reduced waste discharges.

## 7.2 Complete Treatment and Discharge to Watercourses

There may be certain instances where it is preferable to treat the industrial wastes on site and discharge direct to a watercourse. This procedure is normally appropriate where the wastes are incompatible for treatment with domestic sewage, where separate treatment is demonstrably cheaper or where the cost of conveying the industrial waste to regional sewerage systems or works is excessive.

Sri Lankan standards for discharge of industrial wastes to inland waters and marine waters are given in Tables B2 and B3 respectively. Tolerance limits for industrial wastes discharged on land for irrigation purposes are given in Table B5.

## 7.3 Control of Industrial Wastes Discharged to the Sewerage System

It is essential that strict control be exercised over the discharge of industrial wastes to a regional sewerage system. Licences for consent to discharge must be issued to each industrial discharger.

The concept of applying overall standards to cover all industries in an urban area as embodied in the Sri Lankan legislation appears to be administratively easy to achieve but in practice it is not the most logical or equitable means of regulating industrial discharges. The ideal system is to apply standards to each factory in accordance with the general guidelines of protecting the sewers and treatment and disposal systems. In practice the use of individual factory standards is onerous since it requires a large amount of effort on behalf of the regulating agency. A compromise solution is recommended where general guidelines are set down, such as those for incompatible substances in Table B4, and then the more important industries are assessed individually to verify if the general guidelines are realistic.

The logical way of establishing discharge standards is to calculate the maximum allowable concentration for a particular substance taking into account the dilution in the sewer and the reduction occurring during sewage treatment. The procedure for calculating maximum acceptable concentrations is set out in a schematic form in Figure 7.2. The method should be introduced gradually as experience is gained in the operation of the regional system.

In order to recover the cost of conveying and treating industrial wates a charging formula should be applied. A logical formula would typically be as follows:

$$C_i = V_o V_i + b_o B_i + s_o S_i$$

where $C_i$ = charge to industrial users (Rs/y)

$v_0$ = average unit cost of transport and treatment chargeable to volume (Rs/$m^3$)

$b_o$ = average unit cost of treatment chargeable to BOD (Rs/kg)

$s_o$ = average unit cost of treatment (including sludge disposal) chargeable to suspended solids (Rs/kg)

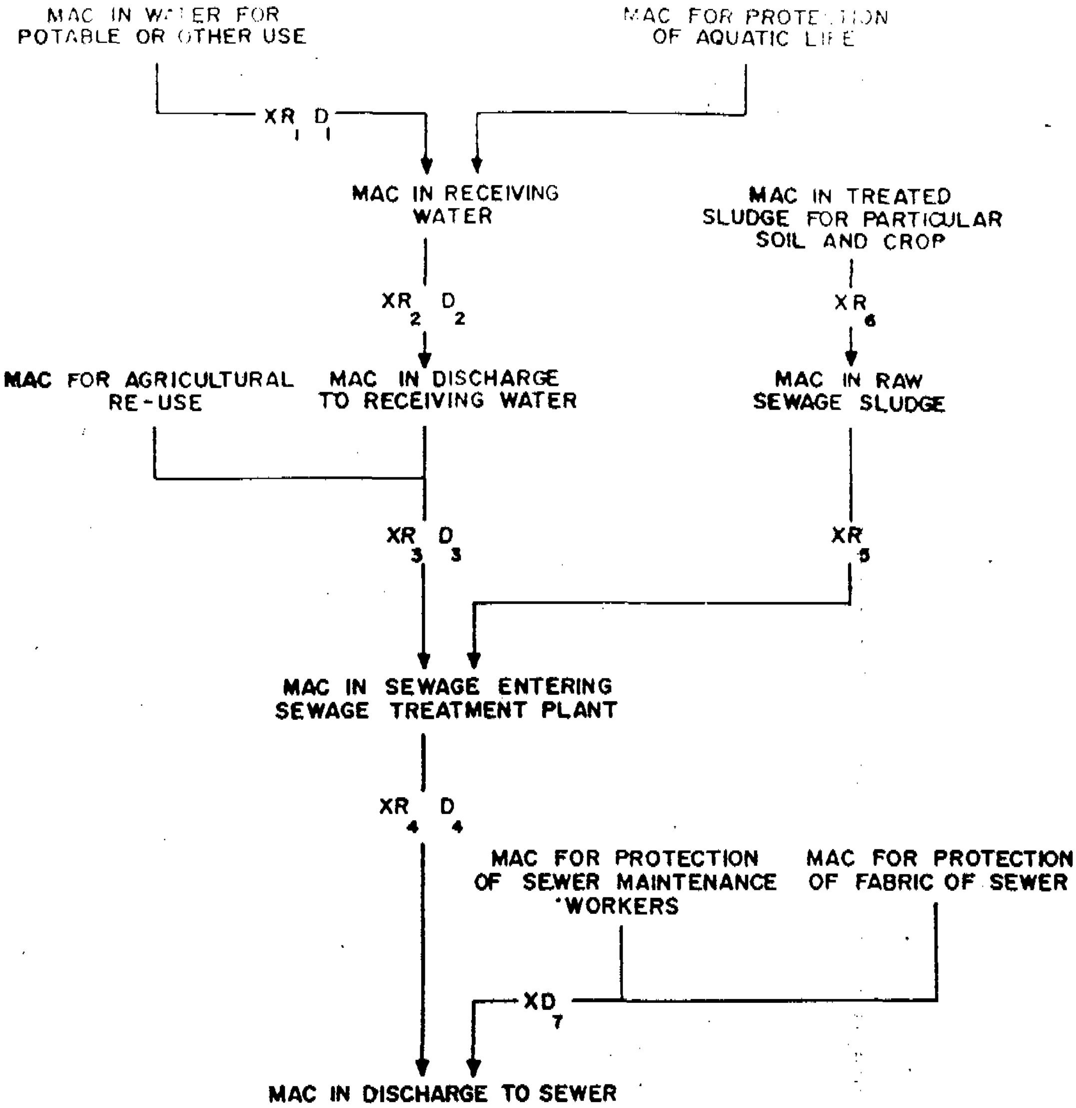
$v_i$ = volume of wastewater from industrial users ($m^3$)

$B_i$ = weight of BOD from industrial users (kg/y)

$S_i$ = weight of suspended solids from industrial users (kg/y)

Two formulae should preferably be used, one to recover capital costs and a second to recover annual operation and maintenance costs. The capital cost formula should be based on maximum industrial waste flow and load which should be taken into account in designing the sewerage and treatment system.

The application of this type of formula requires an existing data base on capital and annual treatment costs and in the early years it would probably be more realistic for the charging formula to be based on flow for sewer costs and on BOD for treatment costs.




| | |
|---|---|
| R | FACTOR TO ALLOW FOR REDUCTION IN CONCENTRATION OF SUBSTANCE ARISING FROM PROCESSES OTHER THAN DILUTION |
| D | FACTOR TO ALLOW FOR REDUCTION IN CONCENTRATION OF SUBSTANCE (NOT EFFLUENT WATER) BY DILUTION. CONCENTRATION OF SUBSTANCE IN DILUTION WATER MUST BE KNOWN. |
| $R_1, D_1$ | RELATE TO REDUCTION AND DILUTION DURING WATER TREATMENT |
| $R_2, D_2$ | RELATE TO REDUCTION AND DILUTION DURING FLOW IN STREAM |
| $R_3, D_3$ | RELATE TO REDUCTION AND DILUTION DURING SEWAGE TREATMENT |
| $R_4, D_4$ | RELATE TO REDUCTION AND DILUTION DURING FLOW IN SEWER |
| $R_5$ | RELATES TO REDUCTION DURING SEPARATION OF SEWAGE SLUDGE VALUE WILL USUALLY BE LESS THAN 1 SINCE MANY SUBSTANCES ARE CONCENTRATED IN SLUDGE. |
| $R_6$ | RELATES TO REDUCTION DURING SLUDGE TREATMENT |
| $D_7$ | RELATES TO IMMEDIATE DILUTION IN THE SEWER |

NOTE: MAC = MAXIMUM ALLOWABLE CONCENTRATION FOR SUBSTANCES DISCHARGED TO SEWERS.

DERIVATION OF MAXIMUM ALLOWABLE CONCENTRATION

FIGURE 7.2

## 7.4 Estimating Industrial Waste Pollution Loads

Major potential waste generating industries in Sri Lanka tend to be located either in industrial estates outside or on the periphery of the main urban areas, or in rural areas. Most industries within residential and commercial development areas are generally small-scale such as vehicle repair and assembly, printing, and metal plating industries. Such industries do not produce significant waste quanities, but they do result in a general environmental degradation which can be controlled by improving internal housekeeping, installing oil traps, etc. A suitable trap design for small-scale industries is shown in Figure 7.1.

There is no substitute for carrying out field surveys to determine industrial waste pollution loads. Few such surveys have been carried out in Sri Lanka although the CEA is gradually accumulating an industrial waste data bank (Ref. CEA Reports and Publications). Data are available from overseas locations but care should be taken in interpreting data from elsewhere because industrial waste loads are heavily dependent on the production process, the avail- ability and cost of water, type of raw material and general attention to housekeeping. For industrial estates, average estate wastes would typically be in the range of 20 to 50 $m^3/d.ha$ and 8 to 15 kg BOD/d. ha. These ranges have been determined from industrial estate surveys in Asia generally but the data do not allow for major polluters such as breweries, and poultry processing factories. For high technology electronics manufac- turing estates the waste loads would be at the lower end of the range. Peak flows should be taken as twice the average values indicated above.

# SECTION 8

## OCEAN OUTFALLS

The discharge of wastewater to the ocean is an accepted means of disposal provided that the assimilative capacity of the ocean at the discharge location is not exceeded and provided that the discharge does not give rise to unacceptable pollution levels in recreation areas or in shellfish harvesting areas.

### 8.1 Discharge Standards

Tolerance limits for domestic and industrial effluents discharged to Sri Lankan marine coastal waters are laid down in Sri Lanka Standard SLS 721: 1985 and are summarised in Table B3. The standard defines marine coastal waters as the sea, ocean, creeks, and tidal waters extending out to 5 km from the mean high water mark and up to the low tide level in estuaries. It should be noted that these tolerance limits should be considered more as guidelines, since SLS 721: 1985 includes the provision that they may be varied in consultation with the CEA.

The objective of a discharge standard is to safeguard the quality of the receiving water for designated uses. In the coastal marine environment such uses are essentially recreation and fishing. It is necessary, therefore, to base discharge standards on receiving water quality criteria.

Recreational activities associated with marine waters encompass water-contact activities (swimming, diving, wading, water skiing, wind surfing, net fishing) and non water-contact activities (boating, aesthetic enjoyment). In Sri Lanka, of course, water-contact fishing such as net fishing is a commercial rather than a recreational activity. Although water-contact activities demand a more stringent discharge standard than non water-contact activities, there are certain quality requirements which form a baseline below which the water quality can be considered unsuitable for any recreational activity. Basically the marine coastal waters should not contain substances attributable to discharges of wastewater that:

- give rise to visible floating or suspended matter, oil, grease or foam;
- produce sludge banks or slime infestation;
- give rise to heavy growths of attached plants or animals or blooms of plankton;
- give rise to discolouration or turbidity or the evolution of gases and odour;
- contain substances which are injurious or toxic to the natural ecology of the disposal area or which can be concentrated in food chains.

Most water quality criteria for marine waters where water-contact recreational activities take place include specific limits for pH and pathogens. The maintenance of the receiving water pH within a range of 6.5 to 8.3 (extreme limits 5.0 to 9.0) is recommended in order to reduce eye irritation. This range is easily achieved by the natural buffering capacity of the sea unless highly alkaline or acidic industrial wastes are present.

Although it is accepted that the presence of pathogens in the receiving water constitutes a health risk, the practical difficulty of isolating and monitoring pathogens results in coliform organisms being used as surrogate indicators for pathogens. There is as yet no proven epidemiological evidence which can be used to stipulate threshold coliform levels in marine waters above which adverse health impacts will arise. Recommended criteria in other countries vary, for example maximum faecal coliform levels of 2000/ 100 ml are stipulated by the EEC, in the USA the Environmental Protection Agency (1976) standard is based on a faecal coliform level of 400/100 ml not being exceeded in 90% of samples.

Coliform levels recorded in the ocean at Colombo, Negombo and Galle in 1979 showed a mean of 11 total coliforms/100 ml. Fecal coliform levels were zero. The samples were collected at a distance of 1000 to 2000m from the shore. However, shoreline samples in Colombo taken at a depth of 0.6 m showed a total coliform range of 14 to $8.12 \times 10^5$/100 ml, with a mean of 18000/100 ml (Engineering-Science, 1981). More recent data for the period October 1987 to January 1988 show a mean total coliform level of 4700/100 ml (range 210 to 110 800/100 ml). At the recreation beach south of Dehiwela Canal to the Mount Lavinia Hotel, the mean total coliform level from the recent data is 1800/100 ml, compared to 4700/100 ml in 1979. The commissioning of the marine outfall at Dehiwela is undoubtedly a contributory factor to this improvement.

There are no coliform criteria for marine waters in the Sri Lankan standards. It is recommended that a guideline faecal coliform value of 1000/100 ml not being exceeded in 80% of samples be adopted. This is comparable to the World Health Organization (1975) criterion for acceptable bathing waters. This standard can be compared to the quality of the recent shoreline samples in the recreation beach previously referred to. The mean total coliform level was 1800/100 ml, with about 80% of the samples being equal to or less than 2400/100 ml. Although an exact relationship between total and faecal coliform organisms cannot be defined, total coliform levels typically exceed faecal coliform levels by at least a factor of two. A tentative conclusion, therefore, is that the shoreline quality at the recreation beach is acceptable for water-contact recreation.

Because of the ability of shellfish to concentrate both pathogens and toxic substances, it is essential that the quality of the water in commercial shell fishing areas be of a high quality. Studies carried out elsewhere on the bacterial pollution of shellfish suggest that the marine water should not contain more than 40 faecal coliforms/100 ml in more than 10% of samples and the shellfish should be regularly monitored for the presence of indicator organisms.

### 8.2 Outfall Design Procedure

The objective in outfall design is to select a discharge location such that the receiving water quality criteria are not exceeded. Usually this will entail ensuring that the outfall is of such a length that the coliform levels at the shore are within acceptable limits under the most adverse conditions of on-shore currents and winds.

The outfall pipe should end in a diffuser section fitted with a number of horizontally-discharging ports. Typical diffuser design criteria are:

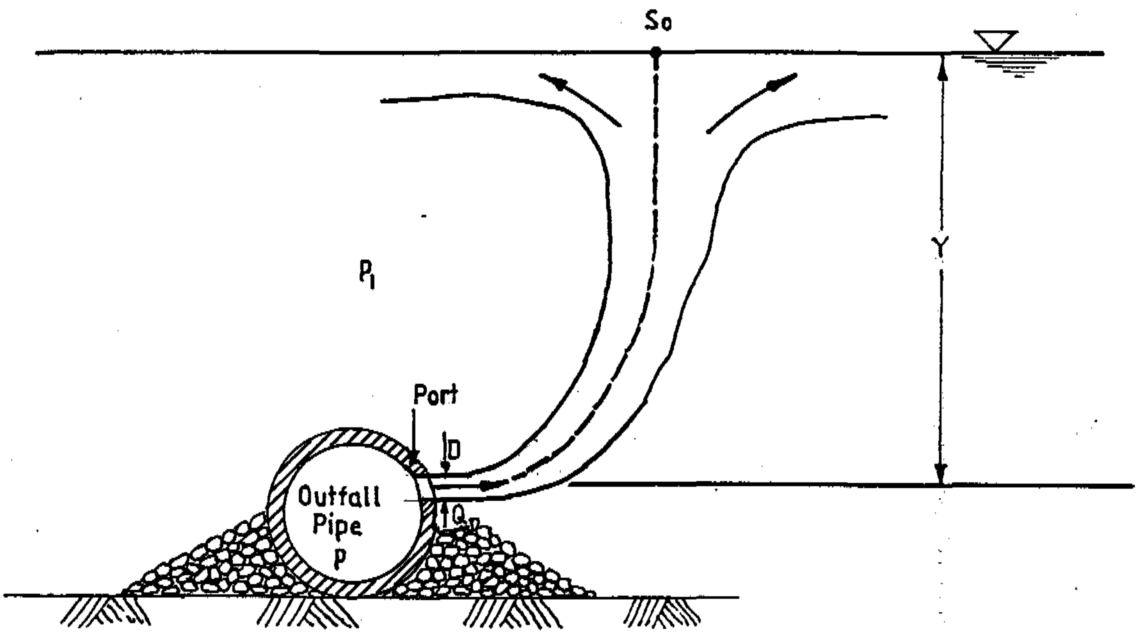
- o Total port area must not exceed the main outfall pipe area.
- o Port diameter > 0.15 m to avoid blockage.
- o Nozzle densimetric Froude number must be greater than unity to avoid ingress of sea water.
- o Port velocity > 2 m/s.
- o Main outfall pipe velocity > 1.5 m/s to avoid settlement of particulate matter.
- o Flow per unit length of diffuser 0.005 to 0.02 $m^3/s.m$

#### 8.2.1 Initial Dilution

When sewage is discharged in the sea it tends to rise to the surface because it is less dense than sea water. As the sewage mixes with the sea water the density of the mixture approaches the density of the receiving water and the sewage loses its momentum and ceases to rise. In some cases density stratification occurs in the receiving water and the sewage remains submerged below the surface of the sea. The dilution of the sewage which occurs as the sewage rises is referred to as the initial dilution and is a function of jet velocity, depth of water, density difference and movement of the receiving water.

A range of design procedures are available for calculating the initial dilution. Some procedures are based on calculating the initial dilution for still water and then modifying for sea water movement, other procedures incorporate factors to take into account the movement of the receiving water. None of the procedures are exact, careful interpretation is necessary for each specific location based on field surveys of actual conditions. The following discussion summarises the most common procedures currently in use.

The initial dilution on the axis of a rising plume (defined as So in Figure 8.1) can be derived for still water conditions from Figure 8.2 in accordance with the procedure of Fan and Brooks (1966). The Froude number (F) is defined as:

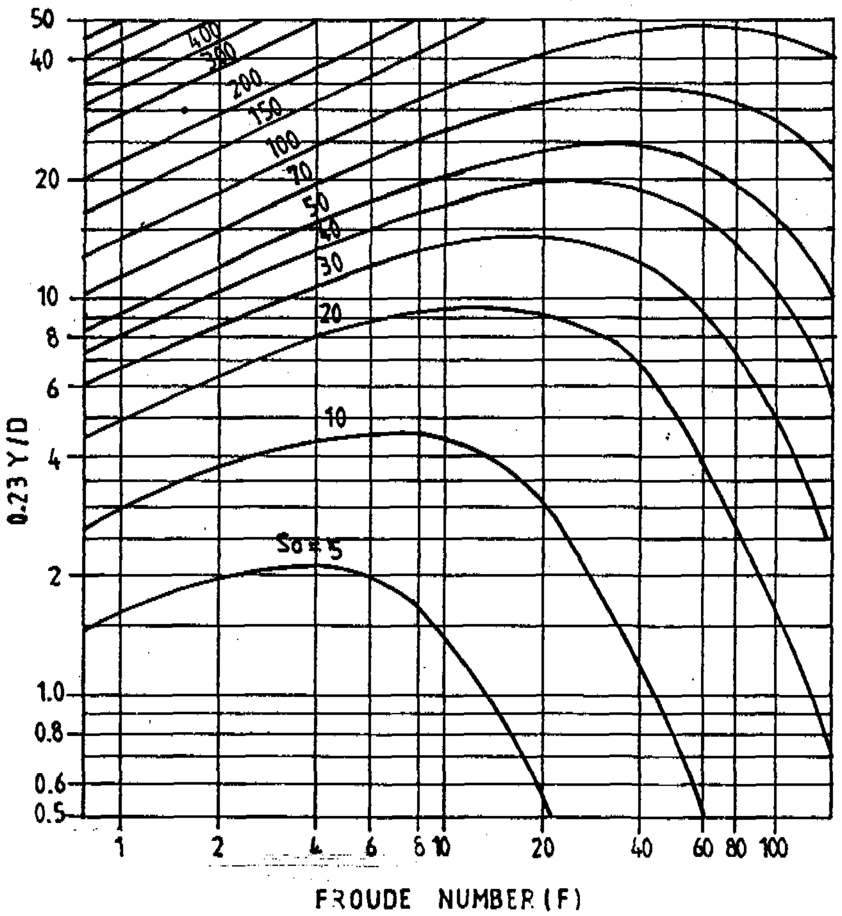



NOTATION :

D diameter of port
Q port discharge
Y height of rise of plume
$P_1$ receiving water density
P wastewater density
S initial centreline dilution

DEFINITION SKETCH FOR RISING JET OF WASTEWATER DISCHARGED HORIZONTALLY INTO OCEAN

FIGURE 8.1




CALCULATION OF INITIAL DILUTION OF A HORIZONTAL DISCHARGE OF WASTEWATER INTO STILL MARINE WATERS

FIGURE 8.2

$$F = Vj\ [gD(P_1 - P)\ P_1^{\ -1}]^{-0.5}$$

where $Vj$ = jet velocity (m/s)

$g$ = acceleration due to gravity

A range of procedures can be used to correct the initial dilution for the movement of the receiving water. Typical correction factors proposed by Agg and Wakeford (1972) and the Hydraulics Research Station (1977) respectively are as follows:

$$\text{Log (correction factor)} = 0.938 \log \left[\frac{Va}{Vj}\right] + 1.107$$

$$\text{Correction factor} = 1 + \left[\frac{6Y}{D}\right]\left[\frac{Va}{Vj}\right]^{1.5}$$

Where $V_a$ = ambient velocity

Field tests have shown that the Agg and Wakefield (1972) equation tends to underestimate the measured initial dilution, whereas the Hydraulics Research Station (1977) equation tends to overestimate the measured dilution.

An alternative procedure to calculate initial dilution which incorporates a correction factor for ambient velocity is that of Roberts (1977) which is summarised in Figure 8.3 (adapted from Ludwig, 1978). The notation in Figure 8.3 is as follows:

Sm = centreline dilution taking into account ambient velocity

Y = height of rise of plume (m)

Q = total wastewater discharge ( $m^3$/s)

L = diffuser length (m)

Va = ocean current velocity (m/s)

q = wastewater flow per unit length of diffuser (Q/L)

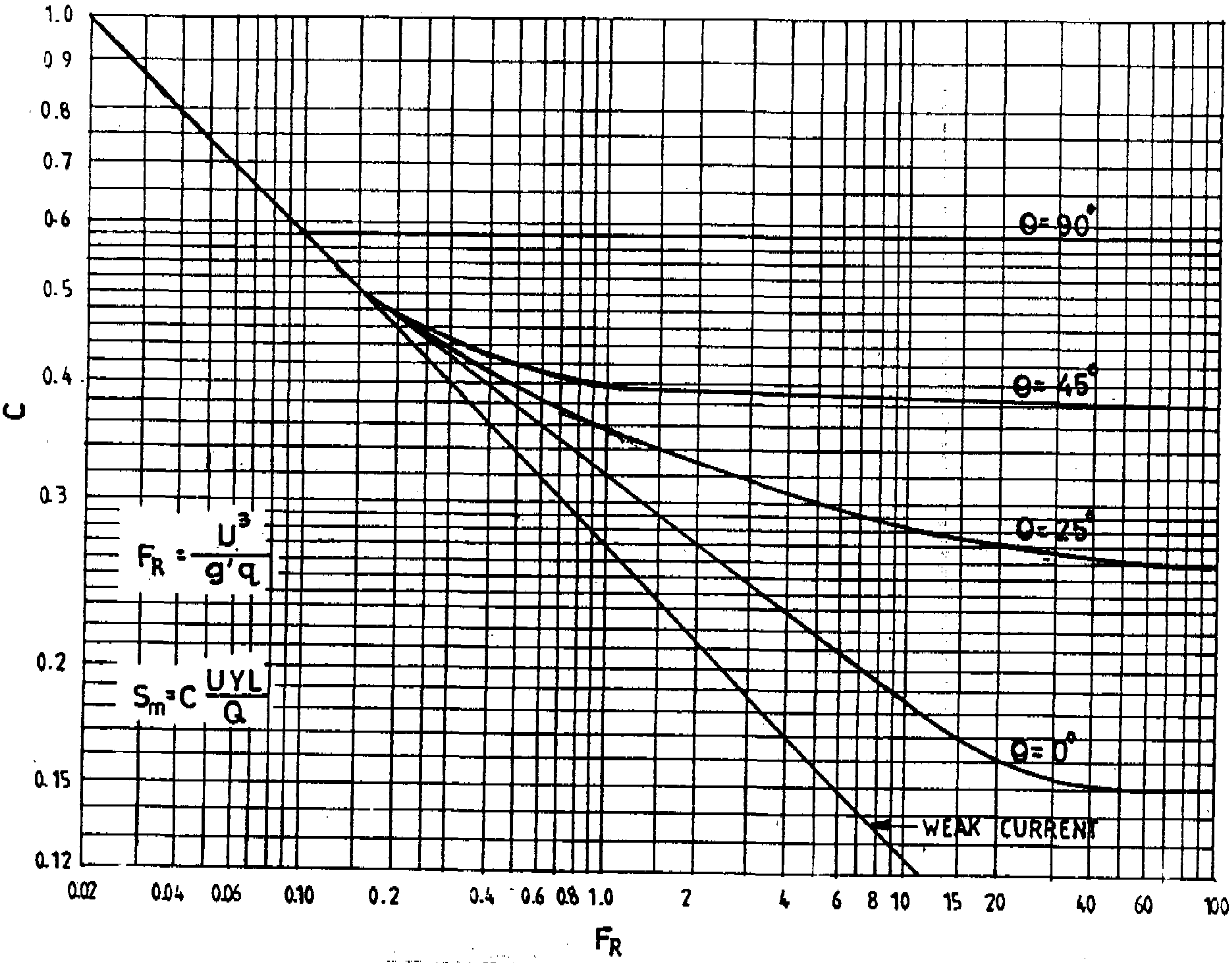
$g^1$ = apparent acceleration due to gravity

= $g\ [P_1 - P]\ P_1^{\ -1}$

$F_R$ = modified Froude number

= $Vj^3\ (\ g^1\ q)^{\ -1}$

$\theta$ = orientation of diffuser line to predominent ocean current




CALCULATION OF INITIAL DILUTION OF WASTEWATER INTO MARINE WATERS TAKING INTO ACCOUNT OCEAN CURRENT

FIGURE 8.3

The procedure in Figure 8.3 is applicable for the following ranges:

$\frac{L}{Y}$ 3.85 to 30.0

$\frac{YN}{L}$ > 5.0

where N = number of discharge ports

The curve marked "weak current" in Figure 8.3 is for still water conditions, for $0 = 90°$ the diffuser is oriented perpendicular to the current, for $0 = 0°$ the diffuser is oriented parallel to the current. Since the values of Sm are minimum centre-line dilutions, actual field dilutions will be greater because of the spreading effect of the plume. An approximate estimate of the field dilution for a line source plume according to Rawn and others (1960) is given by:

$$Sa = Sm.2^{0.5}$$

where Sa = field dilution

### 8.2.2 Coastal Dilution

The dilution which occurs along the coast is more difficult to determine than initial dilution since it depends on coatal currents and turbulent diffusion, both of which are difficult to measure and predict. Various theroretical and experimental studies have been made in an attempt to define turbulent diffusion in the sea, but in coastal areas where the combined influence of tides, wind and currents predominates, no general procedures are wholly adequate. One approach based on field observations of British coastal conditions was developed by Agg (1978) and is summarised in Figure 8.4. The procedure relates initial dilution to the distance of the wastewater discharge from the shore for various amenity values of the receiving water. It is not suggested that the model depicted in Figure 8.4 be applied directly to Sri Lanka, but the same approach can be followed by developing a data base from oceangraphic surveys.

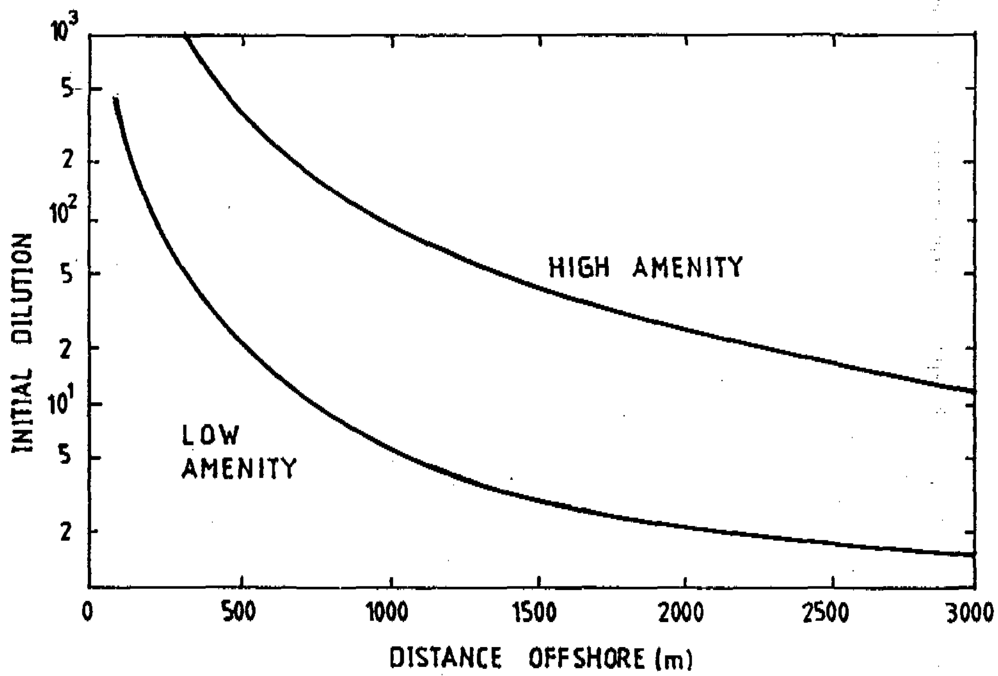
In addition to initial dilution and dispersion, pathogen levels are also reduced by natural die-off in accordance with the following relationship:

$$S_D = 10^{t/T_{90}}$$

where $S_D$ = dilution due to natural die-off

$t$ = time of travel from plume to amenity area (h)

$T_{90}$ = time required for 90% die-off (h)




DESIGN MODEL FOR CALCULATING OUTFALL LENGTH

FIGURE 8.4

For coliform organisms observed valves of $T_{90}$ in the ocean are typically in the range 3 to 5 hours, depending on sunlight, salinity, clarity of the water and other factors. The overall dilution is the product of the individual dilutions resulting from natural die-off, initial dilution (taking ambient velocity into account) and dispersion. For Sri Lankan waters a $T_{90}$ value of 3 hours has been determined from field surveys (Engineering-Science, 1981).

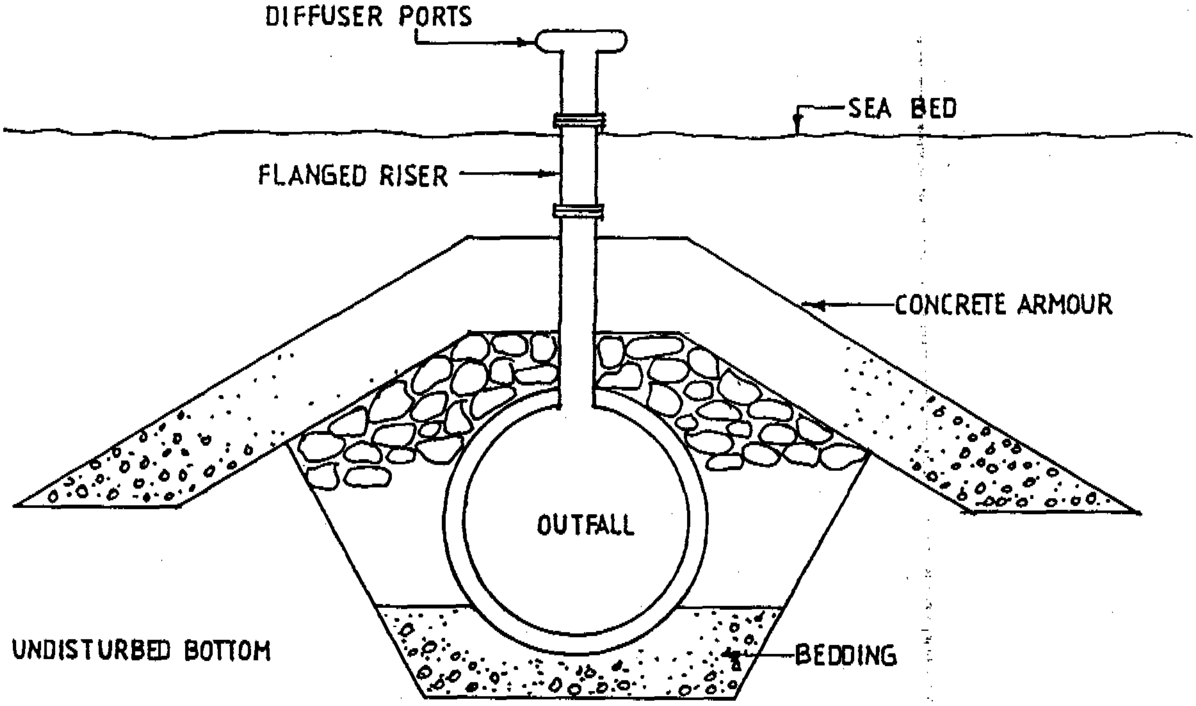
In conclusion, the design of marine outfalls is not an exact science, there being many alternative procedures based on site specific data which may not be applicable elsewhere. Although the procedures outlined in this section can be used for approximate determinations of outfall location and tentative budget estimates, they should not be used as a substitute for the hiring of specialist expertise in conjunction with comprehensive oceangraphic surveys.

## 8.3 Construction

Marine outfalls are generally constructed of either steel or concrete pipes using tow methods or lay barge techniques. The use of high density polyethylene is also popular in some circumstances. The choice of pipe material is a function of the pipe diameter, availability of fabrication areas in the outfall vicinity and availability of construction materials. The protection of the outfall from shipping is necessary in anchorage zones and shipping lanes. In such areas the outfall will have to be buried and protected with concrete armour. The diffuser ports will project above the bottom of the sea and these should be constructed using weaker materials above the bedlevel. This will result in the diffuser being sacrificed if fouled by an anchor, without damage to the outfall. The use of flanges on the riser will also facilitate replacement. A typical cross section detail for a buried diffuser is shown in Figure 8.5.

## 8.4 Pretreatment

In order to prevent floating material from reaching the shore it is advisable to provide preliminary pretreatment facilities consisting of a drum screen to remove particles in excess of 12 mm in size. The screenings should be dewatered and buried in a sanitary fashion.




TYPICAL CROSS SECTION DETAIL
FOR A BURIED DIFFUSER

FIGURE 8.5

## SECTION 9

## SEPTIC TANK SYSTEMS

Where cistern flush-toilets are provided, as opposed to pour-flush latrines or pit privies, and where a local or regional sewerage system is not immediately accessible, septic tank systems are an acceptable means of sewage disposal. A comprehensive Sri Lankan code of practice is available for the design and construction of septic tank systems (Sri Lanka Standards Institution, 1986), the purpose of this section is to summarise the more important aspects of this method of sewage disposal.

### 9.1 Applicability of Septic Tank Systems

The Sri Lankan code of practice recommends that communal septic tanks not be used for populations exceeding 300. The Colombo Metropolitan Region Urban Development Plan (UNDP, 1978) recommends that on-site systems should only be considered when the population density is less than 140 p/ha, with such systems being almost inevitably more economic than waterborne sewerage where the density is less than 100 p/ha. In practice, the key factors are the quantity of wastewater discharged and the method adopted for disposal of the septic tank effluent.

The ideal effluent disposal system is to land, preferably by means of a trench soakaway system. However, land requirements can be considerable if the absorptive capacity of the land is limited. For example, for a septic tank receiving the total household waste of say 150 l/d.p from a family of five with a typical infiltration rate on reclaimed land of 10 l/d. $m^2$ the total land area required for the soakaway would be 112 $m^2$. For a population density of 100 p/ha on such land, the soakways would represent almost 25% of the total development area.

If the septic tank was to receive only toilet waste, the land area required for the soakaway would be much less, say about 25% of that required for the total household waste. In this case sullage wastes (from laundry, bathing, kitchen) would be discharged to a roadside drain, a common practice in many urban areas of Southeast Asia.

The health risks arising from roadside drains containing sullage depend upon the pathogen content of the drain and the possibility of either direct contact between persons, particularly children, and the drain, or contamination of water supplies by ingress of polluted drain water, particularly for shallow well supplies and leaking distribution networks where interrupted supplies are frequent. If it is assumed that pathogens may be present in household wastes in similar proportions to faecal coliforms, the health risks can be assessed from coliform levels, although it is admitted that this is a somewhat tenuous assumption. Studies in varies countries have shown that faecal coliform levels in sullage are significantly lower than in excreta, often by a factor of about five orders of magnitude. For example, if it is assumed that a roadside drain containing raw sewage has a health risk of 100 (on an arbitrary scale) then a drain containing only sullage would have a minimum health risk of 0.001. This comparison gives a general idea of the overall magnitude of potential risk.

If the drains are well designed and maintained and are always free-flowing, the likelihood of stagnant, anaerobic, conditions developing will be minimal. In practice, of course, this is rarely the case and the drains are frequently blocked with refuse and overgrown. The presence of sullage is unlikely to have a significant effect on the mosquito population, with perhaps the exception of the filariasis vector which tends to breed in polluted rather than clean water.

The Sri Lankan code of practice stipulates that septic tank effluent should not be discharged to an open drain without prior chlorination. It is suggested that in the present context of the Sri Lankan urban environment, this stipulation is inappropriate. Chlorination of an effluent with a high organic content is not likely to be efficient. Since septic tanks in Sri Lanka are rarely desludged, unless the household is faced with blockage and severe odour problems, the tank effluent is also likely to contain suspended matter which may encapsulate pathogens and render chlorination virtually ineffective.

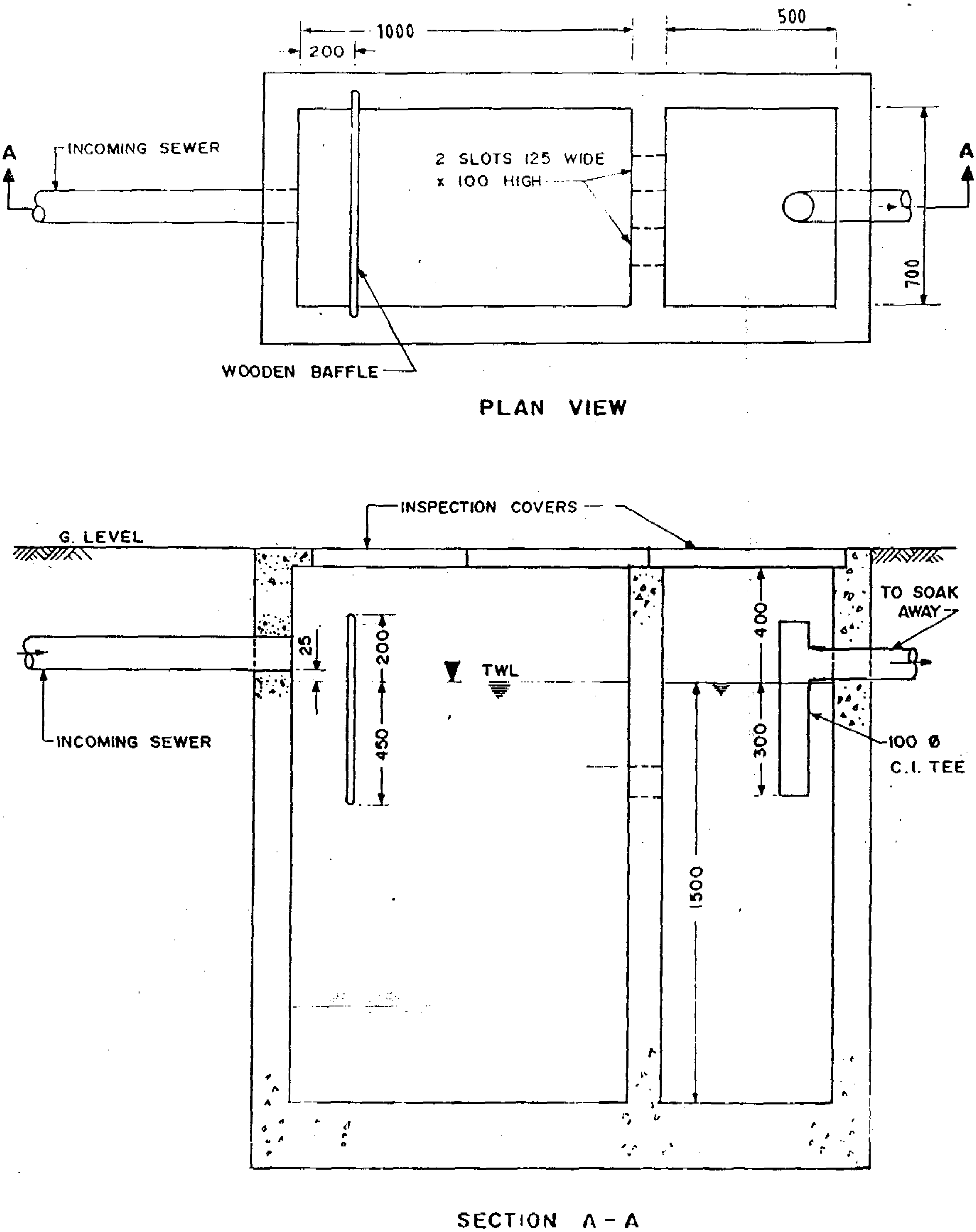
It is suggested that the following strategy be adopted for septic tank use in Sri Lankan urban areas until such time as financial resources are available to construct regional sewerage systems. The strategy is set out in order of preferred priority.

- o Discharge all household wastes to the septic tank and trench soakaway.
- o Discharge toilet wastes only to the septic tank and soakaway, discharge sullage wastes to the drain.
- o Discharge all household wastes to the septic tank and then to the drain (this should not be a widespread practice).

Septic tank desludging frequencies should be maintained in accordance with the design guidelines. The following design criteria refer to tanks serving individual houses with in-house piped water supplies and cistern-flush toilets. Criteria for systems serving pour-flush latrines are presented in the Design Manual "Guidelines for Latrine Selection and Construction."

## 9.2 Septic Tank

Recommended tank dimensions are shown in Figure 9.1 for a total household waste flow of 150 l/d. p and an occupancy of 5 persons. The design is based on a total hydraulic retention time of 24 hours plus sludge storage based on an accumulation rate of 0.04 $m^3$/y.p, a typical sludge build up rate where anal cleaning is with water or soft paper. A desludging frequency of three years should be adopted for calculating the sludge storage capacity. The dimensions can be varied depending upon the occupancy and flow, but the volume of the first chamber should be double the volume of the second chamber. The length to width ratio shown in Figure 9.1 is not mandatory, the tanks may be circular if cheaper. The design shown in Figure 9.1 differs slightly from the standard tank designs currently used in Sri Lanka. The simple wooden inlet baffle as shown in Figure 9.1 removes the risk of blockage of the more conventional inlet tee piece.




# RECOMMENDED SEPTIC TANK DIMENSIONS

FIGURE 9.1

Vent pipes leading from the tank to the atmosphere are not necessary if the house plumbing is provided with a vent stack. When desludging, the tank should not be emptied completely since it is desirable to leave a small amount of sludge (typically 100 mm depth) as a seed to encourage digestion when the tank refills.

As an interim improvement measure to reduce odour problems which arise from discharging tank effluents direct to surface drains rather than to a soakaway, the second chamber can be filled with stones of nominal size 50 mm and the entrance from the first chamber placed at about two thirds of the chamber depth. The outlet T-piece can be removed. The stones act as an upflow anaerobic filter and malodorous compounds are broken down thereby resulting in a tank effluent which can be discharged to a storm drainage system without giving rise to odour problems. The stone filter has no effect on BOD or pathogen removal.

A two-compartment septic tank which is regularly desludged will generally reduce BOD and SS levels in the incoming sewage by about 45 and 80% respectively. Faecal coliform levels in tank effluents are often in the range 1 to 10 million/100 ml.

## 9.3 Soakaway

For septic tank systems receiving the total household waste from dwellings with a relatively high water use, a trench soakaway system is the preferred method since the tank effluent is spread over a relatively large land area. Alternative soakaway systems for pour-flush latrines are described in the Design Manual "Guidelines for Latrine Selection and Construction."

A typical trench soakaway would have a trench width of 0.4 m, a depth of gravel of 0.75 m below the feed pipe in the trench and 0.2 m cover over the pipe. The soakaway should not be covered by an impermeable cover since this eliminates evapotranspiration. The required trench length can be calculated from:

$$L = \frac{PQ}{2Di}$$

where $P$ = population
$Q$ = flow (l/d.p)
$D$ = effective depth of trench walls (0.75 m)
$i$ = infiltration rate (l/d. m²)

Trench spacing should be not less than 2.0 m.

The infiltration rate must be determined from a permeability test conducted on the site. Soakaways are not generally recommended if the percolation time exceeds 4 min/mm, equivalent to a minimum infiltration rate of 10 l/d. $m^2$. Maximum infiltration rates to be used in determining trench lengths should not exceed 50 l/d. $m^2$, regardless of the percolation time. The percolation test is described in the Sri Lankan code of practice.

The land area ($m^2$) required for the soakaway can be calculated from the percolation rate as follows:

$$A = 1.5 \frac{P Q}{i}$$

and $$i = \frac{127}{r^{0.5}}$$

where r = percolation rate (min/cm)

The factor of 1.5 allows for trench spacing.

## 9.4 Septage Disposal

The septic tanks should be desludged regularly in accordance with their design frequency and the septage discharged to a regional sewage treatment works. If regional sewage treatment works are not available, the septage can be discharged either to evaporation/percolation lagoons or to land. Typical design criteria for land disposal are a hydraulic loading rate of 2 to 14 mm/d for areas of moderate permeability where removal of the liquid fraction is by evapotranspiration and percolation without surface run off. The actual rate should be determined from percolation tests and an estimate of the evapotranspiration rate of vegetation growing on the disposal area. In order to avoid ponding the area should be dosed intermittently, say for 6 h followed by 18 h resting. Overall maximum hydraulic loading rates would be about 50 $m^3$/ d. ha. Typical BOD loading rates for land disposal are generally in the range 0.2 to 5.6 kg/d. ha.

Basic guidelines to be followed in selecting a land disposal area for septage are the avoidance of contamination of groundwater supplies, surface run off and ponding, and maintaining discharge rates such that the vegetation is not hydraulically swamped or killed due to an accumulation of trace metals or boron (from detergents) in the plant tissues. Odour problems can be minimised by siting the disposal area away from residential developments and by adding a bag of lime to the tanks prior to commencing the journey to the disposal area. The addition of lime has the effect of stabilizing the organic matter and thereby reducing the sewage odour.

# APPENDIX A

# R E F E R E N C E S

# APPENDIX A

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TABLE B1

TOLERANCE LIMITS FOR INLAND SURFACE WATERS FOR USE

AS RAW WATER FOR PUBLIC WATER SUPPLY

SRI LANKA STANDARD 722 : 1985)

| | Determinant | Tolerance limit |
|---|---|---|
| 1. | Coliform organisms (monthly average, most probable number (MPN) per 100 ml. | Not more than 5000, with less than 5 percent of the samples with value 20,000, and less than 20 percent of the samples with value 5000. |
| 2. | pH range at ambient temperature | 6.0 to 9.0 |
| 3. | Chloride (as Cl) mg/l, max. | 1,200 |
| 4. | Nitrate (as N) mg/l, max. | 10 |
| 5. | Fluoride (as F) mg/l, max. | 1.5 |
| 6. | Phenolic compounds (as phenolic OH) mg/l, max. | 0.005 |
| 7. | Oils and grease mg/l, max. | 0.1 |
| 8. | Pesticide residue | As per WHO/FAO requirements |
| 9. | Arsenic (as As) mg/l, max. | 0.05 |
| 10. | Cyanide (as CN) mg/l, max. | 0.05 |
| 11. | Lead (as Pb), mg/l, max. | 0.01 |
| 12. | Mercury (total as Hg), mg/l, max. | 0.001 |
| 13. | Selenium (as Se) mg/l, max. | 0.05 |
| 14. | Chromium (as Se) mg/l, max. | 0.05 |
| 15. | Dissolved oxygen (DO), mg/l, max. | 4 |
| 16. | Biochemical oxygen demand (BOD) mg/l, max. | 5 |
| 17. | Radio active material : | |
| | a. Alpha emitters uc/ml, max. | $10^{-9}$ |
| | b. Beta emitters uc/ml, max. | $10^{-8}$ |

TABLE B2

TOLERANCE LIMITS FOR INDUSTRIAL EFFLUENTS DISCHARGED

INTO INLAND SURFACE WATERS:

SRI LANKA STANDARD 652 : 1984

| Parameters | Values (Not to Exceed) |
|---|---|
| BOD in 5 days at 20°C | 30 |
| pH | Between 6 and 8.5 |
| Suspended solids, mg/l | 50 particle size below 850 microns |
| Temperature °C | 40 |
| Oils and grease, mg/l | 10 (top 30 cm layer) |
| Phenolic compounds, mg/l | 1.0 |
| Cyanides, mg/l | 0.2 |
| Sulphides, mg/l | 2.0 |
| Fluorides, mg/l | 2.0 |
| Total residual chlorine, mg/l | 1.0 |
| Arsenic, mg/l | 0.2 |
| Cadmium, mg/l | 0.1 |
| Chromium, mg/l | 0.1 |
| Copper, mg/l | 3.0 |
| Lead, mg/l | 0.1 |
| Mercury, mg/l | 0.0005 |
| Nickel, mg/l | 3.0 |
| Selenium, mg/l | 0.05 |
| Zinc, mg/l | 5.0 |
| Ammoniacal Nitrogen, mg/l | 50 |
| Pesticides | Undetectable |
| Radioactive Materials | |
| Alpha emitters, c/ml | $10^{-7}$ |
| Beta emitters, c/ml | $10^{-6}$ |
| Chemical Oxygen Demand mg/l | 250 |

These values are based on dilution of effluent by at least 8 volumes of clean receiving water. If the dilution is below 8 times, the permissible limits are multiplied by 1/8 of the actual dilution.

No increase in the permissible limits for these parameters is to be allowed as a result of increased dilution of the effect beyond 1:8.

Metals: These have an additive effect. If more than one of them is present at the same time, percentage values are calculated for each metal from the actual concentration and the permissible limit. The sum of the percentages should not exceed 100.

TABLE B3

# SRI LANKA TOLERANCE LIMITS FOR INDUSTRIAL AND DOMESTIC EFFLUENTS DISCHARGED INTO MARINE COASTAL AREAS

SRI LANKA STANDARD 721: 1985

| Parameter | Tolerance Limit |
|---|---|
| Domestic and Industrial: | |
| Temperature | 45 °C |
| BOD | 100 mg/l |
| TSS (process waste) | 150 mg/l |
| TSS (cooling water) | influent cooling water TSS plus 10% |
| Particle size of: | |
| Floatable solids | 3mm |
| Settleable solids | 850 m |
| pH | 6.0 - 8.5 |
| Oils and grease | 20 mg/l |
| COD | 250 mg/l |
| Residual chlorine | 1.0 mg/l |
| Ammoniacal Nitrogen (N) | 50 mg/l |
| Industrial only (mg/l) | |
| Phenols | 5.0 |
| Cyanides (CN) | 0.2 |
| Sulphurides (S) | 5.0 |
| Fluorides (F) | 15.0 |
| Arsenic (As) | 0.2 |
| Cadmium (Cd) | 2.0 |
| Chromium (Cr) | 1.0 |
| Copper (Cu) | 3.0 |
| Lead (Pb) | 1.0 |
| Mercury (Hg) | 0.01 |
| Nickel (Ni) | 5.0 |
| Selenium (Se) | 0.05 |
| Zinc (Zn) | 5.0 |
| Organo-phosphorous compounds (P) | 1.0 |
| Chlorinated hydrocarbons (Cl) | 0.02 |
| Radio-active material | |
| Alpha emitters | $10^{-8}$ micro curies m/l) |
| Beta emitters | $10^{-7}$ " " |

TABLE B4

CENTRAL ENVIRONMENTAL AUTHORITY: INTERIM STANDARDS

TOLERANCE LIMITS FOR INDUSTRIAL WASTE WATER DISCHARGED INTO

PUBLIC (COMMON) SEWER FOR FURTHER TREATMENT

Ref. Indian Standard 3306: 1974 with modifications

| Parameters | Values (Not to Exceed) |
|---|---|
| BOD in 5 days at 20°C, mg/l | 200 |
| pH | 6 - 8.5 |
| Suspended solids, mg/l | 500 |
| Temperature °C | 45 |
| Phenolic compounds, mg/l (as $C_6H_5OH$) | 5, (up to 50 if secondary treatment provided) |
| Oils and grease, mg/l | 30 |
| Cyanides, mg/l | 2 |
| Chromium, (Hexavalent) mg/l | 2 |
| Copper, mg/l | 3 |
| Lead, mg/l | 1 |
| Nickel, mg/l | 2 |
| Zinc, mg/l | 10 |
| Ammonical Nitrogen, mg/l | 50 |
| Radioactive materials | |
| Alpha emitters, c/ml | $10^{-7}$ |
| Beta emitters, c/ml | $10^{-6}$ |
| <u>If effluent used for irrigation</u> | |
| Boron, mg/l | 2 |
| Percent sodium | 60 |
| Total dissolved solids | 2,100 |
| Chlorides as Cl, mg/l | 1,000 |

The quality of waste water discharged into common sewer or collection system should be such as to ensure that the waste water:

a. does not damage the sewer by physical or chemical action
b. does not endanger the health of the workers cleaning the sewer
c. does not upset the processes that are normally used in sewage treatment
d. does not overload the common treatment plant
e. does not damage the crops or affect the soil in case the effluent after treatment is used for irrigation; and
f. does not create fire and explosion hazards due to constituents present in the effluent.

Industrial effluents containing solids such as ash, sand, feathers, large floatables, straw, plastics, wood, lime, slurry, beer or distillery slops, chemical or paint residues, gross solids from cannery wastes, tar, hair, rag, metal shavings, garbage and broken glass, shall not be permitted to be discharged into public (common) sewers.

TABLE B5

CENTRAL ENVIRONMENTAL AUTHORITY: INTERIM STANDARDS

TOLERANCE LIMITS FOR INDUSTRIAL

EFFLUENTS DISCHARGED ON LAND FOR IRRIGATION PURPOSES

Indian Standard 3307: 1977 with modifications

| | Parameters | Values (Not to Exceed) |
|---|---|---|
| 1. | pH value | 6 to 8.5 |
| 2. | Total dissolved solids, mg/l maximum | 2,100 |
| 3. | Sulphate (as $SO_4$) mg/l maximum | 1,000 |
| 4. | Chloride (as Cl) mg/l maximum | 600 |
| 5. | Percent sodium, maximum | 60 |
| 6. | Boron (as B) mg/l maximum | 2 |
| 7. | Oils and grease, mg/l maximum | 10 |
| 8. | Biochemical Oxygen Demand mg/l* | 250 |
| 9. | Alpha emitters microcuries per millilitre | $10^{-9}$ |
| 10. | Beta emitters, microcuries per ml. maximum | $10^{-8}$ |
| 11. | Odour | No obnoxious odour |
| 12. | Floatables | No visible large sized solids |

*Can be relaxed or tightened depending on soil conditions and applications rate.

Please Note:

1. It is necessary to limit certain constituents in effluents, especially those considered toxic, so that the effluent may comply with normally accepted irrigation water quality.

2. The Authorities should give due consideration to the local conditions, and in special cases may relax or tighten the limits if need be.

3. Soils on which the effluents are applied are studied periodically from the viewpoint of physico-chemical characteristics to ensure that they are not damaged and the ground waters are not polluted. Similarly, crop yields also need to be studied.

HYDRAULIC LOADING APPLICABLE FOR DIFFERENT SOILS

| Soil Texture Class | Recommennded Dosage of Settled Industrial Effluents Cubic Metre/Hectare/Day |
|---|---|
| 1. Sandy | 225 to 280 |
| 2. Sandy loam | 170 to 225 |
| 3. Loam | 110 to 170 |
| 4. Clay loam | 55 to 100 |
| 5. Clayey | 35 to 55 |

These hydraulic loading rates are only guidelines. In actual practice the loading rates are fixed taking into account the nature of crop, the consumptive use, climate and frequency of application.