REPUBLIC OF KENYA

MINISTRY OF WATER AND IRRIGATION

FINAL PRACTICE MANUAL

FOR

SEWERAGE AND SANITATION SERVICES

IN

KENYA

DECEMBER 2008

CAS CONSULTANTS LTD
P.O. Box 67693-00200,
Tel: 4445240/65; 0722829554; 0733894951
Nairobi.
casconsult@africaonline.co.ke

LIST OF CONTENT

LIST OF ACRONYMS

CBO	Community Based Organization
DELTA	Development Education for Leadership Teams in Action
GOK	Government of Kenya
ITDG	Intermediate Technology Development Group
KWAHO	Kenya Water for Health Organization
OOPP	Objective Oriental Project Planning
PHAST	Participatory Hygiene and Sanitation Transformation
PRA	Participatory Rural Appraisal
PROWWESS	Promotion of the Role of Women in Water and Environmental Sanitation Services
SARAR	Self-esteem Associative Strength, Resourcefulness, Action Planning and Responsibility
SIDA	Swedish International Development Cooperation Agency
UNICEF	United Nations Children,s Fund
WHO	World Health Organization
MOH	Ministry of Health
KEFINCO	Kenya Finland Western Water Supply Program
LBDA	Lake Basin Development Authority
KIWASAP	Kilifi Water and Sanitation Project
PF	Pour Flush (as in “PF toilets”)
ROEC	Reed Odorless Earth Closet
VIDP	Ventilated improved double pit (as in “VIDP latrines)
VIP	Ventilated improved pit 9as in “VIP latrines)
WRMA	Water Resources Management Authority
WSS	Water Supply and Sanitation
NEMA	National Environment Management Authority
BOD	Biological Oxygen Demand

LIST OF TABLES

Table 15.2 Microbiological Water Quality Guidelines for Irrigation water

LIST OF FIGURES

PHOTO GALLERY

1.0 INTRODUCTION

1.1 Background

The provision of sanitation is a key development intervention – without it, ill-health dominates a life without dignity.

The term sanitation in its widest sense covers excreta disposal, sullage and storm water drainage, solid waste management and hygiene and stresses the need to go beyond a concern with the provision of the facilities to consider the services that people receive. However, this Practice Manual does not cover indepth study and analiysis of stormwater drainage and solid waste manangement.

Simply having access to sanitation increases health, well-being and economic productivity. Inadequate sanitation impacts individuals, households, communities and countries.

Despite its importance, achieving real gains in sanitation coverage has been slow. Scaling up and increasing the effectiveness of investments in sanitation need to be accelerated to meet the ambitious targets of Millennium Development Goals.

Nearly 40% of the world's populations (2.4 billion) have no access to hygienic means of personal sanitation. Globally, WHO estimates that 1.8 million people die each year from diarrhoeal diseases, 200 million people are infected with schistosomiasis and more than 1 billion people suffer from soil-transmitted helminth infections. A Special Session on Children of the United Nations General Assembly (2002) reported that nearly 5,500 children die every day from diseases caused by contaminated food and water.

Increasing access to sanitation and improving hygienic behaviors are key factors to reducing this enormous disease burden. In addition, such changes would increase school attendance, especially for girls, and help school children to learn better. They could also have a major effect on the economies of many countries – both rich and poor – and on the empowerment of women. Most of these benefits would accrue in developing Nations.

In Kenya, Environmental sanitation coverage declined in the decade upto 1990 and saw modest gains thereafter. In 1983, the national sanitation coverage was estimated at 49%., 45% in 1990 and 46% in 1996. Difference in access to adequate sanitation between urban and rural environments still persist, with the formally planned urban areas being better served than rural areas, urban slums, and informal settlements. In Kenya, 72% of the excreta disposal facilities are simple pit latrines providing varied degrees of safety, hygiene and privacy.

1.2 Purpose of a Practice Manual

Lack of a National Standard for the Planning and Design, Operation and Maitenance of Sewerage and Sanitation Systems has been singled out as one of the contributors of most sewerage and sanitation services falling into a serious disrepair. Most of the sewerage infrastructure has not been compatible with Kenyan conditions generally and more particularly with specific site conditions. This resulted in a wide variety of methods and designs often leading to the selection of inappropriate technology unsuitable for local conditions, difficult and expensive to operate and maintain.

Consequently**,** the Practice Manual for Sewerage and Sanitation in Kenya has been compiled with the aim of providing guidelines and criteria for all those involved in Water Sector.

The objective is to streamline the Sewerage and Sanitation Sector and guarantee that NEMA standards in Sewerage and Sanitation are achieved.

The aim is to eliminate past mistakes and that resources can be used more effectively if the framework within which they are deployed is clearly defined.

1.3 Scope of Application

The scope of application of this Practice Manual is as follows:-

1) This Practice Manual applies to the planning and design of sewerage system and sanitation in Kenya.
2) The objective of the Manual is to reduce the capital costs through the application of appropriate technology.
3) To be a demand responsive Manual at the community level to significantly increase sustainability of sewerage and sanitation systems and hence the full understanding and cooperation of the community is necessary to ensure that the system functions. It is, therefore, necessary to pay attention to components such as hygiene education, training in operation, maintenance and environmental education.

However, this manual has been prepared through benchmarking which requires for **uncovering Best Practices and learning from others** and hence global information on the same have been obtained and synchronized.

1.4 Target User

This Manual has been prepared for staff of Ministry of Water and Irrigation, Water Service Boards, Water Service Providers, Water Resouces Management Authorities, Consultants engaged in planning, designing and the construction of appropriate sanitation and sewerage facilities in Kenya. Foreign agencies funding these facilities and NGOs who have an interest in supporting the sanitation sector, will find this manual a useful reference register. The Manual will serve as a background document and as a basis for discussion during project preparation and implementation.

1.5 Overview of the State of Water Supply, Sanitation and Sewerage Services in Kenya

1.5.1 Water Supply Services

The water sector in Kenya is facing enormous challenges today as the Government has already recognized that there is poor provision of water supply and sanitation services that has impacted negatively on the economic development of the country by affecting major sectors that include: health, industry and commerce. In addition, women and children labour is now spent more on looking for water rather than for productive purposes. All these have contributed to increase in poverty levels in the country.

The Small Towns Water Supply and Sanitation study found out that the services were of poor quality. Unreliable service had become the rule for most piped systems. Because of a limited focus on comprehensive water resources management, many water supplies face uncertainty with regard to availability and quality of water, especially during the drought years. Lack of investment in maintenance has resulted in the collapse of the WSS infrastructure specially distribution networks, sewers, connections and meters; hence, the need for massive rehabilitation everywhere. There is, therefore, no outstanding Best Practices in Water Supply and Sanitation Services. Schemes owned by some municipalities but managed by private companies portray relatively good management in terms of revenue collection and financial, operation and maintenance management.

In spite of some measures put in place such as forming autonomous water and sewerage companies, the management of the water systems has not been satisfactory in terms of service quality, water loss targets, revenue collection efficiency and flexibility in responding to consumer needs for increased demand and expansion of the distribution network. There are problems in recruiting and retaining management and technical personnel. The Wate Companies are over-established at lower cadre thus absorbing resources that are critically needed to support optimal operations. Poor management of water supply schemes often leads to declining levels which reduces chances of good cost recovery arising out of unwillingness to pay and resulting in sector investments not keeping pace with demand.

Water supplies managed by the Service Board have the poorest operation and maintenance performance. Materials and spare parts for repair of water and sanitation facilities are never availed in time. Documentation of operation and maintenance and field activities is either improperly done or not done at all. The water supplies experience high unaccounted for water and frequent bursts and leakages that take long before they are repaired. This may be attributed to bureaucracies and unavailability of resources. Revenue generated from the sale of water is never ploughed back into the systems. Schemes staff are poorly remunerated and, therefore, lack working morale.

The National WSS strategy paper recognizes that one of the key serious weaknesses in the water sector has been lack of demand management due to lack of attention in the following areas:

- Centralized decision making;
- Weak institutional framework;
- Large quantities of unaccounted for water (over 50%), leakages which are rarely attended to;
- Poor operation and maintenance of facilities;
- Poor designs that do not take into account pressure zones in various schemes;
- Inadequate management and financial skills;
- Lack of monitoring and evaluation of the system;
- Illegal connections and theft;
- Diversion of water revenue to unrelated activities; and
- Tariffs that do not encourage consumers to respond to increased water usage.

The above institutional weaknesses are caused by limited national economic growth; poor organizational structures; lack of autonomy and unclear definition of roles leading to conflicts. Other weaknesses include:

- Poor coordination between sector institutions leading to wastage of resources and duplication of efforts;
- Lack of adequate skills to manage and operate water supplies commercially;
- Inadequate logistical and institutional capacity for effective maintenance, material supply and cost recovery to operate systems;
- Political interference; and
- Diversion of resources for water services to other unrelated activities.

Basic data on water is always available from operators' record books. However, none of these records are used for any situational analysis. Such an exercise would help in forward planning by identifying problems and providing appropriate solutions when they are anticipated.

Most of the water services infrastructure was constructed between twenty and forty years ago. Therefore most of it has outlived its economic design life and are in urgent need of rehabilitation and augmentation to cater for age and increased demand due to increased population. The performances of most of these systems have been below expectation due to the several factors contributing to this state of affairs.

Operation and maintenance of sewerage systems has always been ignored and very little attention is given to effluent quality control. Some of the sewage treatment works are either neglected or abandoned although sewage is allowed to flow into the plants. A random review of events identified as a result of the analysis of press reports on actions of mobilization about wastewater issues in the Kenyan local dailies are as outlined below:

Contamination of water sources: Contamination of water sources was reported on 21st April 1998 in Meru where medical wastes were disposed 100 meters from Nchugi water source. Other reported cases covered Nakuru dumpsite located on the faults of Menengai crater and western fringes of Lake Nakuru National Park where traces of heavy metals such as chromium, zinc, lead, arsenic, copper, iron, manganese and mercury were found. Cases of sewerage systems breakdown were reported in Machakos, Kisumu, Nyandarua, and Embu districts with the effluent finding their ways into water sources used by community members.

A case was reported in Hola district hospital in April 1998 where raw sewerage flooded the hospital and thus hampering service delivery. The situation forced the medical staff and the patients to use the nearby bush for defecation. In Athi River, it was reported that severe water pollution from industry, garbage and faecal effluent had threatened the lives of the water users downstream. The water quality in some parts of the river was found polluted over 2000 times above the WHO standards. The effluent from the Pan Paper mill treatment works in Webuye was reported to be dangerous to both human and aquatic life. The El-nino rains of 1998 led to overflows of the existing pit latrines and consequently the faecal waste found its way to the water sources.

The management and operation of water supply schemes in Kenya are currently facing several problems. The most severe of which are waste of water, lack of adequate technical and commercial management skills, under collection of water revenue, large number of defaulters particularly Government institutions and mismanagement of revenue. The situation characterized by the following among others:

- Frustration and despair on then part of water consumers;
- Intermittent supply of water with the reduction in production output of industries;
- Intermittent supply of water interfering with smooth running of health care institutions;
- Increasing number of defaulters due to weak debt collection procedures by water undertakers;
- Irresponsibility and dishonesty on the part of some key players, particularly the management of water undertakers such as Local Authorities;
- Inadequate investment in water resource management;
- Lack of capacity to manage the operations and maintenance;
- Non prioritization of investment in water resource management; and
- Expensive dirty alternatives to piped water.

Operations and maintenance is an area where the public sector has had a dismal performance across the board. More so because the importance of operations and maintenance has been down played.

Infrastructure in water supply systems consists of intake facilities, generators, pumps, water treatment plants, distribution pipe network, and storage tanks. Maintenance of this infrastructure is the application of management, financial, procurement and engineering skills in pursuit of economic life cycle costs. This involves installation, commissioning, operations modifications and replacement of systems, components or sub-components with feedback of information on design, performance, and costs.

Maintenance is usually undertaken in order to prevent failure or in response to failure or in order to increase availability of a system. Maintenance requires professional expertise and consists of:

- Maintenance planning which includes routine maintenance, scheduled maintenance, un-planned maintenance, emergency maintenance, stand-by capacity plan, spares procurement schedule, systems operations schedule and disaster recover plans;
- Maintenance performance that is the carrying out of actual maintenance at the right time, in the right way, by the right personnel with the correct spares and resources; and

- Maintenance control that ensures that plans are achieved in spite of obstacles, variations and uncertainties in both the organization and the environment. This comprises of budgetary control, production control, inventory control, quality control and performance control.

The ad hoc approach to maintenance of water facilities has resulted in none availability or intermittent supply of water in several towns in Kenya.

1.5.2 Conditions of Sewerage Systems

1.5.2.1 Sewer Reticulation

Sewers for domestic and industrial wastewater are separate from storm water drainage except in the Central Business District of Nairobi where there are combined systems. Most of the Local Authorities in the study areas had no information on the physical condition of sewer reticulation. This makes it impossible to comment on actual sewer rehabilitation needs. The authorities do not have regular sewer inspection programs for data collection.

Newly constructed sewers are generally in good working order despite a general lack of preventive maintenance such as regular cleaning and inspection. Trunk sewers are usually oversized to provide spare capacity for future flows with exception of Webuye Municipality where the size of the trunk sewer to the sewage treatment plant is 225mm diameter. Some towns have not realized the projected wastewater flows as the mode of the water supply system is by rationing or the extension of the sewer network does not cover all the areas already served with water. As a result, accumulation of
Sedimentations in the sewer lines is common as the flows are too low to provide self-cleaning velocities.

Most of the sewers constructed in the last 40 to 50 years ago are in poor structural conditions. Development has exceeded the available hydraulic capacity of older sewers in the old central business districts such as the case of Mombasa City. This has resulted to frequent blockages, overflows and surface flooding.

Lack of adequate sewer network leads to underutilization of the capacity of the existing treatment works. For example, the recently completed Mombasa sewage treatment works has a capacity of treating 17,000m^3/day whereas it receives influent of less than 50% of its design capacity.

1.5.2.2 Wastewater Treatment Facilities

In the publicly operated systems, waste stabilization ponds are used in 27 out of 39 facilities inspected. Field investigations showed that the ponds are robust in operating under harsh conditions when compared to the other systems. Generally these provide problem-free operation with the exception that most of the older ponds are filling with sludge and vegetation that generally reduce retention times and therefore, treatment efficiency. The remaining treatment works use conventional processes such as biological attached growth filters (6No.), three oxidation ditches, or three aerated lagoons. In general, most of the recently constructed sewage works are operating far below their intended design capacity. The low flow rates can be attributed to: low rate of sewer connections and inadequate water supply systems resulting in low per capita water consumption.

1.5.2.3 Effluent Disposal

In Kenya, all treatment plant effluent is discharged into an inland stream or lake except for Mombasa where uncontrolled and untreated sewage is currently being discharged into the Indian Ocean. Homa Bay Town and Kisumu City discharge high BOD_5 and nutrient rich waste directly into Lake Victoria, and Nakuru and Naivasha discharge treated effluent into ensitive ecosystems of lakes Nakuru and Naivasha respectivelyThe algae in a waste stabilization pond effluent contribute to both its suspended solids content and BOD. Low technology systems comprised of rock filters have been installed at Njoro and Old Town treatment works in Nakuru. Rock filters consist of a submerged porous rock bed within which algae settle out as the effluent flows through. The algae decompose releasing nutrients that are utilized by bacteria growing on the surface of the rocks. In addition to algal removal, significant ammonia removal may also take place through the activity of nitrifying bacteria growing on the surface of the filter medium.

In general, where public sewerage systems exist, industries usually discharge their effluents into the sewers. Enforcement of the National Trade Effluent Standards is the responsibility of the WRMA and NEMA. They are severely crippled by a lack of resources to inspect industries and monitor, collect and analyse data. The continued uncontrolled disposal of industrial wastewater that is untreatable in the municipal wastewater treatment works has degraded the existing watercourses around the towns and thus seriously affected the quality of drinking water for the downstream users.

1.5.2.4 Operation and maintenance practice

The Operation and Maintenance practices of the sewerage systems vary widely from one urban centre to the next, depending on the level of funding available; availability of tools and spare parts; equipment; and management and staffing. In several urban centres, sewerage systems are working below capacity due to water rationing and blocking of sewers.

Sewers are not inspected or cleaned regularly and there appears to be no efforts to prevent conditions that will eventually lead to more serious and costly maintenance problems. With the exception of the newer facilities, most sewage treatment facilities have either fallen into serious disrepair or are non-operational and beyond repair.

Maintenance requirements for waste stabilization ponds are very simple; however, they are neglected. Although the treatment process is not immediately affected, the pond environment will suffer, leading to odours, flies and mosquitoes.

The maintenance requirements of ponds are very simple, but they must be carried out regularly. Otherwise, there will be serious odour, fly and mosquito nuisance. Maintenance requirements and responsibilities must therefore be clearly defined at the design stage so as to avoid problems later. The following routine maintenance activities were found lacking in almost all the ponds visited:

(a) Removal of screening and grit from the inlet works;

(b) Cutting the grass on the embankments and removing it so that it does not fall into the pond (this is necessary to prevent the formation of mosquito-breeding habitats; the use of slow-growing grasses minimizes);

(c) Removal of floating scum and floating macrophytes, e.g. Lemna, from the surface of facultative and maturation ponds (this is required to maximize photosynthesis and surface re-aeration and obviate fly breeding);

(d) Removal of any accumulated solids in the inlets and outlets;

(e) Repair of any damage to the embankments caused by rodents, rabbits or other animals; and

(f) Repair of any damage to external fences and gates.

Field observations indicated inadequacy of labourers that led to lack of instructions on the frequency at which the above tasks were done with adequate and constant supervision. There were no records showing whether the supervisor/foreman had a weekly intervals pond maintenance record sheet.

In order that the routine operation and maintenance tasks are properly done, WSP installations must be adequately staffed. The level of staffing depends on the type of inlet works (for example, mechanically raked screens and proprietary grit removal units require an electromechanical technician, but manually raked screens and manually cleaned grit channels do not), whether there are on-site laboratory facilities, and how the grass is cut (manually or by mechanical mowers).

There is a lack of process control and monitoring at most conventional treatment works. These treatment plants require constant monitoring and process adjustments in order to provide the intended performance.

1.5.2.5 Sludge Treatment and Disposal

The field survey revealed that desludging of ponds is rarely done and most ponds are in desperate need of relief. Most of the conventional works surveyed had facilities for digesting and subsequently drying the sludge. Treated sludge is usually disposed of at the municipal dump or sold to farmers for agricultural use where there is cultural acceptable.

Septic sludge of privately owned septic tanks and soak-away pits is removed by local authorities, Ministry of public works or private firms operating on call. Most local authorities own at least one sludge exhauster truck but generally these are unreliable and often unavailable. There is little control over the disposal of septic sludge. Treatment works do not have any special provisions for accepting septic wastes.

Private firms tend to discharge sludge into nearby drainage ditches or open field. The indiscriminate disposal of septic sludge is a serious health hazard and a threat to the environment.

1.5.3 Security

A chain-link fence should surround ponds and gates should be kept padlocked. Warning notices, in the appropriate local language(s), attached to the fence and advising that the ponds are a wastewater treatment facility, and therefore potentially hazardous to health, are essential to discourage people from visiting the ponds, which if properly maintained should appear as pleasant, inviting bodies of water. Children are especially at risk, as they may be tempted to swim in the ponds. Birdwatchers and hunters are also attracted to ponds by the often-rich variety of wildlife, and they may not be aware that the ponds are treating wastewater. The presence of hippopotamuses and crocodiles in a pond makes sampling hazardous, and can be fatal: hippos have killed three pond operators at the Dandora WSP in Nairobi in the past.

1.5.4 Financial analysis and indicators

1.5.4.1 General Experience

Independence from Parent Authority: For those companies which have become independent from their parent institutions, it was found that their operations are smooth and records maintained on a daily basis. For those Councils still attached to the parent institutions take time to keep records on a daily basis. They wait towards the end of the reporting period when they start preparing the accounts. With the current water sector reforms, it is expected that the trend will change.

The Local Authorities schemes run into problems because they had to be controlled by the resolutions of the Council and the arm of the Treasurer. Thus, expenditures do not necessarily have to be prioritized in favour of the Water and Sewerage Department, but the chips fall where personalized interest fall. Therefore, the infrastructure is neglected, and more so the sewerage system. Thus, revenue generated in the WSS is utilized in some unrelated services.

Staffing Levels and Competence: Majorities of the staff in the commercial/accounting division are not professionally qualified. Experience shows that there is one or two qualified while the rest are not. Most schemes have raised concerns about meter-reading process. This has affected billing and therefore levels of revenue collection. No proper records are kept due to corruption in the metering process. The main problem lies in the:

- Recruitment process;
- Lack of incentives;
- Lack of training; and
- Poor supervision.

Commercialization of WSS will see improvement in recruitment and performance.

Most institutions tend to have a bottom heavy kind of structure. For most Local Authorities, the bottom tends to have mostly unqualified non-productive staff. During the study it was found that the unproductive staff tends to be politically protected. Staff rationalization therefore in all areas should be undertaken. It was noted that some of the postings to the sewerage section was on disciplinary basis because of the dirty nature of this section. This trend will have to be reversed and instill professionalism in operations and staff management.

Staff ratio: It was not easy to determine with accuracy the staff in the sewerage section due to high turnover in this section. Most of the staff in the sewerage section are posted on disciplinary grounds. As a result there is a high turnover in the section.

It is not possible to look at sewerage staff ratio per 1000 connection although such a ratio would justify the level of staff in sewerage section compared to connectivity. The most ideal situation could be that for every water connection the must also be a sewer connection, except where there is a communal standpipe. Therefore, based on the water connections, Table 1.1 gives the staff ratio for those towns whose information was available. In the World Bank study (Aide Memoire, 2000), the staff ratio for other African Countries for instance Ghana had a staff ratio of 17.7, Uganda 30.0 and for the Nairobi City 16.2. Thus, the ratios in Table 1.1 compare favourably even though they are generally high.

Table 1.1: Water supply staff ratio per 1000 connection

Town	Number of staff	Number water connections	Staff per 1000 connections
Nanyuki	72	5741	12.5
Kericho	141	6500	21.6
Naivasha	12	1600	7.5
Meru	76	2917	26
Nyeri	136	7019	19
Thika	106	850	12.4
Eldoret	157	15317	10.2

1.5.4.2 Expenditure to Billing

This indicator is of importance as it indicates whether the operators are able to meet their recurrent obligations from the revenue billed. Of the study towns, the following seven towns cannot be able to pay their obligations from the sewerage revenue:

- Mombasa 161%
- Kericho 99%
- Isiolo 117%
- Mavoko 504%
- Kiambu 129%
- Naivasha 92%
- Eldoret 86%

1.5.4.3 Operation and Maintenance relation to Billing

From this study, it is evident that there is minimal expenditure related to sewerage operation and maintenance, yet the sole purpose for any business is to safeguard, improve and maintain operational assets so that they can continue to generate income. Neglect, which is witnessed in most public utilities and thus resulting to loss of revenue and high cost of replacement of the dilapidated assets.

The capital outlay in the water and sewerage utilities is enormous and this outlay or investment is expected to generate income over a long period of time. Sustainability of this income is based on the premise that the assets will continue to be operational. For the assets to perform, they must be maintained on a continuous basis. This is not the case for most water and sewerage operators.

O&M therefore is a factor of production and distribution in a utility such as water and sewerage and has a relationship with volume of sales and generation of revenue. It is therefore important to determine the relationship of O&M to revenue. This relationship however should not be seen on a short-term basis, as its impact is a long-term.

1.5.4.4 Sewerage Revenue to Total Revenue

Revenue from sewerage seems to be too low compared to water revenue. Most towns' sewerage revenue is below 31%. This can only mean that sewerage connectivity in most towns is very low. The odd towns such as Limuru and Homa Bay show percentages of 54 and 50 respectively. The average sewerage revenue for all towns that provided information is about 9.8 per cent.

Generally sewerage connectivity in most towns is very low. Mombasa has the lowest ratio of 0.08 per cent in this regard. The cause of this is due to very low connectivity and a very low tariff (Kshs 15 per connection irrespective of water consumed).

1.5.4.5 Performance based on various scenarios

There are four scenarios in the provision of water supply and Sanitation services:

- Same water supply and sewerage operator - Self-billing and revenue collection for both water and sewerage;
- Different water supply and sewerage operators - Water operator bills and collects for sewerage;
- Different water and sewerage operators - Sewerage operator bills and collects for sewerage; and
- Water Supplier different from operator - Self-billing for both water and sewerage.

The first scenario is the most problem free as the operator for both is the same and there are no hitches in collection of revenue as long as there is effective billing and an appropriate tariff.

The second scenario has proved to be a source of conflict between the water operator and the sewerage operator. In most cases the sewerage operator complains of the water operator not remitting collected revenue on time. The sewerage collector normally would expect full payment not taking into consideration that there are defaulters and part payments. This caused mistrust especially between Local Authorities and the NWCPC.

The third scenario also is a source of conflict. Although the sewerage operator bills and is responsible for collection, there are always conflicts especially where there is no sufficient flow of water and the customers sometimes refuse to pay. It is also not easy to enforce collection, as sewage cannot be disconnected.

The fourth scenario creates a problem when there is a very high tariff differential. The case in point is Nakuru Town where the Municipal Council sold water below the bulk supply rate and therefore incured losses.

During study, these scenarios were noted and the most effective scenario is where the operator is responsible for both water supply and sewerage and can determine the best way to manage the customers.

2.0 COMMUNITY PARTICIPATION IN THE PLANNING OF SANITATION SYSTEMS

2.1 Community Participation and Cooperation of Stakeholders

Sanitation intervention will have a high degree of success if the community is participating in the formulation of the scheme and in some cases they should be prepared to contribute their time and labor during the construction phase. Achieving community acceptance requires considerable effort since in many cases the benefits of sanitation are not apparent and the householders may be completely satisfied with their existing sanitation facilities.

2.1.1 Existing Participatory Methods in Kenya

The key participatory approaches used in the promotion of hygiene in Kenya are the PROWWESS/SARAR, Objective Oriented Project Planning (OOPP), PHAST, DELTA and PRA. These methodologies have been adapted mainly by the MOH, as well as various local and International agencies, namely, UNICEF, WHO and Sida supported projects; Action Aid, Help Age, Intermediate Technology Development Group (ITDG), CARE Kenya, KWAHO, Lake Basin Development Authority (LBDA), Africa Now, NETWAS, RWSG-ESA etc (Refer table 2.1).

Table 2.1 Key Participatory Methods used in the Promotion of Hygiene in Kenya

Method	Institution	Where used
PRA	Action Aid Africa Now CARE KWAHO KEFINCO Kilifi Water and Sanitation Project (KIWASAP) LBDA	Project areas of Eastern, Coast, Nyanza and Western Provinces
PHAST	Africa Now CARA-Kisumu KWAHO MOH	Various regions
SARAR	KEFINCO Africa Now Action Aid KWAHO CARE Kenya ITDG MOH	Various Regions
ZOPP/OOPP	CARE-Kisumu Action Aid Help Age KIWASAP	Siaya, Kakamega Kilifi

From the review undertaken in Kenya by WHO/UNDP, it was clear that most of the methods complemented one another; none of them stood on their own. All the people interviewed on what participatory approaches were, and most defined them as “methodologies which assist trainers to target the local people- making it easier for the different agencies to get information from the community and vice versa”. These methodologies are not used in isolation, but in combination. They are widely used to address health and hygiene problems. Participatory methods are used for training, for material development and in field application at various times, for example, in problem identification, analysis, during planning and implementation, and also for monitoring and evaluation. They are considered a

breakthrough in communication and have been adopted by other sectors such as Agriculture, nutrition, health and housing.
Most of the methods especially SARAR and PHAST are adapted by redrawing the pictures to reflect the local situation and for community diversity purposes.

2.1.1.1 The PHAST Initiative

The PHAST initiative owes its success to all the people who have faith in the capacity of all human beings to be creative and to be leaders of change, if approached in the correct manner.

What is PHAST?

P articipatory
H ygiene
A nd
S anitation
T ransformation
. . . Is an innovative approach to promoting hygiene, sanitation, community management of water and sanitation facilities.

PHAST uses methods and materials that stimulate the participation of women, men and children in the development process.

The PHAST initiative is built on some of the more recently developed principles on how to promote sanitation more effectively. Some of these were expressed in WHO Informal Consultations held in 1992 and1993, and have since been expressed and affirmed elsewhere.

The promotional principles built into the PHAST methodology are as follows:
■ Any sustainable improvement in hygiene and sanitation must be basedon a new awareness of the complex interaction between behaviouraland technological elements.
■ The best way to achieve sustainable improvement is to take an incrementalapproach, starting with the existing situation in a community and building up a series of changes.
■ Improvement in hygiene behavior alone has been shown to have a positive health impact whereas improvement in sanitation facilities alone may not bring health benefits. Therefore, greater emphasis needs to be put on improving hygiene behavior, but the ideal situation would be one where improvement in both behavior and facilities can take place simultaneously.

The following is the strength:-

- Its focus on hygiene behavior change
- Its usage of visual tools; its creation of awareness;
- Its enhancement of community participation and helping in resource identification and improvement of sanitation coverage;
- It enables communities to discuss and generate new ideas; and
- PHAST is easy to use in terms of interpretation and replication, and is relevant for solving problems on sanitation as well as changing community perceptions

The following is the limitation:-

- Concerns that it may open discussions especially on sensitive issues which must be handled skillfully;
- That it is very expensive to implement;
- That the tools can only be used by trained persons;
- That one requires a series of trainings to gain competence;
- That the impact takes long to be felt; and

- PHAST requires skilled artists to draw pictures that are easily understood and interpreted. This finding emphasized the fact that a lot of training is required for trainers to gain skills in the training, interpretation and application of tools in different contexts

2.1.1.2 SARAR

What is SARAR?

As thinking in development, and in health, has evolved it has been recognized that sustained change at community level cannot be achieved without real commitment from and involvement of the community. It is considered that development must respond to the needs felt by the community and that not only should users be involved in the development process but they should choose, manage and own the facilities or services created. This Participatory methodology was developed to facilitate this process. The
Underlying *principle* is that the best way to promote change is to offer communities ways to take more control of their own development.

SARAR is a participatory methodology, developed since the 191Os, which has shown to be effective in enabling people to identify their problems, plan for change and implement and monitor that change. It is based on the philosophy of participatory development, the main beliefs of which are that:

- a high level of personal involvement in decision-making is the root of real, long-term commitment to change;
- people closest to the problem are the best ones to find the solution;
- self-esteem is a prerequisite to decision-making and follow-through; Sustainable learning takes place best in a group context, which contributesto a normative shift;
- Learning should be fun.

The SARAR techniques are not teaching tools which seek to impart knowledge.
They are methods which seek to foster discussions among households and communities. SARAR uses visual materials and role play to facilitate the process. Trainers are trained and then, in turn, train community workers. They learn to use and adapt a series of tools which generate discussion and assist planning. Most importantly they rethink their interaction with the community. They begin to see the community as a source of wisdom - as a group that, when helped to identify its problems and to plan for change, is capable of acting independently to make the desired changes. In water and sanitation programmes, demand for and uptake of services has been seen to increase significantly, as has spontaneous action by the community to construct or upgrade latrines. SARAR stands for Self-esteem, Associative strengths, Resourcefulness,
Action-planning and Responsibility: the five human qualities that the methodology seeks to promote. Planners and community workers can choose to make women a particular focus, but the methodology is relevant to all community members, male and female, young and old. SARAR has been used in programmes addressing a wide range of health or development issues besides water and sanitation, including HIV prevention, diarrhoeal disease control and nutrition.

The implications of using SARAR

The implications for **goals:** Using the SARAR methodology means accepting that people may well identify problems other than those the trainer or manager hoped to focus on. As trainers and policy-makers, we have to ask ourselves whether we can be honestly open-ended in our approach and at the same time hope to generate an increased demand for the particular services our sector offers. We cannot begin with a fixed idea of what the outcome will be. This may mean that different sectors have to coordinate the efforts in relation to the community and allow for multisectoral initiatives.

The implications for programmes

In order to be able to use SARAR, community workers need training and support. They also need time to interact fully with the community. As communities begin to take initiatives for their own development they will need further support. This may mean credit to purchase the materials they need, for example. Traditional systems of
Supply of facilities will no longer be relevant.

The implications for monitoring

Allowing people to define their development
Agenda and to plan for change takes time: annual coverage goals may no longer be a relevant way to monitor change. The programme must necessarily begin slowly and accelerate over time.

The political implications

The SARAR methods allow communities to improve planning skills. This is empowerment and has considerable political implications.
Before applying an approach such as SARAR, community workers and policy-makers must decide whether they are ready to hand some of their traditional control over resources and decision-making to the community.

2.1.1.3 PROWWESS

What is PROWWESS?

It is recognized, that in developing countries women are the principal collectors, managers and often users of water in the home. They are also frequently the guardians of household hygiene and family health. Water collection and use and environmental sanitation may dominate women's daily lives, yet often they are denied a real role in decision-making about water and sanitation.

The PROWWESS programme was created in 1983 to redress this situation.
Its goals have been 'to demonstrate *how* women can be involved, the benefits this will bring to women and their communities and how this experience can be replicated'. PROWWESS stands for Promotion of the Role of Women in Water and Environmental Sanitation Services. Initially the programme was based in the United Nations Development Programme (UNDP), Division for Global and Interregional Programmes (DGIP). Later, in 1990, the programme joined the UNDP/World Bank Water and Sanitation Program.
The PROWWESS programme realized that mechanisms were needed to allow women to participate fully in decision-making about water and sanitation and to plan and monitor change. Many mechanisms for bringing about discussion and stimulating involvement and action were examined.

It was felt that the SARAR methodology, which had originally been developed by Lyra Srinivasan, working with Ron Sawyer, Jacob Pfohl and Chris Srini Vasan, would be particularly effective in achieving these goals.
SARAR has been a cornerstone of PROWWESS efforts to promote community
Participation, and particularly women's participation, in water and anitation Development.

2.1.2.0 Proposed Participatory Methods for Assessing Sanitation Conditions and needs

Participatory Methods will be used to assess local water and sanitation conditions and the possibilities for improving those conditions.

An overview of Participatory Methods arranged in order in which they will likely be used is described:-

- Transect Walk will be used to obtain an initial impression of water and sanitation problems and how people react to them. Informal interviews with community members in the course of such walks will help understand what is being seen and will provide information on what cannot be seen.
- Semi-structured interviews, structured observation, focus group discussions and timelines will help to understand what has happened in the past, what is happening now and what could happen in future.
- Questionnaire surveys and participatory mapping will provide a more detailed understanding of the present situation. Focus group discussions and structured interviews are used to explore specific issues arising from surveys and participatory mapping in more detail. Sanitation ladder exercise is used to obtain an understanding of the assumptions and preference of local people, at the same time providing opportunities for the introduction and discussion of new ideas and helping to inform demand.

2.1.2.1 Transect Walks

This is an initial impression of water and sanitation problems and how people react to them. Informal interviews with community members in the course of such walks help in understanding what is being seen and provide information on what cannot be seen.

At its most basic, a transect walk involve nothing more than walking through an area, observing and recording what you see.

However, it is better if a group of people, including men and women from both within and outside the community and with different areas of expertise and knowledge, make the walk together. Different people see different things. The group should not be too big and group members are encouraged to give individual team members the time and space to break off from the main group in order to explore issues with community members.

Transect walks is timed to provide the maximum amount of information.

2.1.2.2 Informal Interviews

In the course of initial transect walks, people are talked to about their concerns and find out more about the area. Informal interview during this walks provide information on:

- What people think
- Who they perceive to be their leaders and' activists" within the community and
- Aspects of their day to day life that cannot be seen.

Efforts are made when conducting the interviews not to influence people by suggesting to them what one think the answers to questions are.

Before talking to people, some thought to the issues that interest the study and the ways in which introduction of the discussion of these issues is done.. Normally, general question are asked and the community are guided on to more specific issues as the interview progresses
At a later stage in the process, efforts are made to talk to people who may lack either the opportunity or the confidence to speak in more formal meetings. This includes women and people from minority groups. In the case of women, informal interviews are conducted by other women.

Those who carry out reconnaissance surveys are encouraged to discuss their findings with colleagues from different backgrounds and professional disciplines. This help to 'triangulate' interpretation of the findings. In particular, men will explore the implications of their findings with vice versa.

2.1.2.3 Focus Group discussion

Focus group discussion is used to obtain an initial idea of the community's concerns and priorities. A representative of group or groups are brought together from the community and encouraged to talk about their concerns, preferences, hopes and fears. If need be, separate focus group discussions for men and women are arranged.

The Focus groups is used to explore specific issues that have arisen in the course of questionnaire surveys and other general information.

One person facilitates the discussion, with a second person taking notes of what is said. The facilitator is a person from the group that is leading the process of information collection and analysis. The meeting recorder is from the community although this is not essential.

The facilitator aim to:

- Ensure that no individual dominates the discussion
- Make sure that the discussion covers issues of concern.

In early focus group discussion, areas of concern include anything relating to water , sanitation, drainage, solid waste disposal and associated subjects. In later focus group discussions, the discussion is more focused and this requires a greater degree of intervention on the part of the facilitator.

A series of focus group discussions, each attended by representatives of a different group within society are held. The aim is to explore as wide a range of viewpoints and concerns as possible, obtaining the views of people from different genders, ages, social groups and income levels. Attention is focused on the needs of the old and young, who may have special water and sanitation needs.

A checklist of subjects to be covered is prepared by facilitator before start of the focus group discussion.

Overall impressions of problems and not quantitative information are obtained from focus group discussions.

2.1.2.4 Semi-structured interviews with key informants

Informants with special knowledge or who are representative of specific groups within the community provide useful information. The normal approach to key informant interviews is to prepare a checklist of the points upon which information is required and to make sure that those points are raised in the course of the interview. Whenever possible, the information obtained from key informant interviews are checked by interviewing other people who may have a different viewpoint and cross-checking specific points against other sources of information.

Key informants include:-

- Local masons who have been involved in aspects of water and sanitation provision (latrine construction, sewer laying, pipe laying, spring protection, shallow well construction, rainwater harvesting etc)
- Employees of sanitation and water providers (including local authorities, specialist line agencies, and where appropriate, NGOs and CBOs)
- Women, who may face sanitation –related difficulties that are overlooked by men
- Those who have taken action to improve the sanitation facilities available to them
- Those who appear to be content with the status quo.

The information provided by different informants is cross checked.

Government employees working at the local level are included for two reasons:-

- o Have useful insights and information, based on their work in the area and
- o They are likely to be involved in the implementation of decisions made in the course of the planning process. And
- o Possibility of some local government representatives (e.g. school teachers) takes a leading role in planning and implementation at the local level.

Structured observation takes a similar approach in that the aim is to observe a particular activity. The person or team conducting the structured observation prepares a checklist of key points to be noted beforehand. Where the activities being observed are mainly undertaken by women, the observer or observers are also be women.

2.1.2.5 Timelines

People who have lived in the area for a number of years have information on the way in which that area, the facilities provided to it and the problems encountered by the population have developed over time. This information is useful in a number of ways:-

- o Reveal the resources available within the community, in particular, who has taken the lead in trying to address sanitation and water problems in the past.
- o Provide information on previous attempts to solve sanitation and water problems. An understanding of what has gone wrong in the past will help to ensure that similar problems are avoided in the future
- o It may reveal social factors that may be important for water and sanitation provision. Was everyone involved in attempts to improve sanitation? Did some people resist or did some attempt to 'highjack' the process and its benefits.
- o It may reveal changes in the overall situation. For instance, it may be that drainage problems got worse when the water supply was improved.

Like other methods, timelines is used at various stages in the planning process. They may be used to bring people into the process. However, they are likely most useful once a basic rapport has been developed between the planning team and the local community. Timelines are particularly useful in making sure that the knowledge of older people is taken into account in water and sanitation plans.

While information about what has happened in the past is obtained from individuals, people are brought together in groups to think about the past. This enables the different people in the group to reinforce and correct each other's memories. Local people are talked to, to identify those who have knowledge of the history of the settlement and request the people to be part of a timeline discussion. Considerations are given of working separately with groups representing different sectors within society to make sure that everyone's viewpoint is taken into account.

The timeline require facilitator and recorder. The facilitator explains the purpose of the exercise. He has basic list of questions and issues to be raised.

The facilitator allows the participants considerable freedom to introduce their memories and discuss the history of their settlement. Checklist questions are used sparingly and the facilitator aim to guide rather than direct the discussion.

2.1.2.6 Questionnaire Surveys

Questionnaire Surveys have been criticized on the grounds that they tend to reflect the biases of the professionals who prepare them; they require considerable resource and are not really suitable for measuring difference from the norm.

2.1.2.7 Participatory Mapping

Participatory Mapping is a Participatory Rapid or Rural Approach Technique (PRA). In its pure PRA mode, people are asked to draw a map on the ground showing their locality and the features that are important to them. The theory is that by asking people to draw on the ground, everyone feels comfortable and is willing to participate. Experience suggests that this is not necessarily the case in urban areas where most people may feel more comfortable drawing on paper.

This essentially open ended approach to mapping help to establish what is important to people and can also be very effective in drawing women into the planning process.

2.1.2.8 Sanitation Ladders

A sanitation ladder exercise is used for hygiene education and help users make informed choices and/or to help with planning. It involves a facilitator working with a group of community members to assess different sanitation practices and options. Sanitation ladder exercise is carried out with representatives of different groups within society so as to capture their diverse viewpoints. The stages in the exercise are as follows:-

1) A series of pictures, showing different sanitation practices and options will be developed. A group of community members will be brought together and shown the pictures. Only pictures relevant to the area will be used. The group will be asked to look through the pictures and make sure they understand what each picture represents.
2) The groups are asked to rank the pictures from the worst to the best, by laying them out in a line. The groups have to reach consensus on the final ranking and this will prompt discussion on what are 'good' and 'bad' practices or designs.
3) The groups are asked the reason for ranking certain pictures as they did. The resulting discussion will provide the facilitator with opportunities to fill gaps in 'users' understanding and/or discuss hygiene aspects of the different options, thus helping to inform demand.
4) The group is asked about the picture (s) which represent typical situation in their community. If they will not agree, a vote is done to identify the most common option.
5) The group is asked to identify sanitation option they would like to have in an ideal world and which they think would be realistically achievable given local circumstances.
6) The group will finally asked on how to move from "where we are now" to "where "we want to be".

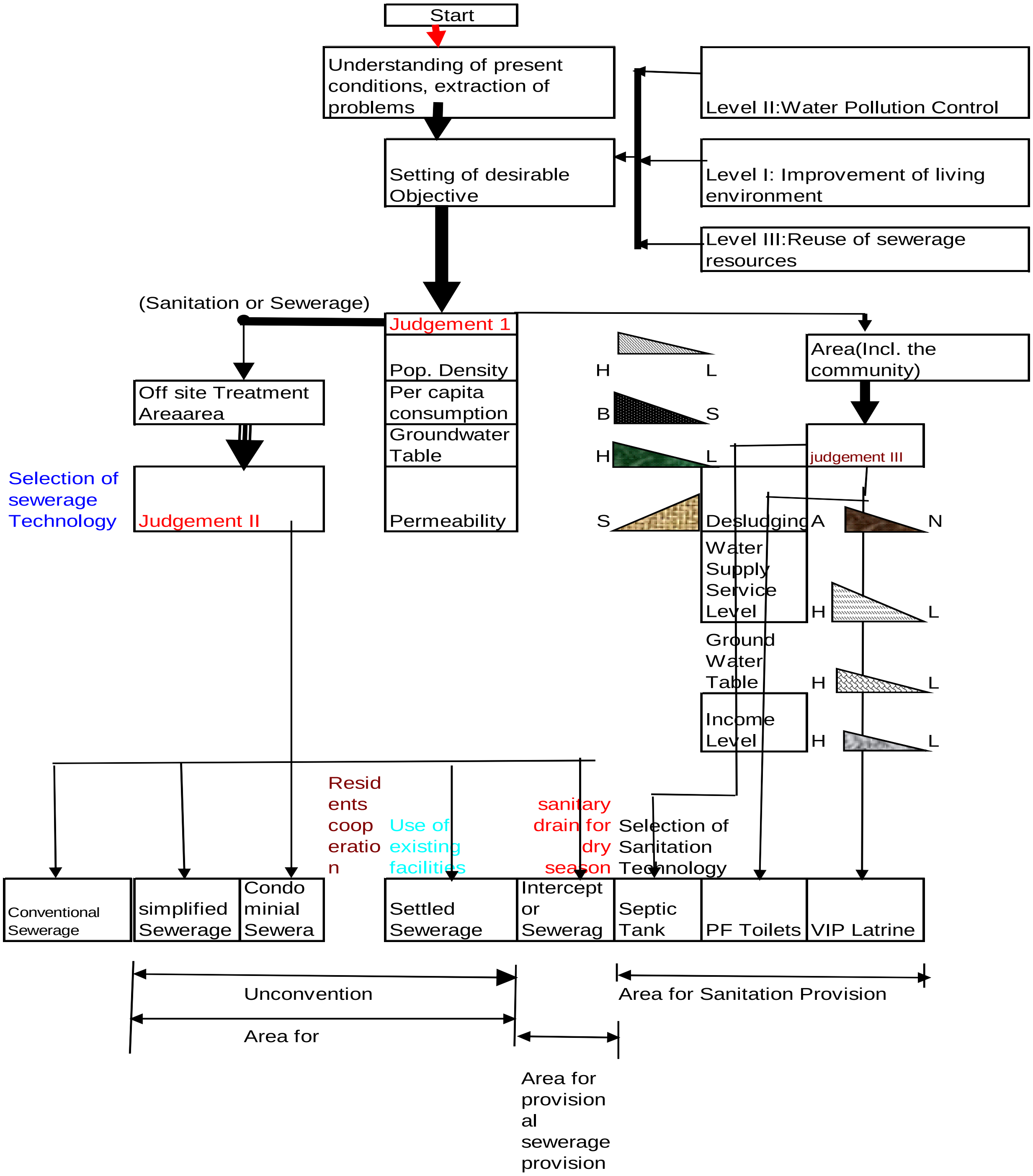
3.0 SANITATION PROGRAM PLANNING

Sanitation Program Planning is the process by which the most **appropriate sanitation technology** for a given Community is **identified, designed**, and **implemented**. The most **appropriate technology** is defined as that which provides the most socially and environmentally acceptable level of service at the least economic cost.

3.1 Existing Sanitation Technology

There are five existing Technologies of Sanitation existing in Kenya. Table 3.1 below has the comparison details:-

Figure 3.1 Algorithm for Selection of Sanitation/Sewerage Technology



Legend	
A	Available
N	Not available
H	High
L	Low
B	Big
S	Small

Figure 3.2 Algorithm for Master Planning

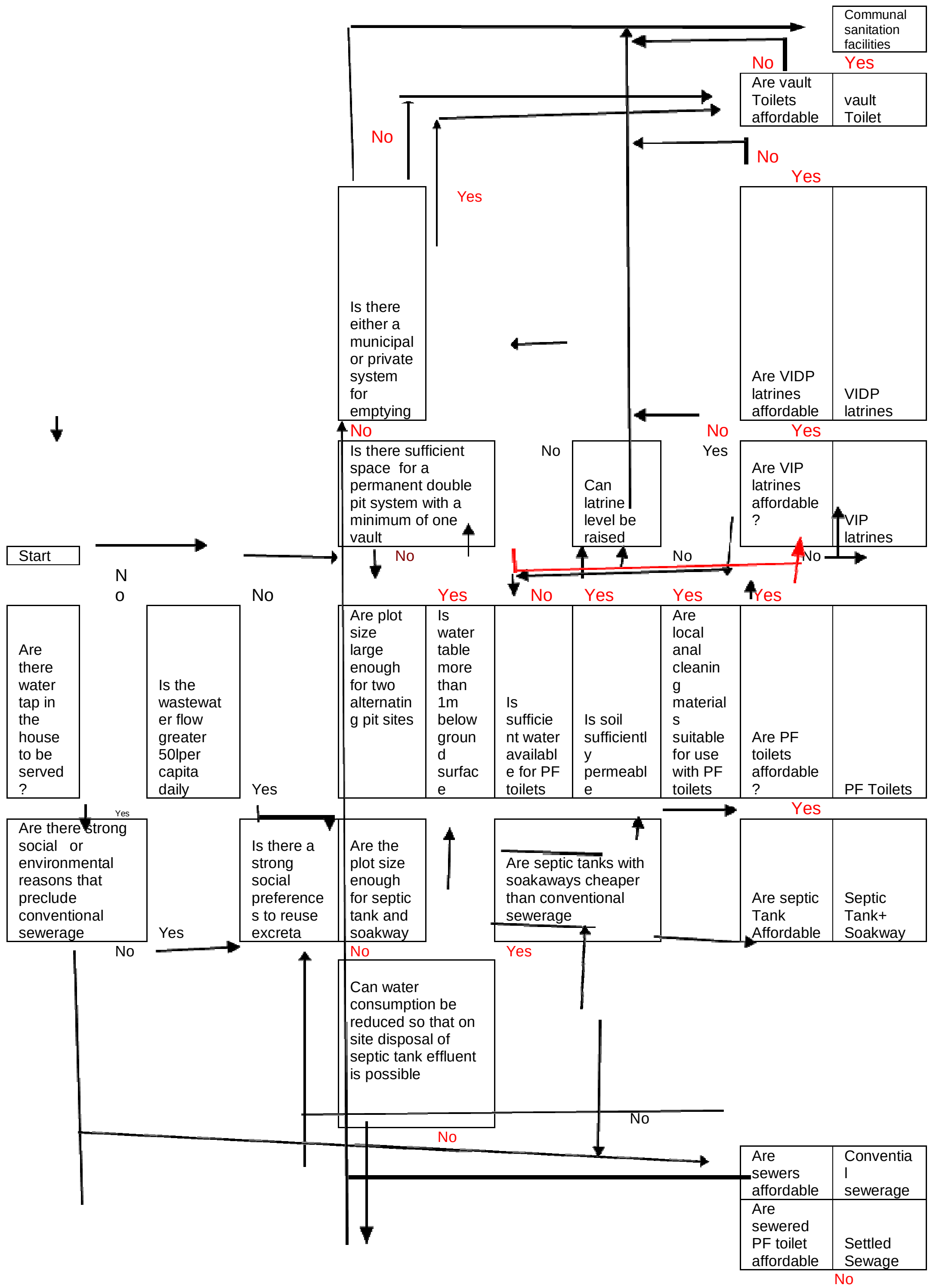


Table 3.1 Comparison of Existing Sanitation Options

Sanitation Technology	Ventilated Improved(VIP) pit latrines	Pour Flush (PF)Toilets	Double vault composting (DVC) toilets	Septic Tanks	Three stage septic tanks	Sewerage
Rural Application	Suitable	Suitable	Suitable	Suitable For Rural Institutions	Suitable	Non suitable
Urban Application	Suitable In Low/Medium Density Areas	Suitable In Low/Medium Density Areas	Suitable In Very Low Density Areas	Suitable In Low/Medium Density Areas	Suitable In Low/Medium Density Areas	Suitable
Construction Cost	Low	Low	Medium	Medium	Medium	Very high
Operating Cost	Low	Low	Low	High	Low	Medium
Ease of Construction	Very easy except in wet or rocky ground	easy	Requires some skilled labour	Requires some skilled labour	Requires some skilled labour	Requires skilled engineer/ builder
Self Help Potential	High	High	High	Low	High	Low
Water requirement	None	Water near toilet	None	Water piped to house and toilet	Water near toilet	Water piped to house and toilet
Required soil conditions	Stable permeable soil, groundwater at least 1m below surface	Stable permeable soil, groundwater at least 1m below surface	None can be built above ground	Stable permeable soil, groundwater at least 1m below surface	Stable permeable soil, groundwater at least 1m below surface	None
Complementary offsite investments	None	None	none	Offsite treatment facilities for sludge	Treatment facilities for sludge	Sewers and treatment facilities
Reuse potential	low	low	high	medium	medium	high
Health benefits	good	Very good	good	Very good	Very good	high
Institutional requirements	Low medium	Low	Low	Low	Low	high

3.2 Basic Studies when Planning for Sanitation / Sewerage

Grasping current sewerage conditions and the state of planning, including conditions and plans for On-site treatment systems, is essential when planning for Sanitation/Sewerage.

Table 3.2 indicates the required information to be collected.

Table 3.2 Important information needed when selecting and designing a sanitation/sewerage system.

Items	Description
Climatic conditions	❖ Temperature Range ❖ Coldest month ❖ Minimum temperature during coldest month ❖ Rainfall (Including drought and food seasons) ❖ Evaporation ❖ Sunshine hours
Site Conditions	➢ Topography ➢ Geology (Including soil Stability) ➢ Hydrogeology (Including seasonal fluctuations of water table) ➢ Vulnerability to floods
Population	✚ Current and projected population ✚ Density (including growth pattern) ✚ Types of residence (including occupancy rates and ownership patterns) ✚ Health Conditions (For all health groups) ✚ Income levels ✚ Local skills that can be utilized (management, Engineering) ✚ Locally available materials, Equipment and components ✚ Municipal services available (including roads and electric power)
Environmental Sanitation	✓ Existing water supply service levels (Including accessibility, reliability, and costs) ✓ Marginal costs of water supply improvements ✓ Existing facilities for night soil disposal, sullage removal, storm water drainage ✓ Other environmental problems (such as garbage and animal wastes)
Sociocultural factors	▪ Residents' understanding of present conditions; their concerns, sensitivity to change ▪ Reasons for acceptance or rejection of past attempts at improvement ▪ Sanitation education level ▪ Religious or cultural factors affecting actual sanitation conditions and selection of technologies ▪ Location and use of facilities by sex,age group ▪ Attitudes towards resource reclamation ▪ Attitude toward communal or shared facilities
Institutional framework	o Sharing Responsibilities o Effectiveness of government, regional, or

	municipal authorities in providing water supply, sewerage, sanitation, street cleaning, drainage, health and education services, residential and urban improvements

Note: The priority of survey items will vary depending on sanitary engineering options under consideration. The above list shows representative areas to be surveyed by planners and designers.

It is necessary to ascertain whether a proposed sanitation system is acceptable socially and whether it is compatible with cultural and religious values. For example, according to Islam, it is forbidden to face or turn ones back to Mecca during defecation.

3.3 Selection of Sanitation/Sewerage Technology

The criterion for selection is the Service Level of Water supply**,** ground permeability, and ground water table, area of the housing lot, population density and ground slope. It is considered that population density, service level of water supply, ground permeability and water table are the minimum criteria**.**

There are many types of sanitation systems currently in use. Formulating a sanitation master plan, the first step is to identify whether an on-site or off-site treatment system should be adopted for each specific development or planning area, and secondly to define the type of sanitation technology to be applied to an on-site treatment area, and what type of a sanitation system to be adopted for an off-site treatment area. A pre-condition to the decision making process is the stated goal that the sewerage system is to provide excreta treatment, excreta removal, improved drainage, preservation of receiving water quality and adhere to NEMA effluent stardands.

Figures 3.1 and 3.2 has a schematic procedure to follow.

3.4 Decision I (On-site treatment or off-site treatment)

1) Population Density

All On-site waste disposal options require adequate space within the lot for their installation. Such space is usually available in rural and low density to medium density urban areas. However, as the density of settlement increases, such space is not readily available, and, even when it is available on-site systems are likely to meet with community opposition especially because they need desludging at some stage during their operation. All forms of pipe networks demonstrate marked reductions in unit household costs as the density of settlement increases, because the same length of pipe work serves an increased number of houses. On-site systems, however, maintain a constant unit household cost irrespective of the density of settlement. At a given density of settlement, piped networks become more economical than on-site systems. Unfortunately, in the case of conventional sewerage this transition only takes place at extremely high population densities. Shallow sewers, being much cheaper than conventional sewers, become cost effective at much lower densities. The population density at which this transition takes place varies with the physical conditions of the settlement (such as soil permeability, topography etc.), but in Natal, Brazil, this transition point occurred at a density of 160 persons per hectare (see figure 3.3). In areas with shallow rock, shallow sewers have even proved more cost-effective than on-site systems at population densities as low as 110 persons per hectare.

Based on review of current practice and experience in other developing countries, conventional sewerage is usually cost effective in urban zones where the population density greater than 120 persons per hectare. Arising from above, onsite sanitation should not be used in Kenya where the population density is at least 120 persons per hectare but adopt sewerage.

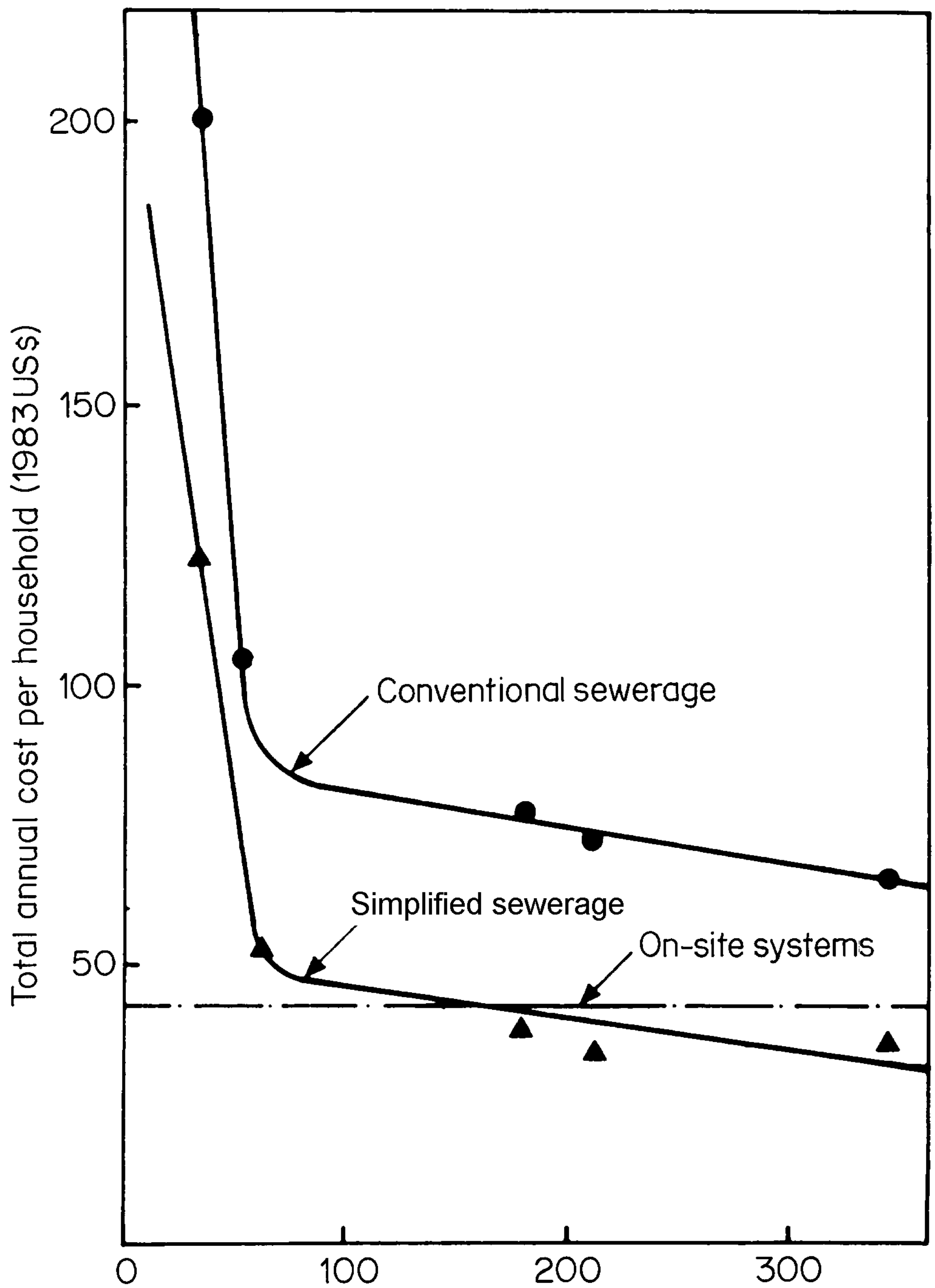


Figure 3.3 Costs of conventional and simplified (condominial in-block) sewerage, and on-site sanitation in Natal in northeast Brazil in 1983. *Source*: Sinnatamby (1983).

2) Service Level of Water Supply

Most low cost, on site waste disposal systems, such as pit latrines and pour flush latrines, only handle the disposal of excreta. Sullage is usually left to infiltrate into the ground at the surface or via some form of on-site sullage soak away. As water consumption increases, the need for such soaks ways increases. In areas of adverse soil conditions and high densities of population, the space for soak ways may not be readily available.

Further, because low income communities do not see the need for investing in soak ways for the disposal of sullage, it is not uncommon to see large quantities of sullage flowing in the streets. Such uncontrolled disposal of sullage can give rise to diseases such as filariasis, as well as erode the usually unsurfaced alleyways found in these settlements. Shallow sewers, because they dispose of both excreta and sullage, can be adopted in settlements where average levels of water consumption are high. A minimum average water consumption of at least 25litres per person per day is required, however, to be available for shallow sewers to operate without blockages.

There are two chief issues to be addressed when considering the service level of water supply namely:-
“How near is a water supply facility located “and
“How much is the per capita consumption?”
The appropriate sanitation system is governed by a combination of these two factors.

Table 3.3 indicates the relationship between per capita consumption and appropriate Sanitation/Sewerage Facilities. With service levels I and II, the supply is hand carried, the minimum volume of water necessary for sustaining a living being carried from wells, tankers, rivers, springs or communal faucets. Therefore, less water using sanitation systems like VIP latrines, etc are desirable. Since the water use of a PF toilet is 6 liters (assuming that one person uses a toilet three times daily with 2 liters per use), it is difficult to use PF toilets effectively with this service level, particularly for large families, except when the housing is located near the water source.

Table 3.3 Relationship between per capita consumption and applicable Sanitation/Sewerage Facilities

	Per Capita Consumption (L)			Applicable Facility	
Service level for water supply(sources)	World Bank	Duncan Mara	Lyonnaise Des Eaux	Excreta	Sullage
(1) Well etc			50		
(2) Communal Faucet	20-25	20-30	10-50	VIP Latrine	Pit Latrine Surface Drain
(3) Yard Tap	50	40-80		PF Latrine+ Pit Latrine PF latrine+Pit Latrine+Sewerage PF latrine + Settling box +Sewerage PF latrine + Septic tank +Soakway PF latrine +Septic tank +Sewerage	 Pit latrine + Sewerage Settling box +Sewerage
(4) Indoor Tap	50-100		40-250	CF Latrine+Septic Tank +Soakway CF latrine+ Sewerage	Settling box +Sewerage Sewerage

3) Water Table

It is recommended that the infiltration capacity of the ground be studied using percolation testing to assess the Feasibility of disposing of sullage by infiltration for service levels III and IV.

In Kenya, effluent from on-site facilities is disposed mainly by infiltration in rural areas, or direct to the street drainage system in urban areas. Potential problems associated with infiltration relate to both pollution of ground water and soil permeability. In this connection, the following guidelines are indicated by WHO and the World Bank.

i) World Health Organisation states

"The pollution extends from the source in the direction of groundwater flow, with only limited vertical and horizontal dispersion. However, this does not apply to pollution in fissured ground where the pollution may flow through the fissures for several hundred metres, often in an unpredictable direction.

In most cases the commonly used figure of a minimum of 15m between a pollution source and a downstream water abstraction point will be satisfactory. Where the abstraction point is not downstream of the pollution, i.e. to the side or upstream, the distance can be reduced provided that the groundwater is not abstracted at such a rate that its direction of flow is turned towards the abstraction point (Fig. 3.4). This is particularly useful in densely populated communities, where shallow groundwater is used as a water supply.

If it is not possible to provide sufficient space between the latrine and the water point, consideration should be given to extracting water from a lower level in the aquifer **(**Fig. 3.4). The predominant flow of groundwater (except fissured flow) is along the strata, with very little vertical movement. Provided the extraction rate is not too great (hand pump or bucket extraction is acceptable), and the well is properly s sealed where it passes through the pollution zone, there should be little or no risk of pollution."

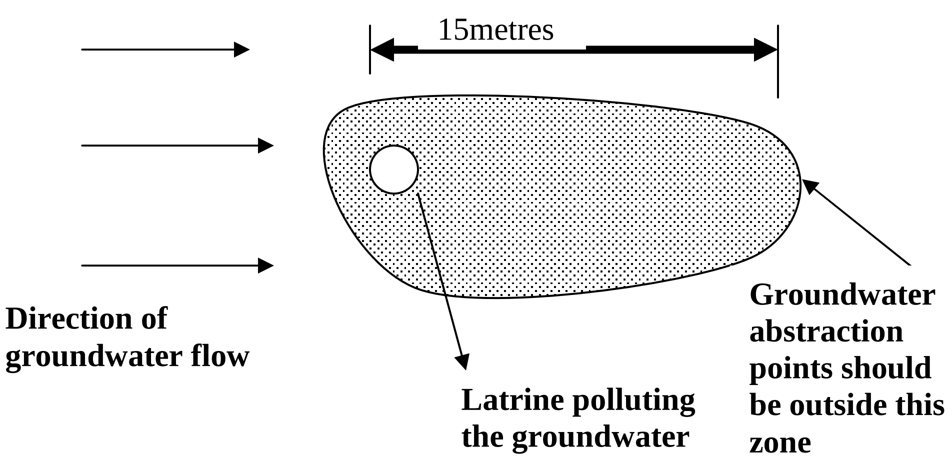


Figure 3.4 Range of water pollution by pit latrine

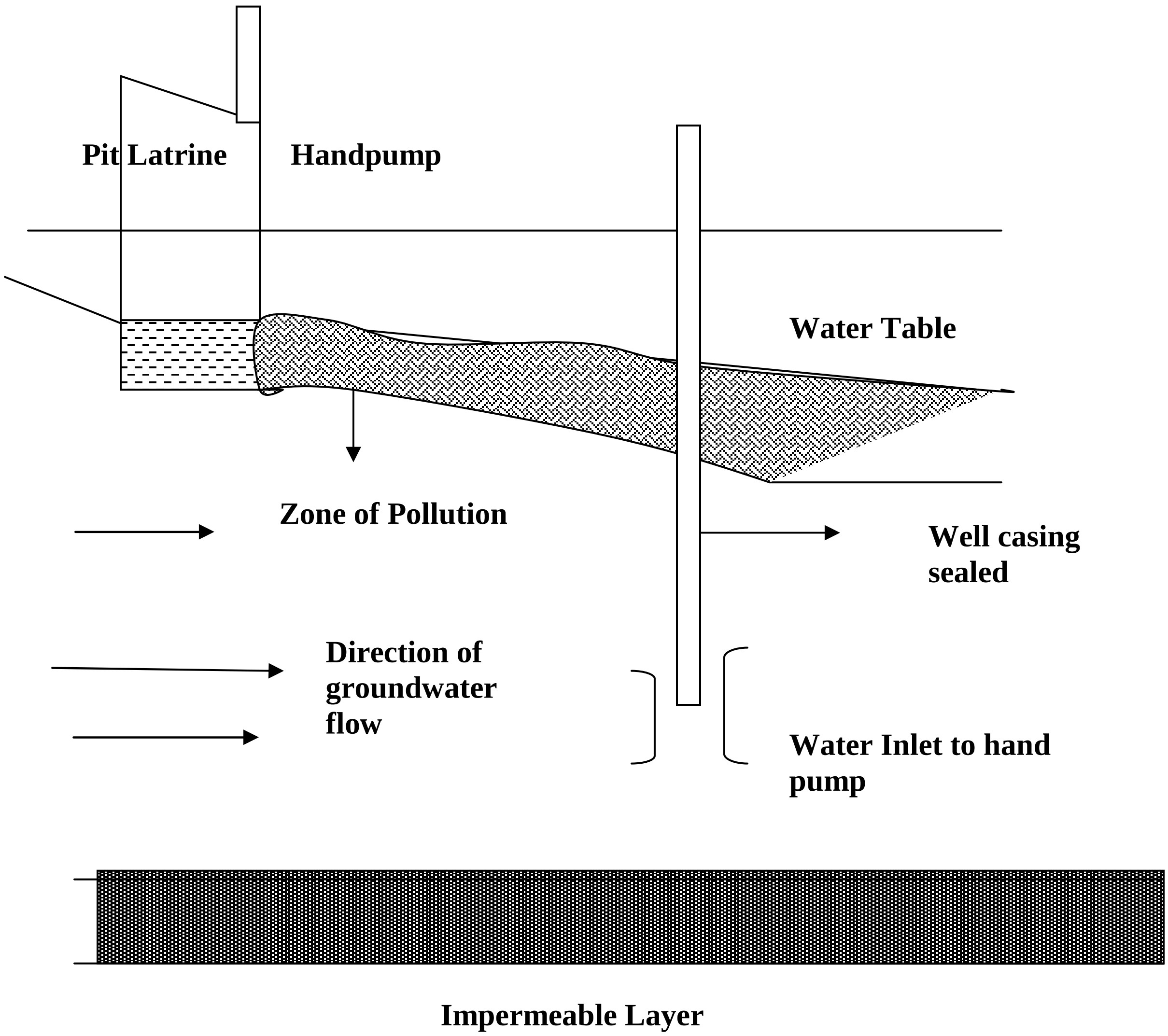


Figure 3.5 Groundwater pollution in hand pump well by pit latrine

"If there is at least 2m between the pit base and the ground water table, little microbial pollutant travel occurs in most unconsolidated soils. A horizontal distance between a well and a latrine of 10m is often satisfactory; if this is not available, specialist hydrogeological advice should be sought. It should be remembered, however, that water supply improvements may be a more appropriate solution."

(ii) World Bank

"If the water table is within 1m of the ground surface, VIP latrines, ROEC's and PF toilets are a doubtful feasibility. However, they may be feasible if the soil is sufficiently permeable such that the water level in a pit is not less than 0.5m below the ground surface.

4) Permeability of Soil

Soil permeability is a key factor in adopting infiltration disposal for either excreta alone or a mixture of excreta and sullage, but reliable permeability data not easy to derive over a wide area since the coefficient is highly variable and can change significantly over a short distance. The infiltration capacities given in table 3.4 (US Environmental

Protection Agency, 1980) are recommended as a possible basis for the sizing of pits and drainage trenches where information about actual infiltration rates is not available.

Table 3.4 Recommended Infiltration Capacities

Type of Soil	Infiltration capacity, settled sewage (L/m^2/day)
Coarse or medium sand	50
Fine sand, loamy sand	33
Sandy loam, loam	25
Porous silty clay and porous silty clay loam	20
Compact silty loam, compact silty clay loam and non expansive clay	10
Expansive clay	<10

Source: US Environmental Protection Agency, 1980

"Since sullage infiltrates at least twice as fast as toilet wastewater, use values for the long term infiltration rate that are twice those given in Table 3.6"

It is possible to reduce the health risk less by handling sullage separately, rather than handling a mixture of partly treated toilet effluent and sullage.Kalbermattenh et al. recommended the provision of a three compartment septic tank, which reportedly doubles the infiltration rate.

"Soil stability is another important factor where the use of VIP latrines, ROEC's and PF toilets is being considered. In unstable soil, pits must be lined and the base left open (this effectively reduces the infiltration area to the base since the walls are closed.) Good soil permeability is also important for these technologies, as well as for septic tank soak ways trenches. In impermeable soils such technologies are infeasible.

5) Per capita consumption, water table and soil permeability

For the example of a septic tank as a typical on-site treatment facility, the relationship between per capita consumption, water table and soil permeability are discussed below.

The septic tank effluent and sullage can be disposed of in essentially three ways as shown below.

For infiltration disposal either soak ways or drain field can be considered, but, the latter require a large area and as a result in urban areas, infiltration is usually by means of soak ways.

Case I	Combined treatment of excreta and sullage by a septic tank	Infiltration disposal by a soak way
Case II	Individual excreta treatment by a septic tank	Infiltration disposal by a soak way
	Sullage	Discharge to a surface drainage system
Case III	Septic tank effluent and sullage	Discharge to a surface drainage system

1) Case 1

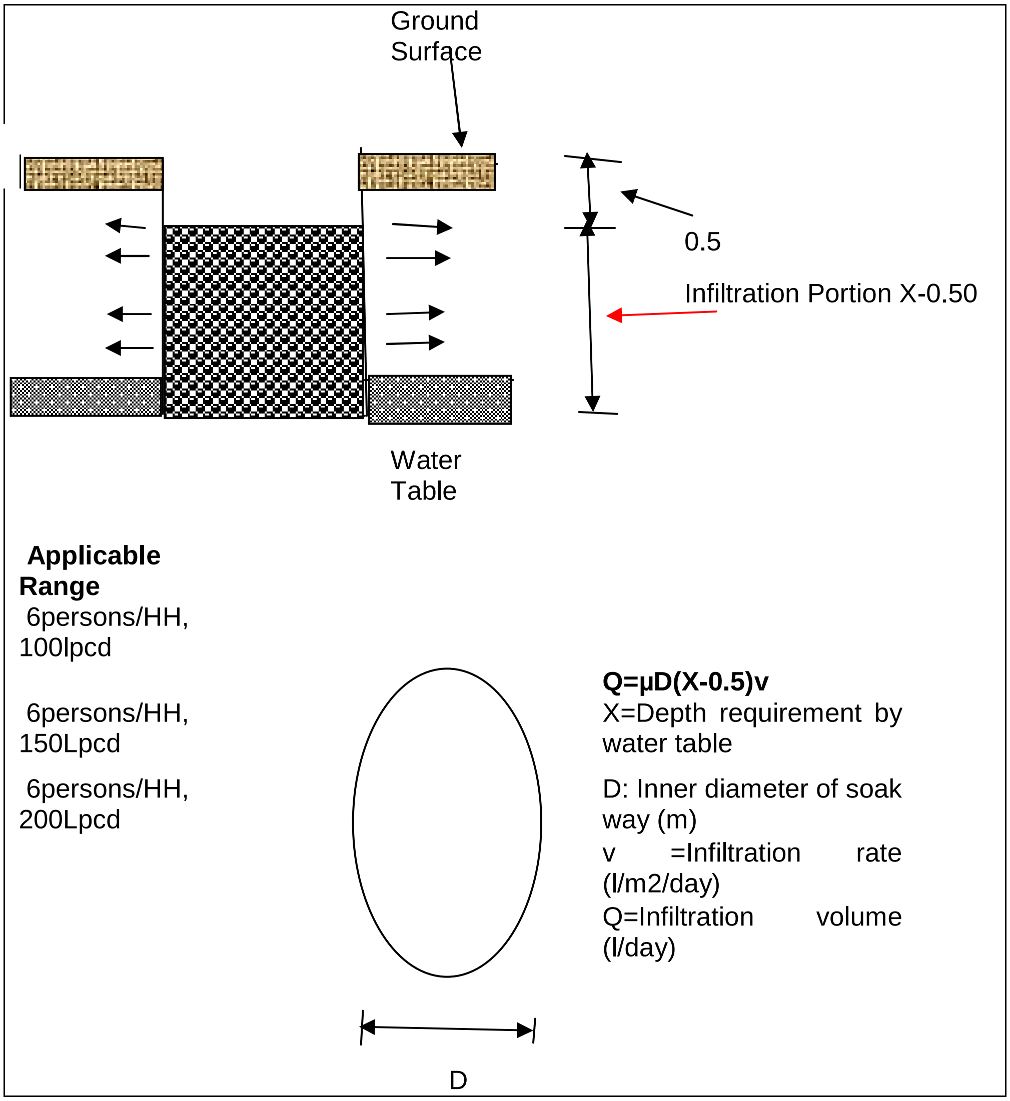
The typical dimensions of a soak way are 1.0 to 2.5m diameter and 2 to 5m deep. Table 3.6 and 2.8 show the possible infiltration volume for internal soak way diameters of 1.0, 1.5, 2.0 and 2 m

Table 3.5 Possible treatment capacity of soak way for mixture of excreta and sullage (l/day)

			Depth from Ground Surface to Water Table (m), X (m)										
Type of Soil	**Infiltration Rate v l/m2/day**	**Inn Dim D (m)**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	4.0	4.5	5	5.5	6
Coarse Sand	50	1	79	157	236	314	393	471	550	628	707	785	864
	50	1.5	118	236	353	471	589	707	825	942	1,060	1,178	1,296
	50	2	157	314	471	628	785	942	1,100	1,252	1,414	1,572	1,928
Medium Sand	50	2.5	196	393	589	785	982	1,478	1,374	1,321	1,767	1,964	2,160
Fine Sand	33	1	52	104	156	207	259	311	363	415	467	518	570
	33	1.5	78	156	233	311	389	467	544	622	700	778	855
	33	2	104	207	311	415	518	622	726	829	933	1,037	1,140
Loamy sand	33	2.5	130	259	389	518	648	778	907	1,037	1,166	1,296	1,426
Sandy loam	25	1	39	79	118	157	196	236	275	314	353	393	432
	25	1.5	59	118	177	236	295	353	412	471	530	589	648
	25	2	79	157	236	314	393	471	550	628	707	785	864
Loam	25	2.5	98	196	295	393	491	589	687	785	884	982	1,080
Porous Silty Clay	20	1	31	63	94	126	157	188	220	251	283	314	346
	20	1.5	47	94	141	188	236	283	330	377	424	471	518
Porous Silty Clay loam	20	2	63	126	188	251	314	377	440	503	565	628	691
	20	2.5	79	157	236	314	393	471	550	628	707	785	864
Compact silty loam	10	1	16	31	47	63	79	94	110	126	141	157	173
Compacted silty clay loam	10	1.5	24	47	71	94	118	141	165	188	212	236	259
	10	2	31	63	94	126	157	188	220	251	283	314	346
Non expanded Clay	10	2.5	39	79	118	157	196	236	275	314	353	393	432

Table 3.6 Possible treatment capacity of soak way for sullage only (l/day)

			Depth from Ground Surface to Water Table (m), X (m)										
Type of Soil	**Infiltration Rate v $l/m^2/day$**	**Inn Dim D (m)**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	4.0	4.5	5	5.5	6
Coarse Sand	100	1	158	314	472	628	786	942	1,100	1,256	1,414	1,570	1,728
Medium Sand	100	1.5	236	472	706	942	1,178	1,414	1,650	1,884	2,120	2,356	2,592
	100	2	314	628	942	1,256	1,570	1,884	2,200	2,504	2,828	3,144	3,856
	100	2.5	392	786	1,178	1,570	1,964	2,956	2,748	2,642	3,534	3,928	4,320
Fine Sand	66	1	104	208	312	414	518	622	726	830	934	1,036	1,140
	66	1.5	156	312	466	622	778	934	1,088	1,244	1,400	1,556	1,710
	66	2	208	414	622	830	1,036	1,244	1,452	1,658	1,866	2,074	2,280
Loamy sand	66	2.5	260	518	778	1,036	1,296	1,556	1,814	2,074	2,332	2,592	2,852
Sandy loam	50	1	78	158	236	314	392	472	550	628	706	786	864
	50	1.5	118	236	354	472	590	706	824	942	1,060	1,178	1,296
	50	2	158	314	472	628	786	942	1,100	1,256	1,414	1,570	1,728
Loam	50	2.5	196	392	590	786	982	1,178	1,374	1,570	1,768	1,964	2,160
Porous Silty Clay	40	1	62	126	188	252	314	376	440	502	566	628	692
Porous Silty Clay loam	40	1.5	94	188	282	376	472	566	660	754	848	942	1,036
	40	2	126	252	376	502	628	754	880	1,006	1,130	1,256	1,382
	40	2.5	158	314	472	628	786	942	1,100	1,256	1,414	1,570	1,728
Compact silty loam	20	1	32	62	94	126	158	188	220	252	282	314	346
Compacted silty clay loam	20	1.5	48	94	142	188	236	282	330	376	424	472	518
Non expanded Clay	20	2	62	126	188	252	314	376	440	502	566	628	692
	20	2.5	78	158	236	314	392	472	550	628	706	786	864



With an internal diameter of 2.5m and depth between the ground surface and water table of 3.0m, the possible infiltration volume of a soak way is 982L/day for porous silty clay and porous silty clay loam. If the family comprises six persons and the water supply return factor (90%, the sustainable per capita water consumption are 205, 135,102 and 82L/capita per day respectively for the different soil types in order to avoid overflowing the soak way. However, these rates will increase by 40% if the water table drops by 1m, and decrease by 40% if the water table rises by 1m. Filtration becomes very difficult in compacted silty loam, compacted silty clay loam, non expanded clay and expanded clay, and under such soil conditions the adoption of sewerage must be considered.

If the dimension (internal diameter) of a soak way used in the study area is within the range noted above, approximate sustainable infiltration volumes can be derived from tables 3.5 and 3.6 for the combination of soil prevailing in the planning area. If the projected infiltration volume calculated from the sustainable infiltration volume and the assumed water supply return factor is less than the design water use rate, infiltration has to be ruled out and sewerage must be considered.

It should be noted that septic tanks are not restricted to individual houses; they can also be used for apartment blocks and small communities. In this case, the number of houses and occupancy rates are key data to determine if infiltration is still a feasible alternative.

2) Case II

The following two options can be considered depending on the kind of toilet being used.

- PF toilet + Septic Tank + Soak way
- CF toilet + Septic tank +soak way

Assuming that one person uses 2litre water per use three times a day, for the combination of PF toilet + septic tank +soak way the per capita consumption is 6 L/day equivalent to a household consumption 36L/day for occupancy of 6 persons. In this case from table 3.6, if the water table is at least 1.5m below the ground surface, a soakway with an internal diameter of 1.5m can be used if the soil is coarse sand, medium sand, fine sand, loamy sand, sandy loam, loam, porous silty clay, or porous silty loam. For compacted silty loam, compacted silty clay, non expanded clay, and expanded clay, infiltration would be difficult and discharge to the surface drainage system has to be considered instead.

Assuming a CF toilet uses 11 to 15 liters of water per flush in a low tank model, with the same assumptions as those used for the PF toilet option; the maximum daily per capita water use is as high as 198liters. In this case infiltration will only be effective with soak way diameter of 1.0m, a depth between the ground surface and water table of 3m, and sandy loam or loam soil. For soils of lower permeability, sewerage should be adopted.

3) Case III

If a surface drainage system is provided, it can be used to receive discharges of septic tank effluent and sullage. However, the septic tank effluent is rarely chlorinated since there is generally no legal requirement to do so, the effluent may contain pathogens that impose a health risk should persons come into contact with the effluent, either directly or through reuse for crop irrigation. From the point of view of minimizing the health risk, it is safer to dispose of both septic effluent and sullage by infiltration.

3.5 Decision II (Selection of Off-site treatment Technology)

Conventional urban sewerage schemes are based on gravity sewers generally constructed under public roads, as a result depths of excavation tend to be considerable at the downstream sections of the network, resulting in increased costs to deal ground water. In heavily trafficked areas where construction might be restricted to certain hours, the construction period tends to be long and the or micro tunneling has to be adopted, again requiring specialized contractors and high costs. According to the World Bank (1992), the per household cost can be as high as US $3,506 for the sewerage system with treatment, compared to US $200 for the water supply system.

1) Investment Needs

Below is the unit cost of various technologies practiced in Kenya which can be used for Master Planning:-

Part (a) and (b) gives the capital cost for each option.

(a) Rural Sanitation

Characteristics of Typical Rural Technologies	Unit	1 I: Lined Double Pit VIP	2 I: Ulined VIP	3 III: Ecosan	4 IV: Abor-loo GI	5 V: Abor-loo Local	6 Septic Tank with Soak Away	7 4-Stance Public Latrine	8 6-Stance School Latrine
Typical pop. Served by System	People	5	5	5	5	5	5	1,000	600
Design life	Years	20	20	20	10	10	20	20	20

The typical technologies selected for rural sanitation include:

1. Lined double pit VIP latrine. The latrine is constructed with 2m deep pits (0.9m x 1.2m) lined with concrete blocks and a superstructure (1.4m x 1.2m) build in concrete block with timber door, corrugated iron roof and ventilation pipe.
2. Un-lined VIP latrine with a 5 m deep (1m x 1m) pit and a concrete slab and concrete block superstructure (1m x 1m) with timber door, corrugated iron roof and ventilation pipe.
3. Ecosan latrine build with concrete blocks with squatting pans, urine separation and evaporation increased by solar heating. The latrine has dimensions of 1m x 1m and a 1m high sub-structure. Sub- and super-structure build with concrete blocks with corrugated iron roofing.
4. Abor-loo with shallow pit and superstructure constructed with corrugated iron sheets
5. Abor-loo similar to 4 but with the superstructure constructed with local available materials.
6. Septic tank with soak away. The septic tank is sized for a household with dimensions 3m x 1.5m x 1.8m deep and with a 1.5m x 1.5m x 2m deep soak away. The tank is constructed in concrete and concrete blocks. 50m of piping to the septic tank and 20 m from the septic tank to the soak away are included in the cost estimate.
7. The design of the public latrine is a pit latrine with 4 stances and 5 pits to allow alternate use of pits to facilitate emptying. The pits are 1.2m x 0.9m and 2m deep and rooms in the superstructure are 1.4m x 1.2m. The latrine is constructed in concrete and concrete blocks.
8. The design of the school latrine is a pit latrine similar to the public latrine but with 6 stances and 7 pits to allow alternate use of pits to facilitate emptying. The pits are 1.2m x 0.9m and 2m deep and rooms in the superstructure are 1.4m x 1.2m. The latrine is constructed in concrete and concrete blocks.

Typical Investment cost of system (kshs)

Rural Technologies		1	2	3	4	5	6	7	8
WSB Area:	Unit	I: Lined Double Pit VIP	II: Unlined VIP	III: Ecosan	IV: Abor-loo GI	V: Abor-loo Local	Septic Tank with Soak Away	4-stance Public Latrine	6-Stance School Latrine
Tana WSB	One structure	58,000	30,000	157,000	21,000	4,000	138,000	181,000	260,000
Athi WSB	One structure	55,000	28,000	148,000	20,000	4,000	130,000	172,000	246,000
Coast WSB	One structure	57,000	29,000	152,000	20,000	4,000	134,000	176,000	252,000
Lake Victoria South WSB	One structure	55,000	28,000	147,000	20,000	4,000	130,000	171,000	245,000
Lake Victoria North WSB	One structure	57,000	29,000	152,000	20,000	4,000	134,000	176,000	252,000
Ewaso Ngiro WSB	One structure	62,000	32,000	169,000	23,000	4,000	148,000	193,000	278,000
Rift Valley WSB	One structure	59,000	30,000	158,000	21,000	4,000	138,000	182,000	261,000

Community Implementation

Rural Technologies		1	2	3	4	5	6	7	8
WSB Area:	Unit	I: Lined Double Pit VIP	II: Unlined VIP	III: Ecosan	IV: Abor-loo GI	V: Abor-loo Local	Septic Tank with Soak Away	4-stance Public Latrine	6-Stance School Latrine
Tana WSB	One structure	35,149	19,240	111,035	15,019	1,372	89,909	112,311	165,367
Athi WSB	One structure	32,900	17,782	104,264	14,259	1,323	83,945	105,485	154,858
Coast WSB	One structure	34,298	18,499	107,271	14,279	1,345	86,878	108,511	159,376
Lake Victoria South WSB	One structure	32,878	17,773	103,539	14,257	1,321	83,906	104,808	154,146
Lake Victoria North WSB	One structure	34,298	18,499	107,271	14,279	1,345	86,878	108,511	159,376
Ewaso Ngiro WSB	One structure	38,125	20,750	120,038	16,507	1,431	97,388	121,337	178,865
Rift Valley WSB	One structure	35,779	19,250	111,764	15,022	1,375	89,951	113,000	166,092

The costs shown in the tables above for on-site sanitation in rural areas are the implementation costs. The costs of sanitation projects can in most cases not be compared to these costs since the sanitation programmes often only promotes household sanitation and in some cases subsidies the construction of latrines with only a concrete slab. Investment planning for sanitation must be based on a strategy that determines how the sanitation programmes are carried out, presently in Kenya, household latrines are not subsidized.

(b) Urban Sanitation

Characteristics of Typical Urban Sanitation Technologies	Unit	1 I: Lined Double Pit VIP	3 III: Ecosan	6 Septic Tank with Soak Away	7 4-stance Public Latrine	8 Sewerage System
Typical population served by system	People	5	5	5	1,000	100,000
Design life	Years	20	20	20	20	30

The typical designs for on-site urban sanitation are similar structures as for the rural technologies. In addition the cost estimate has been prepared for a sewerage system for 100,000 people. The components of the sewerage system include the conveyance system and the sewerage treatment system based on a design similar to the Eldoret system with trickling filters and primary, secondary and tertiary settling ponds.

Typical Investment cost of system (kshs)

Urban Technologies		1	3	6	7	8
WSB Area:	Unit	I: Lined Double Pit VIP	III: Ecosan	Septic Tank with Soak Away	4-stance Public Latrine	Sewerage System
Tana WSB	One structure	58,000	157,000	138,000	181,000	652,873,547
Athi WSB	One structure	55,000	148,000	130,000	172,000	613,006,235
Coast WSB	One structure	57,000	152,000	134,000	176,000	630,448,184
Lake Victoria South WSB	One structure	55,000	147,000	130,000	171,000	611,068,240
Lake Victoria North WSB	One structure	57,000	152,000	134,000	176,000	630,448,184
Ewaso Ngiro WSB	One structure	62,000	169,000	148,000	193,000	703,815,113
Rift Valley WSB	One structure	59,000	158,000	138,000	182,000	655,088,398

Typical Investment Unit Costs (Kshs/capita)

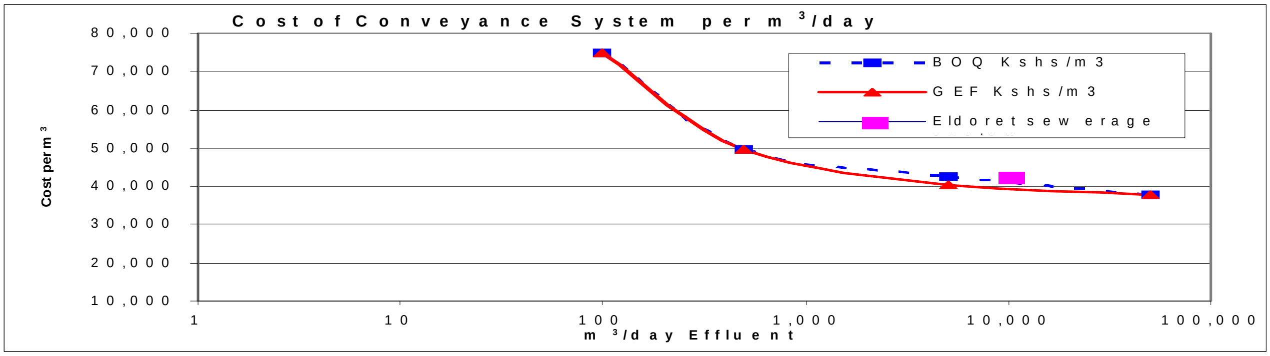
Urban Technologies		1	3	6	7	8
WSB Area:	Unit	I: Lined Double Pit VIP	III: Ecosan	Septic Tank with Soak Away	4-stance Public Latrine	Sewerage System
Tana WSB	Kshs/ capita	11,600	31,400	27,600	95	6,529
Athi WSB	Kshs/ capita	11,000	29,600	26,000	91	6,130
Coast WSB	Kshs/ capita	11,400	30,400	26,800	93	6,304
Lake Victoria South WSB	Kshs/ capita	11,000	29,400	26,000	90	6,111
Lake Victoria North WSB	Kshs/ capita	11,400	30,400	26,800	93	6,304
Ewaso Ngiro WSB	Kshs/ capita	12,400	33,800	29,600	102	7,038
Rift Valley WSB	Kshs/ capita	11,800	31,600	27,600	96	6,551

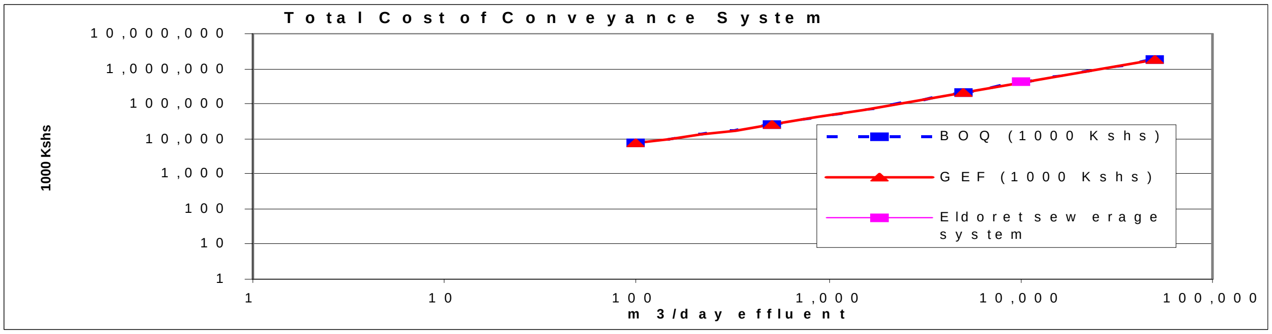
The investment costs per capita for sanitation can not be directly compared. For example the cost of ecosan and septic tanks appear to be at a similar level, however the costs for septic tank and soak away does not include the households investment in toilet facilities, and these are likely to be of a similar magnitude as the cost of the septic tank. Comparison can also not be made directly between the per capita cost of sewerage systems and the on-site sanitation facilities as the sewerage systems would normally need to be accompanied with a private investment by the households in toilet and bathroom facilities of a magnitude much larger than the investment in Ventilated Improved Pit (VIP) Latrines.

C) Cost per Component

Below is the Cost per component:-

Conveyance System

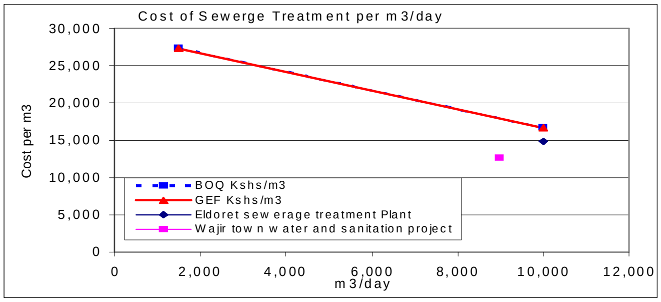


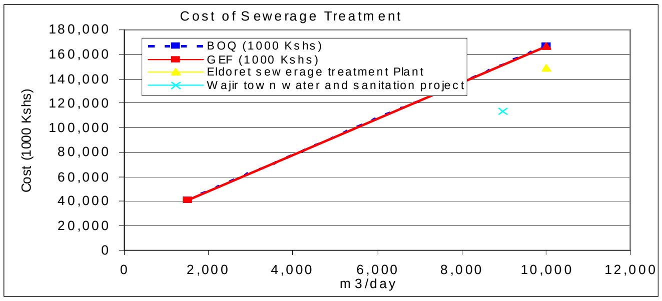


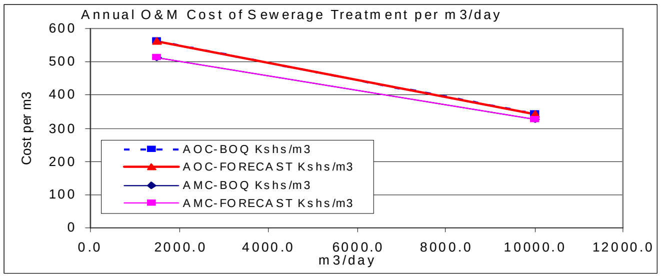
Sewerage Treatment

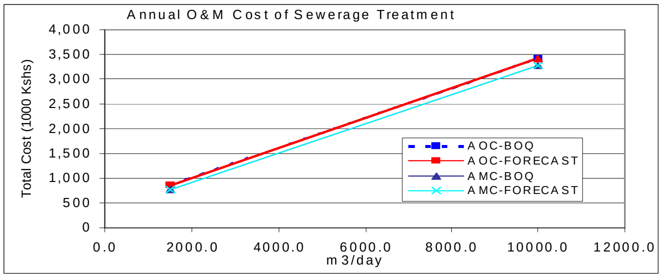
Cost estimate for sewerage treatment was prepared for a system for 10,000 m³/day of effluent based on the design for the Eldoret sewerage treatment plant. The design is based on trickling filters and primary, secondary and tertiary settling ponds.

The cost estimate for a 1500 m³/day plant was been based on the design for Solai Menengai Sewerage Works. This plant is based on waste stabilization ponds only.





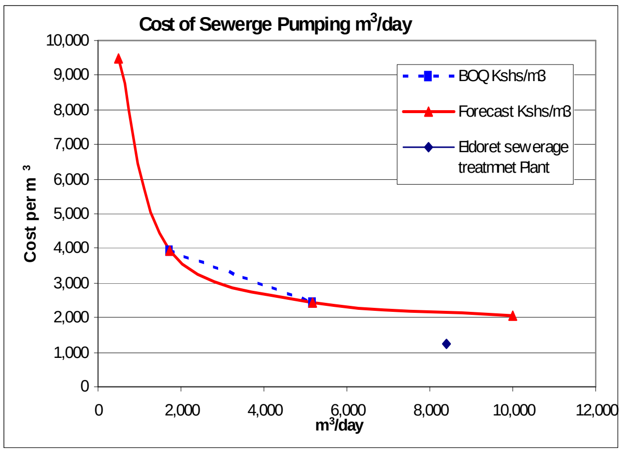


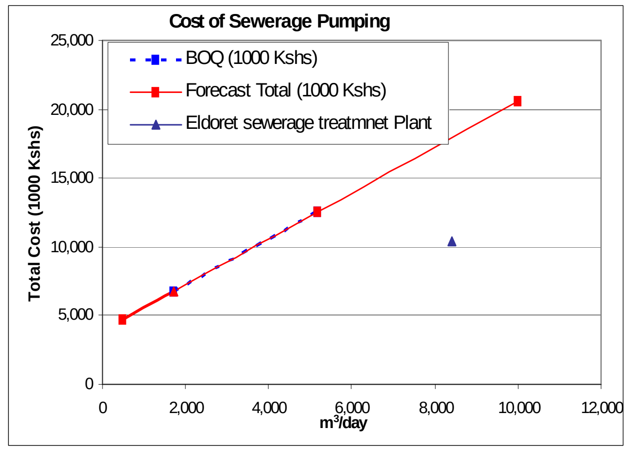


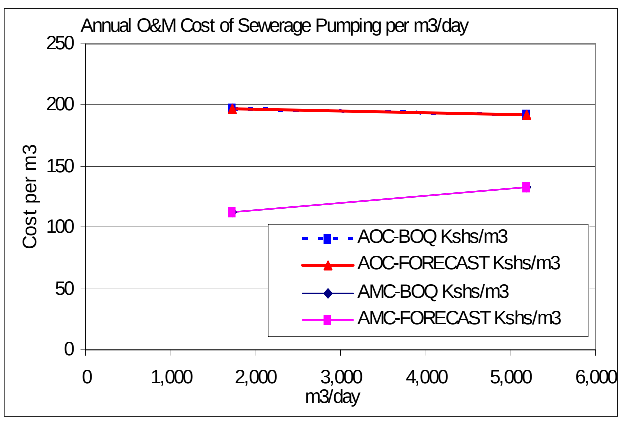
Sewerage Pumping

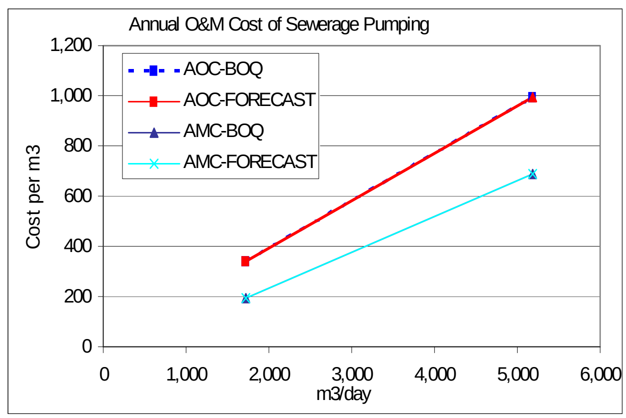
The cost estimates for sewerage pumping stations was prepared on the following assumptions:

- Pipes and fittings cost 50% of pump cost
- Electrical installation, control boards, starters etc. cost 30% of pump cost
- Power source estimated at 30% of pump cost
- 24 hour operation
- 100 % Standby pumping capacity







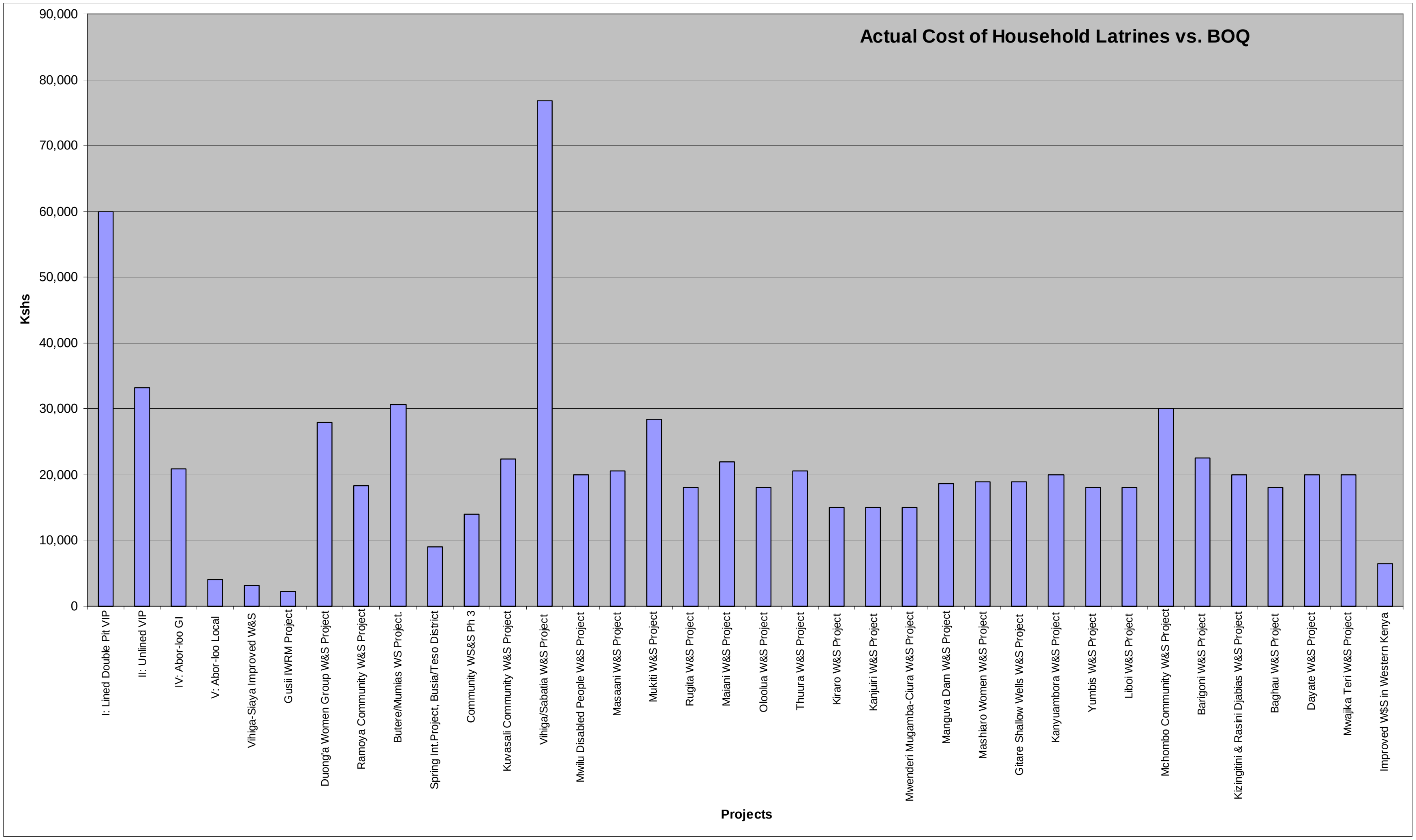


On-Site Sanitation

The cost estimates were prepared for:

- VIP latrines with lined double pits
- Unlined VIP Latrine
- Ecosan latrine with separation of faeces and urine
- Abor-loo latrines where the superstructure is made from corrugated iron roofing sheets and
- Abor-loo latrines where the superstructure is made from locally available materials

The cost estimates include all the costs of building the latrines etc



Source: - **Assessing Unit Costs for Water Supply and Sanitation Services in Kenya, Final Report by Water and Sanitation Program-Africa, World Bank, Nairobi Kenya dated December, 2005**

2) Community Participation

Securing community agreement can take a long but once the community is educated on the importance of hygene and sanitation and an agreement is reached the project can proceed.

3) Effective Use of existing Sanitation Facilities

The adoption of either option assumes that sanitation facilities are functioning properly.

4) Measures for combined drainage of sanitary wastewater and storm water

The storm water which enters the sewerage system should be kept to a minimum, and very carefully controlled; otherwise, the sewers will either be uneconomically large or will flood during wet weather.

Storm water entry should be restricted to the run-off from:-

- Open, paved, public markets;
- The open, paved, yards of slaughterhouses
- The yards of milk collection and/or processing establishments;
- Any similar open, paved yards which, because of organic dirt, should for reasons of public health are washed down daily.

3.6 Decision III (Selection of On-site treatment Facility)

Emphasis has been done on VIP latrines, PF toilets and Septic Tanks since these are the most widely used systems in developing countries, although there are other sanitation facilities that are equally effective.

Of the three systems, the septic tank incorporates a treatment function and is inherently more flexible than the other two systems in that it can also be designed to receive sullage as well as toilet wastes. The system is useful for new development and can also serve housing clusters prior to the later provision of sewerage.

1) Collection (desludging), treatment and disposal of septage from septic tanks

The key factor in septic tank management is the proper collection (desludging), treatment and disposal of septage generated in septic tanks. Unfortunately in many developing countries regular desludging of septic tanks is rarely carried out effectively.

Service Level of Water Supply

If the water supply is based on hand carrying from wells, e.t.c., it is desirable to use VIP latrines or similar systems since only the minimum amount of water is used. If the supply is upgraded to yard taps, then PF toilets are possible, although in very general terms if the capita daily water supply is less than 50 liters, PF toilets will probably not be effective. Where the water is supplied through multiple indoor taps, cistern flushes (CF) toilets can be used. It should be noted that the a aforementioned are only general guidelines; there are many instances where CF toilets function even with a water supply based on communal standpipes if the standpipe is close to the property. Each planning area has to be evaluated separately; in some cases it may be necessary to restrict the upgrading of water supply beyond a single tap in order to avoid problems with the sanitation system. Since health levels are generally satisfactory with a daily water use above 50L/capita, everything else being equal, the justification for greater water provision is not driven by health needs but rather by convenience. Decisions such as these are not easy to make and recourse must be made to local sociologists who can advise on the particular preference of the community.

An important point is the need to check the infiltration capacity of soil when sullage is disposed by infiltration, particularly in those situations where the water supply is by yard taps or house connections.

Possibility of continuing groundwater pollution

In developing countries, shallow wells have been used for drinking water sources for many years and even after a piped supply is provided, the wells may continue to be used, although maybe no longer for drinking purposes, since the water is free.

In areas where groundwater pollution is already present giving rise to public health problems within the community as a result of contaminated well supplies, it is preferable to adopt a septic tank rather than a PF toilet with the tank effluent discharged to the drainage system rather than to the ground. If necessary, the degree of treatment can be upgraded with the provision of an anaerobic up flow filter.

Income Level of Residents

Attention should be paid to the income level of the target population, since all the installation cost is borne by them. According to the World Bank, PF toilets represent the lowest cost in terms of both construction and operation and maintenance, with septic tanks being the most expensive. In economic terms the household cost of a standard septic tank is eight times that of a PF toilet, although this is a generalized average and may not be applicable in all situations depending on the local cost of materials and the degree of community participation in the construction. Surveys need to be carried out to determine household incomes but great skill is required in interpreting correctly between a stated willingness to pay for the upgraded system and the actual ability to pay since most people will readily say that they are willing to pay on the assumption that at the end of the day subsidies will be provided to help defray the cost. Using average population incomes or the common assumption that a system is affordable if the resulting monthly charge costs less than 5% of the household income may give totally misleading results. The incomes of the bottom 20% of the population in many locations can be less than 20% of the average income because of the skewed income distribution. Again, time spent at the commencement of the study in developing a reliable and locally appropriate database is the only way to ensure that the recommendation will result in a satisfactory end product.

Survey of natural conditions

Groundwater levels and permeability coefficients are critical data in a survey of the use of existing sanitation. High groundwater levels may make even underground seepage impossible and cause the areas around sanitation facilities to become wet and therefore breeding grounds for mosquitoes and flies. If the groundwater level is low, a low permeability coefficient may hinder seepage, which is likely to cause the interior of the pit to be wet. Knowing the groundwater level would be very useful if geological and groundwater level data are well compiled during the boring of wells. The permeability coefficient, which is one of the most difficult pieces of data to obtain, is often determined from geological information. It is definitely helpful to have geological data when boring wells.

Layout of property lines, houses and utilities in the block

Condominial sewerage employs a customized service approach (customer oriented approach). Namely, the community residents of residential blocks are allowed to participate in the planning. After they thoroughly understand the merits and demerits of condominial sewerage, including the maintenance responsibility, rate system, etc., the residents are provided with technical options, so that they can discuss them and select an appropriate option. The construction work is done on the basis of the technical option thus determined, and residents will sign an agreement to the effect that they will bear the expenses, pay the rates, and assume responsibility for maintenance. Technical options will be prepared by the project team of the implementing agency for presentation to the residents. For this purpose, it is necessary to survey beforehand the property lines, houses, obstacles, and utilities within each block. The project team must obtain admission into the site during meetings with the committee.

3.6.1 Rebuilding or remodeling of houses in the community

Due attention to residents' future plans during project planning will ensure better arrangements of the network. This is most important when the house connections are to be laid under private land, particularly, in resident's backyards. Residents are often contemplating home improvements such as adding new rooms, expanding the size of the existing house, and if financially possible, rebuilding.

However, such rebuilding will often be done over the area where the network is laid out, or home expansion may have to be over the housing connections because of the small plot. The network is not designed to carry a heavy load, which means that this kind of home expansion or improvement may damage, break, or completely block the sewer connection. These problems become more serious when the network passes through the middle of a backyard instead of near the back boundary or beside the property line where the residents would not expand the house. To minimize this type of future problem, it is important to ask residents beforehand about their future plans for their homes, and to reflect such plans in the network design.

4.0 SOCIOECONOMIC CONDITIONS OF THE COMMUNITY

Community participation is the underlying assumption for a condominial sewerage system. Such a system will only succeed when the cooperation of the community is gained. This means that the socioeconomic conditions for each block in the community have to be grasped.

4.1 Community Layout

Basically, the condominial sewerage system is planned for blocks that are surrounded by roads. Where a block is large or if the terrain is undulating, a subdivision of a block may be more favorable for planning. Negotiations should proceed with the residents of each subdivision (each condominium) of a block.

4.2 Income brackets (Rich and Poor)

As background information for negotiations with each community, it is important to know what income brackets residents are in. A community of wealthy people may attach importance to privacy and prefer a conventional sewerage system over a condominial system.

4.3 Community Organizations, activities, and concerns

The activities of existing residents' associations will indicate how well organized the community is. This is an important aspect of negotiations on condominial sewerage. Also learn who plays a leadership role in the community -for example, an association chairman or elders. Their cooperation is indispensable to the formation of a community wide opinion in the final selection of technical options. Certain communities may not place a priority on sewerage development, in which case development should not be thrust upon them. It is not uncommon that such communities may change their minds when they see the successful achievement of others.

4.4 NGO activities (that might support expansion of sewered areas)

Gain the cooperation of any NGO group in the area and win its understanding of the significance of condominial sewerage. It can help persuade residents that such a system would contribute greatly to the improvement of the regional living environment, the overall objective of NGO activities. This approach is extremely effective because members of these groups are engaged in practical daily activities in the community, and the resident trusts them.

5.0 SANITATION

5.1 Planning

Many methods are used to provide or improve on-site sanitation. At one extreme, a project may involve detailed documentation using the "project cycle"Approach. At the other extreme, sanitation advances when individual householders build their own improved latrines, often because they have seen similar latrines built byneighbors. Many projects and programmes for improving sanitation lie between these extremes. Planning involves consideration of the local situation leading to selection of suitable types of sanitation in line with planning techniques outlined in chapter 3. Designs are prepared and construction follows. On completion, and sometimes at intermediate stages, evaluation takes place.

With some projects, the form of planning and development is laid down by procedures which must be rigidly followed if external funds are to be released. However, in many successful programmes, development depends on the action of householders. Planning then leads to selection of appropriate forms of sanitation. Householders may be encouraged to adopt the selected types of sanitation by health education programmes, by technical or material support, or by other measures. The ways in which the different stages of planning and development may be regarded at various levels are shown in
Table 5.1 below

- Household latrines may be called for because of the convenience they offer to users.

- Good sanitation may be a status symbol.

- Existing excrete disposal methods may result in unacceptable pollution of surface water, soil or groundwater.

- Sometimes a demand for improved sanitation is associated with water supply. For example, a funding agency may require latrines to be constructed before it will provide piped water, or a water authority may wish to protect the catchment area for the supply to a nearby town by eliminating indiscriminate defecation. An increase in the amount of water provided to an area may lead to a demand for better wastewater disposal.

Table 5.1 The project cycle

Government ministries and donor agencies	Implementing agency	Community
Identification		
Definition of target population		Felt need for improved sanitation
1)Determination of economic and health indicators, present service coverage and standards, objectives and policies, financial implications, staffing requirements, and training needs 2) Assignment of planning responsibilities		Exposure to health education
Pre-feasibility surveys		
Consideration of alternative projects to meet objectives taking into account technical social health, environmental financial and economic criteria	Technical and social surveys Planning with the community	Response to questions by health workers and government officials about health wealth water and sanitation
Feasibility demonstration		
Detailed design and analysis of preferred/chosen project price to satisfaction of representatives of proposed target group	Proving of recommended range of technologies at affordable	Discussion regarding experimentation with affordable means of improving sanitation
Appraisal and approval		
Independent check on planning, usually by representatives of funding source Investment decision Release of funds for project implementation		
Implementation		
Consolidation Training, administrative	support procedures, proving technology Determination of financial, material and technical support	Training of local people to assist with programme Invitation to local artisans and contractors to participate
Expansion	Mass promotion in the Community Health education, use of media Demonstration units as" sanitation super- market" Financial, material and technical assistance where appropriate	Publicity about the Programme Systems available to copy Drawings made available Local artisans and contractors available to help with building Household decision as to purchase of sanitation system
Operation and maintenance	Advice on responsibility of household to use and care for on-site system	Use of facilities
Evaluation		

Identification of further projects	Identification of positive and negative aspects; reformulation of design criteria	Comments regarding desired improvements

5.2 The demand for sanitation

The initial demand for provision or improvement of sanitation in a particular area may come from the local people themselves or from a small group of active leaders in the community. Alternatively, the initiative may come from health officials, a government department, the organization responsible for water and sanitation, a bilateral aid agency, or a national or international voluntary organization. Ideally, sanitation improvements should be carried out in accordance with a national or regional sector plan and the adopted primary health care programme. A sector plan often covers both sanitation and water supply. It indicates the number of facilities to be provided, the number of people to be served in each district on a year-by-year basis during the planning period, and the Resources needed. Particular attention is usually given to requirements for internal and external funding and to deficiencies in personnel of various categories.

5.3 Existing Sanitation System in the World

Various Sanitation Systems are in use in the World.

The pit is the prototype whose development proceeds differently depending on whether or not water is used. Those that don't use water have a ventilation pipe to prevent odors and generation of flies. The pit will eventually become full even though night soil stored inside the pit may steadily decrease in volume due to leaching of the liquid portion and progressive settlement, thickening, and anaerobic digestion of solids. The ventilated improved double pit type latrine is a solution for this problem, in which two pits are provided beforehand for alternate use. When one is full, the other is used. This enables permanent use of one or the other. The full pit can be covered with soil for complete sealing and left unused for two years. In this way, pathogens can be completely annihilated, and residue can be taken out in a sanitary condition.

In the same category, but without water use, are the pit latrine or double-pit composting latrine intended for reuse of night soil.

The typical category with water use is the pour flush latrine (PF latrine), made possible thanks to development of a water seal pan that prevents odor and generation of flies. The use of water enables flushing of excreta, which makes it unnecessary to provide a pit directly under the pan. As a result, the latrine may be separated from the pit. The separation of pits, which must be emptied when full, enables placement of toilets inside buildings. This is very significant. It may be a natural trend that the offset PF latrine has two pits similar to the case of the ventilated improved pit. ROEC latrines that use a chute to divert discharge to allow separation of the pan and the pit are among the types that don't use water.

Another important sanitation system that uses water is one is using septic tanks designed to treat night soil inside a watertight tank. As described above, these septic tanks involve settlement, thickening, and anaerobic digestion in the pit, but this is incidental to the principal function, which is to allow leaching of the liquid portion. Septic tanks that lack the leaching function depend on settlement, thickening, and anaerobic digestion, and floating. Efforts to improve the treatment level have resulted in septic tanks evolving from a single vault type to two vault and three vault types, and then to a type with an up flow anaerobic filter. The single vault type tends to cause disturbance of residues (affecting biodegradation and short circuiting sludge) because of intermittent use of the latrine. Two vault and three vault types stabilize the water by alleviating such disturbances.

Regardless of the latrine type, solids accumulate in certain ways, and regular removal or dislodging of such accumulation is the basis of sanitation maintenance.

Table 5.1: The type and development process (WHO)

Composting latrine			No water used
Vaults and cesspits			
Ventilated improved single latrine	Ventilated improved doubled latrine		No water used
Pour Flush(PF) with pit directly below	Offset PF latrine		Water used
Single septic tank	Double septic tank	Double septic tank with aerobic filter	Water used

Sanitation System				
On site treatment		On site/off site treatment	Off site treatment	
Dry type	Wet type	Wet type	Wet type	Dry type
1.Overhang	1.Pour Flash (PF) latrine + saokway	1.Low-volume CF latrine+soakaway/sewer	1.Conventional sewerage	1.Conventional & vacuum car
2.Trench latrine	2.PF + aqua Privy + soakaway	2.Low-volume CF latrine+aqua privy+soakaway/sewer		2.Vaults & cesspit +manual withdrawal+truck/cart
3.Pit latrine	3.PF latrine + septic tank	3.Low-volume CF+septic tank+soakaway/sewr		3.Buckect latrine
4.Reed odorless earth closet (ROEC)	4.Sullage flushing + aqua privy + soakaway			4.Mechanical buckect latrine

Table 5.2: General Classification of Sanitation (World Bank)

5.Ventilated Improved pit(VIP) latrine	5.Sullage flushing + septic tank + soakaway			
6.Batch composting latrine	6.Conventional septic tank			
7.Continous-composting latrine				

Table 5.3 Classification of Sanitation Technologies (World Bank)

Sanitation Technologies	VIP latrine ROEC (Note)	PF latrine	DVC Latrine (Note 2)	Aqua privy	Septic Tank	three Vault Septic Tank (Note 3)	Vaults and cesspits and transport with cart	Sewered PF latrine/septic tank/aqua-privy	Sewerage
Rural application	Suitable	Suitable	Suitable	Suitable	Suitable for rural institutions	Suitable	Not Suitable	Not Suitable	Not Suitable
Urban application	Suitable in Low and Medium Density areas	Suitable in Low and Medium Density areas	Suitable in Low and Medium Density areas	Suitable in Low and Medium Density areas	Suitable in Low and Medium Density areas	Suitable in Low and Medium Density areas	Suitable	Suitable	Suitable
Construction cost	Low	Low	Medium	Medium	High	Medium	Medium	high	Very High
O&m Cost	Low	Low	Low	Low	High	low	High	Medium	High
Ease of Construction	Easy except for wet and rocky	Easy	Easy except for wet and rocky ground	Some skilled labour necessary	Some skilled labour necessary	Some skilled labour necessary	Some skilled labour necessary	Skilled engineer/builder necessary	Skilled engineer/builder necessary
Self Help Potential	High	High	High	High	Low	High	High(for Vault Construction)	Low	Low
Water Requirement	None	Water necessary near latrine	None	Water necessary near latrine	Water supply to the household and to the latrine	Water necessary near latrine	Water necessary near latrine	Water supply to the household and to the latrine	Water supply to the household and to the latrine
Soil Requirement	Stable permeable soil. Groundwater level at least 1m from the ground surface	Stable permeable soil. Groundwater level at least	None (may be constructed on the ground)	Stable permeable soil. Groundwater level at least	Stable permeable soil. Groundwater level at least	Stable permeable soil. Groundwater level at least	None (may be constructed on the ground)	None	None
Sanitation Technologies	VIP latrine ROEC (Note)	PF latrine	DVC Latrine (Note 2)	Aqua privy	Septic Tank	Three Vault Septic Tank (Note 3)	Vaults and cesspits and transport with cart	Sewered PF latrine/septic tank/aqua-privy	Sewerage
Complementary off-site investment	None	None	None	Sludge treatment facilities	Sludge off site treatment facilities	Sludge treatment facilities	Night soil treatment facilities	Sewer network and treatment facilities	Sewer network and treatment facilities
Reuse Potential	Low	Low	High	Medium	Medium	Medium	High	High	High
Benefit of Health	Good	Very Good	Good	Very Good	Very Good	Very Good	Very Good	Very Good	Very Good
Institutional Requirements	Low	Low	Low	Low	Low	Low	Very high	High	High

* Depending on population density, either the onsite or off site sullage disposal facilities become necessary for technologies not connected to sewerage when the water supply exceeds 50 to 100Lcd

**The plinth can be constructed when the groundwater level is less than 1m from the ground surface

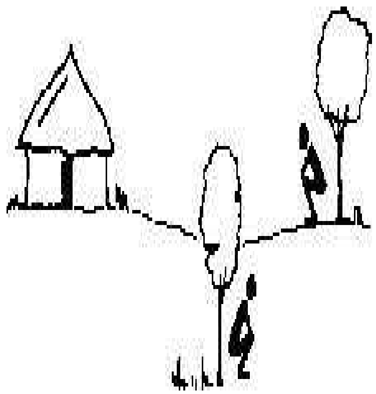
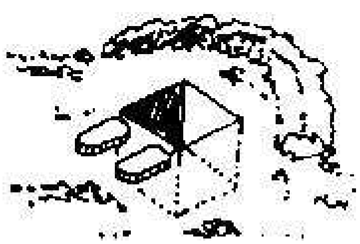
Note 1: ROEC (Reed Odorless Earth Closet) is a kind of ventilated improved pit latrine, in which the pan and pit are offset (decentralized) from each other and connected by a chute.

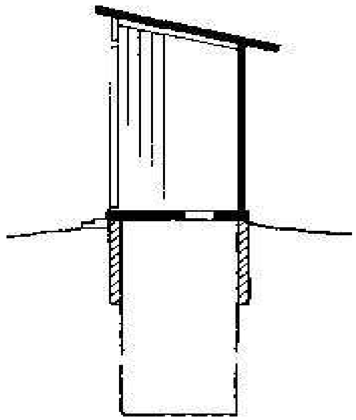
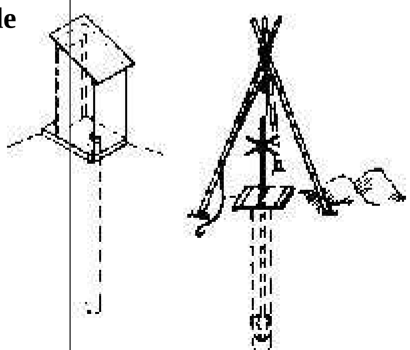
Note 2: VC (double Vault compositing latrine) is the compositing latrine shown in table 5.1; Vaults are used alternately to enable continuous operation

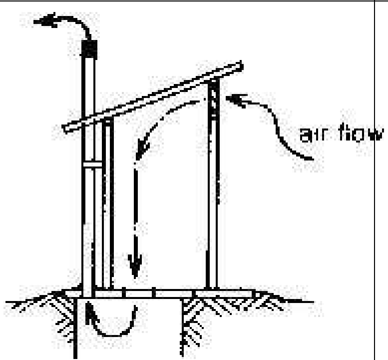
5.4 Different mode of Sanitation

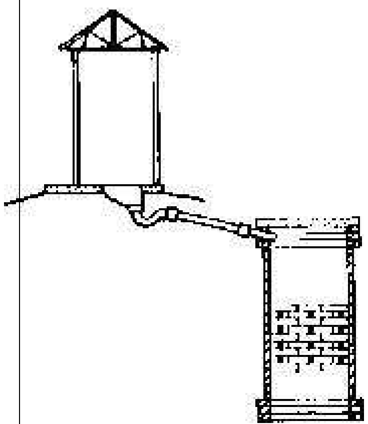
Sanitation includes open defection, shallow pit, simple pit latrine, borehole latrine, ventilated pit latrine, pour flush latrine, single or double pit, compositing septic tank, aqua-privy, overhung latrine, bucket latrine, vaults and cesspits and sewerage. Table 5**.4** below has the details and description.

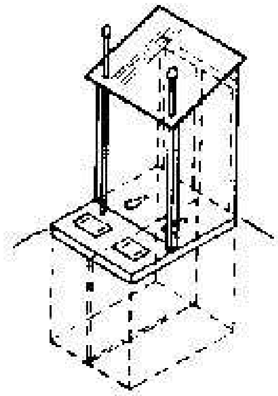
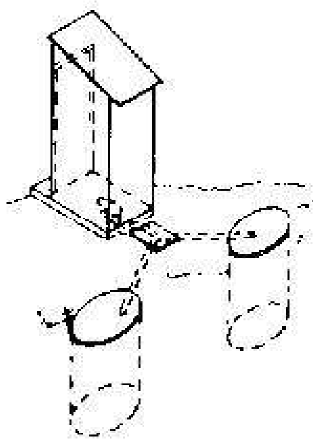
Table 5.4 Types and features of Sanitation (WHO)

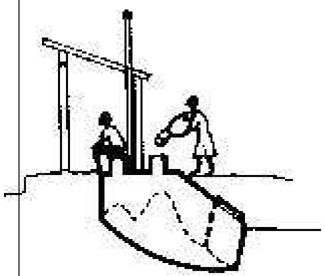
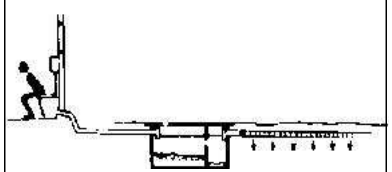
Type	Construction	Description	Advantages	Disadvantages
Open Defection		Where there are no latrines people resort to defecation in the open. This may be indiscriminate or in special places for defecation generally accepted by the community, such as defecation fields, rubbish and manure heaps, or under trees. Open defecation Encourages flies, which spread faeces-related diseases. In moist ground the larvae of Intestinal worms develop, and faeces and larvae may be carried by people and animals. Surface water run-off from places where people have defecated results in water pollution. In view of the health hazards created and the degradation of the environment, open Defecation should not be tolerated in villages and other built-up areas. There are better options available that confine excrete in such a way that the cycle of reinfection from Excrete-related diseases is broken.		
Shallow pit		People working on farms may dig a small hole each time they defecate and then cover the faeces with soil. This is sometimes known as the "cat" method. Pits about 300 mm deep may be used for several weeks. Excavated soil is heaped beside the pit and some is put over the faeces after each use. Decomposition in shallow pits is rapid because of the large bacterial population in the topsoil, but flies breed in large numbers and hookworm larvae spread around the holes. Hookworm larvae can migrate upwards from excrete buried less than I m deep, to penetrate the soles of the feet of subsequent users.	1)No Cost 2) Benefit to farmers as fertilizer	1)Considerable fly nuisance 2)Spread of hookworm larvae

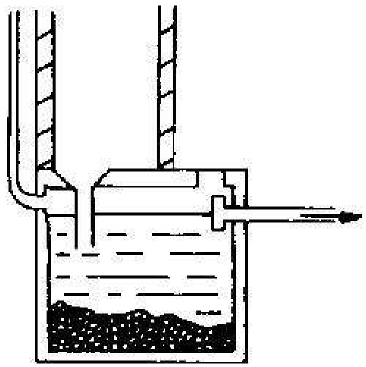
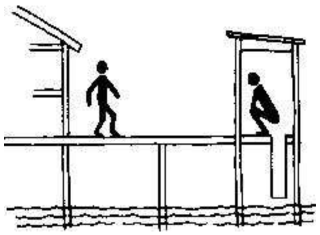
Type	Construction	Description	Advantages	Disadvantages
Simple pit latrine		This consists of a slab over a pit which may be 2 m or more in depth. The slab should be firmly supported on all sides and raised above the surrounding ground so that surface water cannot enter the pit. If the sides of the pit are liable to collapse they should be lined. A squat hole in the slab or a seat is provided so that the excrete fall directly into the pit.	1)Low Cost 2)Can be built by householder 3) Needs no water for operation 4) Easily understood	1) Considerable fly nuisance (and mosquito nuisance if the pit is well) unless there is a tight fitting cover over the squat hole when the latrine is not in use 2) Smell
Borehole latrine		A borehole excavated by hand with an auger or by machine can be used as a latrine. The diameter is often about 400 mm and the depth 6 to 8 m.	1) Can be excavated quickly if boring equipment is available 2) Suitable for short-term use, as in disaster situations	1) Sides liable to be fouled, with consequent fly nuisance 2) Short life owing to small cross sectional area 3) Greater risk of groundwater pollution owing to depth of hole

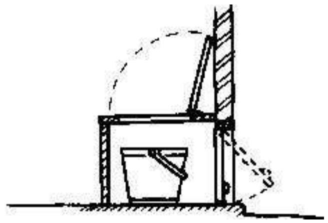
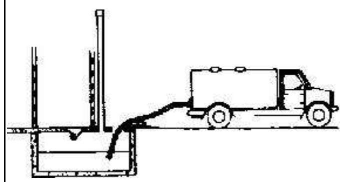
Type	Construction	Description	Advantages	Disadvantages
Ventilated pit latrine		Fly and odour nuisance may be substantially reduced if the pit is ventilated by a pipe extending above the latrine roof, with fly-proof netting across the top. The inside of the superstructure is kept dark. Such latrines are known as ventilated improved pit (VIP) latrines.	1) Low cost 2) Can be built by householder 3) Easily understood 4) Control of flies 5) Absence of smell in latrines	1) Does not control mosquitoes 2) Extra cost of providing vent pipe 3) Need to keep interior dark Easily understood

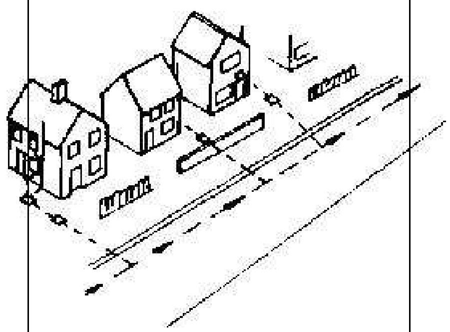
Pour-flush latrine		A latrine may be fitted with a trap providing a water seal, which is cleared of faeces by pouring in sufficient quantities of water to wash the solids into the pit and replenish the water seal. A water seal prevents flies, mosquitoes and odours reaching the latrine from the pit. The pit may be offset from the latrine by providing a short length of pipe or Covered channel from the pan to the pit. The pan of an offset pour flush latrine is Supported by the ground and the latrine may be within or attached to a house.	1) Low cost 2) Control of flies and mosquitoes 3) Absence of smell m latrine 4) Offset type 5) Pan supported by ground 6) Latrine can be in house	1) A reliable (even if limited) water supply must be available 2) Unsuitable where solid anal cleaning material is used 3) Contents of pit not visible 4) Gives users the convenience of a WC 5) Can be upgraded by connection to sewer when sewerage becomes available

Type	Construction	Description	Advantages	Disadvantages
Single or double pit	 	In rural and low-density urban areas, the usual practice is to dig a second pit when the one in use is full to within half a metre of the slab. If the superstructure and slab are light and prefabricated they can be moved to a new pit. Otherwise a new superstructure and slab have to be constructed. The first pit is then filled up with soil. After two years, faeces in the first pit will have completely decomposed and even the most persistent pathogens will have been destroyed. When another pit is required the contents of the first pit can be dug out (it is easier to dig than undisturbed soil) and the pit can be used again. The Contents of the pit may be used as a soil conditioner. Alternatively, two lined pits may be constructed, each large enough to take an accumulation of faecal solids over a period of two years or more. One pit is used until it is full, and then the second pit is used until that too is full, by which time the contents of the first pit can be removed and used as a fertilizer with no danger to health. The first pit can then be used again.	1) Will last for several years if large enough	1) Once constructed the pits are more or less permanent 2) Easy removal of solids from the pits as they are shallow 3) Pit contents can be safely used as a soil conditioner alter 2 years, without treatment

Type	Construction	Description	Advantages	Disadvantages
Composting latrine		In this latrine, excrete fall into a watertight tank to which ash or vegetable matter is added. If the moisture content and chemical balance are controlled, the mixture will decompose to form a good soil conditioner in about four months. Pathogens are killed in the dry alkaline compost, which can be removed for application to the land as a fertilizer. There are two types of composting latrine: in one, compost is produced continuously, and in the other, two containers are used to produce it in batches.	1) A valuable humus is produced	1) Careful operation is essential 2) Urine has to be collected separately in the batch system 3) Ash or vegetable matter must be added regularly
Septic tank		A septic tank is an underground watertight settling chamber into which raw sewage is delivered through a pipe from plumbing fixtures inside a house or other building. The sewage is partially treated in the tank by separation of solids to form sludge and scum. Effluent from the tank infiltrates into the ground through drains or a soakpit. The system works well where the soil is permeable and not liable to flooding or water logging, provided the sludge is removed at appropriate intervals to ensure that it does not occupy too great a proportion of the tank capacity.	1) Gives the users the convenience of a WC	1) High cost 2) Reliable and ample piped water required 3) Only suitable for low-density housing 4) Regular dislodging required and sludge needs careful handling 5) Permeable soil required

Type	Construction	Description	Advantages	Disadvantages
Aqua-privy		An aqua-privy has a watertight tank immediately under the latrine floor. Excreta drop directly into the tank through a pipe. The bottom of the pipe is submerged in the liquid in the tank, forming a water seal to prevent escape of flies, mosquitos and smell. The tank functions like a septic tank. Effluent usually infiltrates into the ground through a soakpit. Accumulated solids (sludge) must be removed regularly. Enough water must be added to compensate for evaporation and leakage losses.	1) Does not need piped water on site 2) Less expensive than a septic tank	1) Water must be available nearby 2) More expensive than VIP or pour-flush latrine 3) Fly mosquito and smell nuisance if seal is lost because insufficient water is added 4) Regular desludging required and sludge needs careful handling 5) Permeable soil required to dispose of effluent
Overhung latrine		A latrine built over the sea, a river, or other body of water into which excrete drop directly, is known as an overhung latrine. If there is a strong current in the water the excrete are carried away. Local communities should be warned of the danger to health resulting from contact with or use of water into which excrete have been discharged.	1)May be the only feasible system for communities living over water 2) Cheap	1) Serious health risks

Type	Construction	Description	Advantages	Disadvantages
Bucket latrine		This latrine has a bucket or other container for the retention of faeces (and sometimes urine and anal cleaning material), which is periodically removed for treatment or disposal. Excreta removed in this way are sometimes termed night soil.	1) Low initial cost	1) Malodorous 2) Creates fly nuisance 3) Danger to health of those who collect or use the night soil 4) Danger to health of those who collect or use the night soil
Vaults and cesspits		In some areas, watertight tanks called vaults are built under or close to latrines to store excrete untill they are removed by hand (using buckets or similar receptacles) or by vacuum tanker. Similarly, household sewage may be stored in larger tanks called cesspits, which are usually emptied by vacuum tankers. Vaults or cesspits may be emptied when they are nearly full or on a regular basis.	1) Satisfactory for users where there is a reliable and safe collection service	1) High construction and collection costs 2) Removal by hand has even greater health risks than bucket latrines 3) Irregular collection can lead to tanks overflowing 4) Efficient infrastructure required

Type	Construction	Description	Advantages	Disadvantages
Sewerage		Discharge from WCs and other liquid wastes flow along a system of sewers to treatment works or directly into the sea or a river.	1) User has no concern with what happens after the WC is flushed 2) No nuisance near the household 3)Treated effluent can be used for irrigation	1)High construction costs 2) Efficient infrastructure required for construction, operation and maintenance 3) Ample and reliable piped water supply required (a minimum of **70** litres per person per day is recommended) 4) If discharge is to a water-course, adequate treatment required to avoid pollution

Note 1:- Drain field has porous pipes laid under the ground. Effluent from septic tanks flows through these pipes for infiltration into the ground for disposal

Note 2:- Pits are unlined holes and used to dispose the liquid content of urine through infiltration

5.5 Classification of technologies in relation to Water Supply

Sanitation and water supply can further be subdivided into improved and none improved types. Table 5.5 below has the details.

Table 5.5 Classification of technologies related to water supply and sanitation

	Improved Technologies	Non Improved Technologies
Water Supply	Household Connection Public Standpipe Borehole (Bore well) Protected dug well Protected Spring Rainwater collection	Non Protected well Non protected spring Vendor provided water Bottled Water Water Truck
Sanitation	Connection to public sewer Connection to septic system Pour flush latrine Pit latrine Ventilated Improved pit Latrines	Bucket latrine Public Latrine Latrines with an open pit

Source: - Global Water Supply and sanitation assessment, 2000 Report, WHO

5.6 Decomposition of Feaces and Urine

As soon as excrete are deposited, they start to decompose, eventually becoming a stable material with no unpleasant smell and containing valuable plant nutrients. Duringdecomposition, the following processes take place.

• Complex organic compounds, such as proteins and urea, are broken down into simpler and more stable forms.

• Gases such as ammonia, methane, carbon dioxide and nitrogen are produced and released into the atmosphere.

• Soluble material is produced which may leach into the underlying or surrounding soil or be washed away by flushing water or groundwater.

• Pathogens are destroyed because they are unable to survive in the environment of the decomposing material.

The decomposition is mainly carried out by bacteria although fungi and other organisms may assist. The bacterial activity may be either aerobic, i.e., taking place in the presence of air or free oxygen (for example, following defecation and urination on to the ground), or anaerobic, i.e., in an environment containing no air or free oxygen (for example, in a septic tank or at the bottom of a pit). In some situations both aerobic and anaerobic conditions may apply in turn. When all available oxygen has been used by aerobic bacteria, facultative bacteria capable of either aerobic or anaerobic activity take over, and finally anaerobic organisms commence activity.

Pathogens may be destroyed because the temperature and moisture content of the decomposing material create hostile conditions. For example, during composting of a mixture of faeces and vegetable waste under fully aerobic conditions, the temperature may rise to 70°C, which is too hot for the survival of intestinal organisms. Pathogens may also be attacked by predatory bacteria and protozoa, or may lose a contest for limited nutrients.

5.7 Volumes of Decomposed human Wastes

As excrete become decomposed they are reduced in volume and mass owing to:

- evaporation of moisture;

- production of gases which usually escape to the atmosphere;

- leaching of soluble substances;

- transport of insoluble material by the surrounding liquids;

- consolidation at the bottom of pits and tanks under the weight of superimposed solids and liquids.

Little information is available regarding the rate at which the reduction takes place although there are indications that temperature is an important factor (Mare & Sinnatamby, 1986). Weibel et al. (1949) measured the sludge accumulation rate in 205 septic tanks in the United States of America, and obtained the results shown in **Fig. 5. 1**; other authors have reported the accumulation rates listed in **Table 5.6.**

Fig. 5.1 Rate of accumulation of' sludge and scum in 205 septic tanks in the United States of America (from Weibel et al., 1949)

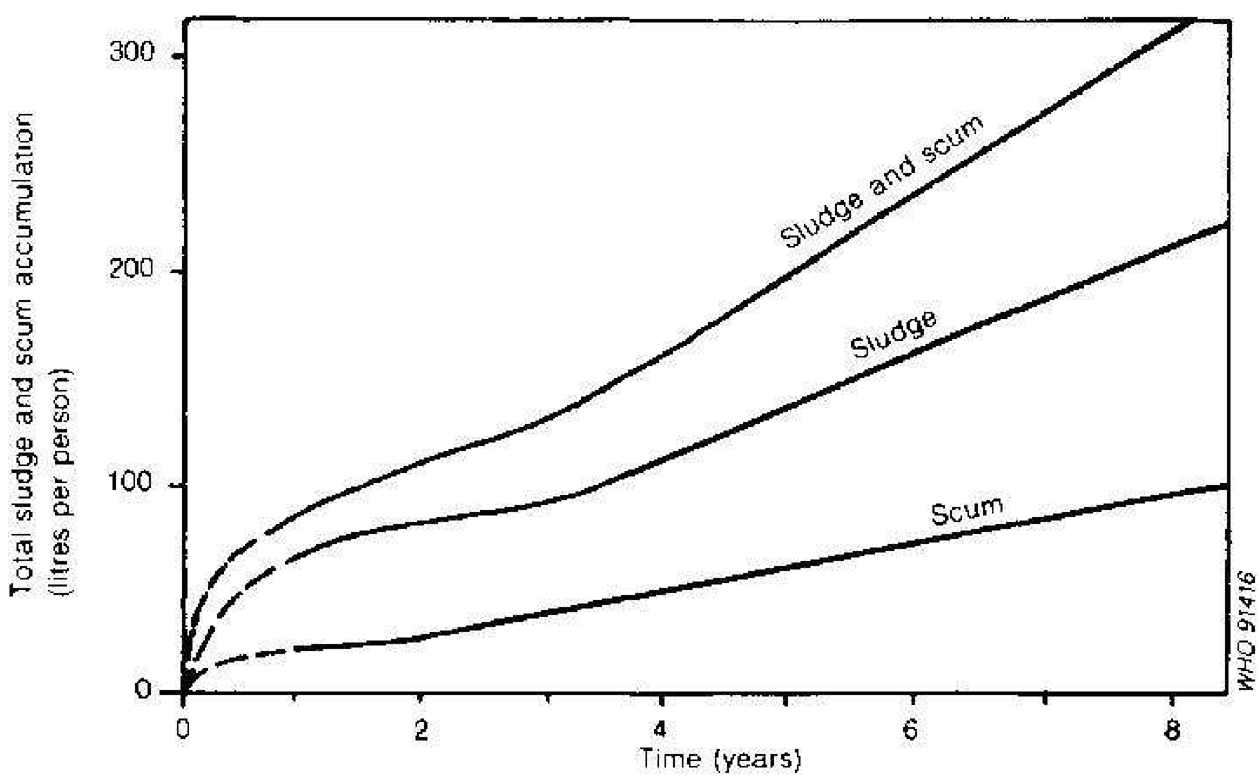


Table 5.6 Excreta accumulation rates (litres per person per year)

Location	Accumulated excreta	Remarks	Reference
Zimbabwe	20	Latrine regularly washed down; degradable cleaning material	Morgan & Mara (1982)
West Bengal	25	Wet pit---ablution water used	Wagner & Lanoix (1958)
West Bengal	34	Wet pit	Baskaran (1962)
Philippines	40	Wet pit; degradable cleaning material	Wagner & Lanoix (1958)
USA	42	Faeces (adult); half amount for children	Geyer et al. (1968)
Brazil	47	Dry pit	Sanches & Wagner (1954)
Philippines	60	Dry pit; degradable cleaning material	Wagner & Lanoix (1958)

The factors with the biggest effect on the sludge accumulation rate are whether decomposition takes place above or below the water table and the type of anal cleaning material used. Decomposition under water produces a much greater reduction in volume than decomposition in air. This is due to better consolidation, more rapid decomposition and removal of the finer material in the water flow. Anal cleaning materials vary widelyaround the world, from those requiring little or no storage space, such as water, to thosehaving a greater volume than the excrete, such as corn cobs, cement bags or stones.

When designing a latrine it is strongly recommended that local sludge accumulation rates should be measured. In the absence of local data, the volumes given in Table 3.8 are suggested as a maximum. There is some evidence to indicate that these figures are on the high side. However, if refuse is added to excrete, the accumulation rate may be much greater.

Where excrete are stored for short periods only, such as in double pit latrines or composting toilets, the reduction process may not be complete before the sludge is removed. In such cases it will be necessary to use higher sludge accumulation rates than indicated above. A 50% increase is tentatively suggested.

Table 5.7 Suggested maximum sludge accumulation rates (litres per person per year)

	Sludge Accumulation Rate
Wastes retained in water where degradable anal cleaning materials are used	40
Wastes retained in water where non-degradable anal cleaning materials are used	60
Waste retained in dry conditions where degradable anal cleaning materials are used	60
Wastes retained in dry conditions where non degradable anal cleaning materials are used	90

5.8 Basis of Design Considerations of On-site Sanitation

5.8.1 Ground Conditions

Ground conditions affect the selection and design of sanitation systems, and the following five factors should be taken into consideration:

- bearing capacity of the soil;
- self-supporting properties of the pits against collapse;
- depth of excavation possible;
- infiltration rate;
- groundwater pollution risk

5.8.2 Bearing Capacity of the Soil

All structures require foundations, and some soils are suitable only for lightweight materials because of their poor load-carrying capacity - marshy and peaty soils are obvious examples. In general, it is safe to assume that if the ground is suitable for building a house it will be strong enough to support the weight of a latrine superstructure made of similar materials, providing the pit is appropriately lined.

5.8.3 Self supporting properties of the pits

Many types of latrine require the excavation of a pit. Unless there is specific evidence tothe contrary (i.e., an existing unlined shallow well that has not collapsed), it is recommended that all pits should be lined to their full depth. Many soils may appear to be self-supporting when first excavated, particularly cohesive soils, such as clays and silts, and naturally bonded soils, such as laterites and soft rock. These self supporting properties may well be lost over time owing to changes in the moisture content or decomposition of the bonding agent through contact with air and/or moisture. It is almost impossible to predict when these changes are likely to occur or even if they will occur at all. It is therefore safer to line the pit. The lining should permit liquid to percolate into the surrounding soil.

5.8.4 Depth of Excavation

Loose ground, hard rock, or groundwater near to the surface limits the depth of excavation possible using simple hand tools. Large rocks may be broken if a fire is lit around them and then cold water poured on the hot rock. Excavation below the water table and in loose ground is possible by "caissoning”, but it is expensive and not usually suitable for use by householders building their own latrines.

5.8.5 Infiltration Rate

The soil type affects the rate at which liquid infiltrates from pits and drainage trenches. Clays that expand when wet may become impermeable Other soils such as silts and fine sands may be permeable to clean water but become blocked when transmitting effluent containing suspended and dissolved solids.

Opinions vary regarding the areas through which infiltration takes place. For example, Lewis et al. (1980) recommended that only the base of pits or drainage trenches should be considered and that lateral movement (the sidewall influence) be ignored. Mara (1985b) and others have assumed that infiltration takes place only through the side walls as the base rapidly becomes blocked with sludge. Until more evidence is available, it is recommended that the design of pits and trenches should be based on infiltration through the side walls up to the maximum liquid level. For trenches, the area of both walls should be used.

The rate of infiltration also depends on the level of the groundwater table relative to the liquid in the pit or trench. In the unsaturated zone, the flow of liquid is induced by gravity and cohesive and adhesive forces set up in the soil. Seasonal variation may produce a change in the amount of air and water in the soil pores and this will affect the flow rate. Conditions at the end of the wet season should normally be used for design purposes as this is usually the time when the groundwater level is at its highest. In the saturated zone all pores are filled with water and drainage depends on the size of the pores and the difference in level between the liquid in the pit or trench and the surrounding groundwater.

Soil porosity also affects infiltration. Soils with large pores, such as sand and gravel, and rocks such as som sandstones and those containing fissures, drain easily. Silt and clay soils, however, have very small pores and tend to retain water. Soils containing organic materials also tend to retain water but the roots of plants and trees break up the soil, producing holes through which liquids can drain quickly.

The rate of groundwater flow in unsaturated soils is a complex function of the size, shape and distribution of the pores and fissures, the soil chemistry and the presence of air. The speed of flow is normally less than 0.3 m per day except in fissured rocks and coarse gravels, where the speed may be more than 5.0 m per day, with increased likelihood of groundwater pollution.

5.8.6 Pore Clogging

The soil type affects the rate at which liquid infiltrates from pits and drainage trenches. Clays that expand when wet may become impermeable. Other soils such as silts and fine sands may be permeable to clean water but become blocked when transmitting effluent containing suspended and dissolved solids.

5.9 Determining Infiltration Rates

It is rarely possible to measure accurately the rate of flow of effluent from pits and drainage trenches, especially as the flow often decreases as soil pores become clogged.
Consequently various empirical rules are used. Some recommendations are based on the rate of percolation of clean water from trial holes dug on the site of a proposed pit or drainage field using various design criteria to allow for differences in infiltration rates (US Department of Health, Education, and Welfare, 1969; British Standards Institution, 1972).
Laak et al. (1974) found that, for a wide range of soils, the infiltration rates of effluent were 130 litres per m² per day. A conservative rate of 10 litres per m² per day was recommended for general application. On the other hand, rates of up to 200 litres per m² per day are considered applicable in practice in the United States of America (US Department of Health, Education, and Welfare, 1969), and Aluko (1977) found that, in Nigeria, designs with a maximum of 294 litres per m² per day have proved satisfactory.
The infiltration capacities given in Table 2.5(US Environmental Protection Agency, 1980) are recommended as a basis for the sizing of pits and drainage trenches where information about actual infiltration rates is not available. The capacities given for coarse soils are restricted to prevent possible groundwater pollution and therefore may be unnecessarily conservative in areas where this is not a problem. Gravel is capable of much higher infiltration rates, which may be a problem in areas where shallow groundwater is used for human consumption.

5.10 Pit Latrine

1. When calculating the dimensions of a hole for a pit latrine, three conditions must be satisfied.

2. The pit should have sufficient storage capacity for all the sludge that will accumulate during its operational life or before its planned emptying.

3. At the end of the pit's operational life there should still be sufficient space left for the contents to be covered with a sufficient depth of soil to prevent surface contamination with pathogenic organisms (soil seal depth).

4. There should be sufficient wall area available at all times to enable any liquid in the pit to infiltrate the surrounding soil.

Storage volume

The storage volume required to accommodate the sludge that accumulates in the pit during its operational life can be calculated from:

V = N x P x R
where
V= the effective volume of the pit (m^3)
N = the effective life of the pit (years)
P = the average number of people who use the pit each day
R = the estimated sludge accumulation rate for a single person (m^3 per year).
Once the effective volume of the pit has been calculated, the plan area is decided. This should be based on local preference, ground conditions and construction materials, and is generally circular or rectangular in shape. Note that only the area inside the lining is utilized for sludge accumulation, not the excavated area.

Having determined the plan shape and area, the depth of pit required for sludge accumulation is calculated as follows:

Sludge depth = [total sludge volume (V)]/(plan area)

Soil seal depth

This is usually taken as 0.5 m. In the case of double pit latrines it is the depth to the bottom of the inlet drain.

Infiltration area

In communities where people use water for anal cleaning or bathe in the toilet, a considerable amount of water may enter the pit. If it is assumed that the soil pores below the sludge surface are blocked, then additional wall area must be allowed for infiltration of the liquids above the sludge.

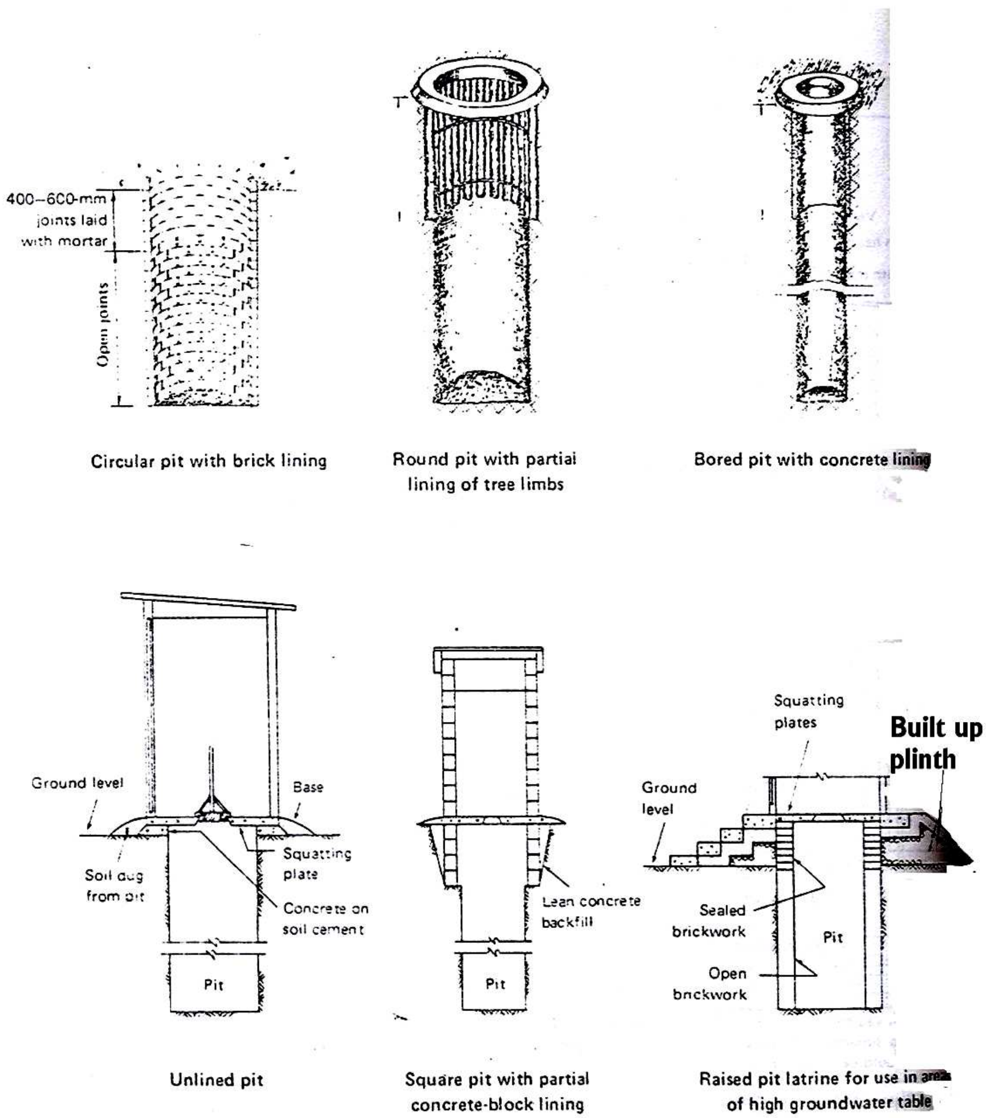
The infiltration area cannot include the soil seal depth since the top 0.5 m of a pit has a fully sealed lining.

Assuming that all the liquid entering the pit lies on top of the sludge, then the liquid depth will rise until the area of contact between liquid and soil is large enough to permit infiltration of the daily intake of liquid.

Pit depth

The total depth of the pit is calculated as follows:
Pit depth = sludge depth + infiltration depth + soil seal depth
Some traditional pit designs are shown in figure 5.2 below.

Figure 5.2 Alternative Pit Designs(mm)



5.11 Superstructures

The function of the toilet superstructure is to provide privacy and to protect the user and the toilet from the weather. Superstructure design requires assessment of whether separate facilities are required for men and women in the same household. Local customs and preference often influence its location, orientation, shape, construction material, design (e.g., without roof, window details0, and size. Color may be very important to householder use and maintenance of the facility. These details should be designed in consultation with the user. The technical design requirements of the superstructure are relatively straightforward and may be stated as follows:-

Size: the plan area should be at least 0.8 cubic meter to provide sufficient space and generally not more than 1.5 cubic meters. The roof height should be a minimum of 1.8meters Ventilation: there should be several openings at the top of the walls to dissipate odors and, in the case of VIP latrines and ROECs, to provide draft required for functioning of the vent pipe. These openings should be about 75 to 100mm by 150 to 200m in size; often it is convenient to leave an open space between the top of the door and the roof.

The door: this should open outwards in order to minimize the internal floor area. In some societies, however, an outward opening door may be culturally unacceptable, and an open entrance with a privacy wall may be preferred. In either case, it must be possible to fasten the door from the inside, and it may also be necessary to provide an external lock to prevent use by unauthorized persons. At its base, the door should be just clear of the floor in order to provide complete privacy while preventing rot of the bottom of the door planks.

Lighting: natural light should be available and sufficient. The toilet should be sufficiently shaded, however, to discourage flies; this is particularly important in the case of VIP latrines and ROECs.

The walls and roof: these must be weatherproof, provide adequate privacy, exclude vermin, and to architecturally compatible in external appearances with the main house. In urban areas especially an L-shaped wall in front of the door may be regarded by the community as desirable or essential for privacy

A wide variety of materials may be used to construct the superstructure, e.g.; brick or concrete blocks, with tile or corrugated iron or asbestos cement roof; mud and wattle, bamboo or palm thatch, with palm thatch roof; ferrocement, sheet metal, or timber with corrugated iron or asbestos roof. The choice depend on cost, material availability, and community preferences.

5.12 VIP and VIDP Latrines

In the case of VIP latrines, the pit is around square $1m^2$ in cross section and its depth is then readily calculated from the required volume. Depths are usually in the range from 3 to 8m although pit depths of 12m or more are found where soils are particularly suitable. With VIP latrines, it may be advantageous to use enlarged pits provided the ground conditions are suitable.

The upper part of the pit should be lined so that it can properly support the squatting plate and superstructure. If this is not done, the pit may collapse. In suitable soil conditions it may be necessary to extend this lining down to the bottom of the pit (Figure 5.3), but care must be taken to ensure that the lining does not prevent percolation.

A VIDP latrine differs from a VIP only in that it has two alternating pits. When one is full, the pit should rest at least one year before it is emptied to ensure pathogen destruction. Pit depths can be varied to reflect soil condition (i.e. ease of construction) and desired emptying frequency. To facilitate emptying and prevent collapse of the partition wall, however, the pit should not be as deep as that of a VIP.

All pits should be constructed to prevent surface water from entering. This requires grading to ensure diversion of surface drainage. In cases where the pit is partially offset from the superstructure, it should normally be constructed on the downhill side.

5.13 Reed Odorless Earth Closets (ROECs)

These latrines normally have the advantage over VIP latrines that the pit, being completely offset, can be larger and thus lasts longer. The design lifetime should be 15 to 20 years. The width of the pit is generally about 1m and, for easy desludging, its depth should not exceed 3m; its length can thus be readily calculated from the equation given in section 5.10 (see figure 5.3 and 5.4 below).

Figure 5.3 Reed Odorless Earth Closet (ROEC) in mm

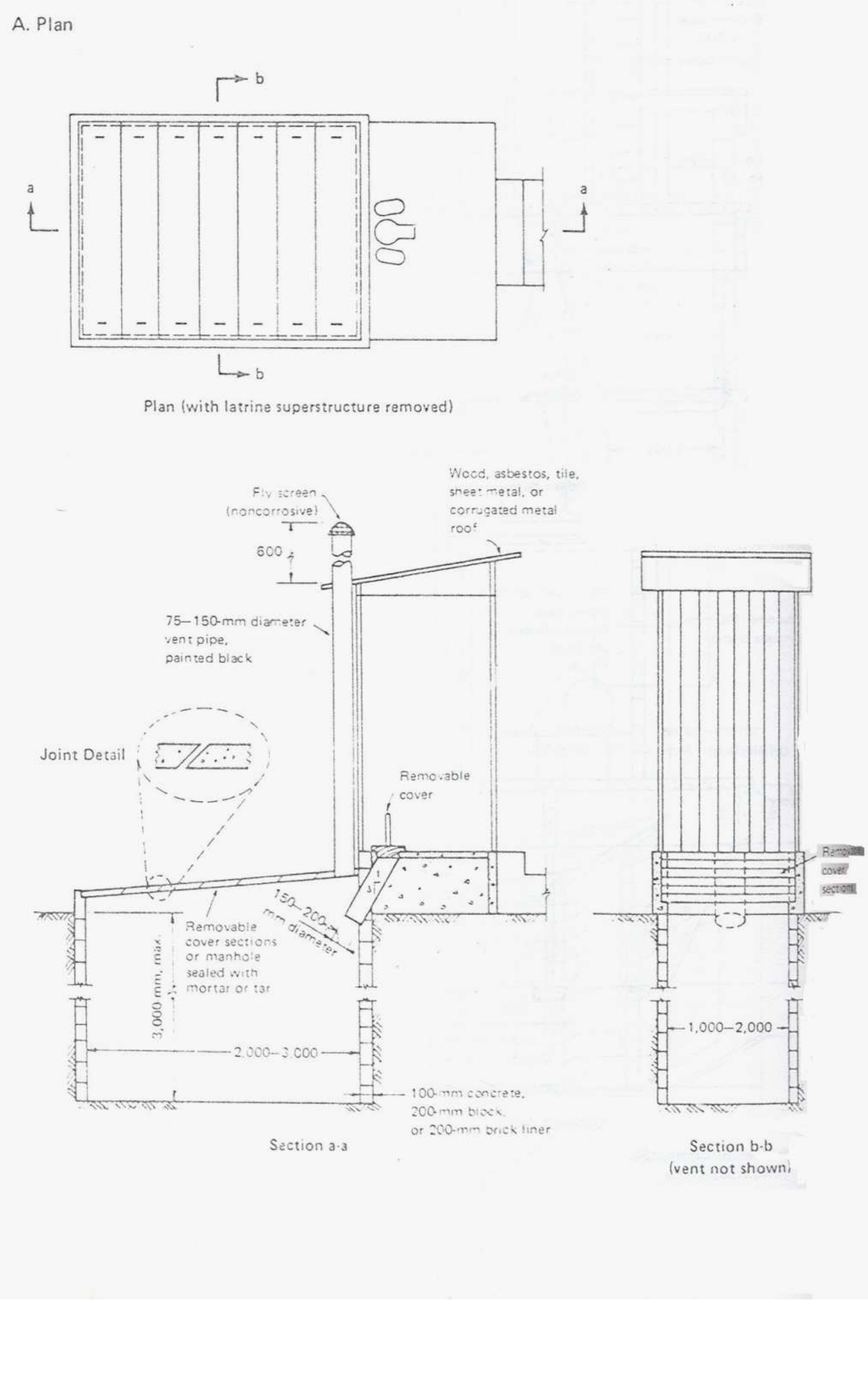
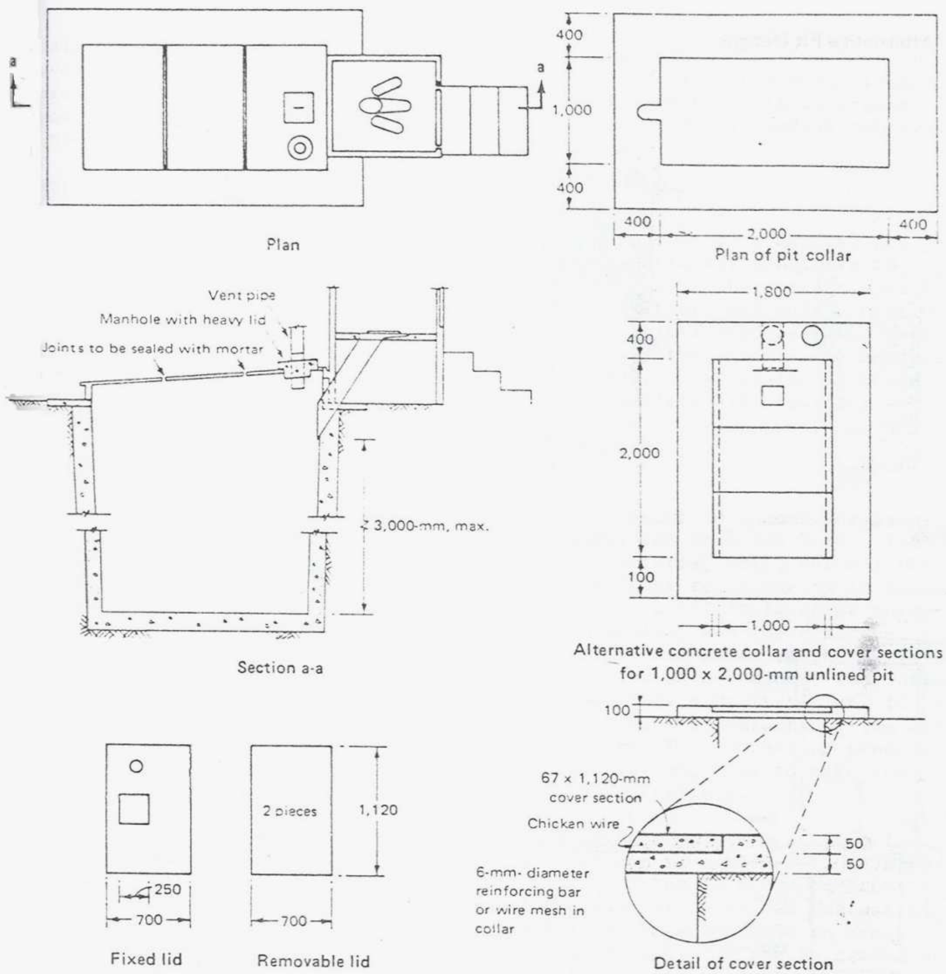


Figure5.4 ROEC

B. Structural Details



Note: Pedestal seat with curved chute may be substituted for squatting plate. Construction materials and dimensions for superstructure may vary according to local practice. The vent should be placed for maximum exposure to sunlight.

5.14 Borehole Latrines

This type of pit latrine is not recommended as a household sanitation facility since it is too small (usually only 400mm in diameter and upto 4m deep for hand augers) and cannot be ventilated. Borehole latrine thus have a short lifetime (Usually 1 to 2 years) and generally unacceptable levels of fly and odor nuisance. Where mechanical augers are available, greater depths and lifetimes can be provided but ventilation is still a problem (see figure 5.2).

5.15 Compositing Toilets

Household systems for composting night soil and other organic materials are used under a variety of conditions. They are successful in both developing and industrial countries when they receive a high degree of user care and attention. This is most likely to occur when there is an urgent need for fertilizer or when there is a high degree of environmental concern. There are two types of systems, continuous and batch.
Continuous Composting Toilets

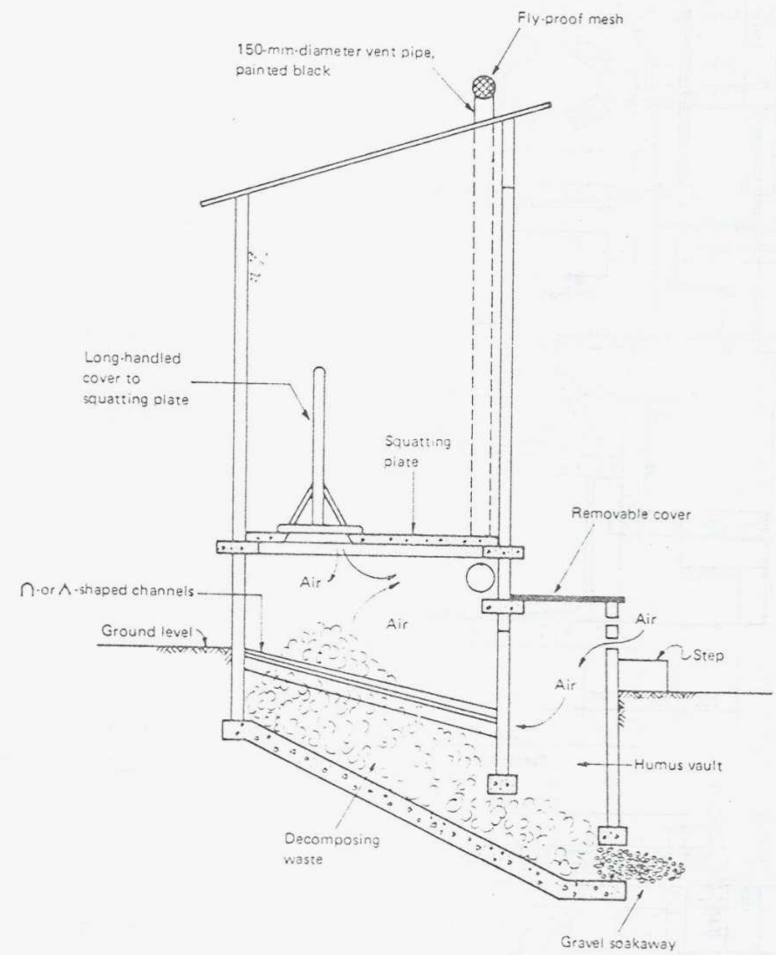
Continuous composting toilets are developments of a Swedish design known as a "multrum"(see figure 5.5 below.) They are extremely sensitive to the degree of user care: the humus has to be removed at the correct rate, organic matter has to be added in the correct quantities, and only a minimum of liquid can be added. Even with the required sophisticated level of user care, short circuiting may still occur within the system, and viable excreted pathogens can be washed down into the humus chamber. The results of these field trials indicate that continuous compositing toilets are presently not suitable for use in developing countries.

5.16 Batch Composting Toilets

Double vault composting (DVC) toilets are the most common type of batch composting toilet. Designs are shown in Figure 5.6 and 5.7. The design details , such as fixed or movable superstructures, vary, but all DVC toilets have certain design principles and operational requirements in common. There are two adjacent vaults, one of which is used until it is about three quarters full, when it is filled with earth and sealed, and the other vault is then used. Ash and biodegradable organic matter are added to the vault to absorb odors and moisture. If ash or organic matter is not added, the toilet acts either as a VIP latrine, if it is unsealed, or as a vault toilet, if it is sealed. When the second vault is filled and sealed, the contents of the first vault are removed and it is put into service again. The composting process takes place anaerobically and requires approximately one year to make the compost microbiologically safe for use as a soil fertilizer.

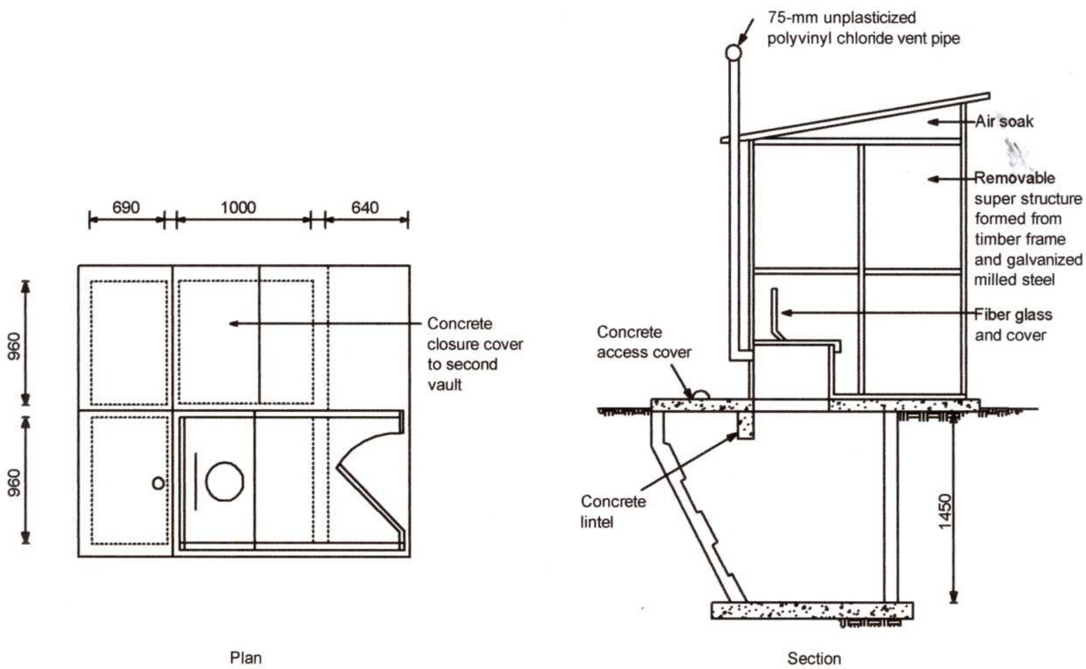
To produce good composted humus, the optimum moisture content in the vault should be between 40 and 60 percent. This can be achieved in several ways. In the Vietnamese DVC toilet (figure 5.7) urine is excluded from the vault and either drained to a small gravel soakaway or collected for use as a nitrogenous liquid fertilizer. This is unlikely to be acceptable in areas where the prevalence of urinary shistosomiasis is high. In the Botswanan and Tanzanian DVC toilet (see figure 5.6) the base of the vault is permeable, permitting infiltration and percolation of urine and water; clearly this approach is not applicable in areas where there is a high groundwater table.

Figure 5.5 'Multrum" continuous Compositing Toilet

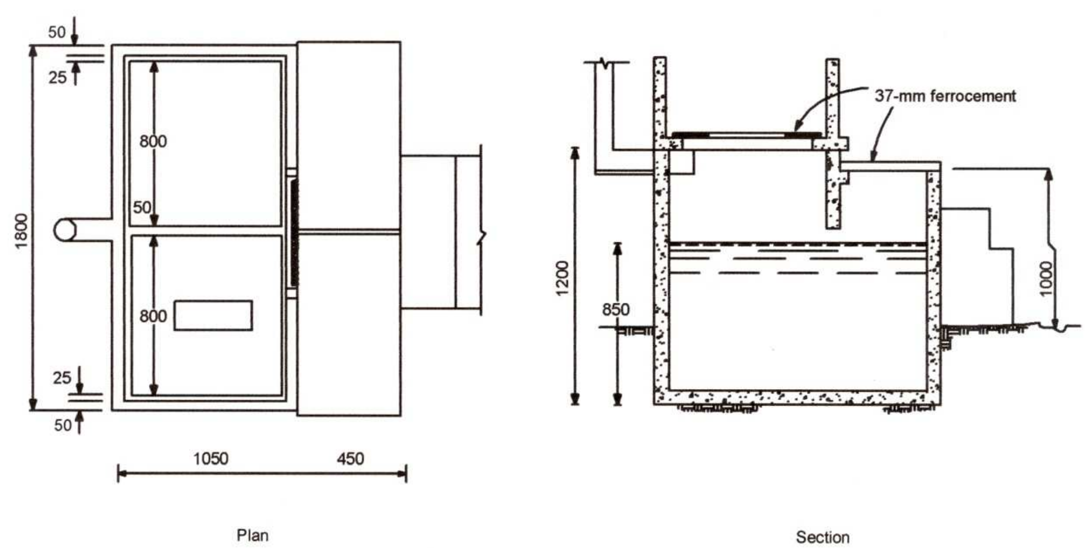


Source: Adapted from a drawing by U. Winblad.

Figure 5.6 Double-vault composting Toilets



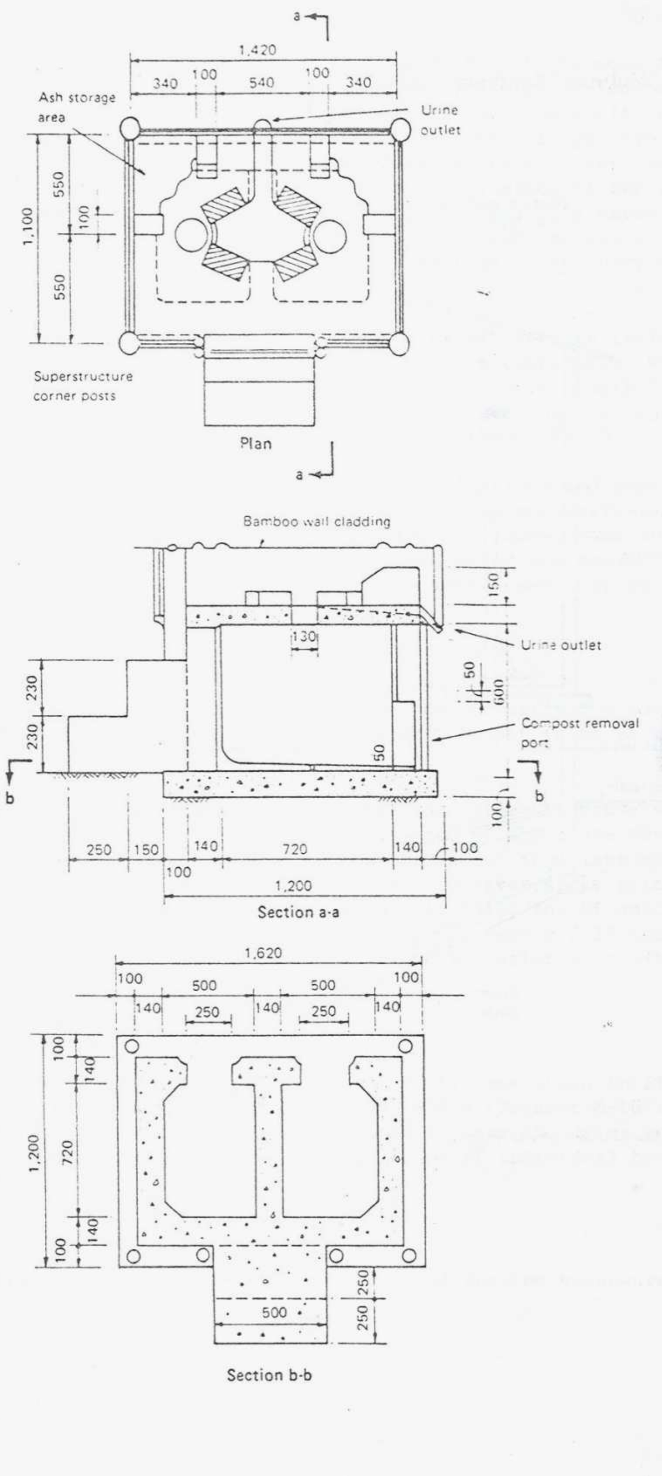
Model used in Botswana



Model used in Tanzania

Source: From a drawing by R.A Boydell

Figure 5.7 Double-vault Composting Toilet Used in Vietnam



5.17 Pour-Flush Toilets

Pour flush (PF) toilets have seals beneath the squatting plate or pedestal seat and are available in many different designs. Two basic types are shown on figure 5.8: the direct discharge and the offset pit design. In both design approximately 1 to 2 liters of water (or sullage) are poured in by hand to flush the excreta into the pit. They can be used for several sanitation service levels. The first type is a modification of the pit latrine in which the squatting plate is provided with a simple water seal. This type is often used with wet pits since the water seal prevents odor development and mosquito breeding. It is especially suitable where water is used for anal cleansing.

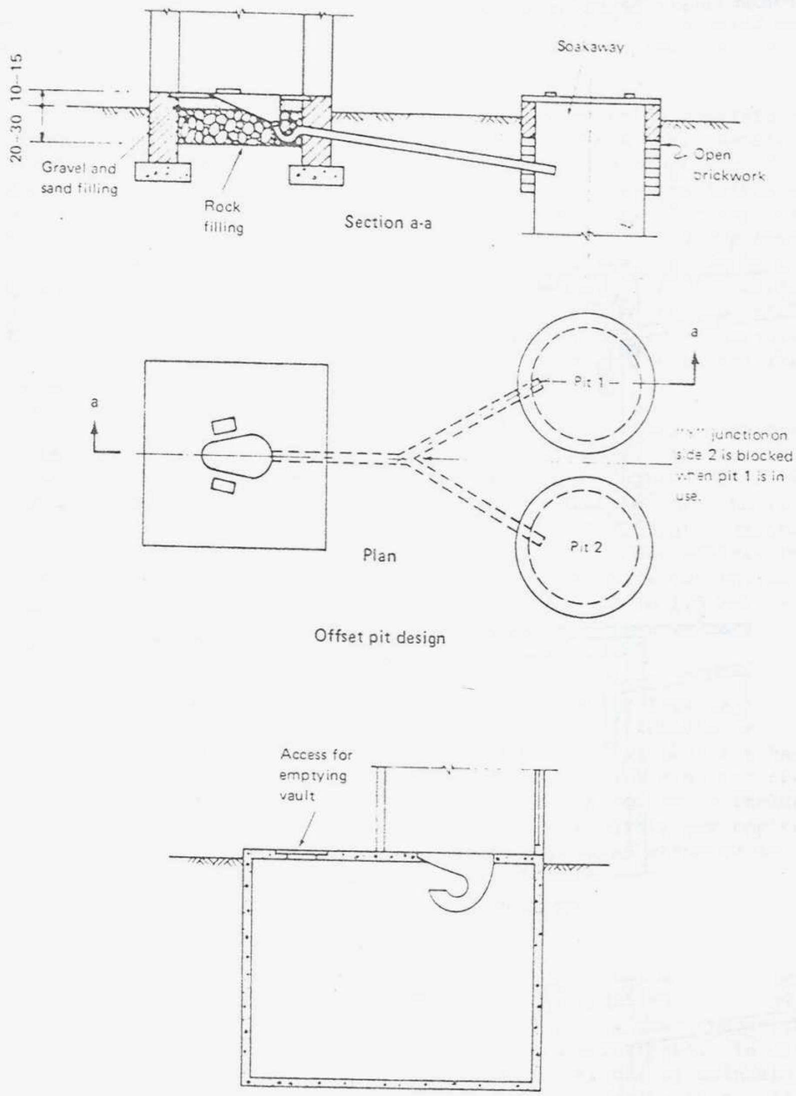
The second type of PF toilet, which is widely used in India, South East Asia, and some parts of Latin America, is used in combination with a completely offset pit. The PF bowel is connected to a short length (8m maximum) of 100m diameter pipe that discharge into an adjacent pit. The slope of the connecting pipe should not be less than 1 in 40.

The pit is designed as described for VIP latrines and provided with a concrete or ferrocement cover slab and wall lining as necessary. Because the digestion of excreta solids proceeds more rapidly in wet than in dry pits, however, a design capacity (C) of 0.04 m3 per person yearly can be used. The volume (V) of pits less than 4m deep may be calculated from the equation given in section 5.11.

This type of PF toilet may be installed inside the house since it is free from both odors and fly and mosquito nuisance; it therefore obviates the need for a separate external superstructure, and it can thus meet social aspirations for a “inside” toilet at low cost. Wherever space permits, two pits should be built. Then, when the first pit is full, the PF unit can be connected to the second pit. When the second pit is nearly full the first one can be used almost indefinitely.

Figure 5.8

Alternative Designs for Pour-flush Toilets



Offset pit design

Direct discharge design

Note: In the offset pit design, the pit is placed at site of "Y" junction if only one pit is installed.

5.18 Sewered PF systems

The sewered PF system is a conceptual development of the sewered aquaprivy system that not only overcomes certain drawbacks inherent in the design concept of the latter while retaining its inherent economic advantages, but also provides more technically appropriate sanitation system in areas where the wastewater flow exceeds the absorptive capacity of the soil. The sewered PF system can either be developed from an existing PF pit latrine or it can be installed as a new facility. There are minor technical differences between these alternatives and only the latter will be considered.

The sewered PF toilet system has five parts:-

- The PF bowl, with a vent pipe and inspection chamber;
- A short length (8m maximum) of 100mm pipe laid at not less than 1 in 40;
- A small two compartment septic tank;
- A network of small bore sewers; and
- A sewage treatment facility

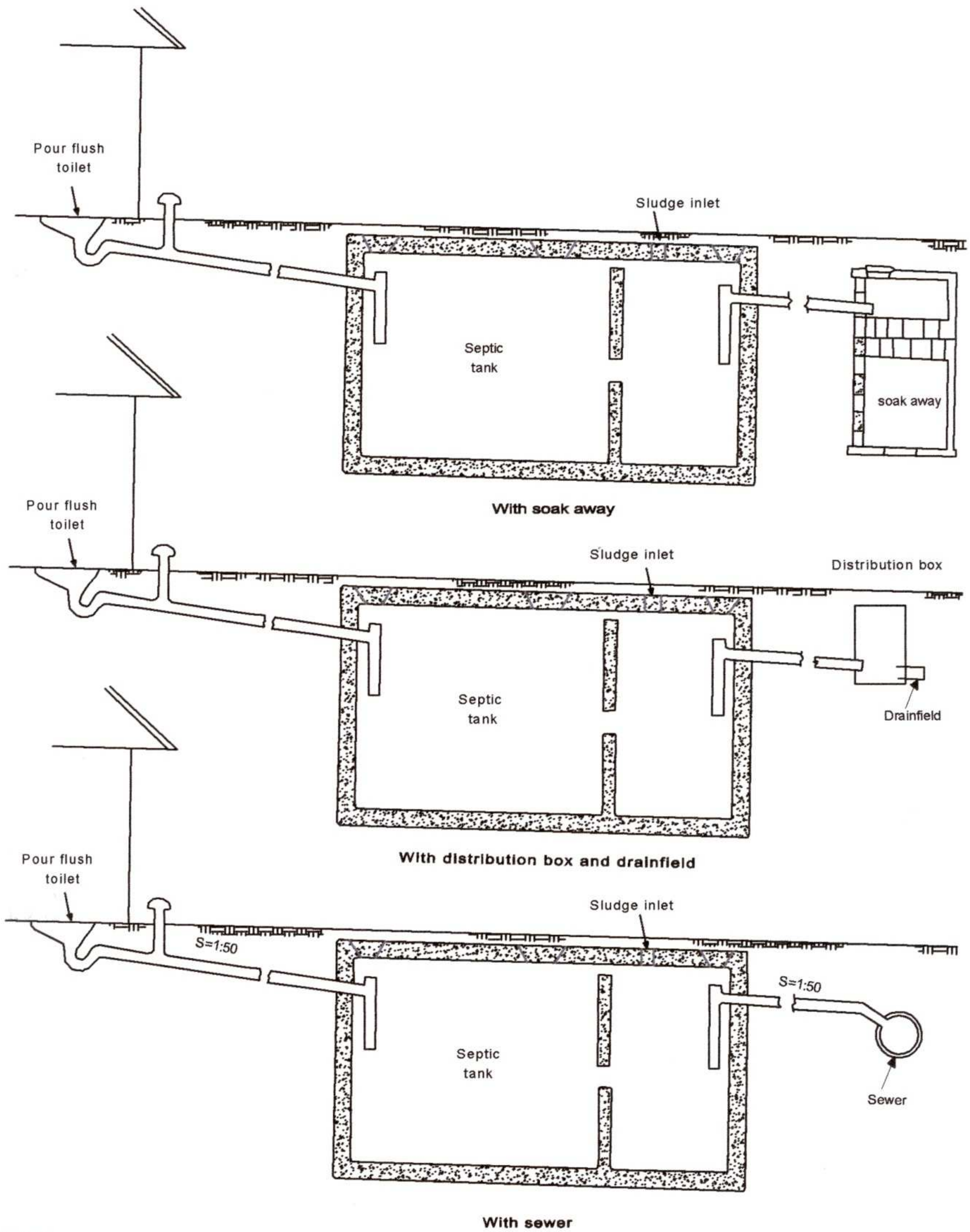
A typical arrangement is shown in diagrammatic form in figure 5.9.

Only excreta and PF water are discharged into the first compartment of the septic tank and only sullage into the second. The two compartments are interconnected by a double T-junction, the invert of which is a nominal 30m above the invert of the exit pipe of the second compartment, which is connected to the street sewer. Thus the contents of the first compartment are able to overflow into the second, but sullage cannot enter the first compartment. This arrangement effectively eliminates the very degree of hydraulic disturbance caused by high sullage flows that, in single compartment tanks, would resuspend and prematurely flush out some of the settled excreta; it thus permits a considerably higher retention time of excreta in the tank and hence is able to achieve a substantially increased destruction of excreted pathogens.

Guidelines for the size of the two compartment septic tanks may be developed as follows:-
Assuming a per capita daily production of excreta of 1.5 liters and a maximum pour flush water usage of 6liters per capita daily, the maximum toilet wastewater flow amounts to 7.5 liters per capita per daily. Allowing a mean hydraulic residence time of 20 days in the first compartment implies a volume requirement of 0.15 m^3 per user, which compares well with the recommendation that the first compartment should be calculated on the basis of $0.15m^3$ per user, subject to a minimum of $1m^3$. The minimum recommended size tank ($1.5m^3$ working volume) is thus suitable for up to seven users and a water consumption of 140liters per capita daily. Desludging of the septic tank is required when the first compartment is half full of sludge, which occurs every 22months assuming a sludge accumulation rate of $0.04m^3$ per person yearly and a capacity of $0.15m^3$ per user.

Since all but the smallest solids are retained in the septic tank, it is not necessary to ensure self cleansing velocities of 1m per second in the receiving sewers. Small bore sewers of 100 to 150mm diameter can be used and these can be laid at flat gradients of 1 in 150 to 300. Sullage water ordinarily carries no solids that could clog sewer pipes. Consequently, manholes need only be provided at pipe junctions. Thus the sewered PF system achieves considerable economics in pipe and excavation cost compared with a conventional sewerage system. Taking into account this savings, the extra cost of the small septic tank, the savings in water usage , and the lower cost of the toilet fixtures, the annual economic cost of a sewered PF system can be expected to be considerably less than that of a cistern flush toilets connected to a conventional sewerage system. In addition, treatment costs will be less because of the enhanced pathogen removal and biochemical oxygen demand (BOD) reduction (approximately 30 to 50%) in the septic tank

Figure 5.9 Pour-Flush Toilet-Septic tank Systems



S= slope

5.19 Aqua privies

There are three types of aqua privies; the simple or conventional aqua privy, the self topping or sullage aqua privy, and the severed aqua privy. The second and third types are simple modifications of the first type designed to accept sullage, which the first type cannot.

The conventional aqua privy toilet (figure 5.10) consists essentially of a squatting plate situated immediately above a small septic tank that discharges its effluent to an adjacent soakway. The squatting plate has an integral drop pipe, in diameter 100 to 150mm, the bottom of which is simple water seal is formed between the squatting plate and the tank contents. In order to maintain this water seal, which is necessary to prevent fly and odor nuisance in the toilet, it is essential that the tank be completely watertight and the toilet user add sufficient water to the tank via the drop pipe to replace any losses. A superstructure is provided for privacy and a small vent pipe is normally incorporated in the design to expel the gases produced in the tank.

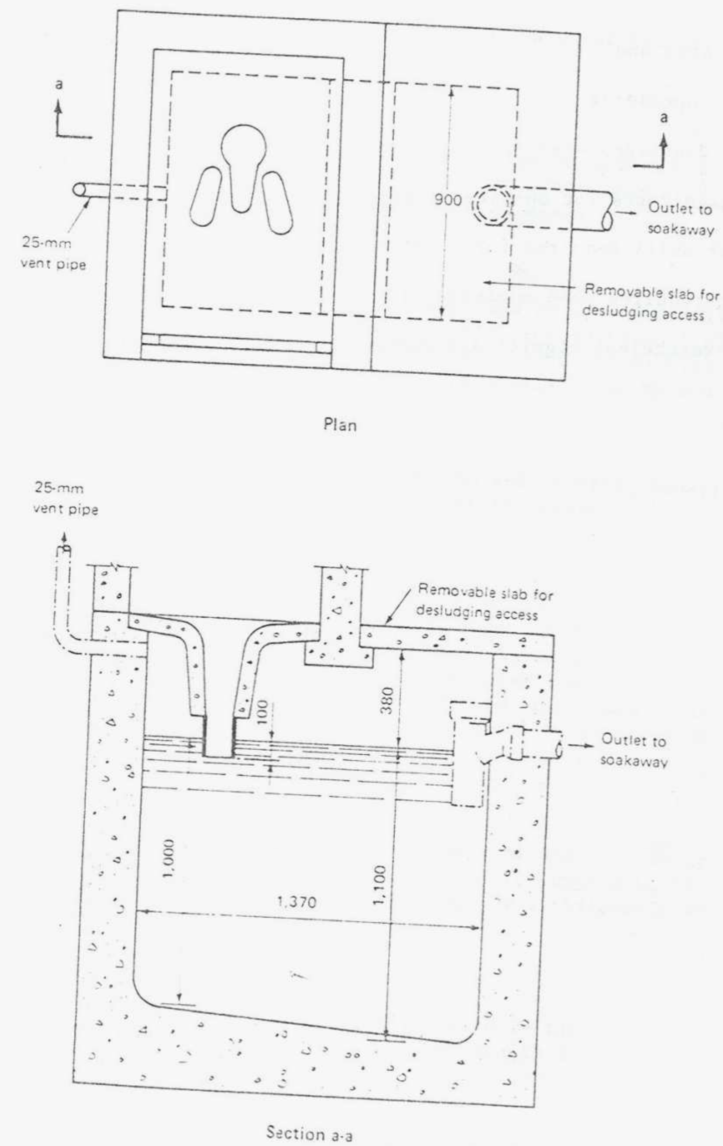
The excreta are deposited directly into the tank where they are decomposed anaerobically in the same manner as in a septic tank. There is, as with septic tanks, a gradual accumulation of sludge (approximately 0.03 to 0.04m^3 per user per year), which should be removed when the tank is two thirds full of sludge. The tank volume is usually calculated on the basis of 0.12m^3 per user, with a minimum size of 1m^3. Desludging is normally required every 2 to 3 years when the tank is two thirds full of sludge. The liquid depth in the tank is normally 1.0 to 1.5m in household units; depths of up to 2m have been used in large communal aqua privies.

The volume of excreta added to the aqua privy tank is approximately 1.5liters per capita daily, and the water used for ‘flushing” and maintenance of the water seal is about 4.5liters per capita daily; thus the aqua privy effluent flow is around 6liters per capita daily. The soakaways should therefore be designed on this basis, although it is common to include a factor of safety so that the design flow would be, say, 8liters per capita daily. The sidewall area of the soakaway should be calculated assuming an infiltration rate of 10 liters per m^2 daily.

Technical Appropriateness

Maintenance of the water seal has always been a problem with conventional aqua privies, except in some Islamic communities where the water used for anal cleansing is sufficient to maintain the seal. Even there, however, it is necessary for the vault to remain water tight. In many other communities people are either unaware of the importance of maintaing the seal or they dislike being seen carrying water into the toilet. If the seal is not regularly maintained, there is intense odor release and mosquito nuisance.

Figure 5.10 Conventional Aquaprivy

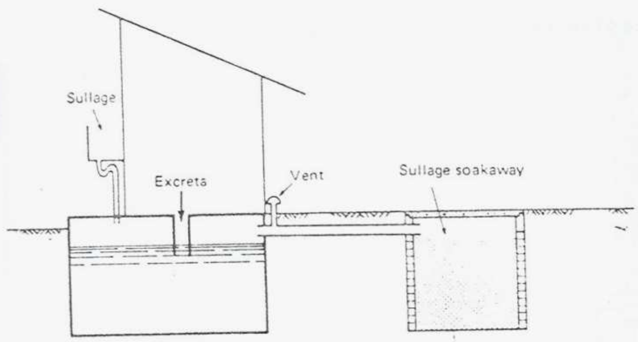


The conventional aqua privy (Figure 5.10) suffers major disadvantages: in practice the water seal is rarely maintained. As a consequence, it cannot be recommended as a viable sanitation technology option. Although the problem of water seal maintenance may be overcome in both the sullage and sewered aqua privies as shown by figures 5.11 and 5.12, and in spite of the evidence that these two systems have had success (notably in Zambia), the basic design of the aqua privy system is questionable because of the expensive watertight tank needed to maintain the water seal. Experience has shown that the water seal may not always be maintained (usually because of failure or inadequacy of the water supply), so that the system has a relatively high risk of intermittent malfunction.

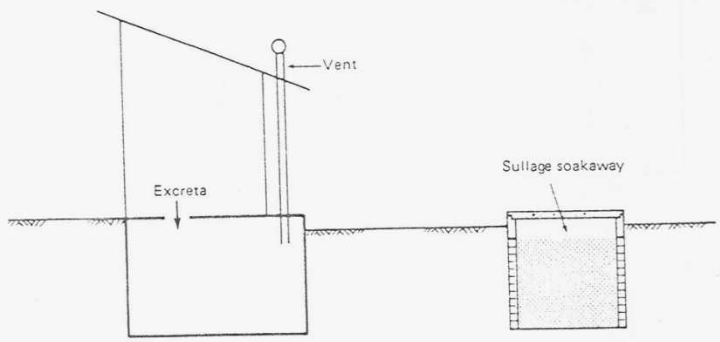
.As shown in figure 5.11, the sullage aqua privy is operationally equivalent to either a VIP latrine (or ROEC) with an entirely separate soakage pit for sullage disposal or a PF latrine with a completely offset pit that can also be used for sullage disposal. The latter alternatives cost less than the sullage aqua privy and in fact are superior because of their reduced risks of odor and fly nuisance and operational malfunctions. The PF toilet has a much more reliable water seal, which does not require a watertight pit, can be located inside the house, and is more easily upgraded to a cistern flush toilet.

The logic of the sewered aquaprivy system is similarly questionable. An aquaprivy is sewered not because of any need to transport excreta along sewers, but as a method of sullage disposal in areas where the soil cannot accept any or all of the sullage produced. As shown in figure 5.12, the sewered aqua privy can be considered as functionally equivalent to a sewered PF toilet. The sewered PF toilet is the superior system; it is also marginally cheaper.

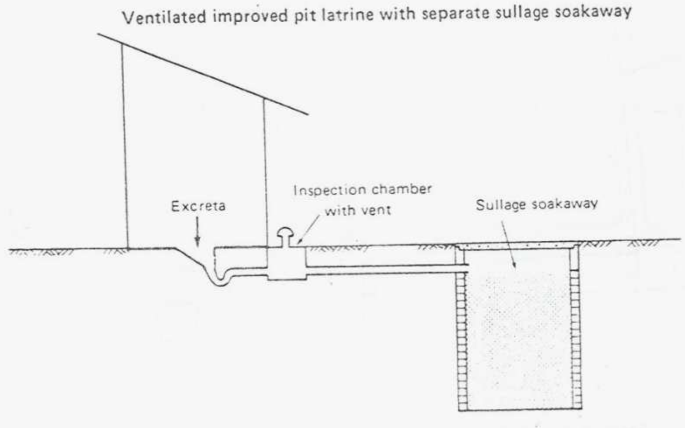
Figure 5.11 Formal Equivalence of Sullage Aquaprivy to Ventilated Latrine with Separate Sullage Soakaway or Toilet



Aquaprivy

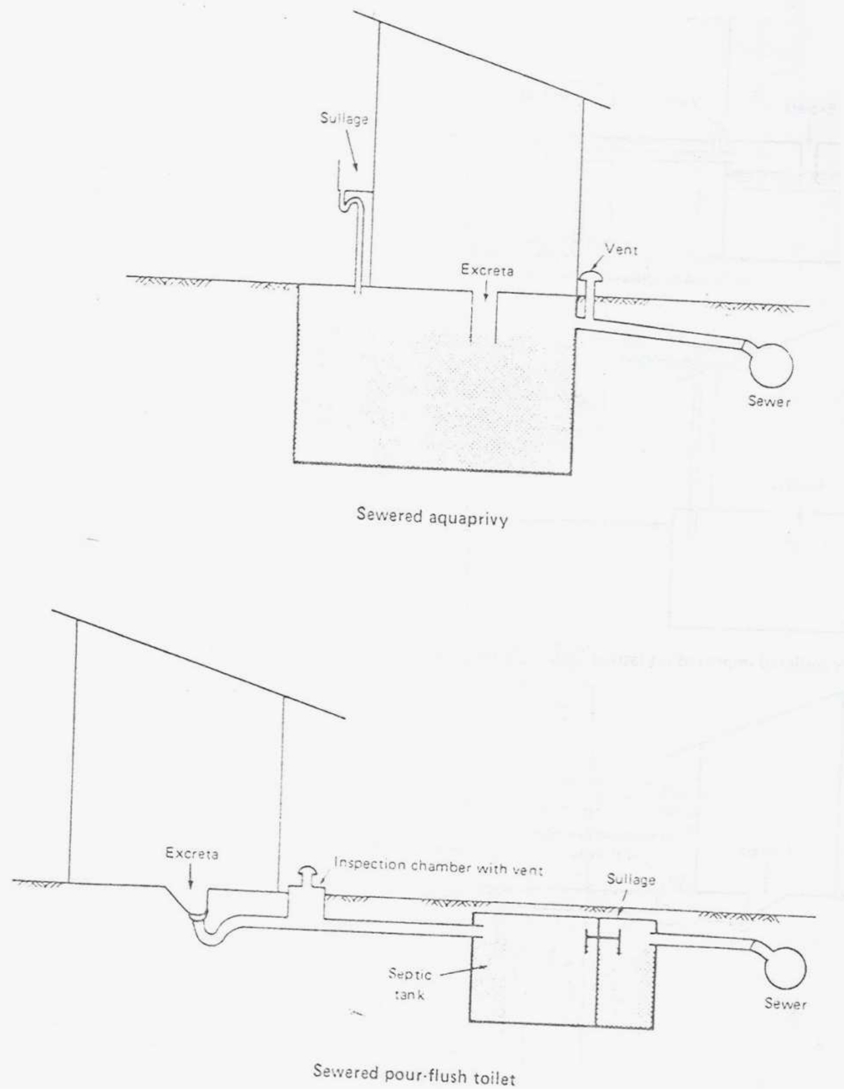


Ventilated improved pit latrine with separate sullage soakaway



Pour-flush toilet

Figure 5.12 Formal Equivalence of Sewered Aquaprivy to sewered pour flush toilet



5.20 Septic Tanks, Soakways and Drainfields

Septic tanks are rectangular chambers, usually sited just below ground level; they receive both excreta and flush water from flush toilets and all other household wastewater. The mean hydraulic retention time in the tank is usually 1 to 3 days. During this time the solids settle to the bottom of the tank where they are digested anaerobically, and a thick layer of scum is formed at the surface. Although digestion of the settled solids is reasonably effective, some sludge accumulates and the tank must be desludged at regular intervals, usually once every 1 to 5 years. The effluent from septic tanks is, from a health point of view, as dangerous as raw sewage and so is ordinarily discharged to soakways or leaching fields. **Although septic tanks are most commonly used to treat the sewage from individual households, they can be used as a communal facility for population up to about 300.**

A two compartment septic tank (Figure 5.13) is now generally preferred to one with only a single compartment because the suspended solids concentration in its effluent is considered lower. The first compartment and the overall length to breadth ratio are 2 to 3 to 1. Experience has shown that in order to provide sufficiently quiescent conditions for effective sedimentation of the sewage solids, the liquid retention time should be at least 24 hours. Two thirds of the tank volume is normally reserved for the storage of accumulated sludge and scum, so that the size of the septic tank should be based on 3 day's retention at start up; this ensures that there is at least 1 day retention just prior to each desludging operation. Sludge accumulates at a rate of 0.03 to 0.04 m^3 per person yearly; thus, knowing the number of users, the interval between successive desludging operations (which are required when the tank is one third full of sludge) is readily calculated.

Figure 5.14 shows a variety of alternate designs, including an experimental septic tank in which an anaerobic up flow filter is substituted for subsurface systems for effluent disposal.

The following guidelines can be used to determine the internal dimensions of a Rectangular tank.

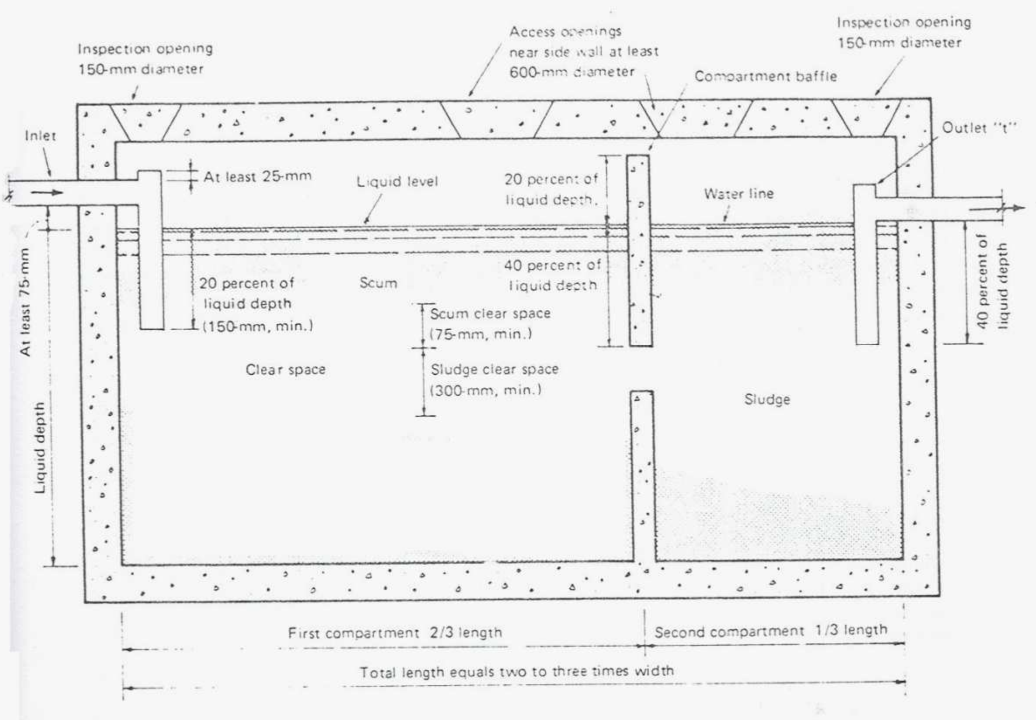
1. The depth of liquid from the tank floor to the outlet pipe invert should be not less than 1.2 m; a depth of at least 1.5 m is preferable. In addition a clear space of at least 300 mm should be left between the water level and the under-surface of the cover slab.

2. The width should be at least 600 mm as this is the minimum space in which a person can work when building or cleaning the tank. Some codes of practice recommend that the length should be 2 or 3 times the width.

3. For a tank of width W, the length of the first compartment should be 2 W and the length of the second compartment should be W (Fig. 5.20). In general, the depth should be not greater than the total length.

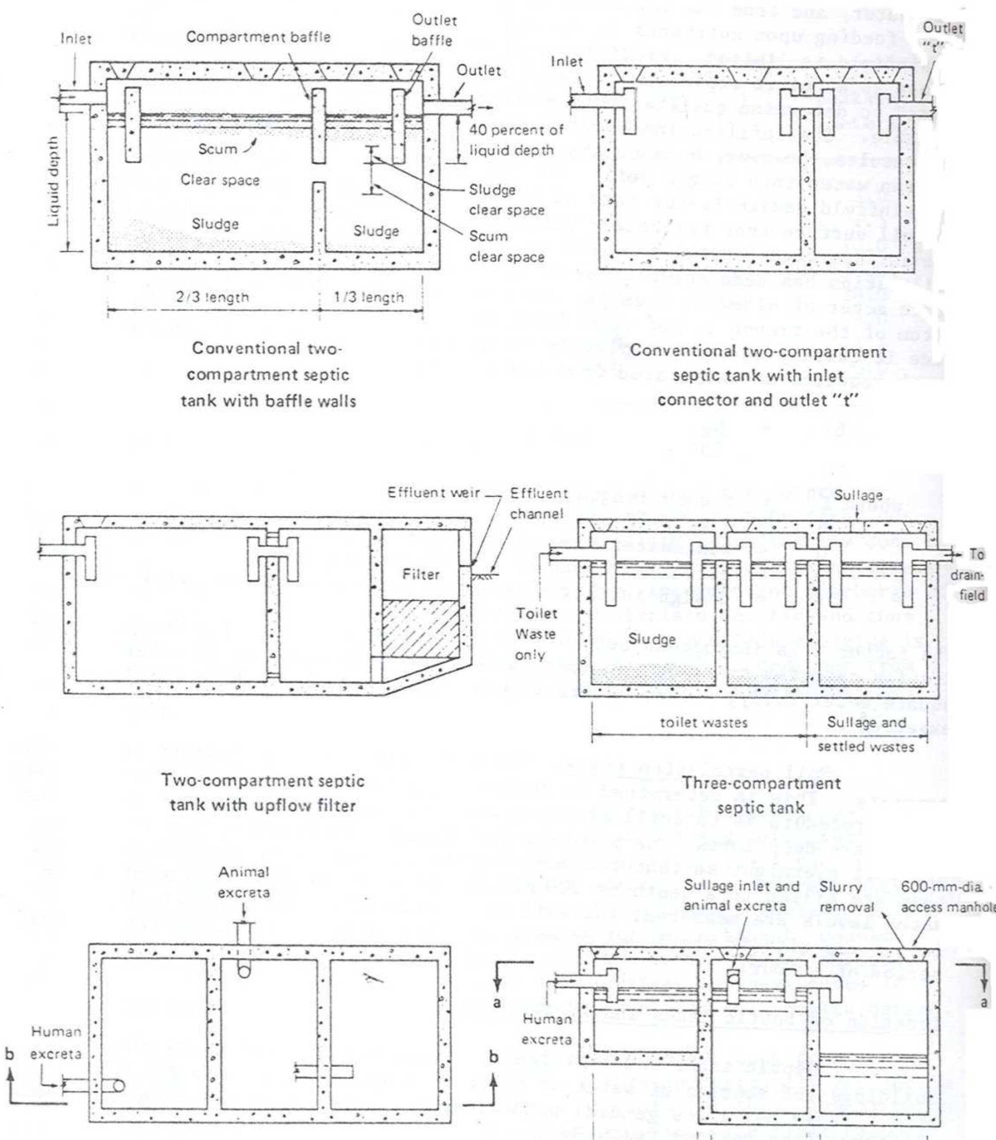
These guidelines give the minimum size of tank. There is no disadvantage in making a tank bigger than the minimum capacity. It may be cheaper to build larger tanks using whole blocks, rather than cutting blocks

Figure 5.13 Schematic of Conventional Septic Tank (mm)



Note: If vent is not placed as shown on figures 5.2, 3& 4 septic tank must be provided with a vent.

Figure 5.14 Alternative Septic Tank Designs



Three-compartment septic tank for resource recovery

5.21 Effluent Disposal

Subsurface disposal into soakaway pits or irrigation in drain field trenches (soakaways) is the most common method of disposal of the effluent. The soil must be sufficiently permeable; in impermeable soils either evapo-transpiration beds or up flow filters can be used, although there is little operational experience with either of these systems. For large flows, waste stabilization ponds may be more suitable.

5.21.1 Drain field Design

The tank effluent is discharged directly to a soakaway (Figure 5.15) or, with larger flows or less permeable soils, to a number of drainage trenches connected in series. Each trench consists of open joint agricultural drainage tiles of 100mm diameter laid on a 1m depth of rock fill (20mm to 50mm grading0. The effluent infiltrates into the soil surrounding the trench, the sidewalls of which are smeared and partially clogged during excavation. Further clogging of the effluent soil interface results from slating (hydration) and effluent into the interface, from chemical deflocculation of clay particles when the effluent water has more sodium than the original interstitial groundwater, and from the formation of an organic mat made up of bacterial slimes feeding upon nutrients in the effluent. This means that the life of a drain field is limited. Provision must therefore be made to set aside land for use as a future replacement drain field. The trench length required is calculated from the equation:

$$L=NQ/2DI$$

Where L=trench length in m;

N=number of users;

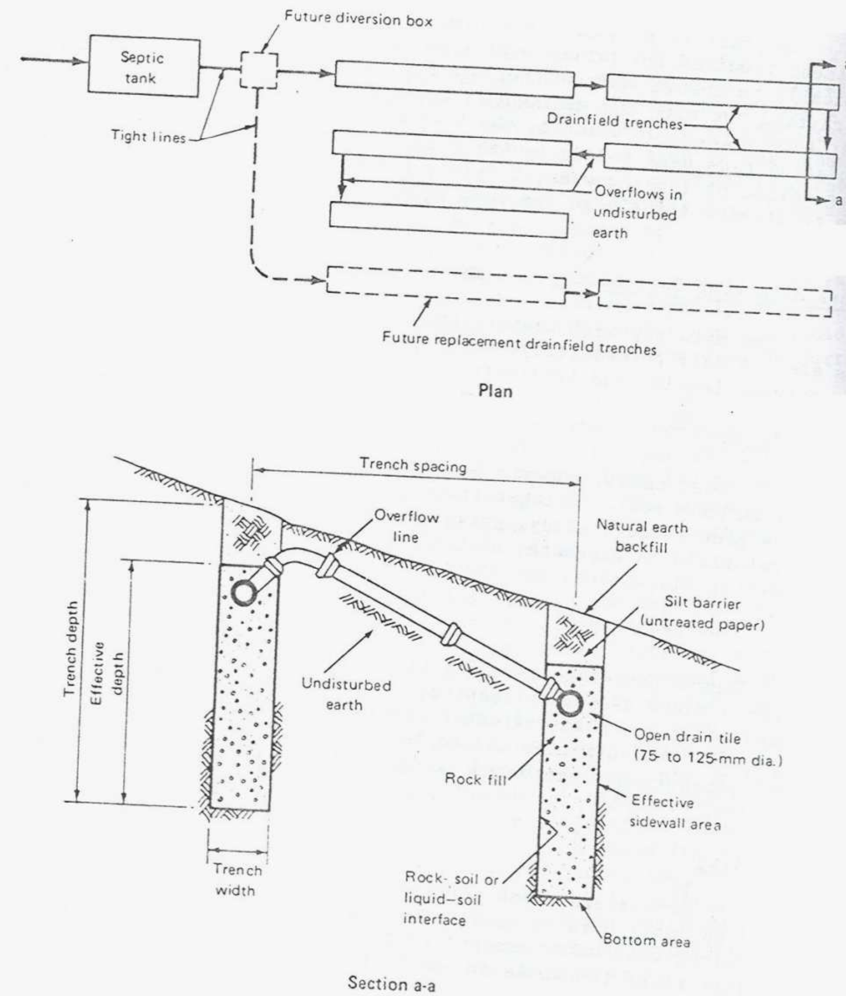
Q=wastewater flow, liters per capita daily;

D=effective depth of trench, m; and

I= design infiltration rate, liters per square meter daily

The factor 2 is introduced because the trench has 2 sides. The design infiltration rate for soakaways or drain fields should be taken as 10 liters per m^2 daily, unless a more accurate figure is known from the local experience.

Figure 5.15 Drainfield for Septic-tank Effluent



5.21.2 Location of Septic Tanks and Drain fields

Septic tanks and drain fields should not be located too close to building and sources of water or to trees whose growing roots may damage them. Table 5.8 gives general guidelines for location in the form of a minimum distances from various features.

Table 5.8 Minimum Distance Requirements for Septic Tanks and Soakways

Item	Septic Tank (m)	Soakaway (m)
Buildings	1.5	3.0
Property boundaries	1.5	1.5
Wells	10.0	10.0
Streams	7.5	30.0
Cuts or Embankments	7.5	30.0
Water Pipes	3.0	3.0
Paths	1.5	1.5
Large Trees	3.0	3.0

5.22 Construction

5.22.1 Latrine

Many components of sanitation systems are common to different types of latrines. The technical details of the following components are considered:

- pits and pit linings;
- latrine floors, which may be cast directly on the ground where the pit or vault is offset;
- slabs, supported over direct or offset pits;
- footrests and squat holes;
- seats;
- water seals, pans, pipes and junction chambers;
- vent pipes;
- Superstructures.

5.22.2 Pits

Excavation

Most pit latrines provide sanitation for a single household, usually necessitating a pit about 1 m across and 3 m or more in depth (although much larger pits are common in some areas), or two shallow pits of up to 1.5 m in depth. The pit may be circular, square or rectangular in plan. Circular pits are more stable because of the natural arching effect of the ground around the hole, with no sharp corners to concentrate the stresses.

However, people often find that square or rectangular pits are easier to dig. The depth of the pits often follows local traditions. It is usually advantageous to dig the pit as deep as possible, but this depends on soil conditions, cost of lining and the level of the groundwater.

Pit linings

The need for a pit lining depends upon the type of latrine under construction and the condition of the soil. In septic tanks and aqua privies, for example, which require watertight compartments, the pit is always lined. However, in pit latrines it is only necessary to have a lining if the soil is likely to collapse during the life span of the latrine.
It is not easy to decide in advance whether a soil will be self supporting. If other excavations in the locality (such as shallow wells) have proved to be self-supporting over a number of years, then it is probably safe to assume that a pit for a latrine can be dug without support. Granular soils such as sands and gravels normally require support.
Cohesive soils, such as silts and clays, and soils with a high proportion of iron oxides, such as laterites, are often self-supporting. However, silts and clays may lose their self-supporting properties when wet, particularly where there is a varying water table.

If there is any doubt about the conditions it is better to assume that the soil is not self-supporting. Increasingly it is recommended that all pits should be lined, especially where the design life is over five years. Failure of an unlined deep pit can be extremely hazardous for the person excavating it. If the failure occurs some years later it can be expensive for the owner and disturbing for the users. In all cases the top 300 500 mm should be lined and sealed to support the slab (and where necessary the superstructure) and to prevent contamination of the surface and entry of vermin.

The lining may be of any material that supports the soil and that will last as long as the design life of the pit. Commonly, materials such as fired bricks, concrete blocks, concrete, ferrocement and local stone are used, but stabilized soil blocks, old oil drums (though with a limited life in corrosive groundwater) and unglazed fired clay pipes have also been successful.

Quarried stones, where available cheaply, makes a satisfactory lining. The more regular blocks should be used for the top 500 mm with mortar joints. Less-regular stone can be used for the remainder of the lining without mortar in the vertical joints. The builders or masons must be skilled and experienced if the lining is to last a reasonable length of time. Where local stone is used, its durability must be confirmed. Some stone will deteriorate when exposed to air or water or to frequent changes between wet and dry conditions.

The use of timber or bamboo is not generally recommended, since they are subject to insect and fungal attack and often have a limited life. Some hard woods can be satisfactory provided they are treated with tar, creosote or other preservative to lengthen their life. Care must be taken to ensure that none of the preservatives leach into the groundwater as even low levels of some preservatives can be toxic (WHO, 1984). Woven cane and bamboo have been used for the lower part of a lining with stronger materials used for the top 500 mm. However, unless the pits are designed to have an extremely short life, cane and bamboo should be avoided.

Backfilling

Any space around the outside of the lining should be backfilled with compacted earth taken from the pit or, where available, with sand and gravel. If the ground is particularlyweak, the top of the pit may be backfilled with weak concrete or a soil-cement mixture to give additional strength. Strengthening may be important if the top of the pit has become overly enlarged during excavation.

5.22.3 Latrine floors

Floors of latrines, whether laid on the ground or supported over a pit, should be smooth and impervious so that they may be cleaned easily and have a satisfactory appearance to users. The upper surface should be at least 150 mm above the surrounding ground level (Fig. 5.16) to prevent rain and surface water entering the latrine.

The floor surface should slope gently to facilitate cleaning and to prevent surplus wash water from collecting in puddles. The slope is normally from the outer-edge of the floor towards the squat hole or pan at the centre, so that the water used for cleaning flows into the pit and does not foul the area surrounding the slab. A fall of about 20 mm between the edge and the centre of a slab up to 1.5 m across is sufficient to prevent pools of liquid forming (Fig. 5.8). Where seats are used, the floor should slope away from the seat support so that any wash water flows towards the latrine entrance.

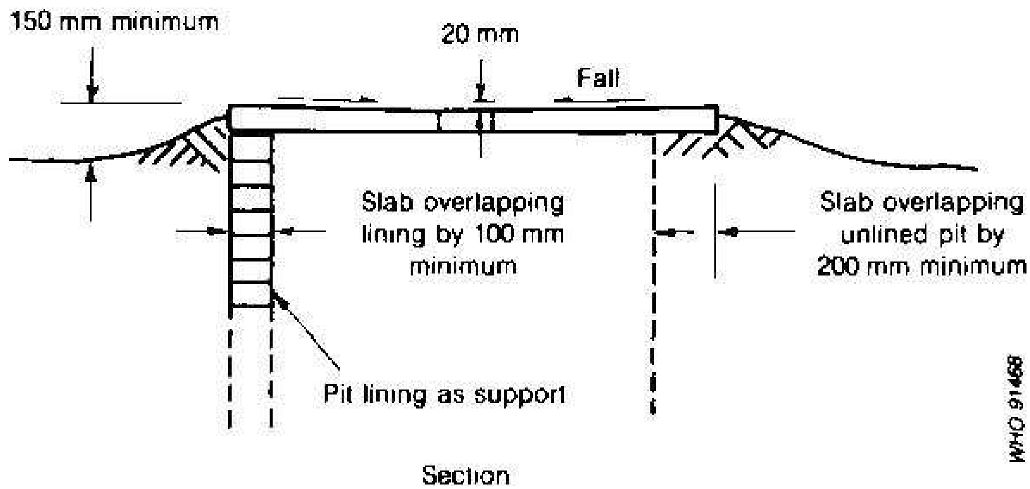
If a precast slab is smaller than the inside floor area of the superstructure, an impervious surface is normally provided to seal the area between the slab and the inside wall of the building. Any area around the slab which is left as bare earth could be fouled, thus becoming a possible site for hookworm infestation. However, in order to minimize costs, the space around the squatting area inside the superstructure should be limited. This reduces building costs for the superstructure as well as flooring materials. But the squat hole or pan should not be so close to the superstructure that users are forced to lean against the wall when they are trying to defecate. A minimum floor space of 80 cm in width and I m from front to back is normally acceptable (Mare, 1985b)

5.22.4 Slabs

Requirements

A latrine slab serves two main purposes, as a support and as a seal. It has to support the weight of the person using the latrine and, possibly, the weight of the superstructure. It also seals the pit, with the exception of the squat hole and, where required, the vent-pipe hole. This facilitates control of flies and smells and reduces the likelihood of rodents and surface water entering the pit. Where the slab has been made in sections (for ease of placing and emptying) or has a removable cover, the joints should be sealed with a weak mortar such as a lime or mud mortar.

Fig. 5.16 Requirements of slabs



To support the weight of a person over a latrine pit the suspended slab has to act structurally in the manner of a bridge. Where seats are provided, the extra weight has to be allowed for when designing the slab. Depending on the design of the slab, the materials may have to be able to resist forces in tension as well as in compression . The materials needed to carry the tensile forces are often more expensive than those commonly used in low-cost buildings. The slab is often the most expensive individual component that has to be paid for by the user. It is therefore important to ensure that it is carefully designed to serve its purpose with a minimum of costly material.

The slab normally rests on a foundation or on the top of the pit lining (see Fig. 5.16). This ensures that the weight of the slab and the weight of the person using it are spread evenly on the soil. Particular care must be taken where the slab also has to carry part of the weight of the superstructure. If the ground is weak, the foundation prevents subsidence or collapse of the ground underneath the load. Any gaps between the slab and the pit lining should be sealed with earth or a weak mortar to prevent ingress of water. This seal also prevents small animals and insects getting into and out of the pit.

Where a pit is excavated to a larger diameter than planned; precast slabs are occasionally supported on timber poles. This practice is not advisable as the heavy load on the poles is likely to lead to early failure. However, small slabs (approximately 500 mm square), designed to provide a hygienic squat hole for existing latrines at minimum cost, will not overload a timber support.

Shapes of direct pit slabs

The shape and size of the pit are the first factors to be considered when designing a supported slab. Latrine pits can be round, square or rectangular and it is usual to find that a particular shape becomes the accepted design for a particular area.

Reinforced concrete

Because of the weakness of concrete in tension it is often reinforced with other materials. Most commonly it is strengthened by the inclusion of steel bars. Details of the reinforcing steel required for common sizes of slab are shown in Table 5.9. Mild steel bars, 6 mm in diameter spaced at intervals of 150 mm, or 8 mm in diameter spaced at intervals of 250 mm in each direction, are normally sufficient for 80-mm thick slabs of up to 1.5 m in span. This span distance is measured at the point of minimum span, that is, the shortest distance between two points which fully support the slab. Where used correctly, reinforcement in a concrete slab will support at least six adults on a 1.5-m span slab. For the small spans illustrated, extra steel is not required for trimming around the pit opening.

Table 5.9 Spacing of steel reinforcement bars for concrete slabs[a]

Slab	Steel bar	Spacing of steel bars (mm) for minimum slab span of				
thickness	Dia. (mm)	1m	1.25m	1.5m	1.75m	2m
65	6	150	150	125	75	50
	8	250	250	200	150	125
80	6	150	150	150	125	75
	8	250	250	250	200	150

5.23 VIP Latrine

5.23.1 Vent pipes

The vent pipe, i.e., the tube connecting the latrine pit to the open air above the pit, serves two purposes: (I) to create a draught of air from the superstructure, through the squat hole and out of the pit, passing up the vent; (2) to act as a light source which will attract flies to the screen trap which is attached to the top of the vent. Normally the vent pipe is straight and rises vertically above the pit so that the daylight at the top can be seen directly by any flies in the pit (Fig. 5.17). A straight pipe also maximizes the air flow; bends in the vent absorb part of the energy in the air movement.

Fig. 5.17 Straight vent pipe

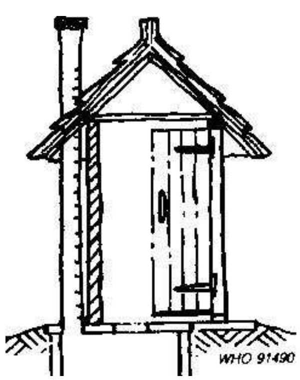
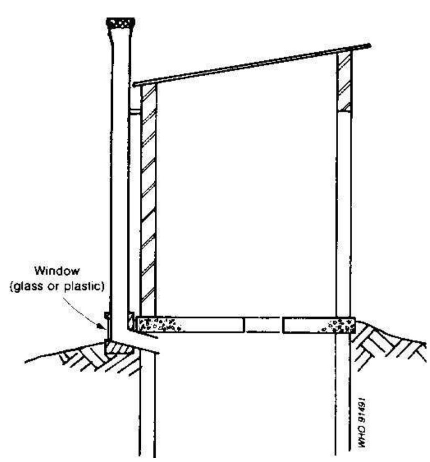


Fig. 5.18 Angled vent pipe with window



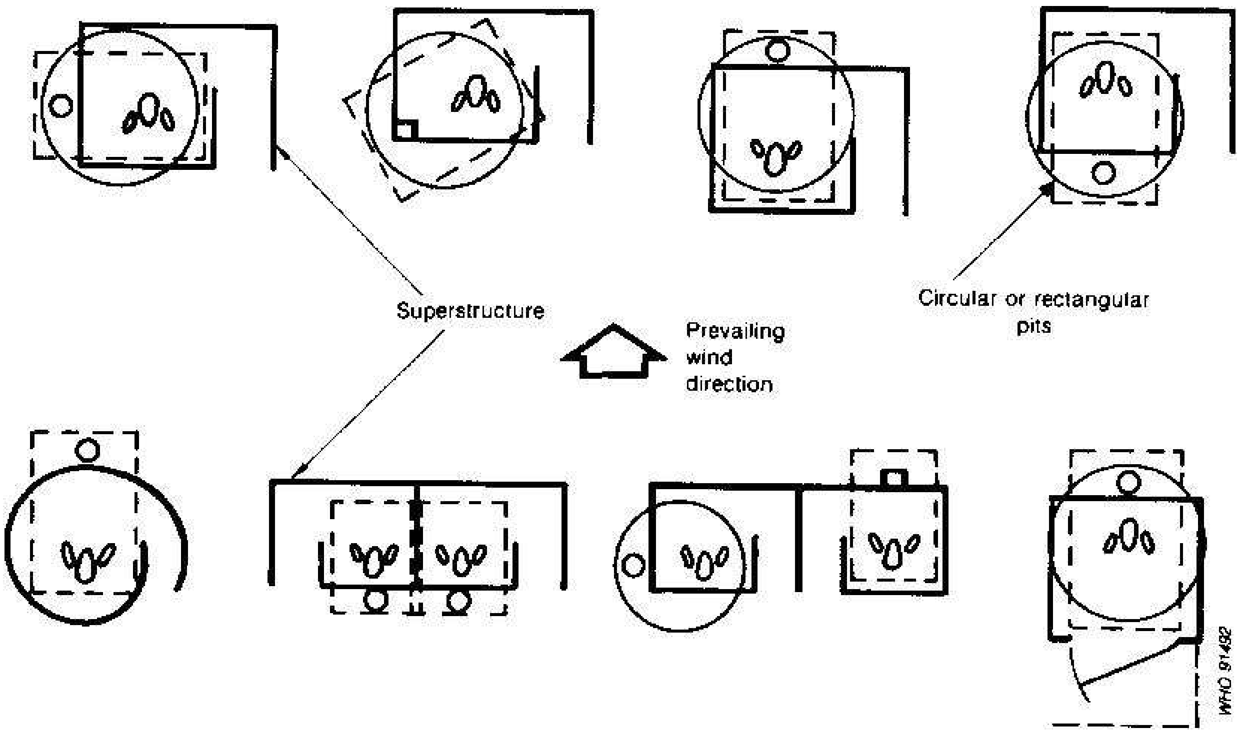
With certain types of slab, or where existing slabs require upgrading with a vent, there may be a need to bring the pipe out horizontally underneath the slab before turning to the vertical. In this situation an ancillary light source is required in the form of a glass or perspex window at the bend (Fig. 5.18). Flies in the pit are first attracted to the light source at the window. They cannot escape from the vent at that point so, following the air flow upwards, they then go towards the light at the top of the vent.

The draught through the vent is created primarily by the movement of wind across the top of the pipe. This air movement creates a suction effect, sucking air out of the pit and up the vent. To achieve satisfactory air movement, the top of the vent should be at least 500 mm above the highest part of the roof, except where the roof is conical, in which case the pipe should reach at least the height of the roof apex. However, if the pipe can be extended even higher, a stronger updraught will be created in the vent. Wind speed increases even at slightly higher elevations above the ground, which creates a stronger suction effect. Also, the higher the vent, the less likely it is to be shielded by buildings or other obstructions which may cause air turbulence and reduce or even reverse the updraught in the vent. Any large trees or overhanging branches close to the vent may significantly affect air movement and thus reduce the effectiveness of the ventilated latrine. Similarly, a rain cowl should not be placed on top of the vent, as it will reduce the air flow; the amount of rain entering the pit is not likely to be significant.

The vent should therefore be located in the best position to catch any air movements across the upper end of the pipe. Vent pipes are normally placed outside the superstructure, particularly where the building materials available make it difficult to construct a watertight joint where the pipe would pass through the roof. Free-standing pipes may be secured to the wall of the superstructure using standard pipe fittings, strips of galvanized steel, galvanized wire or other noncorrosive material. Where possible, the vent should be located on the side of the building which faces the equator that is the side which receives most sunlight. The warming of the surface of the vent pipe raises the temperature of the air in the pipe, increasing the upward draught. Painting the vent black aids this thermal effect. However, the air movement over the top of the vent is the most significant factor in causing updraught and a vent placed inside the building will still work effectively.

The updraught may also be increased by using a spiral design for the superstructure, which funnels the air into the structure. If there are no other ventilation holes, this produces a positive pressure inside the structure, thus forcing air through the squat hole and the pit and up the vent. However, where the winds are particularly variable and often blow from a direction away from the superstructure opening, a negative pressure may be created which will suck foul air out of the pit and into the building (Fig. 5.19).

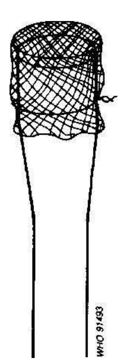
Fig. 5.19 Layouts for superstructures, vent pipes and pits



Dimensions of the vent pipe

Vents may be square or round and can be constructed from a wide variety of materials.
Circular vent pipes should normally have an internal diameter of at least 150 mm for
smooth materials (PVC or asbestos cement) or 230 mm for rough surfaces (such as locally produced cement-rendered pipes), although in exposed places with high wind speeds a smaller diameter may be sufficient. It is normally advantageous to enlarge the top of the vent pipe by about 50 mm to account for the head losses, that is, the reduction in energy and therefore in updraught caused by the air passing through the fine mesh of the flyscreen (Fig. 5.20). There is a danger that cobwebs, dirt or insect matter may build up on the screen, restricting air flow. Belling the top of the pipe can serve to balance these restrictions.

Fig. 5.20 Belled vent with fly screen



Materials

Materials suitable for vent pipes include asbestos cement, unplasticized PVC, bricks, blocks, hollowed-out bamboo, ant-hill soil, cement rendered reeds or bamboo, and cement-rendered hessian. The choice of material will need to take into account durability, availability of materials and skills, cost, and availability of funds. Ordinary PVC becomes brittle when exposed to strong sunlight, so material with a special stabilizer should be used if possible. Because galvanized steel corrodes in a humid atmosphere, the use of thin sheets is not recommended for vent pipes except in very dry climates.

Brick and block chimneys

Vent pipes may be made from bricks or blocks with cement mortar joints in the form of a chimney that is at least 230 mm^2 internally. The flyproof screen should be stretched over the top surface of the highest bricks. If it is built into the course joint one brick down, a receptacle is created which catches leaves and other debris. The chimney may be free-standing or built into the corner of the superstructure.

Locally made vent pipes

Reeds, poles, thin bamboo or strips of 1020 mm of large bamboo can be tied together with wire or string to make a mat which forms a base for cement mortar. The mat, about 2.5 x 1.0 m, is rolled round rings made of green sticks to form a tube about 300 mm in diameter. Flyproof netting is fixed over one end of the tube, which is then laid on the ground. The upper part of the pipe is covered with a layer of cement mortar made with one part of cement to three parts of sand. When the mortar has dried the tube is put in position with the mortared part against the wall of the latrine. Then the outer part of the pipe is plastered with cement mortar. Alternatively the pipe may be rotated on the ground and completely plastered before erection.

A vent pipe can also be made with hessian. First, a 250-mmdiameter tube is formed of spot-welded steel mesh made of 4-mm bars at 100-mm centres (100 mm apart, centre to centre). Hessian or jute cloth is stitched tightly round the outside of the tube and flyproof netting is stitched over one end. Cement mortar, made of one part of cement to two parts of sand, is then brushed over the tube in several layers until a total thickness of about 10 mm is formed. The vent pipe is then fixed in place. Alternatively, a pipe may be made from ferrocement with three or four layers of mesh plastered with cement mortar and without any hessian.

Fly screens

Fly screens should be made of material that will not be affected by temperature, sunlight, or the corrosive gases that are vented from the pit. Stainless steel or aluminium is considered to be best. Their comparatively high cost may be justified by their long life, especially as the screen accounts for a very small proportion of the total cost of the latrine. PVC-coated glass-fibre netting is relatively cheap.. However, it tends to become brittle

Ordinary plastic screens deteriorate quickly in sunlight. Painted mild steel mesh, commonly sold as window screening against mosquitos, and galvanized mild steel mesh last only a few months before corrosion by the pit gases renders them ineffective. Gases and sunlight weaken the screens but the actual tearing of the material is assumed to be caused by birds alighting or possibly by lizards which frequent the top of rough-walled vent pipes or simply by the tension within the flexing screen

5.24 Septic Tank

The construction of a septic tank usually requires the assistance and supervision of an engineer or at least an experienced construction foreman. The design of the inlet and outlet is critical to the performance of the tank. Careful checking of levels is particularly important for large tanks that include complicated inlet, outlet and baffle board arrangements.

For small household tanks, the floor is usually made of unreinforced concrete thick enough to withstand uplift pressure when the tank is empty. If the ground conditions are poor or the tank is large, the floor may have to be reinforced. The walls are commonly built of bricks, blocks or stone and should be rendered on the inside with cement mortar to make them watertight. Large reinforced concrete tanks serving groups of houses or institutions must be designed by a qualified engineer to ensure that they are structurally sound.

The tank cover or roof, which usually consists of one or more concrete slabs, must be strong enough to withstand any load that will be imposed.

Removable cover slabs should be provided over the inlet and outlet. Circular covers, rather than rectangular ones, have the advantage that they cannot fall into the tank when removed.

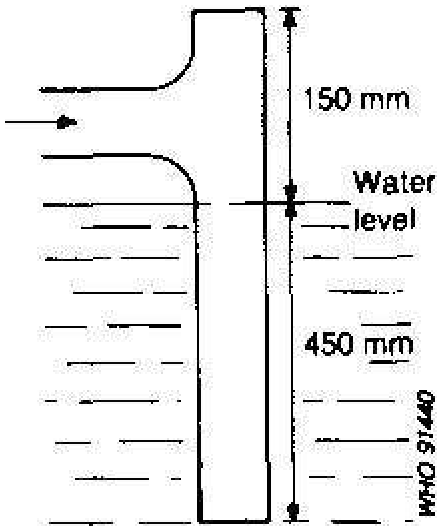
Septic tanks have been constructed from a variety of prefabricated sections, including large-diameter pipes. Experience has shown that the problems involved in fixing the inlet and outlet outweigh the advantages of using pipes. A number of proprietary designs of tank are manufactured from asbestos cement, glass-reinforced plastic and other materials and are sold commercially.

Inlet

The sewage must enter the tank with the minimum possible disturbance to the liquid and solids already in the tank. Surges and turbulence reduce the efficiency of settlement and can cause large amounts of solid matter to be carried out in the tank effluent. Suitable inlet arrangements are shown in Fig. 5.21 and 5.24

Surges are caused by flushing of the WC and emptying of sinks and baths. Their effect can be minimized by using drainpipes of not less than 100 mm in diameter and ensuring that the gradient of the pipe approaching the septic tank is flatter than about I in 66. Sizes and gradients of pipes between the building and the septic tank may be specified in local building regulations.

Fig. 5.21 Septic tank Inlet pipe



Outlet

For septic tanks less than 1.2 m wide, a simple T-pipe arrangement can be used for the outlet. A removable cover above the T-pipe should be provided to permit clearance of any blockage. An alternative to the T-pipe is a baffle plate made of galvanized sheet, ferrocement or asbestos cement fitted round the outlet pipe (Fig. 5.22). A deflector may be provided below the outlet to reduce the possibility of settled sludge being resuspended and carried out of the tank. For tanks wider than 1.2 m, a full-width weir can be used to draw off the flow evenly across the tank. A scum board should be fitted to prevent the scum washing over the weir (Fig. 5.23).

Fig. 5.22 Septic tank outlet baffle plate

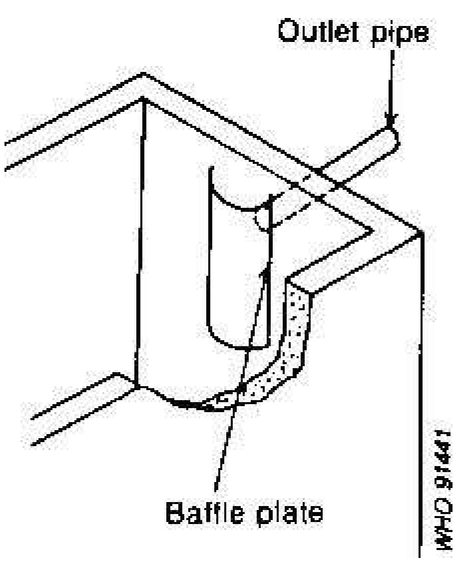
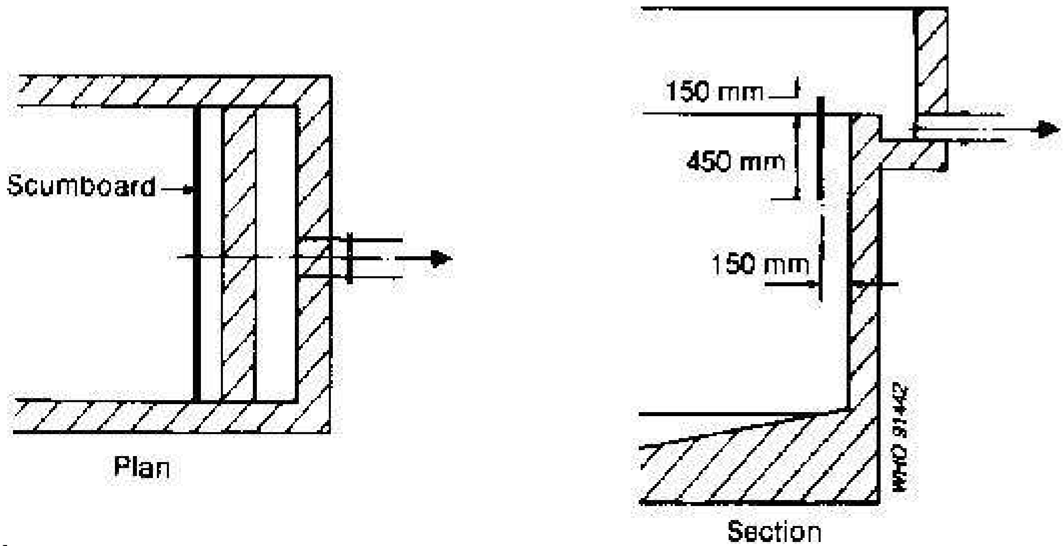


Fig. 5.23 Septic tank outlet using full width weir



Dividing wall

If a tank is divided into two or more compartments, slots or a short length of pipe should be provided above the sludge level and below the scum level, as shown in Fig. 524. At least two should be installed to maintain uniform flow distribution across the tank.

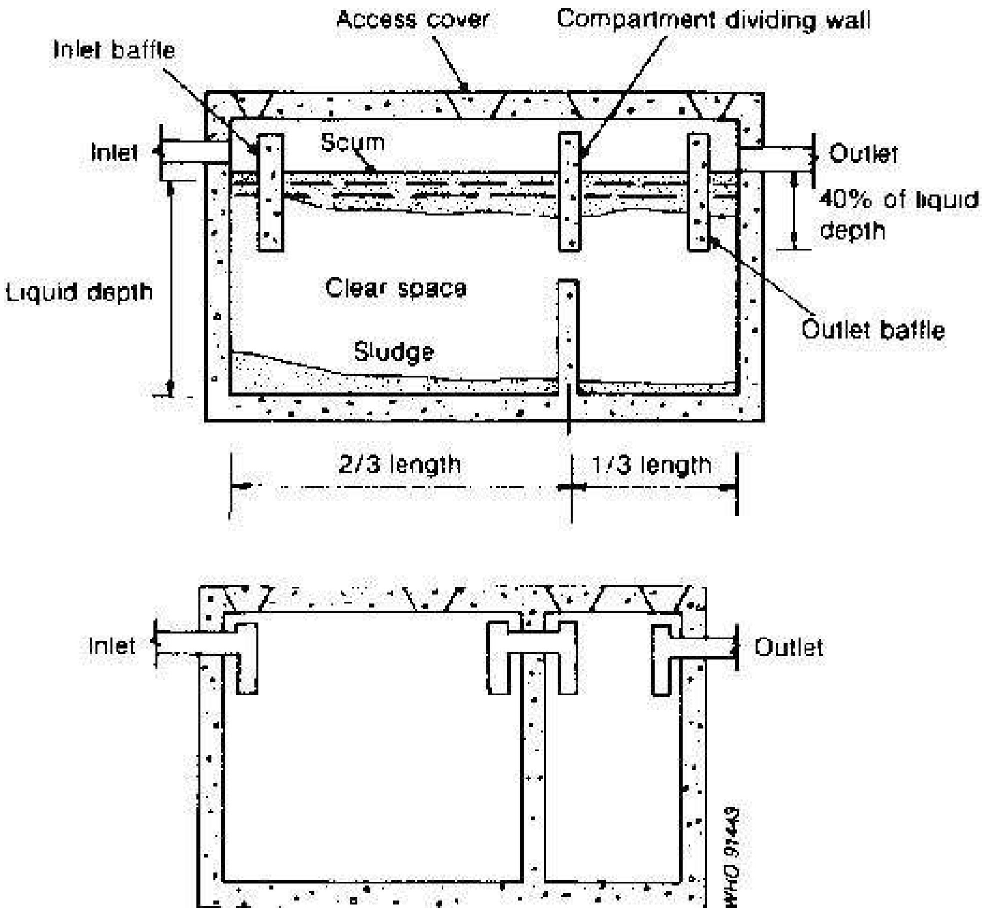
Ventilation of the tank

The anaerobic processes that occur in the tank produce gases which must be allowed a means of escape. If the drainage system of the house or other building has a ventilation pipe at the upper end, gases can escape from the septic tank along the drains. If the drainage system is not ventilated, a screened vent pipe should be provided from the septic tank itself.

The tank floor

Some codes of practice recommend that the floor of a septic tank should slope downwards towards the inlet. There are two reasons: firstly, more sludge accumulates near the inlet, so a greater depth is desirable; secondly, the slope assists movement of sludge towards the inlet during desludging. For a two-compartment tank, the second compartment should have a horizontal floor and the first compartment may slope at a gradient of 1 in 4 towards the inlet. When calculating the tank volume, it should be assumed that the floor is horizontal at the higher level. The effect of sloping the floor provides extra volume. The disadvantages of providing a sloping floor are that additional depth of excavation is required, the construction is made more complicated, and the cost of construction is increased.

Fig. 5.24 Septic tanks showing options for connections between compartments



5.1 Operation and Maintenance

5.1.1 Pit latrines

The principle of all types of pit latrine is that wastes such as excrete, anal cleaning materials, sullage and refuse are deposited in a hole in the ground. The liquids percolate into the surrounding soil and the organic material decomposes producing:

- gases such as carbon dioxide and methane, which are liberated to the atmosphere or disperse into the surrounding soil;
- liquids, which percolate into the surrounding soil;
- A decomposed and consolidated residue.

In one form or another, pit latrines are widely used in most developing countries. The health benefits and convenience depend upon the quality of the design, construction and maintenance. At worst, pit latrines that are badly designed, constructed and maintained provide foci for the transmission of disease and may be no better than indiscriminate defecation. At best, they provide a standard of sanitation that is at least as good as other more sophisticated methods.

Simplicity of operation and construction, low construction cost, the fact that they can be built by householders with a minimum of external assistance, and effectiveness in breaking the routes by which diseases are spread, are among the advantages that make pit latrines the most practical form of sanitation available to many people. This is especially true where there is no reliable, continuous and ample piped water supply.

Unfortunately, past failures, especially of public facilities, discourage some sanitation field workers from advocating their widespread use. Objections to the use of pit latrines are that poorly designed and poorly constructed latrines produce unpleasant smells, that they are associated with the breeding of undesirable insects (particularly flies, mosquitos and cockroaches), that they are liable to collapse, and that they may produce chemical and biological contamination of groundwater. Pit latrines that are well designed, sited and constructed, and are properly used need not have any of these faults.

5.2

5.2.1 Emptying pits

The emptying of single pits containing fresh excretes presents problems because of the active pathogens in the sludge. In rural areas, where land availability is not a constraint, it is often advisable to dig another pit for a new latrine. The original pit may then be left for several years and when the second is filled it may be simplest to re-dig the first pit rather than to excavate a new hole in hard ground. The sludge will not cause any health problems and is beneficial as a fertilizer. However, in urban areas, where it is not possible to excavate further holes and where the investment in pit-lining and superstructure has been substantial, the pit must be emptied.

From the public health point of view, manual removal should be avoided. Where the groundwater level is so high that the pit is flooded or where the pit is sealed and fitted with an effluent overflow, the wet sludge can be removed by ordinary vacuum tankers. These tankers are the same as those used for emptying septic tanks

5.2.2 Ventilated pit latrines

These are also known as ventilated improved pit (VIP) latrines. The major nuisances that discourage the use of simple pit latrines - smell and flies - are reduced or eliminated through the incorporation of a vertical vent pipe with a flyscreen at the top Wind passing over the top of the vent pipe causes a flow of air from the pit through the vent pipe to the atmosphere and a downdraught from the superstructure through the squat hole or seat into the pit. This continuous flow of air removes smells resulting from the decomposing excrete in the pit and vents the gases to the atmosphere at the top of the vent pipe rather than through the superstructure. The flow of air is increased if the doorway of the superstructure faces the prevailing wind. If a door is fitted it should be kept shut at all times (except when entering or leaving) to keep the inside of the latrine reasonably dark, but there should be a gap, normally above the door, for air to enter. The area of this gap should be at least three times the cross- sectional area of the vent pipe.

In addition to removing odours from the pit, the screened vent pipe significantly controls flies. Flies are attracted to the pit by the odour coming from the vent pipe but are unable to enter because of the screen. A few flies enter the pit through the squat hole or seat, and lay eggs in the pit. New young flies attempt to leave the pit by flying towards the light. If the latrine superstructure is kept sufficiently dark, the major source of light is at the top of the vent pipe, but the screen prevents the flies from escaping there and they eventually fall back into the pit to die.

Well-constructed and maintained VIP latrines combat all the problems associated with simple pit latrines, except mosquitos. However, they are considerably more expensive than simple pits, since a ventilation pipe and full superstructure are required. Because the defecating hole is directly over the pit they accept any form of anal cleaning material without blocking. Routine operation is limited to keeping the superstructure clean, ensuring that the door (where fitted) is kept closed, occasionally checking that the fly-proof netting on top of the vent pipe is not blocked or broken, and pouring water down the vent pipe once a year to remove spiders' webs.

5.2.3 Ventilated double-pit latrines

Although it is usually best to provide large deep pits, this may not be possible where rock or groundwater lies within one or two metres of the ground surface. A variation of the VIP latrine suitable for such situations has two shallow pits side by side under a single superstructure. The pits are usually lined with bricks or blocks. Each pit may have its own squat hole or seat. Alternatively, slabs may be movable, one with a hole for the pit in use and a plain slab for the other pit. Whichever design is used, only one hole must be available for defecation at any time. The latrine may be provided with two ventilation pipes (one for each pit) but more usually only one is fitted, to the pit in use.
The hole for the ventilation pipe for the pit not in use is sealed. As with single VIP latrines, the superstructure must be kept partially dark at all times to discourage flies.

5.2.4 Operation

One pit is used until it is filled to within about half a metre of the top. The defecation hole over the full pit is then sealed and the one over the empty pit opened. Where necessary, the ventilation pipe is moved from the full to the empty pit, and the vent hole in the slab of the full pit sealed. The second pit is then used until filled to within half a metre of the top. The contents of the first pit can now be removed and the pit reused. The pits must be large enough to allow each pit to be used for at least two years. This ensures that when the pit contents are dug out most of the pathogenic organisms have died.

Double-pit latrines can be considered as permanent installations. The small effective capacity (0.72 m^3 for a family of six, using a sludge build-up rate of 60 litres per person per year, enables pits to be relatively shallow, and therefore easier to empty than deep pits. The pits should extend beyond the superstructure, either to the sides or at the back, with removable slabs for emptying. These slabs should be easy to lift, but should be sealed to prevent flies getting in or out. The central wall between the two pits should be made with full mortar joints and may be rendered with cement mortar on both sides.

As with the single-pit VIP latrine, the double-pit VIP latrine has the advantages of reduced smell and fly nuisance. Also the contents of the latrine dug out every two years or so are valuable soil conditioners. Double-pit VIP latrines are usually (but not always) more expensive than single-pit VIP latrines, and require a greater operational input from the user, particularly in changing over pits. Some societies have shown resistance to handling the decomposed contents of the pit but this can often be overcome with education and time. Allowing people to see (and handle) the contents of a pit as it is emptied is the strongest persuader for those concerned.

All projects involving the construction of double-pit latrines must allow for a prolonged support programme. Householders need to be reminded to change pits at the right time and should be assisted in doing so. This assistance will probably have to be available for at least the first two pit changes to ensure that the complete cycle is covered.

5.2.5 Pour-flush latrines

The problems of flies, mosquitos and smell in simple pit latrines may be overcome simply and cheaply by the installation of a pan with a water seal in the defecating hole The pan is cleared by pouring (or, better, throwing) a few litres of water into the pan after defecation.

The amount of water used varies between one and four litres depending mainly on the pan and trap geometry. Pans requiring a small amount of water for flushing have the added advantage of reducing the risk of groundwater pollution. The flushing water does not have to be clean. If access to clean water is limited, laundry, bathing or any other similar water may be used.

Pour-flush latrines are most appropriate for people who use water for anal cleaning, and squat to defecate, but they have also proved popular in countries where other cleaning materials are common. However, there is a likelihood of blockage where solid materials such as hard paper or corncobs are put in the pan. The placing of solid cleaning materials in a container for separate disposal is not generally recommended unless careful attention can be given to the handling of the waste and sterilizing of the container. Blockage may also be caused by material used by menstruating women. This should be disposed of separately, e.g., by burying or burning. Efforts to clear blockages often result in damage to the water seal.

In most cases, because of the small quantity of water required for flushing, pour-flush latrines are suitable where water has to be carried to the latrine from a standpipe, well, or other water source. There is no justification for the belief that the pit should be ventilated to prevent the build up of gases. A vent pipe adds to the cost of the latrine and any gases produced easily percolate into the surrounding soil.

5.2.6 Septic tanks

Septic tanks are commonly used for wastewater treatment for individual households in low-density residential areas, for institutions such as schools and hospitals, and for small housing estates. The wastewater may be waste from toilets only, or may also include sullage.

The septic tank, in conjunction with its effluent disposal system, offers many of the advantages of conventional sewerage. However, septic tank systems are more expensive than most other on-site sanitation systems and are unlikely to be affordable by the poorer people in society. They also require sufficient piped water to flush all the wastes through the drains to the tanks.

Treatment processes

Wastes from the toilet, and possibly kitchens and bathrooms, pass through drains into a sealed, watertight tank, where they are partially treated. After a period - usually 1-3 days- the partially treated liquid passes out of the tank and is disposed of, often to the ground through soakpits or tile drains in trenches.. Many of the problems with septic tank systems arise because inadequate consideration is given to the disposal of the tank effluent.

Settlement

A principal aim of septic tank design is to achieve hydraulically quiescent conditions within the tank to assist the settlement by gravity of heavy solid particles. The settled material forms a layer of sludge on the bottom of the tank which must be removed periodically. The efficiency of removal of solids by settlement can be high. Majumder et al. (1960) reported removal of 80% of suspended solids in three tanks in West Bengal; similar removal rates were reported in a single tank near Bombay (Phadke et al., undated). However, much depends upon the retention time, the inlet and outlet arrangements, and the frequency of desludging. Large surges of flow entering the tank may cause a temporarily high concentration of suspended solids in the effluent owing to disturbance of the solids which have already settled out.

Flotation

Grease, oil, and other materials that are less dense than water float up to the liquid surface, forming a layer of scum which can become quite hard. The liquid moves through the tank sandwiched between the scum and sludge.

Sludge digestion and consolidation

Organic matter in the sludge and scum layers is broken down by anaerobic bacteria witha considerable amount of organic matter being converted into water and gases. Sludge at the bottom of the tank is consolidated owing to the weight of liquid and solids above. Hence the volume of sludge is considerably less than that of raw sewage solids entering the tank. Rising bubbles of gas cause a certain amount of disturbance to the liquid flow.

The rate at which the digestion process proceeds increases with temperature, a maximum rate being achieved at about 35 °C. The use of ordinary household soap in normal amounts is unlikely to affect the digestion process (Truesdale & Mann, 1968). The use of abnormally large amounts of disinfectant causes bacteria to be killed off and thereby inhibits the digestion process.

Stabilization of liquids

The liquid in the septic tank undergoes biochemical changes, but there are few data on the removal of pathogens. Both Majumder et al. (1960) and Phadke et al. (undated) found that although 80-90% of hookworm and Ascaris eggs were removed by the septic tanks studied, in absolute terms very large numbers of viable eggs were contained in the effluent, with 90% of effluent samples containing viable eggs.

Since the effluent from septic tanks is anaerobic and likely to contain large numbers of pathogens which can be a potential source of infection, it should not be used for crop irrigation nor should it be discharged to canals or surface-water drains without the permission of the local health authority.

Design principles

The guiding principles in designing a septic tank are:

- to provide sufficient retention time for the sewage in the tank to allow separation of solids and stabilization of liquid;
- to provide stable quiescent hydraulic conditions for efficient settlement and flotation of solids;
- to ensure that the tank is large enough to store accumulated sludge and scum;
- to ensure that no blockages are likely to occur and that there is adequate ventilation of gases

Liquid retention volume

If the septic tank accepts sullage as well as toilet waste, the sewage flow from a house or institution usually represents a high proportion of the water supplied. If the water supply per person is known, the sewage flow may be taken as 90% of the water supply. If the water supply exceeds about 250 litres per person per day, the excess is likely to be used for watering gardens. In most developing countries, the maximum sewage flow may be assumed to be between 100 and 200 litres per person per day.

If only WCs are connected to the septic tank, the sewage flow is estimated from an assumption about the number of times each user is likely to flush the WC. For example, each person may flush a 10-litre cistern four times a day.

The minimum capacity required for 24 hours' liquid retention is:

A = P x q litres
Where;

A = required volume for 24 hours' liquid retention;
P = number of people served by the tank;
q = sewage flow per person (litres per person per day).

Volume for sludge and scum storage
The volume required for the accumulation of sludge and scum is obtained from the following formula:

B = P x N x F x S
Where;

B = the required sludge and scum storage capacity in litres;
N = the number of years between dislodging (often 2-5 years; more frequent dislodging may be assumed where there is a cheap and reliable emptying service);
F = a factor which relates the sludge digestion rate to temperature and the dislodging interval,
S = the rate of sludge and scum accumulation which may be taken as 25 litres per person per year for tanks receiving WC waste only, and 40 litres per person per year for tanks receiving WC waste and sullage.

5.3 Starting up the tank

The process of anaerobic digestion of the sewage solids entering the tank can be slow in starting and it is a good idea to "seed" a new tank with sludge from a tank that has been operating for some time. This ensures that the necessary microorganisms are present in the tank to allow the digestion process to take place in a short time

5.3.1 Maintenance

Routine inspection is necessary to check whether desludging is needed, and to ensure that there are no blockages at the inlet or outlet. A tank needs to be desludged when the sludge and scum occupy the volume specified in the design. A simple rule is to desludge when solids occupy between one-half and two-thirds of the total depth between the water level and the bottom of the tank. One of the difficulties with septic tanks is that they continue to operate even when the tank is almost full of solids. In this situation the inflow scours a channel through the sludge and may pass through the tank in a matter of minutes rather than remaining in the tank for the required retention time.

The most satisfactory method of sludge removal is by vacuum tanker. The sludge is pumped out of the tank through a flexible hose connected to a vacuum pump, which lifts the sludge into the tanker. If the bottom layers of sludge have cemented together they can be jetted with a water hose (which may be fitted to the tanker lorry) or broken up with a long-handled spade before being pumped out.

If a vacuum tanker is not available, the sludge must be bailed out manually using buckets. This is unpleasant work which exposes the operatives to health hazards.

Care must be taken to ensure that sludge is not spilled around the tank during emptying. Sludge removed from a septic tank includes fresh excrete and presents a risk of transmission of diseases of faecal origin. Careful disposal is therefore necessary.

When a septic tank is desludged it should not be fully washed out or disinfected. A small amount of sludge should be left in the tank to ensure continuing rapid digestion.

5.4 Aqua-privies

An aqua-privy is a latrine set above or adjacent to a septic tank and is useful in situations in which there is a limited water supply (Fig. 5.25). Where the latrine is above the tank, a chute drop-pipe, 100 to 150 mm in diameter, hangs below the squat hole or latrine seat so that excrete drops directly into the tank below water level. The bottom of the pipe should be 75 mm below the liquid level in the tank, providing a seal which prevents gases escaping into the latrine superstructure and limits the access of flies and mosquitos to the tank. Alternatively the toilet may be fitted with a pan with a water seal.

Where the latrine is adjacent to the tank, the pan with water seal is connected by a short pipe. Effluent from the tank goes to a soakpit, drainage trench or sewer. There is usually only a small flow of effluent and it is therefore very concentrated.

Fig. 5.25 Aqua-privy

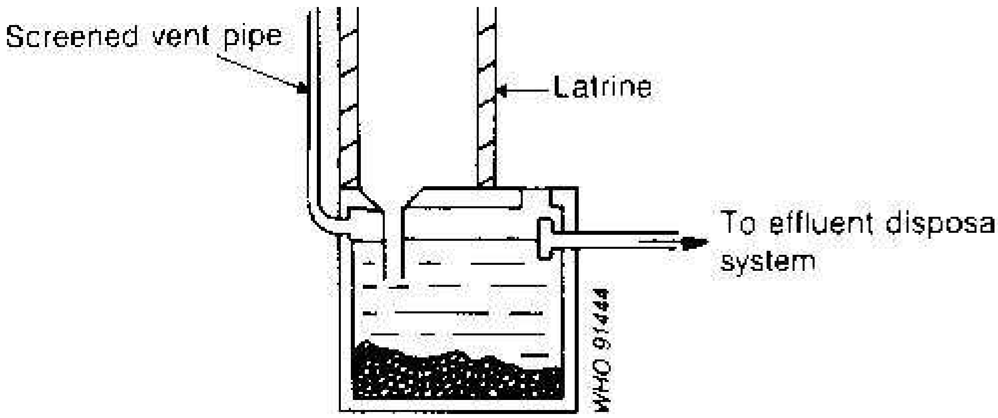
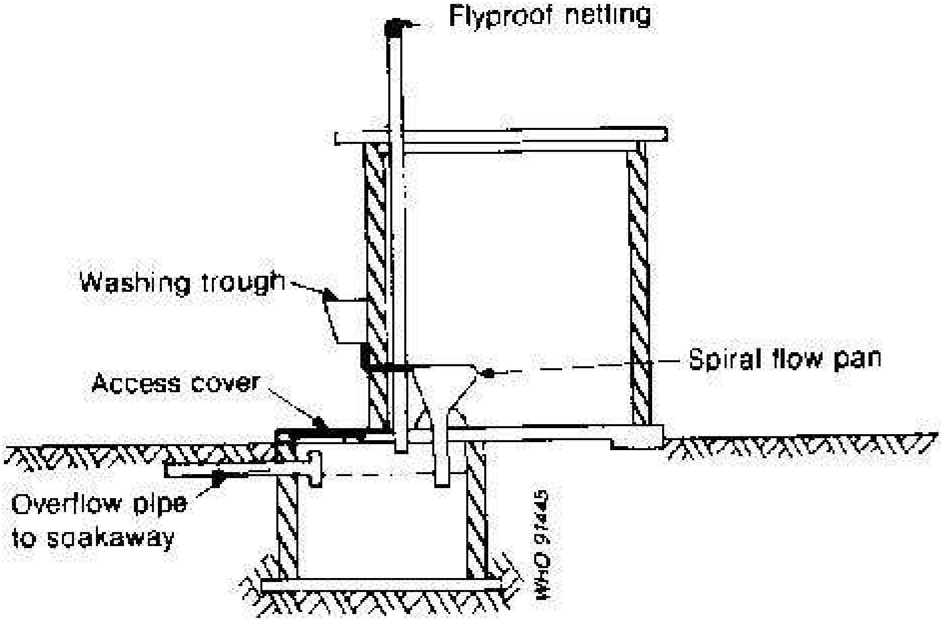


Fig. 5.26 Aqua-privy with pan flushed by waste from a washing trough



In order to keep a seal at the bottom of the drop-pipe it is essential that the water level in the tank is maintained. If the tank is completely watertight, a bucketful of water every day, used to clean the latrine, is sufficient to compensate for any losses due to evaporation. However, it has been found in practice that many tanks leak. In some places sullage is discharged into the tank (Fig. 5.26), but even this has not proved sufficient to ensure that the water level is above the bottom of the drop pipe at all times.

The design capacity of aqua-privy tanks may be calculated by the same procedure as for septic tanks. Regular removal of sludge and scum is essential, so a removable cover for desludging is required. A vent pipe is usually provided.

5.5 Disposal of effluent from septic tanks and aqua-privies

A septic tank or aqua-privy is simply a combined retention tank and digester; apart from losses through seepage and evaporation, the outflow from the tank equals the inflow. The effluent is anaerobic and may contain a large number of pathogenic organisms. Although the removal of suspended solids can be high in percentage terms, the effluent is still concentrated in absolute terms, and the need for safe disposal of septic tank effluents cannot be too strongly stressed.

The effluent from large tanks dealing with sewage from groups of houses or from institutions may be treated by conventional sewage treatment processes such as percolating filters. Effluent from septic tanks and aqua-privies serving individual houses is normally discharged to soakpits or drainage trenches for infiltration into the ground.
Unfortunately it is not possible to predict the useful life of such disposal systems, which depend on the efficiency of the septic tank and the soil conditions. Pools of stagnant liquid often form when both toilet wastes and sullage are discharged to a septic tank and then to a drainage field which is too small or is clogged. This creates a potential health risk. Overloading of the drainage field may be avoided by allowing only toilet wastes to go to the septic tank. Sullage can be dealt with separately with fewer health risks than a mixture of partly treated toilet waste and sullage. A three-compartment septic tank, where sullage is introduced into the final compartment should be used since the effluent infiltration rates may be double those for two-compartment tanks.

5.6 Soakpits

Pits used to dispose of effluent from septic tanks are commonly 2-5 m deep with a diameter of 1.0-2.5 m. The capacity should be not less than that of the septic tank.

Depending on the nature of the soil and the local cost of stone and other building material, soakpits may either be lined or filled with stones or broken bricks. Linings are generally made of bricks, blocks or masonry with honeycomb construction or open joints (Fig. 5.27), as for the linings of pit latrines. The infiltration capacity of the soil may be increased by filling any space behind the lining with sand or gravel hard material such as broken rock or broken kiln-dried bricks not less than 50 mm in diameter may be used to fill an unlined pit (Fig 5.28).

Whether the main part of the pit is lined or filled, the top 500 mm should have a ring of blocks, bricks or masonry with full mortar joints to provide a firm support for the cover. The ring may be corbelled to reduce the size of the cover. Covers are usually made of reinforced concrete and may be buried by 200-300 mm of soil to keep out insects.

Increasing the diameter of the pit results in a disproportionate increase in the volume of excavation and in the cost of the cover slab compared with the increase of wall area. Therefore, if the required infiltration area is large, it may be more economical to provide drainage trenches.

Fig. 5.27 Lined soakpit

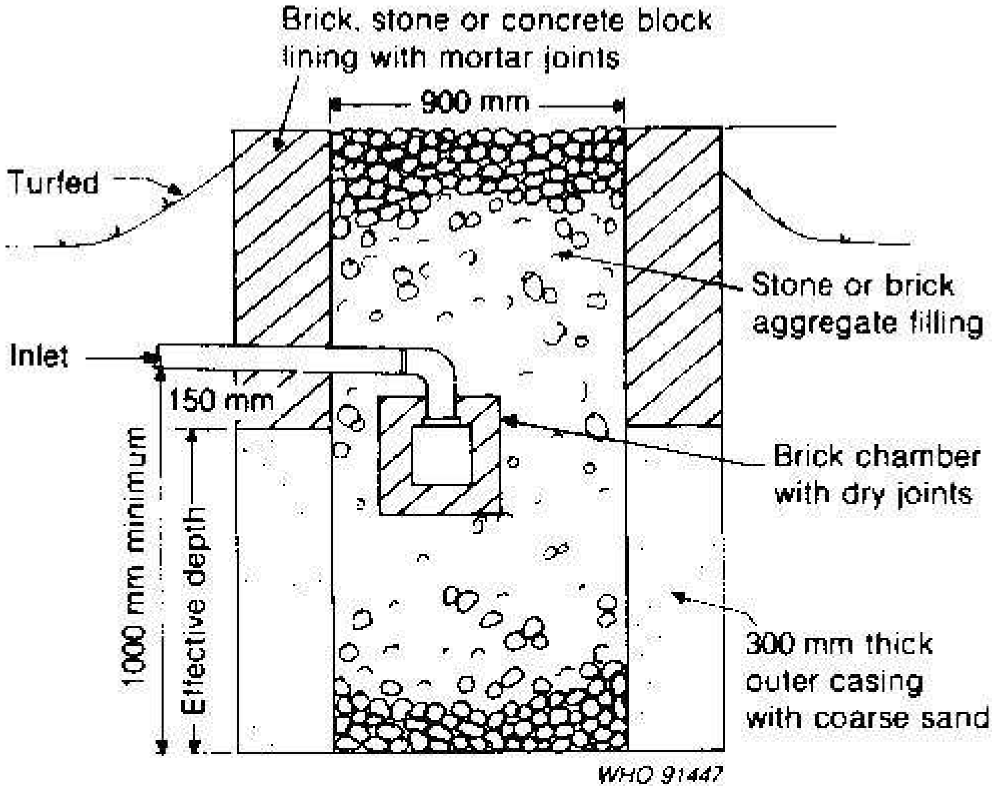
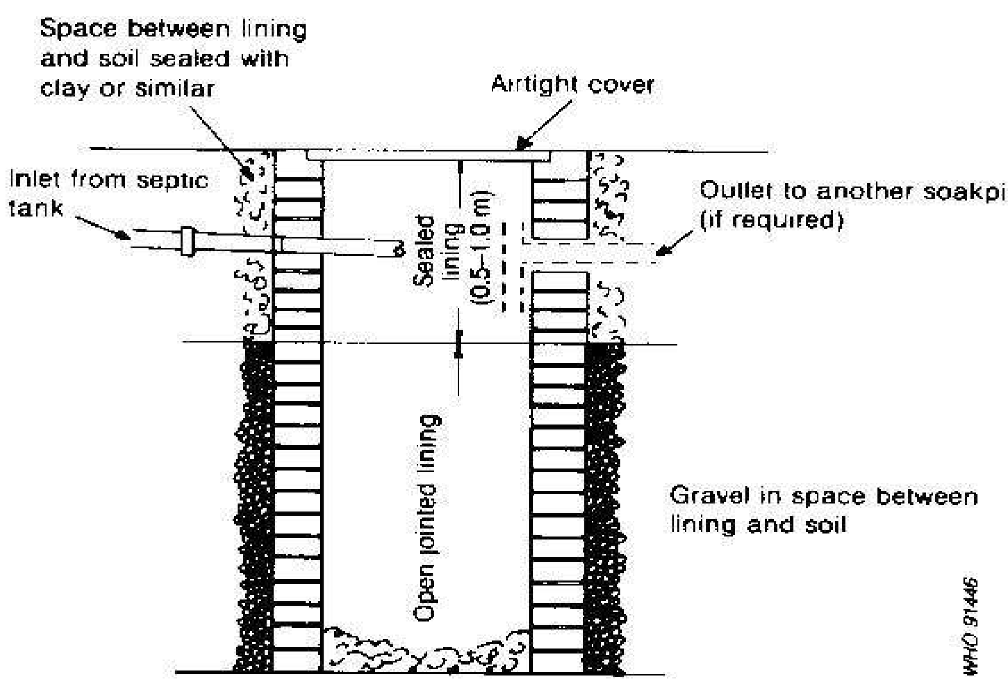


Fig. 5.28 Unlined Soak pit

5.7



5.8 Drainage trenches

The disposal of the large quantity of effluent from septic tanks is often effected in trenches which disperse the flow over a large area, reducing the risk of overloading at one place. The trenches make up a drainage field. The effluent is carried in pipes which are normally 100 mm in diameter with a gap of about 10 mm between each pipe. Unglazed stoneware pipes (tile drains) are often used, either with plain ends or with spigot and socket joints. The upper part of the gap between plain end pipes may be covered with strips of tarred paper or plastic sheet to prevent entry of sand or silt. With spigot and socket pipes, a small stone or cement fillet can be placed on each socket to centre the adjoining spigot (Fig 5.29).

Drainage trenches are usually dug with a width of 300- 500 mm and a depth of 600 1000 mm below the top of the pipes. A common practice is to lay the pipes at a gradient of 0.2-0.3% on a bed of gravel, the stones with a diameter of 20- 50 mm. Soil is returned to a depth of 300 500 mm above the stones, with a barrier of straw or building paper to prevent soil washing down (Fig. 5.30).

If more than one trench is needed it is recommended that the drains be laid in series. Drains in series are either full or empty, allowing the soil alongside empty drains to recover under aerobic conditions (Fig. 3.31). If drains are laid in parallel, there is a tendency for all trenches to contain some effluent. Trenches should be 2 m apart, or twice the trench depth if this is greater than 1 m.

Fig. 5.29 Open pipe joint in a drainage trench

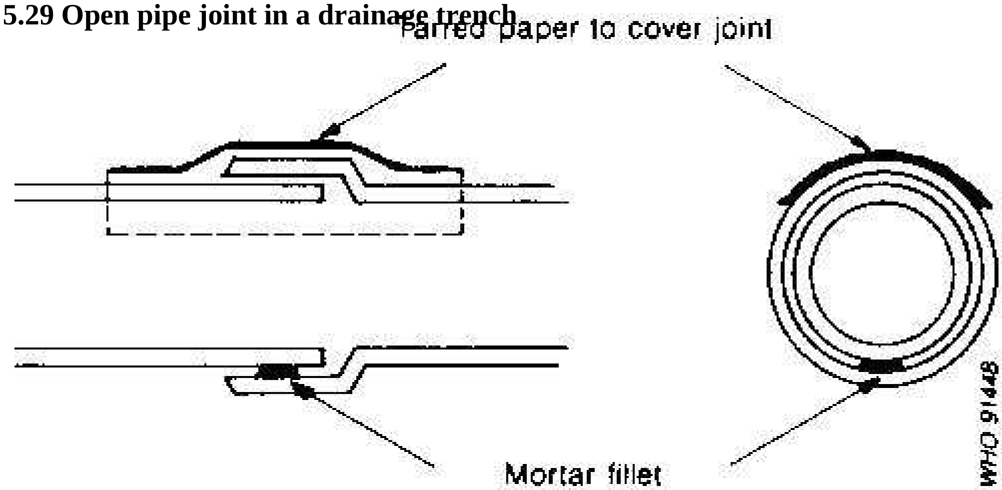


Fig. 5.30 Drainage trench

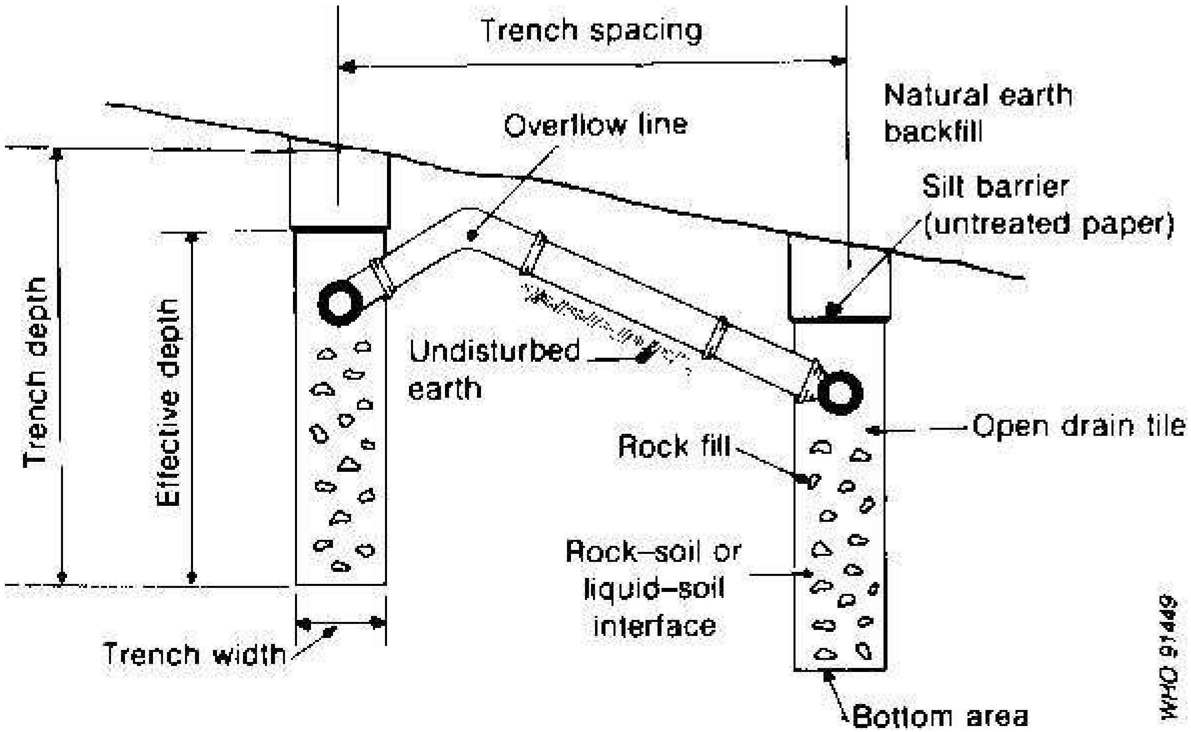
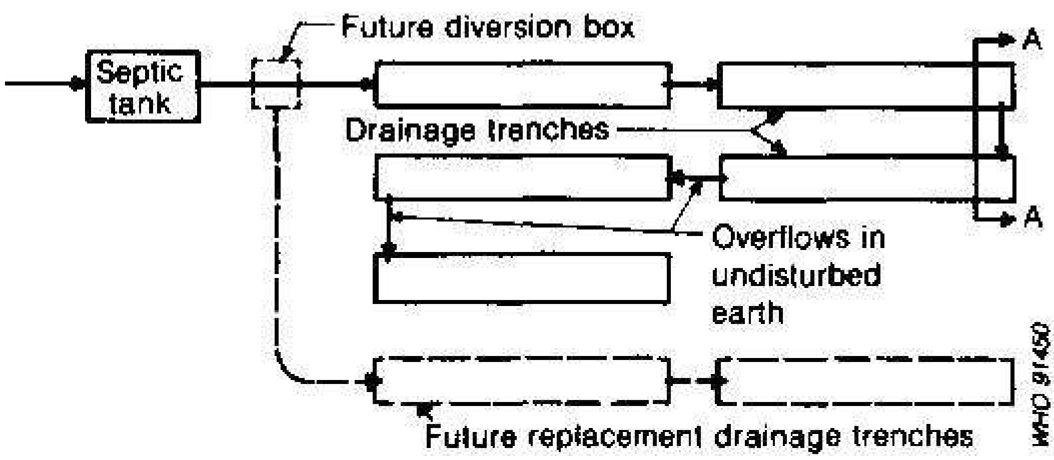


Fig. 5.31 Drainage trenches laid in series in a drainage field. A-A indicates section shown In Fig. 5.30



The length of trench should be calculated by dividing the flow of effluent by the infiltration rate, allowing for the area of both sides of the trench.

5.9 Other latrines

5.9.1 Bucket latrines

The system in which excrete are removed from bucket latrines (also called nightsoil latrines or earth closets) is one of the oldest forms of organized sanitation. Bucket latrines are still found in many towns and cities in Africa, Latin America and Asia, because their low capital cost makes them attractive to underfunded local authorities.

In some rural and periurban areas, members of households take nightsoil to manure heaps or apply it directly to fields as fertilizer. In towns and cities, nightsoil is often collected by sweepers engaged by householders on contract, or by the local authorities. Buckets are usually emptied into larger containers near the latrine. In some places labourers carry these containers by hand or on their heads; hand-carts, animal-drawn carts, bicycles and tricycles are also used.

The number of bucket latrines is declining rapidly. However, for many years to come, some people will have to rely on bucket latrines as their only form of sanitation. The following paragraphs give suggestions for improvements to existing systems until they can be replaced by more acceptable forms of sanitation.

Good operation

A container made of non-corrosive material is placed beneath a squatting slab or seat in the bucket chamber, with rear doors which should be kept shut except during removal and replacement of the bucket.

The bucket chamber should be cleaned whenever the bucket is removed. The squat hole should be covered by a fly proof cover when not in use. The cover of the seat should be hinged (Fig. 5.32) and the cover of the squatting slab should have a long handle.

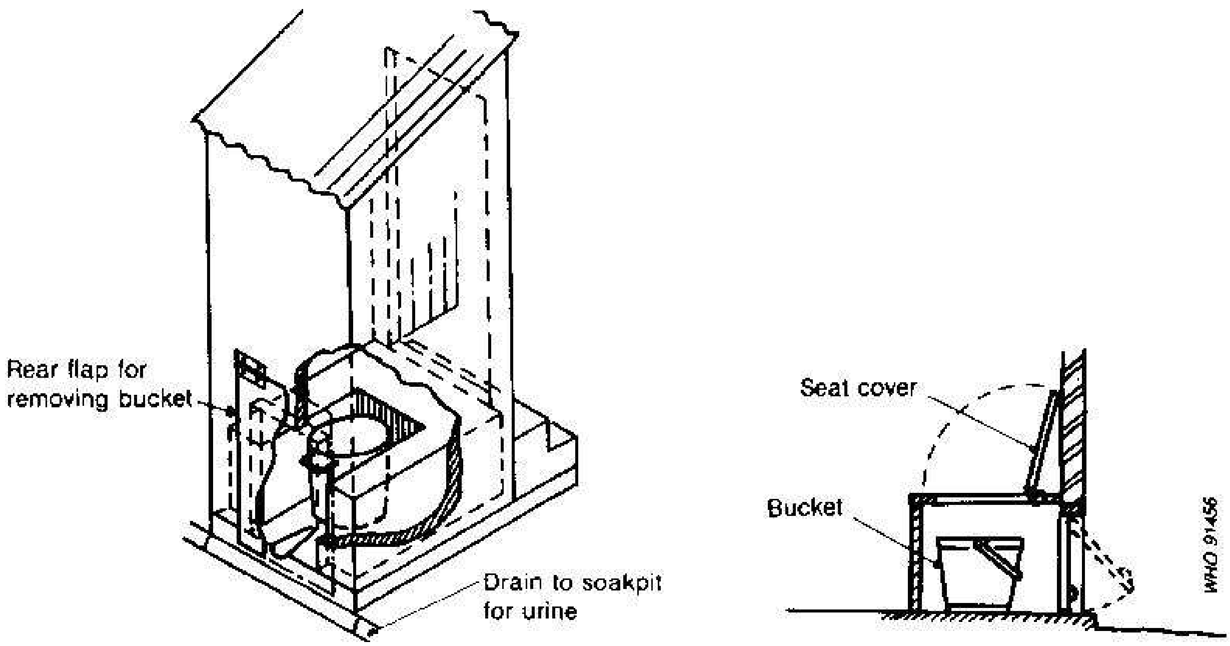
At regular intervals (preferably each night) the container should be removed and replaced by a clean one. Full containers should be taken to depots or transfer stations where they are emptied, washed and disinfected with a phenol or cresol type of disinfectant. In some towns it is the practice to provide two buckets painted in different colours for each latrine. Containers should be kept covered with tight-fitting lids while in transit and the operators should be provided with full protective clothing. Proper supervision and management are essential. Defective buckets should be repaired or replaced and transport vehicles should be kept in good order.

In some systems, urine is diverted away from the buckets to reduce the volume to be dealt with. It is usually channelled to soakpits, but may be collected separately and used directly as fertilizer. Water used for washing latrines and bucket-chambers should pass to soakpits, and should not be allowed to pollute the ground around the latrines.

Disposal methods

The practice of dumping nightsoil indiscriminately into streams or on open land is objectionable and causes health hazards.

Fig. 5.32 Bucket latrine



Sewers

Bucket latrines are sometimes found in towns that are partially provided with sewers, in which case it may be convenient to discharge the nightsoil into a main sewer. Tipping points on sewers require careful design to prevent contamination of surrounding areas and should be as near to the sewage works as possible. Extra water may have to be added to prevent blockage of the sewers.

Sewage treatment works

Nightsoil may be discharged into the sewage flow at the works inlet, at sedimentation or aeration tanks, or directly to waste stabilization ponds or sludge digestion tanks.

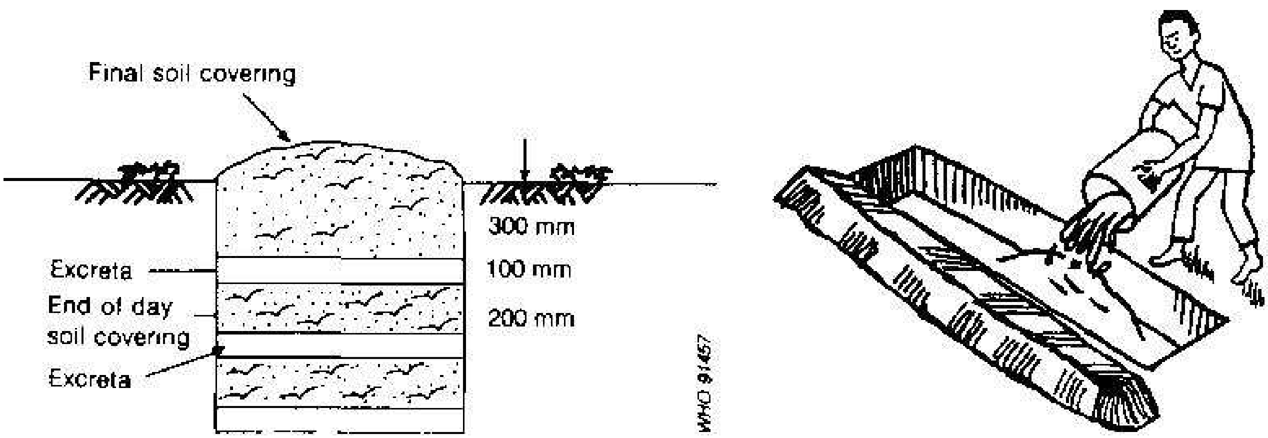
Trenching

Trenches about 1 m deep and I m wide may be filled with nightsoil to within not less than 300 mm of the top. The trench is then backfilled with excavated soil, which should be well compacted to prevent the emergence of flies or excrete being dug up by animals (Fig. 5.33). At the end of each day any exposed excrete must be covered with at least 200 mm of soil, well compacted. After backfilling, the trench should remain untouched for at least two years, after which it can be excavated for reuse and the contents used as fertilizer. The trenching site should be close to the collection area but away from residential areas. It should have deep and porous soil, be well above the water table, and not be subject to flooding.

Reuse

Nightsoil can be used as a fertilizer after all pathogens have been destroyed. It may also be added to ponds for fish cultivation.

Fig. 5.33 Disposing of excreta from bucket latrines by trenching



5.9.2 Vault latrines

Vault latrines are a way of overcoming the problem of frequent emptying needed with bucket latrine systems. A watertight tank or vault below or close to a latrine is used to collect faeces, urine and sometimes sullage. The capacity of the vault is often sufficient for 2-3 weeks' accumulation of excrete, after which time the vault is emptied. The system is satisfactory if collection is reliable and hygienic, and the vaults are properly flyproofed, vented and fitted with water-seal toilets.

In some places, the contents of vaults are bailed out by hand and taken away in tanks mounted on carts. This is highly undesirable. Trials with manually operated pumps to empty vault contents have not been very successful because with a low pumping rate (about 400 litres per hour) complete evacuation of the vault is a long and tedious operation. This method is obviously also undesirable.

Motorized vacuum tankers can provide safe removal but must be backed up by good institutional support for operation and maintenance. Most vacuum tankers cannot lift vault contents if the proportion of solids exceeds about 12%, but some have facilities for adding water to vaults before lifting the contents.

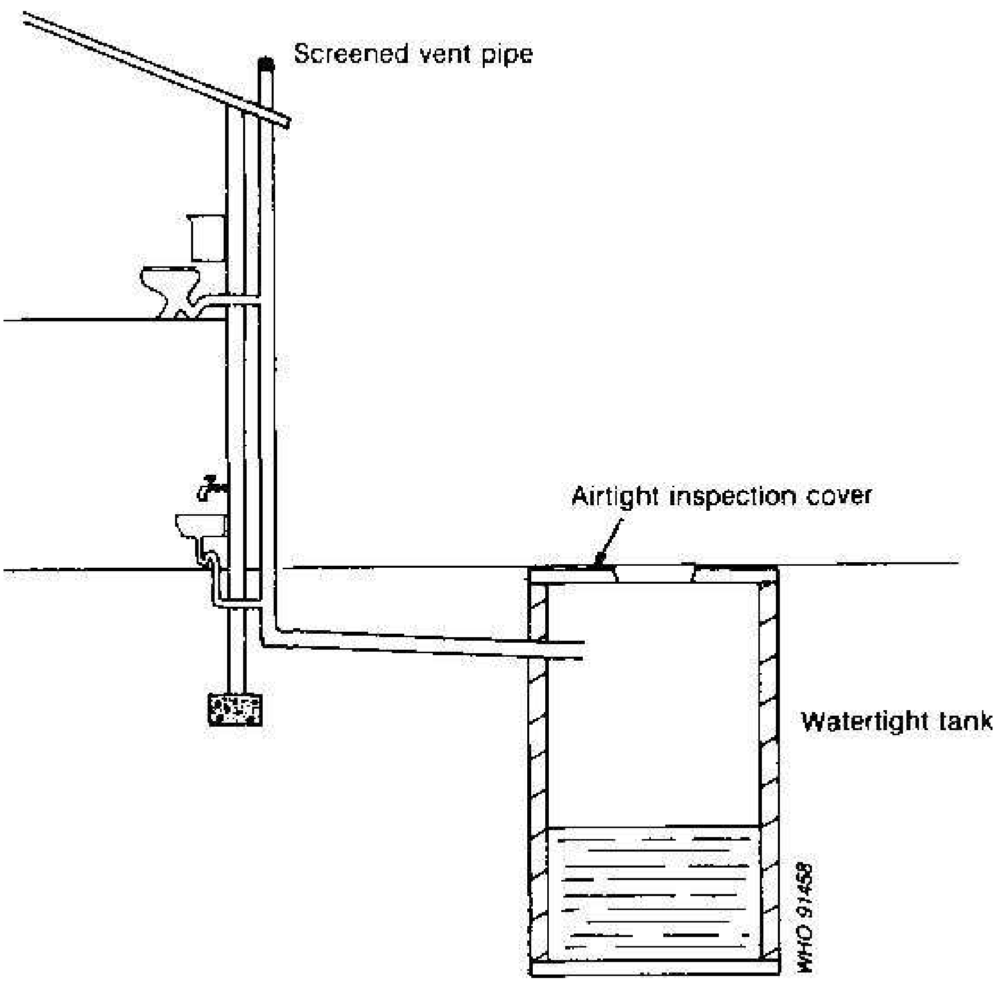
Sufficient extra space to allow for irregularities in collection time should be planned for in designing vault capacity. In communities where finance, spare parts and good maintenance are available, the additional space needed may be only 15-20%. However, where vehicle maintenance is poor, an allowance of 50% may be advisable.

The performance of vaults has been mixed, mainly dependent on the levels of finance and vehicle maintenance. Poorly constructed vaults are common, leading to problems with odour and flies, ground pollution and thickening of the vault contents. It is not recommended that new vault latrines be constructed.

5.9.3 Cesspits

Cesspits, like vaults, are watertight tanks with sealed covers (to keep out mosquitos). They differ from vaults in that they are usually located outside the premises and collect sullage as well as the wastes from water closets. The capacity may be sufficient for up to several months' use (Fig. 5.34).

Fig. 5.34 Cesspit



The cost of providing a regular removal service for all the wastewater from a house with a good supply of piped water can be very high, making cesspits an expensive form of sanitation

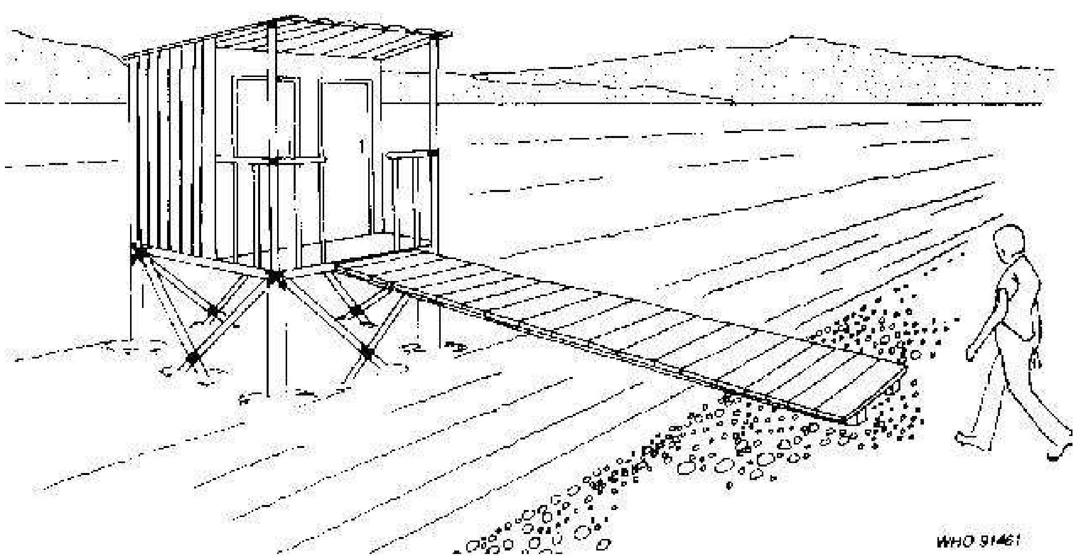
5.9.4 Overhung latrines

An overhung latrine consists of a superstructure and floor built over water (Fig. 5.35). A squat hole in the floor allows excrete to fall into the water. A chute is sometimes provided from the floor to the water. Overhung latrines should never be built in places where pit latrines can be provided. However, they may be the only possible form of sanitation for people living on land that is continuously or seasonally covered with water.

Such latrines might be acceptable provided the following conditions are met:-

- The receiving water is of sufficient salinity all year round to prevent human consumption.
- The latrine is installed over water that is sufficiently deep to ensure that the bed is never exposed during low tide or the dry season.
- Every effort is made to select a site from where floating solids will be carried away from the village.
- The walkways, piers, squatting openings, and superstructures are made structurally safe for adults and children.
- The excrete are not deposited in still water or into water that will be used for recreation.

Fig 5.35 Overhung latrines



5.10 Design Examples

5.10.1 Introduction

The design of a latrine is governed by both consumer expectations and public health requirements. Although basic design factors remain the same (pit volume, septic tank retention time, etc.), the factors that govern the final cost of the latrine are controlled by local circumstances and requirements.

It is not feasible to illustrate all the possible design options. However, this example gives details of how to determine the basic dimensions for the most common designs and illustrate the design procedure.

5.10.2 Pit latrine design

Pit size

When calculating the dimensions of a hole for a pit latrine, three conditions must be satisfied.

1. The pit should have sufficient storage capacity for all the sludge that will accumulate during its operational life or before its planned emptying.
2. At the end of the pit's operational life there should still be sufficient space left for the contents to be covered with a sufficient depth of soil to prevent surface contamination with pathogenic organisms (soil seal depth).
3. There should be sufficient wall area available at all times to enable any liquid in the pit to infiltrate the surrounding soil.

Storage volume

The storage volume required to accommodate the sludge that accumulates in the pit during its operational life can be calculated from:

$V = N \times P \times R$

Where;

V= the effective volume of the pit (m^3)

N = the effective life of the pit (years)

P = the average number of people who use the pit each day

R = the estimated sludge accumulation rate for a single person (m^3 per year).

Once the effective volume of the pit has been calculated, the plan area is decided. This should be based on local preference, ground conditions and construction materials, and is generally circular or rectangular in shape. Note that only the area inside the lining is utilized for sludge accumulation, not the excavated area.

Having determined the plan shape and area, the depth of pit required for sludge accumulation is calculated as follows:

Sludge depth = [total sludge volume (V)]/ (plan area)

Soil seal depth

This is usually taken as 0.5 m. In the case of double pit latrines it is the depth to the bottom of the inlet drain.

Infiltration area

In communities where people use water for anal cleaning or bathe in the toilet, a considerable amount of water may enter the pit. If it is assumed that the soil pores below the sludge surface are blocked, then additional wall area must be allowed for infiltration of the liquids above the sludge.

The infiltration area cannot include the soil seal depth since the top 0.5 m of a pit has a fully sealed lining.

Assuming that all the liquid entering the pit lies on top of the sludge, then the liquid depth will rise until the area of contact between liquid and soil is large enough to permit infiltration of the daily intake of liquid.

Pit depth

The total depth of the pit is calculated as follows:

Pit depth = sludge depth + infiltration depth + soil seal depth

• Example 5.1

A family of six intends to dig a pit latrine with an operational life of 20 years. The family uses newspaper and corncobs for anal cleaning, and sullage is disposed of separately.
• Sludge volume

V = N x P x R

The values of N and P are given (20 years and 6 people) but the sludge accumulation rate (R) is not. In the absence of local information the rate given in table 5.8 can be used. The accumulation rate cannot be determined without some knowledge of the depth to the water table. Assuming this is greater than the likely pit depth, an accumulation rate of 901/year is used (see Table 5.8). Sludge volume = 6 x 20 x (90/1000) (1 m^3 = 1000 1itres) = 10.8 m^3 If it is found that the pit does enter the groundwater, then the calculation should be done again using the appropriate sludge accumulation rate (601/year, from Table 5.8).
• Plan area
The pit will be rectangular, with internal dimensions of 1.2 m by 2.0 m. Thus the depth required for sludge is:
(10.8/1.2x2.0) = 4.5

• Infiltration area
Since solid objects are used for anal cleaning and sullage is disposed of elsewhere, there will be very little liquid to infiltrate. Accordingly the infiltration area can be ignored.
• Soil seal depth Assumed to be 0.5 m. Therefore the designed pit depth is:
4.5m + 0.5 m = 5 m
This is very deep and consideration could be given to increasing the plan area or reducing the life of the pit.

• Example 5.2

A family of six intends to construct a pit latrine to last 20 years. The family uses water for anal cleaning and intends to use the toilet as a bathing area. The ground is mainly a fine sand with a water table 3 m below the surface:

• Sludge volume

Using the figures given in Table 5.8, the sludge accumulation rate will be 601/year above the water table and 401/year below. First assume that the pit will be mainly above the water table. If it is found that it enters into the groundwater by more than 1.0 m then the volume can be recalculated.

Volume (V) = N x P x R

= 6 x 20 x (60/1000)

= 7.2 m^3

• Sludge depth

If the pit is to be circular, with an inside diameter of 1.3 m, the sludge depth will be:

Sludge volume/Plan area = (7.2 x 4)/ (p x 1.3^2) = 5.42 m

A pit of these dimensions would mean that most of the sludge would collect below the water table. Therefore the volume should be recalculated using a sludge accumulation rate of 401/year.

V = 6 x 20 x 0.04 = 4.8 m^3

Therefore the new sludge depth will be:

(4.8 x 4)/ (p x 1.3^2) = 3.62 m

• Infiltration rate

The infiltration capacity of a fine sandy soil is about 331/m^2 per day (see Table 3.4)

Assuming the volume of water entering the pit each day is 200l then the infiltration area required will be:

200/33 = 6.1 m^2

Therefore liquid will build up in the pit until a contact area of 6.1 m^2 is achieved. Water depth = infiltration area pit circumference = (6.1)/(p x 1.3) = 1.49 m

Assuming a soil seal depth of 0.5 m, the total depth required for the pit is:

3.62 + 1.49 + 0.5 = 5.61 m

This is a slight underestimate of the required depth because some of the sludge will accumulate above the groundwater level. Bearing in mind the inaccuracy of the basic design data, however, it is not necessary to carry out a more accurate calculation.

• **Example 5.3** Offset Pour Flush Double Pit Latrine

An offset pour-flush double-pit latrine is to be constructed for a family of six who use water for anal cleaning. The groundwater table is within 0.5 m of the surface during the rainy season and the soil is sandy silt.

• Sludge volume

As for the previous examples:

V= N x P x P

In a large pit the value of R would be taken as 401/year (see Table 5.8) but as this is a double pit, full consolidation of the sludge is unlikely to have taken place within the time taken to fill the pit (generally 2 years). Therefore a higher sludge accumulation rate (such as 60 l/year) should be used.

Sludge volume = 6 x 2 x (60/1000) = 0.72 m^3

• Sludge depth

If each pit is 1.2 m wide and 1.2 m long, the sludge depth will be:
0.72/ (1.2 x 1.2) = 0.5 m

• Infiltration depth

An offset pour-flush toilet uses about 31 of water per flush. Assuming
20 flushes per day the total liquid inflow will be:
3 x 20 = 60 litres
If 6 l of urine enter the pit each day, the total daily inflow of liquid will be 661. The infiltration rate for sandy silt is about 25 l/m² per day (see Table 3.4); therefore the infiltration area required is:
66/25 = 2.6 m²
The perimeter length of each pit is 1.2 x 4 = 4.8 m, therefore the liquid depth will be:
2.6/4.8 = 0.5 m

• Pit depth

The pit depth is the sum of the component depths, i.e.:

h to bottom of inlet pipe	0.2m
liquid depth	0.5m
sludge thickness	0.5m

Total depth of each pit below ground level 1.2 m

• **Example 5.4** Septic tank design

Design a septic tank suitable for a household with up to eight occupants in a low-density housing area in which the houses have full plumbing, all household wastes go to the septic tank and the nominal water supply is 200 l per person per day. Water is used for anal cleaning and the ambient temperature is not less than 25°C for most of the year.

• Stage 1
Volume of liquid entering the tank each day
A = P x q

Where;
A = volume of liquid to be stored in the septic tank
P = number of people using the tank
q = sewage flow = 90% of the daily water consumption per person (Q).
q = 0.9 x Q = 0.9 x 200 = 180 litres per person per day.
Therefore A = 8 x 180 = 1440 litres
• Stage 2
The volume of sludge and scum is given by
B = P x N x F x S
Where;
B = volume of sludge and scum
P = number of people using the tank
N = period between desludgings

F = sizing factor (taken as 1.0)

S = sludge and scum accumulation rate (see table 5.8)

Assume N is 3 years; F= 1.0; as all wastes go to septic tank S = 401 per person per year.

Therefore:

B = 8 x 3 x 1.0 x 40 = 960 litres

• Stage 3

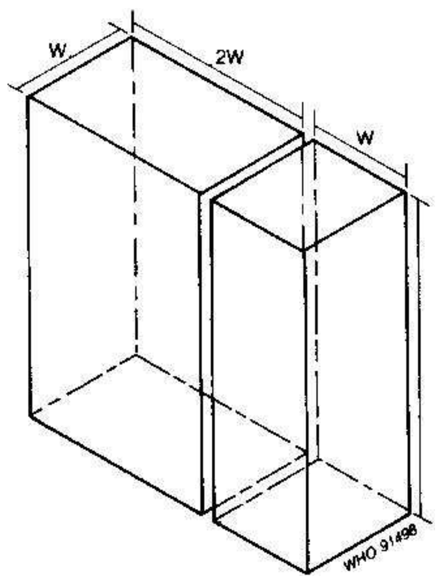
Total tank volume = A + B = 1440 + 960 = 2400 litres (2.4 m^3)

• Stage 4

Assume liquid depth = 1.5 m

Assume tank width is W m

Fig. 5.36 Internal dimensions of the septic tank designed in example 5.4



Assume two compartments, length of first = 2W; length of second = W

This tank is illustrated in Fig. 3.36

Volume of tank (V) = 1.5 x (2W + W) x W = 4.5 W^2

Thus 4.5

W^2 = 2.4 m^3

W = 0.73

Therefore:

width of tank = 0.73 m

length of first compartment = 1.46 m

length of second compartment = 0.73 m
Depth of tank from floor to soffit of cover slab
= liquid depth + freeboard
= 1.5 + 0.3
= 1.8 m

Flotation

Since septic tanks have sealed walls and floor, the design must be checked to make sure that the tank does not float out of the ground. Flotation will occur if the total mass of the empty septic tank is less than the mass of the water it displaces. This will only happen if the groundwater level is higher than the bottom of the tank.

Calculate the mass of the walls, floor, roof and any baffle walls (concrete: 2400 kg/m3; brickwork: 1500 kg/m3). Measure the volume of the tank (outside dimensions) between the highest groundwater level and the bottom of the tank. Multiply the volume by the density of water (1000 kg/m^3). This gives the mass of water displaced.

If the mass of water displaced is greater than the total mass of the empty septic tank then the tank may float. This can be prevented by increasing the mass of the structure (e.g., by increasing the thickness of the floor or walls) or reducing the amount of the tank that is below the water table.

Soil pressure

For large tanks, such as for a school or a number of houses, it is necessary to check that the side walls of the tank are not likely to collapse owing to the outside soil and water pressure. This is most likely when the tank is empty. Such a calculation is beyond the scope of this book, and reference should be made to a manual on reinforced concrete or masonry design.

• Example 5.5

Design a septic tank for a household having five occupants in a medium density housing area in which the houses have full plumbing. Only WC wastes go to the septic tank, and paper is used for anal cleaning. The ambient temperature is more than 10°C throughout the year.

• Stage 1

Daily volume of liquid
A = P x q
If the WC has a 10-litre cistern and each person flushes it four times a day, the sewage flow q = 4 x 10 = 40 litres per person per day and A = 5 x 40 = 200 litres.

• Stage 2

Volume for sludge and scum
B =P x N x F x S
Assume N is 3 years, F = 1.0; as only WC wastes go to septic tank S = 25 litres per person per year.
So B = 5 x 3 x 1.0 x 25 = 375 litres

• Stage 3

Total tank volume V = A + B =200+375 = 5751(0.575 m^3)

As this is less than the minimum recommended volume of 1.0 m^3, the dimensions for theminimum volume should be calculated.

• Stage 4

Assume liquid depth = 1.5 m.
Assume tank width is Wm.
Assume two compartments:

length of first = 2W
length of second = W
Volume of tank = 1.5 x (2W + W) x W = 4.5 W^2

If 4.5 W^2 = 1.0 m^3, then W = 0.47 m

As this is less than the recommended minimum width of 0.6 m, assume W = 0.6 m.
Length of first compartment (2W) = 1.2 m
Length of second compartment (W) = 0.6 m
Depth of tank from floor to soffit of cover slab

= 1.5 m (liquid depth) + 0.3 m (freeboard) = 1.8 m

The tank volume (excluding freeboard) is: (1.2 + 0.6) x 0.6 x 1.5 = 1.62 m^3 which is larger than the required volume calculated in stage 3. This is no disadvantage; in practice the minimum retention time will be greater than 24 hours or the tank will provide longer service than three years before requiring desludging.

5.10.3 Aqua-privy design

Aqua-privies are basically small septic tanks. They have the same purpose as septic tanks and work in the same way. It is recommended therefore that they are designed in the same way as septic tanks. It is also recommended that the minimum size of tank should be 1.0 m^3. This is because smaller tanks are more difficult to build and the turbulence produced by the inflow will prevent proper settlement.

• **Example 5.6** Disposal of effluent from Septic tanks and Aqua-privies

Determine the size of soakpit required in porous silty clay to dispose of the effluent from the septic tank considered in Example 5.5.

From Example 5.5, the sewage flow is 200 litres per day.
From Table 3.4, the infiltration rate for sewage is 201 per m^2 per day.
Therefore, the wall area required is 200/20 = 10 m^2

If the pit is 1.5 m in diameter, then the depth required from the bottom of the pipe from the septic tank to the bottom of the pit is:

10/ (p x 1.5) = 2.12 m

• Example 5.7

Determine the size of drainage field required in porous silty clay to dispose of the effluent from the septic tank considered in Example 5.4.

From Example 3.4 the sewage flow is 1440 l per day. From Table 3.4, the infiltration rate for sewage is 20 l per m^2 per day.

So the wall area required is 1440/20 = 72 m^2

If the effective depth of the trench (the depth from the bottom of the pipe to the bottom of the trench) is 0.6 m, the length of trench required is: 72/ (0.6 x 2) = 60 m
This allows for infiltration on both sides of the trench.

If the plot is large enough, the drainage field should consist of two trenches, each 30 m long, connected in series.

5.10.4 Composting toilets

5.10.5 Double-vault latrines

The design of a double-vault latrine is similar to that of a pit latrine, i.e., the volume of each vault is calculated using the formula:

$V = N \times P \times R$

Where;

- V= the effective volume of the vault (m^3)
- N = the number of years the vault must last before becoming full
- P = the average number of users
- R = the estimated sludge accumulation rate for a single person (m^3 per year).

The difficulty with vault design is that very little information exists on the sludge accumulation rate in vaults where excrete are mixed with ash and other organic material, and there has been little research into the pathogen survival rate in such an environment.

Desludging period

Pit latrines are usually designed such that excrete are not handled for two years. Since the inside of a composting toilet is similar to that of a pit latrine, it is reasonable to assume that it should be designed using similar parameters. However, some researchers disagree with this, saying that the low moisture content of the compost produces very alkaline conditions that destroy the pathogens in a much shorter time. Time as low as four months have been suggested. In the absence of more accurate information, however, a two-year retention time is recommended.

Sludge accumulation rates

The accumulation rate for the excrete component of the compost can be determined in the same way as for a double-pit latrine. In the absence of more accurate local information, figures 50% greater than those given in Table 5.3 are suggested.

Estimating the volume of ash and other organic material is more difficult. Experience in Viet Nam indicates that approximately twice the volume of faeces has to be added (Jayaseelan et al., 1987). Rybczynski (1981) suggested five times the volume of faeces, and Kalbermattan et al. (1980) recommended allowing 0.3 m^3 per person per year for all wastes.

In the absence of information to the contrary, it is suggested that the total sludge build- up rate is calculated as three times the estimated faecal build-up rate.

• Example 5.8

Design a doub1e-vault composting toilet for a family of six who use paper for anal cleaning.

The effective volume of each vault (V) must be:
$2 \times 6 \times (0.06 \times 1.5 \times 3) = 3.24\ m^3$
Vaults are usually sealed when they are three-quarters full, therefore the actual volume of the vault must be:
$4/3 \times 3.24 = 4.32\ m^3$
If the vault has a plan area of 1.3 x 1.3 m, the depth will be: $4.32/(1.3 \times 1.3) = 2.56$ m

Continuous composting toilets

Even fewer design data are available for continuous composting toilets than for double- vault latrines. Past designs have been empirical and little published information exists indicating the level of their success. It is suggested that, until more data are available, the size of the primary tank in the toilet should be based on the formulae and factors used for double-vault latrines. The second tank should be 10-20% of the size of the first tank. The floors of both tanks should slope at an angle of 30° to the horizontal. No design data exist for calculating the size and number of aeration channels or the diameter and height of the ventilation pipe.

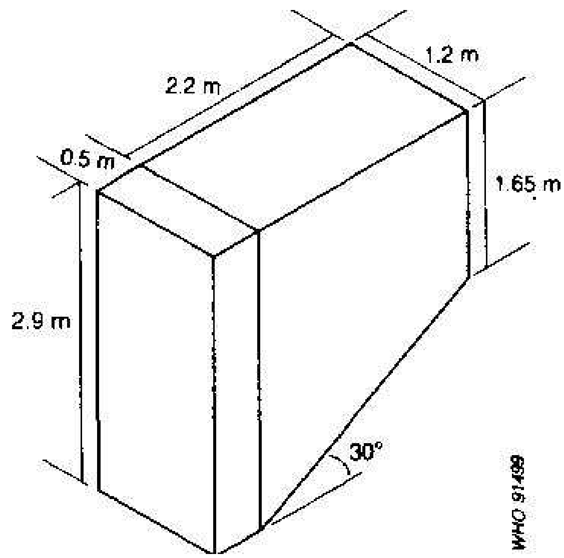
• Example 5.9

Using the information given in Example 3.8, design an appropriate continuous composting toilet.
From Example 5.8 the volume of the primary tank should be 4.32 m^3.
The volume of the second tank will be: 4 3 x 0.1 5 = 0.65 m^3
Assuming the first tank is 1.2 m wide and 2.2 m long then its depth will be 1.65 m.
The length of the second tank will be:
$0.65/ (1.2 \times 1.65) = 0.33$ m
This is short and would make emptying very difficult; increase the length to 0.5 m.
Since the vault floor must slope at an angle of 30°, the depth of excavation at the outlet end will be greater than the depth at the inlet.

Assuming the floor of the second tank is horizontal the internal floor level will be at a depth of:

$1.65 + 2.2 \tan 30° = 2.9$ m Fig. 5.37shows the final internal dimensions of the tank.

Fig. 5.37 Internal dimensions of the continuous composting toilet designed in example 5.9



6.0 PLANNING FOR SEWERAGE

6.1 Introduction

A Sewerage Project matures through distinct stages in its life which extends from identification as a crude idea through to its conversion into routine operation. Sewerage Projects normally mature through the following stages:-

(a) Project Identification
(b) Project Preparation (Feasibility)
(c) Project Appraisal and Selection
(d) Project Financing
(e) Project Implementation and
(f) Project Monitoring and Evaluation.

These stages are essentially sequential in nature, and are unique, entailing unique sets of actions which are necessary for the development of a mature project. The first four stages (a to d) form the project planning while the other two form the implementation of the project.

The planning phase of a project deals with all the pre-investment activities which attempt to identify the project idea and formulate it into a set of action plans that can effectively achieve the intended objectives within the specified time period. The implementation phase covers the actual investment period when resources are assembled together for the productive process of the projects which are to follow. Without properly implemented projects, good plans become only empty objectives. During the operation phase, actual generation and distribution of output takes place and the benefits of a project are realized and used.

For effective Planning, data is required. Table 6.1 has the detailed planning data required

6.2 Design Period

1) The Design Period of sewerage scheme will be set reasonable by taking into account the urban growth rate and future prediction accuracy

   In conventional design, it is common to design trunk sewers and interceptors for the projected peak flow expected for a period of 25 to 50 years or for the saturation population of the area. Such long design periods make it possible to benefit from economics of scale in sewerage systems. However, these have to be balanced against the opportunity cost of capital, uncertaintities in presiding future land use patterns or directions of growth in developing country cities, and the high cost of maintaining large sewers with low flows.

   The use of shorter design periods avoids such problems and reduces the large capital requirements in sewerage systems, facilities financing, and enhances prospects of achieving greater coverage with a given investment. With shorter design periods and construction by phases, starting from upstream ends, the effects of errors in forecasting population growth and their water consumption can be minimized and corrected. For these reasons, it is recommended a design period of **10-20** be adopted.

2) Most Feasibility/Preparatory Studies are made on the basis of estimates for the next five to ten years.

Kenya being a developing country the design periods should be shorter than those conventionally employed for advanced industrial countries due to.

- The quality of the facilities is likely to be poor and their life relatively short
- Systems are readily extended
- The rate of population growth is uncertain, but it is possible that rapid growth will accompany the installation of water and sewerage facilities
- The rate of interest for borrowing can be quite high
- The need for the works would be such that they would be nearly loaded to capacity at an early stage

In consideration of the above, the design period for an emergency development plan will be set at about three to five years. To achieve early investment effects for decentralized sewerage, a design period of five and ten years is considered desirable.

Table 6-1 Data related to the planning and design of sewerage facilities

Item	Descriptions
Social and economic conditions	Population trends (social and natural dynamics), tourist population, factory locations/types of industries, and production output
National Measures	National policy on sewerage development
Sewerage master plan development	Design district, design parameters, location map of treatment plants, sewers and pumping station drawings
Sewerage development	Scope of the existing sewerage development area, collection systems, and sewer main conditions, location, section, flow capacity, manhole construction, storm drain development
Connection condition	Sewage main connection methods, connection conditions for homes
Condition of pumping stations	Locations, structures of pumping stations; their specifications, suppliers, ages, auxiliary equipment (generators, etc.), running record, equipment failure/repair records
Condition of wastewater treatment plants	Design documents and as built drawings, operation records, equipment failure and repair records of plant facilities, and auxiliary equipment (generators, test equipment, etc.), sludge disposal sites and treatment/disposal records
Sewerage facility management and maintenance	Operation and maintenance systems, personnel, and equipment owned by each treatment plant, pumping station maintenance method, pipe cleaning method and frequency, subcontracted to private sector or not, personnel qualifications, maintenance budget and expenditures
System, organization and operation	Sewerage related laws, sewerage design standards, organization and personnel situation, organization policy/decision making system, division of duties, water and sewer rate system and procedures for related decisions and changes, fee collection system, and fee collection ratio, penalty for non payment, accounting system, and financial condition (cash flow, source of revenue, balance sheet)
Areas not served by sewage works	Topographical conditions, formation of blocks, condition of streets and traffic volume, buried utilities, river pollution, pollutant inflow into public waters, waste treatment and disposal
Trends among donor nations and agencies	Type of assistance given, amounts of funds involved for aid related to sanitation, including sewerage
Environmental	Water pollution, measures to prevent water pollution, water quality regulations and actual enforcement
Others	Prices and land acquisition conditions for wastewater treatment plants and pumping stations, communities' independent activities to improve sanitation, NGO efforts

6.3 Design Area

The design area is determined by taking into account the following requirements;

As a rule, it should be an area where urbanization is expected within the design period, including the development plan. For areas equipped with existing wastewater discharge/treatment facilities, including those similar to sewerage (Household wastewater treatment tanks, etc), the review must cover assistance to promote self reliance of the area.

It must be clearly indicated that the area may differ depending on the type of the plan.

As Regards (1):

The area concept must take into consideration the following:-

- The design area must conform to a higher plan related to sewerage when one exists. In Kenya, sewerage is lagging behind due to financial constrains, sewerage development should proceed independently in high priority urban areas, and the concept of a higher plan, such as a basin wide sewerage system plan, has yet to be developed.
- For a densely populated area away from the center of the design area and for thinly populated areas located between them, on site treatment must be planned in satisfactory combination and with due consideration for cost effectiveness
- Decentralized sewerage is intended for subdivided blocks in the design area to achieve early effects in multiple highly urgent treatment areas targeted on the basis of the degree of worsening of the living environment and the effects of public water pollution.

As regards (2):

The master plan should establish districts to be covered by the overall plan for the urban area, including those expected to be urbanized within the design period. This design area naturally includes districts where wastewater is already treated and disposed of according to certain methods. One of great challenges in developing an off site treatment (sewerage)/on site treatment plan is how to handle these areas. If existing treatment and disposal methods present problems, one approach is to abolish or merge them as the areas in question are integrated into the sewerage scheme. Another approach is to improve and maintain existing methods as an independent system if there are no problems with them or problems that can be solved through minor improvements. Abolishing and merging systems would present timing problems. Integration of these methods into the sewerage scheme should not be attempted in the course of the feasibility study. The feasibility study must limit the area addressed-not unnecessarily expand it. This principle is applied to enhance the feasibility of the selected area. Septic tanks may be widely used in certain areas, with sullage discharged without treatment. Incorporation of these areas into the sewerage design area must be considered only when such a situation can no longer be left as it is due to severe worsening of the living environment and public water pollution.

6.4 Type of Collection Systems

The type of sewerage system must be determined by taking into account the topography, meteorology and the present condition of wastewater/storm drainage facilities.

There are separate and combined sewerage systems. Separate systems carry sanitary wastewater and storm water in different conduits, while combined systems receive and convey both sanitary wastewater and storm water in the same conduit. Some areas combine these two types.

In Kenya, Partially separate sewerage systems are recommended as an acceptable compromise. Bigger sewers than for a separate system will be required, but against this these larger sewers may often be laid at flatter gradients and thus at shallower depths.

The storm water which enters the sewerage systems should be kept to a minimum, and very carefully controlled; otherwise, the sewers will either be uneconomically large or will flood during wet weather.

Stormwater entry should be restricted to the run-off from:-

(a) Open, paved, public markets;
(b) The open, paved, yards of slaughterhouses;
(c) The yards of milk collection and/or processing establishments;
(d) Any similar open, paved yards which, because of organic dirt, should for reasons of public health be washed down daily

Any area drained in this way should be surrounded by kerbs in order to exclude the drainage from surrounding areas. Under no circumstances should the run off from any unpaved area be accepted. Domestic yards should not be connected to sewers; sullage gulley in such yards may be connected, but these should always be kerbed so that yard drainage is excluded.

The drainage from streets, pavements and the forecourts of garages and filling stations should enter storm water drains, not sewers; petrol/oil interceptors are required between forecourts and drains.

The partially separate system has advantages:-

- The occasional storm flows in the sewers will flush them; in particular, they will wash away any corrosive acid deposits.
- They will have no relief overflows which can cause pollution and contamination of water resources.
- Virtually all the pathogens in the towns, liquid wastes will be collected and carried to the sewage treatment works.

6.5 Design Population

The design population is determined on the basis of the following data after prediction of growth in the design area over the design period.

1) An estimate of the total design population

The Design Population is based on the population estimate separately established in the urban plan.

If such an estimate does not exist, the design population is estimated by using the average annual growth rate on the basis of past population trends.

2) A population Distribution Estimate

The total design population is distributed by zoning on the basis of population density in representative areas.

3) Mobile Population

For sightseeing areas and the other areas with a seasonal mass influx of population, the influx, for example, the tourist population, must be taken into account.

As regards (1):

The design population is used as the basis for calculating design wastewater flow for sanitary wastewater treatment and disposal. The total population and its distribution within the design area are estimated by assuming growth will be projected during the design period. Also refer to city planning and other long term plans for the design area.

Various demographic data can be obtained in Kenya, including the 1962,1969, 1979, 1989 & 1999 census population, the respective 1962,1969,1979, 1989 & 1999 census of residents by town, natural dynamics, and social dynamics. Development of these data is generally insufficient. Population estimates often have to be based on past annual average growth rates.

6.6 Design Wastewater Flow

Wastewater generated can be roughly classified into domestic, commercial and industrial.

6.6.1 Household Usage in Developing Countries

Table 6-2 Data on the average daily water supply per capita in developing countries

		Daily water supply per capita (L/person/day)			**GNP per capita (US $)**
Countries	**Cities**	**1993**	**1997**		
East Asia	Beijing	149	96		
China	Shanghai	193	143		**860**
	Tianjin	125	101		
	Chongju			260	
	Hongkong	111	112		
Taiwan	Taipei	281	262		
Korea	Seoul	180	209		10,550
	Ulsan	166	157		
The philippines	Manila	133	202		1,200
	Cebu	139	173		
	Davao		145		
Vietnam	Hanao	157	45		310
	Ho Chi Minh	131	136		
Cambodia	Phnom Penh		32		300
Laos	Nationwide	140			
	Vientiane		172		400
Myanmar	Yangon	120	67		
Thailand	Mandalay	153	110		
	Nationwide	172			
	Bangkok	217	265		2,740
	Chiang Mai		135		
	Chon Buri		145		
Malaysia	Kuala Lumpur	222	200		4,530
	Penang	203	244		
	Johor		193		
Singapore	Singapore	168	183		32,810
Indonesia	Jakarta	148	135		1,110
	Bandon	96	120		
	Tiruttani	153			
	Medan		131		
Papua New Guinea	Lae		146		
Mongolia	Ulan Bator		177		390

Country	Cities	Daily water supply per capita (L/person/day)			GNP per capita (US $)
		1993	1997		
Southwest Asia					
India	Deli	257	209		370
	Calcutta	213	202		
	Mumbai		178		
Pakistan	Karachi	172	157		500
	Faisalabad		170		
	Lahore		213		
Bangladesh	Dacca	44	95		360
	Chiittagong		139		
Nepal	Katmandu	97	91		220
Bhutan	Thimphu		93		430
Sri Lanka	Colombo	168	165		800
Maldives	Male		16		1,180
Central asia					
Uzbekistan	Tashkent	109			1,020
Kazakhstan	Almaty	186			1,350
Kirghiz	Bishkek	112			480
Near East					
Algeria	Nationwide			46 (90)	1,260
Central and South America					
Costa Rica				197	2,680
Columbia	Bogota			208	2,180
Chile	Santiago			204	
Brazil	Nationwide			151	
	Brazilia			211	4,820
	Sao Paulo			237	
	Santa Catarina			143	
	Minas			154	
Oceania					
Cook islands	Rarotonga	464	267		
Samoa	Nationwide	475			
	Apia		337		1,140
Tonga	Nuku alofa	81	78		1,810
Solomons	Honiara	184	251		870
Fiji	Suva	203	135		2,460
Vanuata	Port Villa		273		1,360

Though some values are extremely high while others are extremely low, most values may fall within a range of about 100 to 250 L/person/day.

Water supply service levels are roughly classified into (1) Manual carrying from a public tap, well, water service tanker truck, river, or spring, (2) one yard tap or manual pump (well) per house, and (3) muiltiple faucets per house from indoor tap. Daily water supply per capita varies naturally according to the difference in convenience of water usage.

In view of the above, table 6.3 below has the household water use segregated as per type of housing for Developing Countries.

Table 6.3 Daily Average Water Supply per capita according to Class of Dwelling

Class of dwelling	**Description**	**Daily average water supply per capita (L/person/day)**
High class detached houses, exclusive apartment houses	Two or more toilets and three or more taps per household	150-260
Middle class ordinary houses, apartment houses	One toilet and two taps per household	110-160
Low level room for rent, substitute houses provided by government, divided houses	Minimum one tap per household, shared toilet	55-70*

* High frequency of use because of waste

6.6.2 Consumption Rates in Kenya

Table 6.4 below has the recommended consumption rates which are applicable in Kenya.

However, where the facilities are existing and there is adequate water supply with coverage of over 80% and the supply unlimited, the water usage should be obtained from the water service provider was possible.

Table 6.4 Recommended Consumption Rates for Kenya

Consumer	Unit	Rural Areas			Urban Areas		
		High potential	Medium potential	Low potential	High class housing	Medium class housing	Low class housing
People with individual connections	1/head/ day	60	50	40	250	150	75
People without individual connections	1/head/ day	20	15	10	_	_	20
Boarding schools	1/head/ day	50					
Day schools with WC Without WC	1/head/ day	25 5					
Hospitals Regional District Other	1/bed/ day	400) 200) 100)			+20 1 per outpatient and day (Minimum 5000 1/day)		
Dispensary and Health Centre	1/day	5000					
Hotels High class Medium class Low class	1/bed/ day	600 300 50					
Administrative offices	1/head/ day	25					
Bars	1/day	500					
Shops	1/day	100					
Unspecified	1/ha/				20,000		

Source: Ministry of Water and Irrigation Water Practice Manual, 2005

If **Table 6.4** does not have per capita water consumption are there is inadequate design data, it is recommended that per capita water consumption values as recommended in **section 6.6.3** below be adopted.

6.6.3 Consumption Rates obtained through Surveys

There are two methods of obtaining per capita water demand namely: - **Average Domestic** per capita consumption where a 24hour piped supply is provided and surveys covering representative samples of consumers in all supply zones.

Arising from the above, during the Service Demand Forecast of Nairobi City and adjacent seven urban centers of Ruiru, Kikukyu, Ngong, Kiserian, Ongata Rongai, Mavoko and Kitenkella undertaken by Athi Water Services Board in 2007, per capita Survey were undertaken which is represented in Tables 6.5 to 6.7

Table 6.5 Commercial Per Capita Water Demand in M^3/Day

Commercial Entity	**Per Capita**
Financial Institutions	
Large Financial Institution Bank Head Office, branch money lender, hire purchase company, insurance company, real estate developer, financial co building, premises over $300km^2$ with over 25 employees	2.95
Medium Financial Services **From 6 to 26 employees**	0.569
Small Financial(Up to 5 Employees)	0.157
Hotels/Lodging Houses	
Large High Standard Lodging House/Hotel/D Class over 100 Rooms	0.130
Small High Standard Lodging House/Hotel/ upto 40rooms	12.5
other Catering and Accommodation	3.0
Bar and Lodging	0.734
Middle Class Hotel	0.954
Butchery	3.267
Petrol Station	
Large petrol Station (Over 6 pumps or with garage workshop and retail shop	21.214
Medium petrol Station (from 4 to 6 pumps or with garage workshop and retail shop	4.9112
Small petrol Station(upto3 pumps and without garage workshop and retail shop	1.279
Kiosk(saloon)	0.0374
Communication Company	1.32
Dry Cleaner	1.70
Churches	2.05
General Merchant /wholesaler Shop	0.872
Hyper Super Market	10.56
Shop	0.257

Table 6.6 Industrial Per Capita Water Demand in M^3/DAY

Class	ISIC No.	Process	
Class 1	3111	Slaughtering, preparing and preserving meat	
	3112	Manufacture of dairy products	
	3113	canning and preserving of fruit and vegetables	
	3114	Canning, preserving and processing of fish	
	3115	Manufacture of vegetables and animal oil and fats	
	3116	Grain mill products	
	3117	Manufacture of bakery products	
	3119	Manufacture of cocoa, chocolate and sugar confectionery	
	3121	Manufacture of food products	
	3122	Manufacture of prepared animal feeds	
	3131	Distilling, rectifying and blending of spirits	
	3133	Malt liquors and malt	
	3134	Soft Drinks and carbonated waters industries	
Class 2	3140	Tobacco manufacture	
	3231	Tanneries and leather finishing	
	3240	Manufacture of footwear	
	3411	Manufacture of pulp, paper and paperboard	
	3419	Manufacture of pulp, paper and Articles	
	3420	Printing, publishing and allied industries	
Class 3	3700	Basic Metal Industries	
	3811	Manufacture of cutlery, hand tools and general hardware	
	3812	Manufacture of furniture and fixtures primarily of metal	
	3813	Manufacture of Structural Metal Product	24
	3819	Manufacture of fabricated metal products except machinery and equipment	20
	3830	Manufacture of electrical machinery, apparatus, appliances and supplies	48
	3843	Manufacture and assembly of motor vehicles	37
	3845	Manufacture of aircraft and repair	
Class 4	3211	Spinning, weaving and finishing textiles	57
	3212	Manufacture of made up textile goods except footwear	132
	3220	Manufacture of wearing apparel except footwear	
	3219	Manufacture of textiles	
	3511	Manufacture of basic industrial chemical excluding fertilizer	57
	3514	Manufacture of fertilizer and pesticides	15
	3521	Manufacture of paints, varnishes and lacquers	27
	3522	Manufacture of drugs and medicines	25
	3523	Manufacture of soap perfumes, cosmetics and other preparations	27
	3529	Manufacture of chemical products NEC	35
	3530	Petroleum products	1.0
	3550	Manufacture of rubber products	13
	3560	Manufacture of plastic products	19
	3620	Manufacture of glass and glass products	23
	3691	Manufacture of structural clay products	77
	3699	Manufacture of non metallic products NEC	

Table 6.7 Per Capita Water Demand in Health Institutions in M^3/DAY

	Special Hospital	District Hospital	Health Centre	Dispensary	Remarks
Proposed per capita Water Demand to be adopted	1545	65	5	5	

Source: - Service Demand Forecast of Nairobi and surrounding urban areas of Ngong, Kiserian, Ongata Rongai, Kikuyu, Mavoko & Ruiru by Athi Water Services Board dated December, 2006

6.7 Design Wastewater Flow

Generally about 85% of water used ends up as wastewater.Table 6.8 has the correlation between water use and wastewater flow.

Table 6.8 Factor of Water Used that ends up as Wastewater

	Category	Factor (As a percentage)
1.	High class housing	75
2	Average Urban Housing	80
3	Low Cost Housing	85
4	Communal ablution/latrine block	85
5	Day schools, shops and offices	85
6	Other Institutions	80

Source: - W.H.O. Report No. 9

6.7.1 Average per Capita Waste Water Generated

The Average per capita wastewater generated is obtained by multiplying the respective water demand with respective value in table 2.35 less 10%, which is the assumed value of minor leakages.

6.7.2 Design Pollution Load

Wastewater treatment plants, mostly define design wastewater influent quality in terms of a $BOD_{5.}$ The strength of sewage is governed by the water consumption. According to Duncan Mara, the strength of sewage in terms of BOD_5, is low at 200 to 250mg/L in the US, where the level of daily water consumption is high (350-400L/person) and high at 400 to 700 mg/L in tropical countries with relatively low water consumption (40-100L/person) (**See Table 6.9**)

Table 6.9 Sewage Content in Tropical and Genial Regions (mg/L)

	Kenya (Nairobi)	Kenya (Nakuru)	India (Kodungaiyur)	Peru (Lima)	Israel (Herzliya)	USA (Allentown)	UK (Yeovil)
BOD_5	448	940	282	175	285	213	324
SS	550	662	402	196	427	186	321
TDS	503	611	1,060	1,187	1,094	502	-
Chloride	50	62	205	163	163	96	315
NH_4 -N	67	72	30	76	76	12	29

Apart from the view that water quality will be high because of low water consumption, there is also the interesting observation that daily amount of feaces excretion per capita is 400g in areas with a tropical climate where people's diet is mainly vegetables as compared to 120g in areas with a genial climate where the diet is high in protein (see **Table 6.9**)

Domestic wastewater is classified into night soil (toilet effluent) and sullage.

For the daily BOD_5, pollution load per capita, **Table 6.10** shows data for various countries and regions.

Table 6.10 Daily SS and BOD_5 pollution loads per capita (g/person)

Region and country	SS	BOD_5	Source
Zambia		36	Duncan Mara
Kenya		**23**	
Southeast Asia		43	
India		30-45	
France, rural area		23-34	
United Kingdom		50-59	
USA		45-78	D.A.Okun & G. Ponghis
Industrial Areas	90	54	
Uganda, Kampala (Residential Area)	43	63	
India	67	35	
Brazil, Guanabara	75		
United Kingdom, UK	62	59	WHO
Developing Countries		45	
Brazil, Sao Paulo		**44**	
Kenya	**80**	**55**	**WHO Report No. 9**

Arising from 6.10, it is recommended that for design purposes, the strength of domestic sewage in Kenya be taken as **55grammes** of **BOD_5 and 80 grammes of suspended solids per head per day.**

Duncan Mara's data showing a breakdown of nightsoil and sullage in tropical zones **(**see **Table 6.11) shows 22g/person (55%)** of the daily per capita **BOD_5** pollution load (40g/person) is from night soil while 18g/person (45%) is from sullage.

Table 6.11 Average breakdown of daily BOD_5 load per capita (g/person)

	USA	Tropical Zone
Washup	9	5
Dish washing	6	8[2]
Garbage disposal	31	-
Washing	9	5
Toilet –feaces	11	11
-urine	10	10
-paper	2	1
Total(average adult male)	**78**	**40**

1. From garbage disposal unit in kitchen sink
2. Including Food debris

Feaces and urine excreted by individuals vary depending on the water consumption, climate, intake and occupation. The only way feasible to obtain the exact amount in specific location is actual measurement. The average feaces excretion amount (g/person/day) is reported in **Table 6.12**

Table 6.12 Amount of wet feaces excreted by adult

Location	Amount of wet feaces (g/person/day)	Source
China (male)	209	Scott (1952)
India	255	Macdonald (1952)
India	311	Tandon & Tandon (1975)
Peru (Rural Indians)	325	Crofts (1975)
Uganda (Villages)	470	Burkitt et al, (1974)
Malaysia (agricultural community)	477	Balasegaran & Burkitt (1976)
Kenya	520	Cranston & Burkitt (1975)

The feaces excretion amount varies over a wide range even in a relatively homogenous group. Generally, active adults who live in rural areas and whose diet is rich in fiber excrete large amount of feaces than children and elderly persons who live in urban areas and eat foods low in fiber.

Table 6.13 has data on the chemical content of feaces and urine.

Table 6.13 Chemical content of feaces and urine

	Urine	Feaces
Excretion per capita	1.2L	150g
Nitrogen	11g/person/day	2g/person/day
Phosphorus	1g/person/day	0.6g/person/day
Potassium	2.5g/person/day	0.6g/person

The annual amount of feaces per capita ranges from 25 to 50kg. The latter amount contains 0.55kg of nitrogen, 0.18kg of phosphorus and 0.37kg of potassium. Though the nutrient content is less than that of urine, feaces function as an extremely favorable soil conditioner.

The movement to develop toilets that will separate urine and feaces for the purpose of 100% effective utilization of urine for fertilizer and feaces for soil conditioner is known as Ecosan (Ecological Sanitation)

6.8 Design Effluent Quality

6.8.1 Performance of existing sewerage Schemes

6.8.1.1 Influent BOD

On average, the influent BOD is strong at most treatment works indicating that individual water consumption is low. The samples obtained at the treatment works ranged from 875mg/l to 200mg/L. The average of 26 samples taken at different treatment plants was 480mg/L and approached the value of 550mg/L often used in Kenya

The estimated BOD per capita varied tremendously from 13grams/day to 134 grams/day.
The normal range in Kenya is 48 to 60g/c/day with a value of 50g/c/day used in design

6.8.1.2 Performance of the Treatment Works-BOD Removal

Only two treatment works, Bungoma and Kapsabet had a value of 20mg/L BOD. Others hand a range from 20 to 100mg/l.

6.8.1.3 COD

The BOD/COD ratio is an indicator of the biodegradability of the wastewater. Domestic wastewater is highly degradable with a typical ratio of 0.4 to 0.8. A comparison of data available for 27 facilities indicates that 9 facilities had a ratio greater than 0.5 indicating high organic content. The majority of treatment works had low ratios indicating high levels of non degradable industrial wastes or acute water shortages causing biologically degradable solids to settle in sewers.

COD values ranged from medium to strong indicating that there is potentially a large component of industrial or toxic liquid waste being discharged into public sewers. Effluent COD's in all cases exceeded standards.

6.8.1.4 Performance of the Treatment Works -Pathogen Removal

The level of fecal coliform is an indicator of the amount of pathogenic organisms in the wastewater. The results of the survey ranged between 0 to 1,275MPN per 100mL of sample indicating that most treatment works are effective at reducing coliform counts. Only two treatment works exceed 1,000MPN per 100mL: Homa Bay and Kariobangi Nairobi. In Kariobangi's case there are no maturation ponds and the high fecal counts are expected. In Homa Bay, the treatment works have maturation ponds but the facility is hydraulically overloaded probably because of high storm water flows and accumulation of sludge.

6.8.2 Receiving Streams

Samples were taken at 20 treatment facilities, upstream and downstream of the discharge point in order to gain a very crude measure of surface water quality issues. In 9 cases, the BOD of the receiving watercourse upstream of the treatment plant was higher than the effluent standard of 20mg/L indicating a serious surface water quality problem from other sources of pollution upstream. Samples of the Nairobi River taken upstream of the Dandora treatment works had BOD values that are similar to those of strong sewage. At 10 treatment works, the BOD downstream of the treatment plant was lower than the BOD upstream. This result appears to indicate that the effluent is contributing a significant amount of flow and diluting the natural watercourse (Source:-Aftercare Study by JICA, 1998).

6.8.3 Design Effluent Quality

6.8.3.1 Water Courses

To safeguard aquatic life, NEMA has prepared a standard which all sewerage works has to treat there wastes before discharging into any water course.

The characteristics of the standard are as follows:-

BOD5 at 20^0c	-30mg/l
COD	-50mg/l
Suspended Solids	-30mg/l
Coliform	-1,000 per 100ml
Helmith Eggs	-1,000 per llitre
Dilution at Discharge Point	-8 to 150 times dry weather flow (Adopted from Royal Commission 8th Report, England requirement)

Table 6.14 below has the details.

Table 6.14 Standards for Discharge into Natural Water Courses

Parameter	Maximum allowable (Limit)
pH	6.5-8.5
BOD (5days at 20^0c) not to exceed	30mg/l
COD not to exceed	50mg/l
Temperature not to exceed ^{0}c	±3 of ambient temperature of the water body
Total Coliform Count /100ml	1000
E.coli (Counts/100ml)	Nil
Colour	15 Hazen units (H.U.)
Total dissolved solids	1200mg/l
Total suspended solids	30mg/l
Oil and Grease (mg/l)- where conventional treatment shall be used	Nil
Ammonia , ammonium compounds, NO_3, compounds and NO_2 compound (Sum total of ammonia –N times 4 plus –N- and Nitrite-N)	100mg/l
Arsenic	0.02mg/l
Arsenic and its compounds	0.1mg/l
Benzene	0.1mg/l
Boron	1.0mg/l
Boron and its compound-non marine	30mg/l
Mercury (mg/l)	0.05
Cadmium not to exceed	0.01 mg/l
Cadmium and its compound	0. 02 mg/l
Carbon tetrachloride	0. 1 mg/l
Lead	0.01mg/l
Lead and its compound	0.1mg/l
Chromium VI	0.05 mg/l
Cis-1,2-dichloro ethylene	0.4 mg/l
Copper	1.0 mg/l
Zinc	0.5 mg/l
Dichloromethene	0.2 mg/l
Dissolved iron	10 mg/l
Dissolved manganese	10 mg/l
Fluoride	1.5mg/l
Fluoride and its compound (Marine and non marine)	8mg/l
Selenium	0.01 mg/l
Selenium and its compound	0. 1 mg/l
n-Hexane extracts (animal and vegetable fats)	30 mg/l
n-Hexane extracts (mineral oil)	5 mg/l
Nickel	3.0 mg/l
Oil and Grease	5 mg/l
Nitrate	20 mg/l
Phosphates	30 mg/l
Cyanide	2 mg/l
Sulphide	0.1mg/l
Hexavalent chromium VI compounds	0.5mg/l
Phenols	0.001mg/l
Simazine	0.03mg/l
Detergents	nil
Tetrachloroethylene	0.1mg/l
Thiobencarb	0.1mg/l
Thiram	0.06mg/l
Total Cyanogen	Nd
Mercury	0.005
Trichloroethylene	0.3mg/l
Whole effluent toxicity	Nil
Total Phosphorous	2mg/l
Total Nitrogen	2mg/l

Source: - The Environmental Management and Co-ordination (Water Quality) Regulations, 2006

Note:-

1. The standard adopted for Coliform count is that of ministry of water and irrigation since that of 30m/100ml suggested by NEMA is not achievable anywhere in the world.

6.8.4 Industrial Effluents Draining into Public Sewers

Both the quantity and quality of industrial effluents vary considerably, not only with the type of factory but also between neighboring factories producing similar end products.

Therefore, each individual factory on a proposed sewerage system must be considered separately. If the factory is existing, then the volume and also the concentrations of the more important constituents in the factory effluent should be measured; for a new factory; forecasts based upon factories elsewhere, which produce the same end products using similar working processes, probably give the most accurate estimates obtainable.

It is a requirement that industrial effluent be pretreated before discharging into public sewers.

Table 6.15 below has the details of the standards to be attained before discharging into public sewer.

Table 6.15 Standards for Industrial Discharge into Public Sewers

Parameter	**Standard**
pH	6.0-9.0
BOD (5days at 20^0c) not to exceed	500mg/l
COD not to exceed	1000mg/l
Temperature not to exceed ^{0}c	20-35
Oil and Grease (mg/l)- where conventional treatment shall be used	5
Ammonia Nitrogen (mg/l) not to exceed	20
Substance with an obnoxious smell	Shall not be discharged into the sewers
Arsenic (mg/l)	0.02
Mercury (mg/l)	0.05
Cadmium not to exceed	0.5 mg/l
Lead	1.0mg/l
Chromium (Total)	2.0 mg/l
Copper	1.0 mg/l
Zinc	5.0 mg/l
Selenium	3.0 mg/l
Nickel	3.0 mg/l
Nitrate	20 mg/l
Phosphates	30 mg/l
Cyanide	2 mg/l
Sulphide	2mg/l
Phenols	10mg/l
Detergents	15mg/l
Colour	Less than 40 Hazen Unit
Alkyl Mercury	Not Detectable
Free and Saline Ammonia as N	4.0mg/l
Calcium Carbide	Nil
Chloroform	Nil
Inflammable solvents	Nil
Radioactive residues	Nil
Degreasing solvents of mono-di-trichloroethylene type	Nil

Source: - The Environmental Management and Co-ordination (Water Quality) Regulations, 2006

7.0 SEWERS

7.1 General

Public sanitary sewers perform two primary functions:
• Safely carry the design peak discharge,
• Transport suspended materials to prevent deposition in the sewer.
It is essential that a sanitary sewer has adequate capacity for the peak flow and that it functions at minimum flows without causing operational problems.

Design all sanitary sewers as a separated system.

Wastewater can originate from four primary sources: 1) domestic, 2) commercial, 3) industrial development and 4) extraneous inputs. Extraneous flow such as infiltration and inflow enters a sewer through system defects and illicit connections.

7.2 Planning

Sewers should be planned in line with recommendations of chapter 7.

However, the following additions should be considered.

7.2.1 Alignment of Sewers

The alignment of sewers shall be determined by the need for sewer service, environmental constraints and economic feasibility. There are three major elements to a sewer alignment:

(1.) the route selected,
(2.) the horizontal alignment and
(3.) the vertical alignment

Each element needs to be considered in detail to ensure an economic alignment that provides the service required.

7.2.2 Route Selection

The route or streets to be traversed by a proposed sewer should be based upon sewer deficiency Studies and demand for sewage service.

7.2.3 Route Investigation

The Designer should review the sewer project route immediately upon assignment of the project. Environmental considerations should be considered. The need for the sewer project and the Economic feasibility should not be different than originally intended.

7.2.4 Field Reconnaissance

A field reconnaissance should be made to identify any changes in the conditions since the sewer Project was initially conceived.

a. Depending on the project size and complexity the following may be investigated:

b. Area to be served
c. General Topography. (Preliminary investigation may vary from a casual observationof field conditions to a detailed topographic and geological study).
d. Nearest available outlet sewers
e. Location and size of large surface and subsurface obstructions and improvements
f. Size and number of existing buildings (including basements)
g. Zoning
h. Future development of the area

The following references would be useful to prepare preliminary plans:

a. Drainage Maps
b. Past and Present Physical Development Plans
c. SK Maps scale 1:50,000 sheets
d. Respective Toposheets
e. "S" (Sewer) Maps
f. Substructure Maps
g. City Planning and Regional Planning data
h. Aerial Photographs
i. Flow Gauging Records
j. Census Data
k. Construction plans of existing improvements

7.2.5 Use of Photographs

Photographs should be taken of the route selected for reference purposes. Surface culture such as large overhanging trees, as well as, congested vehicular traffic, classes of property development, etc., should be recorded with the photographs.

7.2.6 The Horizontal Alignment

The most economical horizontal alignment will, generally, be the shortest length possible. The alignment may be varied to accommodate utilities, to maintain traffic safety and convenience, to equalize HC lengths on either side and to minimize other appurtenant work. The goal should be a horizontal alignment that fulfills the City's obligations to the public and utility companies, yet represents the shortest and most economical length possible. The Engineer should use all surveys and existing records, such as, cadastral maps, Sewer Maps, record and past surveys and other data available in identifying the most feasible alignment.

7.2.7 Substructures

Except under unusual circumstances, the sewer shall be located so that no portion of the existing Substructure lies closer than 0.3m (horizontally and vertically) from the limits of the sewer trench.

Close proximity to parallel electrical conduits and to high pressure water mains, gas, and oil lines and other high pressure mains shall be avoided, if possible, because of the hazards involved during construction. If the sewer to be constructed is in close proximity to a thrust block of a pressure main, the owner of the main shall be consulted and involved.

When practical, avoid long-skewed crossings under existing substructures. They are very costly because of the amount of tunneling, difficult excavation and special supports required.

7.2.8 Criteria for Separation of Sewers and Water Mains

To safeguard waterborne disease outbreaks attributed to the entry of sewage-contaminated groundwater into the distribution systems of public water supplies, the following separations are recommended:-

a. Parallel Construction: The horizontal distance between pressure water mains and sewer lines shall be at least 3m.
b. Perpendicular Construction (Crossing): Pressure water mains shall be least 0.3m above sanitary sewer lines where these lines must cross.
c. Separation distances specified in (a) and (b) shall be measured from the nearest edges of the facilities.
d. Common Trench: Water mains and sewer lines shall not be installed in the same trench.

When water mains and sanitary sewers are not adequately separated; the potential for contamination of the water supply increases. Therefore, when adequate physical separation cannot be attained, an increase in the factor of safety should be provided by increasing the structural integrity of both the sewer pipe materials and joints.

7.2.9 Right-of-Way

Sewers shall be placed in public right-of-way whenever possible.

7.2.10 Sulfates Attack

In certain locations, sulfates, alkalies, and salt water may be present. These are harmful to concrete and steel if found in sufficient quantities. Chemical analysis of the groundwater or soil sample should be made to determine the quantity of these substances present. The most active of these harmful substances are the sulfates, which shall be considered in the design when they are found in sufficient concentrations as shown in Table 7.1

Table 7.1 Sulfate Concentration Harmful to Concrete

% Water Soluble sulfate (as SO_4) in Soil Samples	**Ppm sulfate (as SO_4) in Water**	**Relative degree of attack on concrete**	**Suggested Type of resistant Concrete**
To 0.10	To 150	Negligible	-
0.10 to 0.20	150 to 1000	Positive	Type II
0.20 to 0.50	1000 to 2000	Considerable	Type V
0.50 and over	2000 and over	Severe	Type V

7.3 Design Criteria

Sewerage over the years has faced investment crunch more than Water Supply.

As means of Sewerage Promotion, there is need to embrace new and emerging “Appropriate Technologies” viz:-

- Shallow Sewer Systems and
- Simplified Sewerage

Further, some of past Designs have adopted design recommendations from W.H.O Report No. 9 on minimum velocities of 0.75m/sec and minimum gradient varying with the diameter of the pipe leading in some cases of over excavations beyond 5m hence making them too deep for connection. A case in point is the existing Garissa Sewerage System which was constructed over 30 years ago but the number of connections has stagnated at 400 due to sewers being too deep for any meaningful connections.

This is not an isolated case and I believe other examples exist in the Country.

In view of the above shortcomings, sections 7.3.1 to 7.3.2.5 have been lifted from:-

- The Design of Shallow Sewer Systems Manual by UNCHS HABITAT dated 1986 and
- World Water Magazine of January/February 1986 Magazine titled " Shallow Systems Offer Hope to Slums" in Northern Brazil and Pakistan by Gehan Sinnatamby, Duncan Mara and Michael MC Gary and
- Simplified Sewer by Delft University, Netherland downloaded from the Internet

To cope with the emerging trend, lower per capita cost and address past shortcomings and experience.

7.3.1 House Connections

All household wastewaters are drained to an inspection chamber located along the length of a common block sewer line, usually through two or more pipe connections. One of these is for the water closet connection and, depending on the distribution and number of sullage generating areas within each house, one or more sullage connections are also provided. Where a grit/grease trap is provided, it is usual to connect all the sullage to it and provide a single connecting pipe to the inspection chamber.

7.3.2 Water Closet House Connection

This connection is the short length of pipe that drains the water closet to the inspection chamber. A vertical ventilation column is usually provided somewhere along the length of the connection, often adjacent to the exterior wall of the house. The following specifications are recommended for the water closet house connection:

Pipe materials for house connection and ventilation column	*PVC*
Pipe diameter for house connections and ventilation column	*Minimum 75mm*
Pipe gradient for house connection	*Minimum, 1 in 50*

Manually flushed squat pans, using 3liters of water per flush, have been shown to be capable of transporting simulated waste materials an average distance of over 5meters along 75mm diameter pipes laid at a gradient of 1 in 100. Since the layouts of shallow sewers are such that the common block sewer line is adjacent to the toilets, the water closet connections are usually short in length and rarely exceed 5m. even so, a minimum gradient of 1 in 50 is recommended to take account of variations in site and sewer layout. Clearly, the shallowest gradients may be adopted where pedestal seats using large flush volumes are adopted. In any event, a gradient not flatter than 1 in 100 is recommended.

7.3.3 Sullage House Connections

Where water consumption is low (25 to 30Lcd) and where sand, brickbats and other abrasive materials are customarily used for cleansing utensils, all sullage house connections must be preceded by some form of grease/grit trap, especially for kitchen wastewaters. Even where water consumption is high and detergents are used for cleaning utensils, it is advisable to provide this preventive maintenance device. When the overflow pipe from these devices, i.e., the sullage house connection, is free from gross accumulation of solids, it may be laid at a very flat gradient. Direct connections to the common block sewer inspection chamber may also be made from laundry and bathing areas, including also kitchen wastewaters, in areas where the average water consumption is over 75Lcd. The following specifications are recommended for sullage house connections:

Pipe material	***PVC***
Pipe Diameter	***Minimum 38mm but usually 50mm***
Pipe gradient	***Minimum, 1 in 200***

7.3.4 Common Block and Street Collector Sewer

Within a block, all water closet and sullage house connection pipes are connected to a single pipe known as the common block sewer. Where the natural drainage path within the block is unidirectional, only a single block sewer is necessary. Where this is not the case, in order to minimize excavation and follow the natural fall of the land, it may prove necessary to adopt more than one common block sewer.

The common block sewers are designed to be flushed frequently, and therefore as many houses as possible should be connected to them. They should be located adjacent to the toilets they serve and are, hence, usually located at the backs of houses. In certain housing layouts where the toilets are located at the front, block sewers are also provided at the fronts. When the block sewer emerges from the block, it is connected to a street collector sewer. Since block sewers and street collector sewers operate in a similar manner, criteria for their design are discussed together below.
Traditionally, the design of sewers has been based on the concept of ensuring that peak daily flows carry away any solids deposited during periods of low flow. These self cleansing velocities occur at least once a day in the large downstream sewers but not usually in the small diameter house connections and street collector sewers in the upper reaches of the network where the flow is intermittent. Recently, however, the concept of self cleansing velocities in sewers as a basis for their design has been questioned by many researchers. It has been found that, although solids are deposited in pipes, they become re-suspended and transported on subsequent flushing, as the pressure force caused by the difference in depth of water across the solids increases. Solids are therefore shifted along the pipes by the flushing action of sequential waves of wastewater, and, as solids deposit in the pipe invert, wastewater backs up until the pressure is sufficient to move the solids forward. Based on these findings, it has been suggested that there is no reason to attempt to design a totally deposit free house connection or sewer.

A comprehensive study of the causes of blockages in house connections, conducted in the United Kingdom, concluded that most sewers experienced intermittent stoppages during normal operation and that these were removed by wave action rather than by the maintenance of a minimum scour velocity. Blockages were observed to be removed efficiently as the depth of flow increased and were primarily the result of poor workmanship and infrequent use of the system, rather than of limitations in pipe diameter or gradient. It would appear, therefore, that recent studies undertaken on the operation of sewers do not support the current design practice of restricting the use of flat gradients by working to a minimum mean velocity.
Unable to model the true mode of operation of sewers, especially at the upper reaches of a network, engineers have resorted to rules of thumb which have become increasing conservative over the years, thus adding unnecessarily to the cost of sewerage. Current knowledge still offers no alternative to using the self cleansing velocity design concept. It is on this principle that procedures have been developed for the design of shallow sewers, although recent research findings have enabled substantial changes to be made in design standards and criteria to promote cost reductions. These are discussed below.

7.3.5 Minimum and Maximum Velocities

Minimum peak daily velocities in pipes have been said to lie between 0.76m/sec and 1.0m/sec.

However, laboratory studies have shown that solids do not decelerate until they reach a threshold velocity below which no further motion is possible; this velocity, depending on solid shape, pipe size and slope, relative pipe roughness etc., was found to be between 0.2 and 0.4m/sec. Inert material will, however, be deposited at velocities below 0.3m/sec. Accepting this findings and assuming that sewers do not operate on the **principle** of **self cleansing velocities** and that, when solids are deposited they become re-suspended and transported by subsequent flushes, a minimum flow velocity of 0.5m/sec is considered adequate. In Brazil, sewers have been designed for this value of self cleansing velocity for over two decades. Maximum velocities of flow have, in the past, been specified, in order to reduce the possibility of pipe erosion. Such effects were said to occur at flow velocities in excess of 4.0m/sec, but studies have shown that erosion effects observed at velocities greater than this threshold value are only minimal, and hence no upper limit of flow velocity is recommended. Thus, the minimum and maximum **sewage design velocities** can be stated as follows,

Minium Velocity *0.5m/sec*
Maximum Velocity *No limit required*

7.3.6 Minimum and Maximum depth of flow

Minimum and maximum proportional depths of flow, at peak flows, are usually specified for pipes conveying sewage, in order to ensure that a sufficient sewage flow for solid transport and an adequate surplus capacity are provided in the pipe, a minimum proportional depth of **0.2** is considered necessary for solid transport, and a maximum proportional depth of **0.8** in order that approximately 8% surplus capacity is available to provide a small margin of safety against underestimation of the sewage flow.

However, in the absence of exact design methods, the maintenance of a minimum and a maximum proportional depth is recommended as follows:

Minimum depth of flow in pipe	*0.2 times pipe diameter*
Maximum depth of flow in pipe	*0.8 times pipe diameter*

7.3.7 Minimum Sewer Diameter

Since small diameter pipes have shown to have better solid transport properties than large diameter pipes, because of their ability to build greater depths of water behind solids, there is no justification for using pipe diameters in escess of 100mm, except when required by the magnitude of the flow. In fact, it is definitely not advisable to increase the pipe diameter under the reduced falls proposed for shallow sewers, since this decreases the depth of flow, and solids will thus tend to be deposited frequently in the pipeline. The following recommendation is made regarding the minimum sewer diameter:

*Minimum sewer **diameter 100mm***

7.3.8 Minimum Sewer Gradient

A wide range of minimum gradients has been recommended in the past. The Study of the causes of sewer blockages in the United Kingdom found that sewer gradients, as with pipe diameters, were not determining factors in causing sewer blockages. House connections laid at gradients as flat as 1 in 1200 were found to operate satisfactorily for many years. In fact, the investigation discovered most blockages on the commonly recommended gradients of between 1 in 30 and 1 in 70. British Practice recommends that a 100mm diameter pipe be laid at a gradient not flatter than 1 in 70, provided it serves the equivalent of at least one water closet, and 1 in 80, when serving when serving up to five houses. As pipe diameter increases, it is often common to see recommendations to adopt shallower gradients: a minimum gradient of 1 in 150 is often recommended for 150mm diameter sewer pipes. **Minimum pipe gradients are, however independent of pipe diameters**. Using Manning's equation, it can be shown that the gradient required for a pipe is given by the following equation:

$$\mathbf{I = Q^{-2/3} V^{8/3} 1.59 n^{2} (\o \sin \o)^{-2/3} \o^{4/3}}$$

Where;

- I =Gradient, m/m
- Q =Flow Rate, L/sec
- V =Velocity of flow, m/sec
- Ø =angle subtended at the centre of the pipe by the wetted perimeter, radians
- N =roughness coefficient

Taking the limiting conditions (in the upper reaches of a sewer network) of a minimum depth of flow of 2.2 times the diameter (0=1.85459) and a velocity of flow of 0.5m/sec, and assuming Manning's roughness coefficient n=0.013, the minimum pipe gradient is given by:

$$\mathbf{I = 0.01 Q^{-2/3}}$$

It follows that, for a given discharge the minimum permissible gradient is independent of the pipe diameter.

The above equation may be used to obtain minimum gradients for a range of flows. Brazilian practice recommends that the flow from a single cistern flush unit of 2.2L/sec be used to represent the minimum discharge at the head of any sewer system. Clearly, search discharges are rapidly attenuated in the house connection as the solids move away from the water closet, and they are never obtained in practice in the sewers, but no problems have resulted from following this assumption. It is interesting to note that over 200 households with an average of five persons per household and an average flow of 100Lcd would be required to generate a flow equivalent to 2.2L/sec. The minimum gradient corresponding to a discharge of 2.2L/sec is 1 167(0.6 per cent), and this attains a maximum self cleansing velocity of 0.5m/sec. The recent success with minimum gradients of 1 in 167 in shallow sewer systems in Pakistan, in settlements where water consumption averaged only 27Lcd and where manually flushed squat pans were utilized for excreta deposition coupled with the fact that sewers do experience intermittent formations of blockages, would suggest that very flat gradients may be safely adopted in areas of medium and high water consumption.

However, in the absence of additional field data, a minimum sewer gradient of 1 in 167 is recommended.

Minimum sewer gradient *1 in 167*

7.3.9 Minimum Depth of Sewer

Usually, the minimum depth of block sewers is determined by the depth to invert of water closet and sullage house connections. Because the length of these connections is usually short, it is possible to lay pipes at a minimum depth to soffit of pipe of only 0.2m. However, in order that accidental damage to pipework may be avoided, it is prudent to adopt a minimum soffit cover of 0.3m. This implies that a 100mmdiameter block sewer will be laid at a minimum trench depth only marginally greater than 0.4m. While such depths are acceptable in areas not subject to vehicular loadings. Continuation of the use of such depths, once the block sewer emerges into the street, necessitates careful location. Where it is inevitable that vehicular traffic pass over the line, pipes must be laid with a minimum cover to soffit of 0.8m or, alternatively, the pipe must be protected with a concrete collar.

7.3.10 Pipe Flow Formula

Out of several formulae commonly used to design sewers, the manning formula is probably the most widely used. Although recent research has demonstrated some limitations of the Manning formula; for example, it takes insufficient allowance for the independence to flow caused by slimes which can build up in sewers, nevertheless it is recommended that it be used for sewer designing in Kenya.

The Manning formula is $(1/n)R^{2/3}S^{1/2}$

Where; n= Manning Constant
R=Hydraulic Radius in m
S=Hydraulic Gradient in m/m

The Manning formula contains a friction coefficient "n" in the Manning which varies with condition of interior surface of the sewer. Proposed values of "n" for various types and sizes of pipes are given in Table 7.2.

If the value 0.013 is substituted for "n" in the Manning formula, it becomes Crimp and Bruges! Formula; the results of calculations based on this formula are available in convenient tabular form so that sewer design is very much simplified.

Where the value of "n" differs from 0.013, the Crimp and Bruges! Tables may still be used provided that the sewer capacities and flow velocities they give are reduced by an appropriate percentage.

It is suggested that, when designing smaller diameter sewer, the values given in the Crimp and Bruges! Tables should be modified in order to allow for possible changes to sewer gradients during construction, to carter for unforeseen localized estate-type development and also to allow for inaccuracies in forecasting the points where the sewage draining from future development will enter the sewerage system.

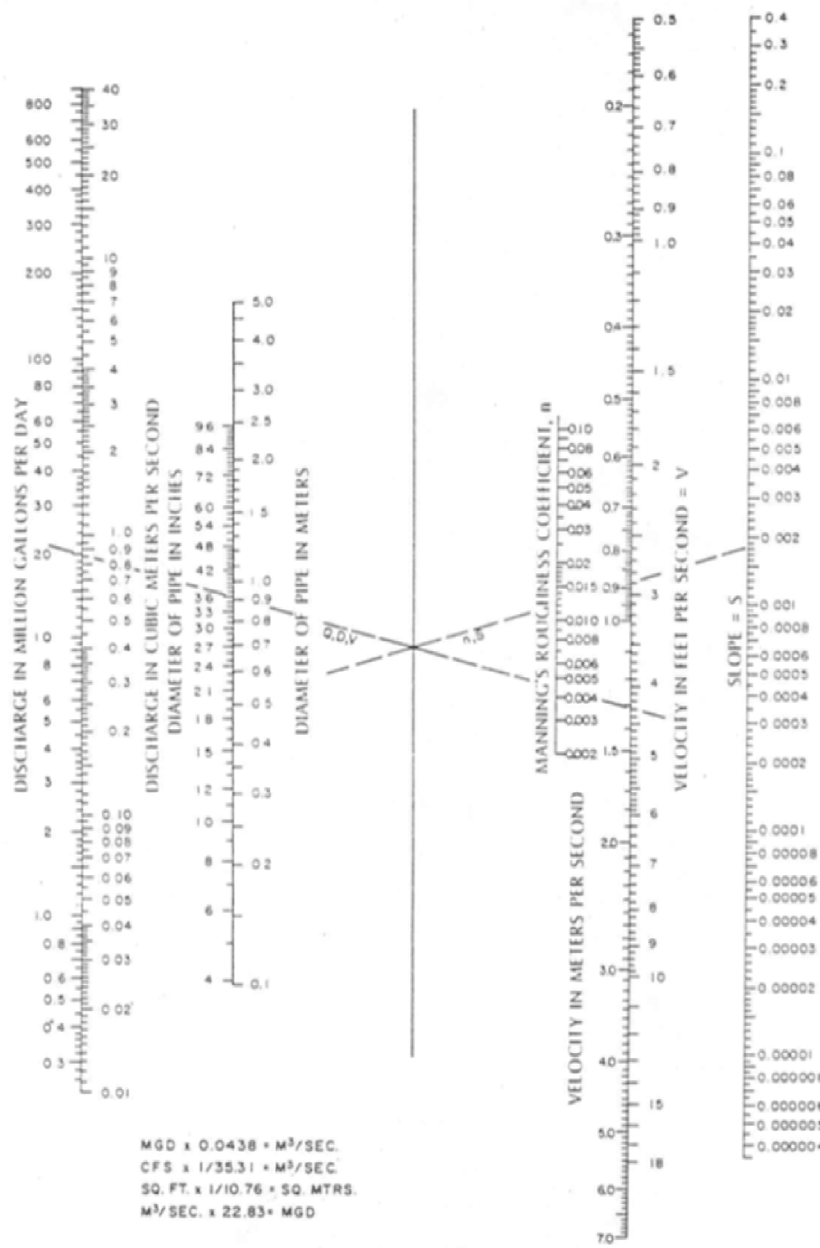
Table 7.2 also shows the factors which it is proposed the Crimp and Bruges! Values should accordingly be multiplied.

It should be noted that the Crimp and Bruges! Tables apply to pipes running full (but not surcharged).

Figure 7.1 shows how the discharge and velocity in a sewer vary with depth of flow; particular points are that the discharge capacity of a sewer running full is exactly twice that of the sewer running half-full, but the velocity of flow is the same in each case.

Table 7.2 Multiplying Factors for use with Crimp and Bruges Tables

Pipe Material	Pipe Diameter		Friction Coefficient "n"	Percentage adjustment to Crimp and Bruges flows and velocities because of varying "n" values	Percentage adjustment for smaller size sewers (see Text)	Multiplying factor (rounded off)
	mm	**Inches**				
Spun Concrete	300 or less	12 or less	0.015	87	75	65
	Greater than 300 but less than 600	Greater than 12 but less than 24	0.015	87	90	80
	600 or more	24 or more	0.014	93	100	95
Cast Concrete	300 or less	12 or less	0.018	72	75	65
	Greater than 300 but less than 600	Greater than 12 but less than 24	0.018	72	90	75
	600 or more	24 or more	0.013	100	100	75
PVC	300 or less	12 or less	0.013	100	75	75
	Exceeding 300	Exceeding 12	0.013	100	90	90
Pitch Fibre	100 and 150	4 and 6	0.014	93	75	70



MANNING'S FORMULA: ENGLISH $V = \frac{1.486}{n} R^{2/3} S^{1/2}$

METRIC $V = \frac{1}{n} R^{2/3} S^{1/2}$

Fig. 7.1 Alignment Chart for Flow in Pipes, Manning Formula

GRAVITY SANITARY SEWER

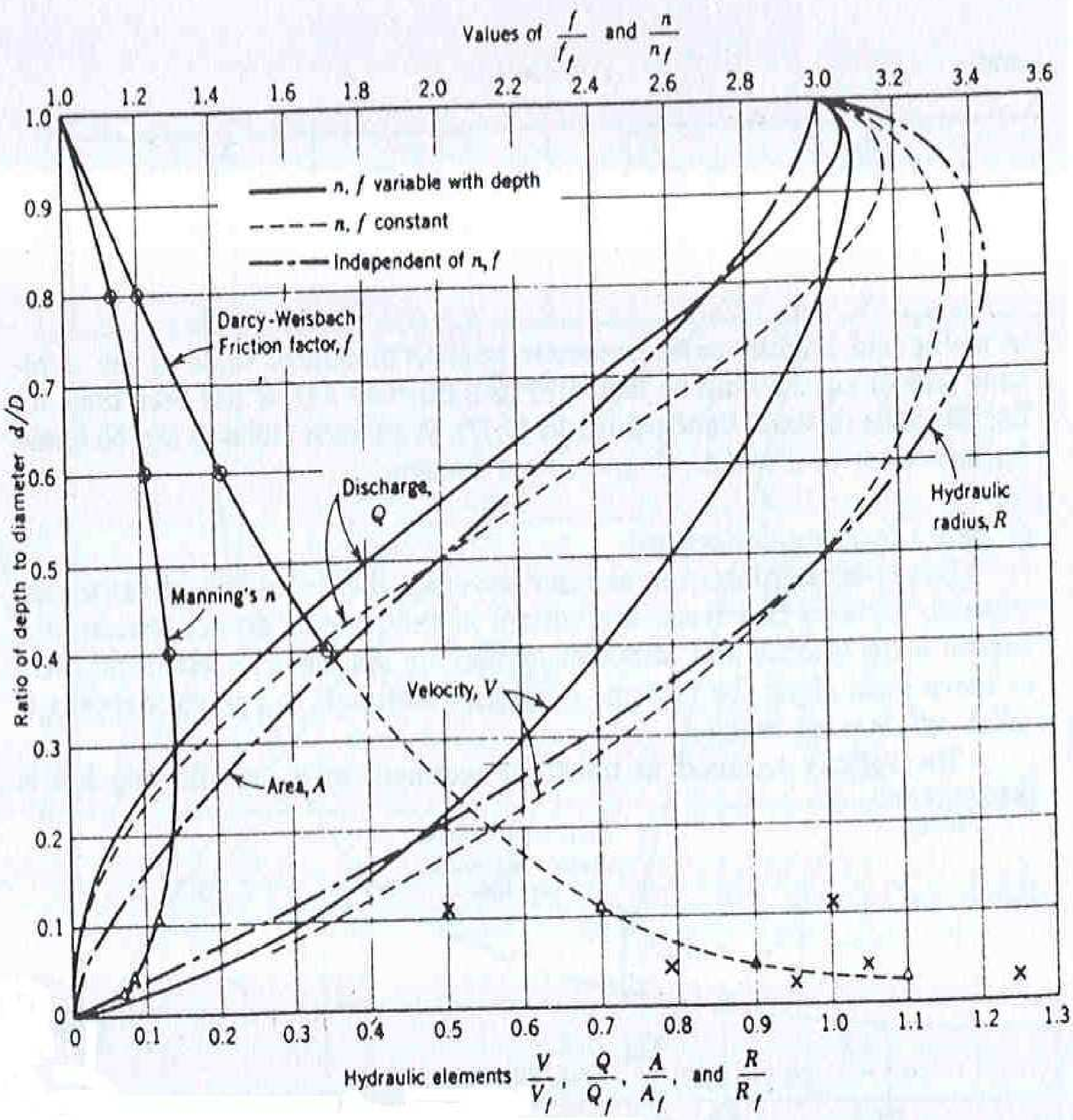


Fig. 7.2 Hydraulic Elements Graph for Circular Sewers

7.4 Flows

7.4.1 Domestic Sewage

Forecasts of the quantity of Domestic Sewage likely to be discharged into each length of sewer may be made using the data given in Section 6 of the Report.

In Kenya, the ratio between the peak and daily rate of flow and the average daily flow of domestic sewage is of the order of 2½. However, it would be uneconomical to use this factor to design all the sewers in a large sewerage network.
Sewage takes a finite time to flow through a sewerage system; as a result, peak flow rates tend to decrease as the extent of the network and therefore the population served, increase.
Babbitt, in his book "Sewerage and Sewage Treatment", suggests that the relationships between peak sewage flows and population served is

$$\mathbf{M=5/p^{1/5}}$$

Where M is the ratio of peak to average flow rate and P is the contributing population in thousands. In practice, it is usually more convenient to link peak flows to pipe sizes and the following peak flow factors are suggested for use in Kenya.

Table 7.3 Peak Flow Factor

	Peak Flow Factor
A For sewers 12 inches (300 millimeters) in diameter or less	2½
B For sewers exceeding 12 inches (300 millimeters) but less than 24 (600 millimeters) in diameter	2
C For sewers 24 inches (600 millimeters) in diameter or more	1½

The designations A, B and C above are identical to those given in sub-section 4.4 below.

If, as an example, an 18 inch (450 millimeter) diameter sewer will receive neither industrial effluent nor storm water nor groundwater, then, in accordance with Criteria 1 in sub-Section 6.4, the sewer should have a capacity equivalent to the average daily design flow times 2 times 2. Thus, when running full, the sewer will have capacity equivalent to double the peak domestic sewage flow; that is, it will take the peak domestic sewage flow when running half full.

7.4.2 Industrial Effluents Discharging into Public Sewers

When designing sewers, it is important to distinguish between wet weather and dry weather flows from those factories which have outside yards draining into sewers. In this sub-section, only dry weathers effluent flows are considered; storm water run-off from factory yards as discussed under 6.

Few generalizations can be made about either the average daily flow or the maximum instantaneous rate of flow from industrial premises; each factory should be considered separately.
Records of readings of the meters on supply mains give some guidance to factory sewage flows; however, when estimating industrial effluent quantities, care must be taken to include not only for water taken from public mains but also for the water privately abstracted by the factory from surface or underground sources.

In sub-section 6 below, the varying rates between the instantaneous and the daily average rates of flow were denoted by the letters “D”,”E” and “F”, etc: in practice, it may be restricted to a maximum or three by effluent discharge agreement; that is, if necessary, factories will be forced to install flow “balancing” arrangements.

As in the case of domestic sewage, sewers should be designed to deal with the peak dry weather industrial effluent flow when running half-full.

It is appropriate to assume that, as the necessary sewer size increases, the industrial peak flow factors will decrease in the same way, and by the same ratios, as for domestic sewage(See Clause 6.3.4.1 above)

7.4.3 Storm Water Entering Public Sewers

When designing sewers, storm water should be categorized as follows:-

i) Unauthorized and unavoidable storm water entry, such as from illicit connections or because of leakage through manhole covers, etc.
ii) Authorized storm water from paved open markets and like and from certain factory yards.

Unauthorized and Unavoidable Storm Water

A sewer designed as suggested in the section has spare capacity equivalent to peak flows of domestic sewage plus peak industrial effluent flows of domestic sewage plus the peak industrial effluent flows. It is customary, and reasonable, to assume that this surplus capacity is sufficient to deal with all authorized and unavoidable storm water which finds its way into sewers; therefore, this type of storm water may be ignored for design calculations.

It must be appointed out, however, that this spare capacity will be sufficient only if the utmost care is taken to prevent the entry of storm water into sewers, except where specifically authorized. The fact that such an allowance has been made must not be made as an excuse to permit the indiscriminate entry of storm water into sewerage systems; such practice must always be considered illegal if sewers are not to be overwhelmed during periods of heavy rain.

Authorized Storm Water

An estimate of this must be made in order to apply Criteria 1 given in sub-section 4.4.
Estimates of the quantity and rate of flow of storm water from any particular area may be made using the formula:-

$$Q=0.85I\,W$$

Where; Q the quantity of storm water entering the sewer,
I the rainfall intensity,
W the size of the area which is draining

The units used for Q, I and W must correspond; for example, if Q is in cubic feet per second, “I” must be in cubic feet per acre and A must be acres.

The value of “I” varies throughout Kenya; everywhere, it is inversely proportional to the duration of storms. When designing a partially separate sewerage system, “I” can be taken as rainfall intensity of a 30 minute storm with an average frequency of five years, for the particular town under consideration.

The intensities of 30 minute storm with average frequencies of five years for particular locations in Kenya are as shown in Fig. 7.3 below:-

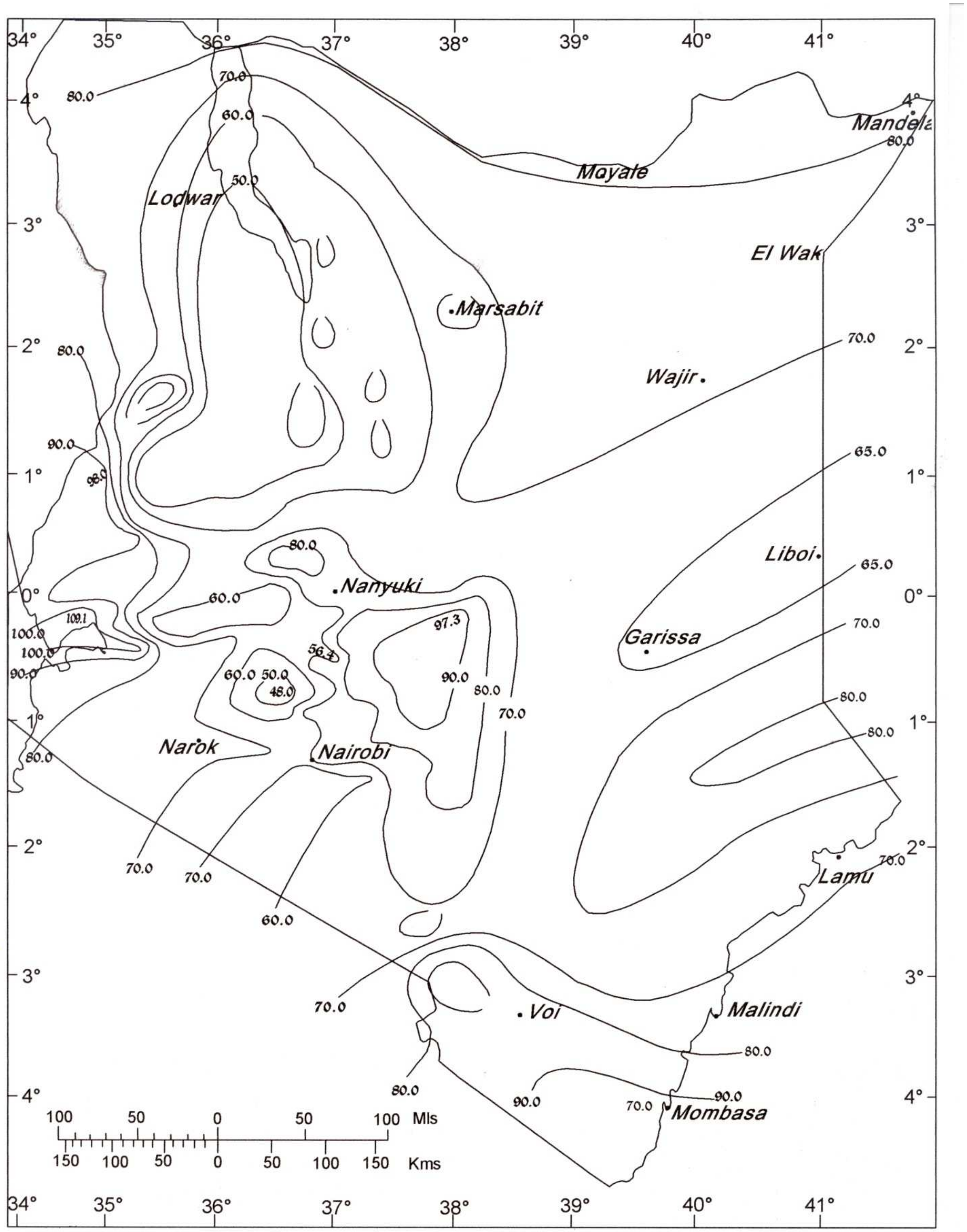


Fig. 7.3 Five-year 30-minute Rainfall Intensity in mm/hr

3 inches depth of rain falling at a uniform rate during one hour on one acre produces a run-off of 3.024 cubic feet per second.

Substituting this intensity in above formula, Q=0.85X19W=16W (where Q is in liters per second and W is in acres) Or Q=178 W (where Q is in liters per second and W is in hectares)

It should be noted that run-off from a yard measuring one acre is equivalent to approximately 1.3 million gallons per day, or the average daily volume of sewage produced by 1000 acres of flow density, high class housing, or 1000 acres of high density, low cost housing with individual water closets and ablution facilities. This serves to emphasis that the flow of storm water into sewers should be kept to an absolute minimum, consistent with maximum safeguards to the public health.

By the same reasoning which allows the peak domestic sewage flow factor to be decreased as sewers increase in size, so the rainfall intensity on any particular area may be considered to reduce by 20 per cent for every mile (1.6 kilometers) length of trunk sewer below each yard discharging storm water into the sewer; that is, if the volume of storm water is discharged into a sewer from a particular yard is calculated to be at a rate of one million gallons per day, then when determining the size of a sewer located one mile below the yard, the run-off may be taken as 800 000 gallons per day only; for a sewer two miles below the yard, the storm water flow may be taken as only 640 000 gallons per day, and so on.

Groundwater infiltration

Where a sewer is located below the water table, groundwater infiltration depends upon the water tightness of manholes, property drains and sewers, and particularly that of their joints, and upon the nature of sub-soil; infiltration is greater in gravels and coarse sands than in cohesive clays.

Infiltration into properly constructed sewerage system should be relatively low and will tend to decrease with time as fine particles from the sewage or sub-soil block small leaks.

The United Kingdom "Code of Practice for Sewerage" recommends that, during testing with water at 4 feet (1.2 meters) head, a sewer should not lose more water than 0.2 gallons per inch nominal diameter of the sewer per hour per meter length of sewer per meter of nominal diameter). Unless the depth of sewer below the water table greatly exceeds 4 feet (1.2 meters) infiltration should be of the same order.

For a 9 inch (225 millimeter) diameter sewer, this means that infiltration should be the order of 2000 gallons per mile per day (equivalent to approximately 5000 liters per kilometer per day). In comparison to the likely sewage inflow, infiltration is relatively small in quantity and it is customary to assume that it is catered for by the surplus capacity normally built into each sewer (as in the case of unauthorized and unavoidable storm water – see sub-section 6.3.4.3).

This assumption cannot however be made in the case of existing sewerage systems in Kenya; poor workmanship on sewers and poor control of property drains connected into sewers has inevitably resulted in sewerage systems which are not watertight. Therefore, when dealing with existing sewers in water-logged ground, possibly in order to incorporate them into a new sewerage system, it is essential that the actual infiltration rates are measured so that so that appropriate allowances can be made.

These measurements may normally be made at about 3.a.m following a period when the rainfall has been heavy, and the water table is therefore high; at this time of night, it may be assumed that sewage flows will be negligible (except that allowance must be made for industrial effluents if any factories work overnight) so that the flow in sewers are then virtually entirely due groundwater infiltration.

7.5 Pipes and Manholes

There are several Kenya Standards covering sewer pipes and manholes which correspond to the British Standard Specifications.

However, it seems that some products which are claimed to be in accordance with these Specifications are frequently of inferior quality; example, in the case of concrete sewer pipes, control over the cleanliness of concreting sand and the proportion of water added is not always sufficiently strict.

Types of Sewer Pipes Available in Kenya

Only three types of sewer pipes are manufactured in this country; Viz Concrete, PVC pipes, Pitch Fiber pipes. The sizes are as per table 7.4 below.

Table 7.4 Standard Pipe, Sewer and Fittings diameters in mm

Ref Letter	Pipe material stated	Clay	Concrete	Ductile Iron	Welded Steel	UPVC (NOD)	Asbestos Cement	GS or GI	Polythene
a	10	75	150	40	80	20	50	6	10
b	15	100	225	50	100	25	60	8	15
c	20	150	300	((60))	150	32	75((80))	10	20
d	25	225	375	65	200	40	100	15	25
e	32	300	450	80	250	50	125	20	32
f	40	375	525	100	300	63	150	25	40
g	50	450	600	125	350	75	175	32	50
h	65	525	675	150	400	90	200	40	65
i	80	600	750	200	((450))	110	250	50	80
j	100	675	825	250	500	125	300	65	100
k	125	750	900	300	((550))	140	350	80	125
l	150	825	975	350	600	160	400	100	150
m	200	900	1050	400	((650))	200	450	125	200
n	250		1125	-450	700	225	500	150	250
o	300		1200	500	((750))	250	600		300
p	350		1350	600	800	280	700		
q	400		1500	700	((850))	315	800		
r	450		1650	800	900	355	900		
s	500		1800	900	1000	400	1000		
t	600		1950	1000	1200	450	1200		
u	700		2100	1200	1600	500	1400		
v	800		2250	1600	1800	560	1500		
w	900		2400	1800	2000	630	1600		
x	1000		2550	2000			1800		
y	1200		2700				2000		
z	1400								

Source: Modified Civil Engineering Standard Method of Measurement for Kenya (Modified from Civil Engineering Standard Method of Measurement, 1999 by Institution of Civil Engineers, London).

Concrete Pipes

There are local companies which, manufacture concrete sewer pipes in all sizes from 6 inches (150 millimeters) diameter up to 48 inches (1 200 millimeters) in three inch (75 millimeters) increments; four inch (100 millimeters) diameter pipes are also made. All locally manufactured concrete sewer pipes are nominally in accordance with KS 02-548:1986 (Concrete Pipes -drainage).

Some manufactures are spun; pipes are available in lengths of up to 6 feet (1.8 meters), even for the larger sizes. Some manufacturer, whose pipes are cast in vibrated moulds, provides flexible joints, incorporating rubber rings; the other manufacturers supply spigot and socket pipes which need jointing with hessian and cement mortar.

The sewer pipes are normally unreinforced and only one wall thickens per pipe diameter are available; however, some of the manufacturers say they could provide extra strength pipes if required.

Occasional hydraulic and crushing pipe tests are carried out, on request, by the Ministry of Works. The manufacturers do not normally test their own products.

PVC Pipes

There are three types of drainage pipe depending on the thickness of the walls viz: - :Low, Medium and High Grade. Pipes up to 12 inches (300 millimeters) in diameter are made in Kenya from PVC imported in bulk. Joints are flexible and are sealed by rubber rings; each length of pipe has a socket affixed to the pipe during manufacture.

These sewers pipes are nominally in accordance with KS 06-217:1981(Specification for unplasticized PVC pipes and fittings for buried drain and sewer but there have been occasional cases of eccentricity resulting in variable thicknesses of the two pipe walls.

When the total cost of transport plus laying and bedding sewers is taken into account, PVC sewers are competitive with concrete sewers.

Pitch Fiber Pipes

Pitch fiber pipes up to 8 inches (200 millimeters) in diameter are shipped from Europe, to receive their final machining in Kenya. This procedure probably ensures that they are in accordance with B.S. 2760.

Joints are made by pushing the machine-tapered ends of the pipe into Polypropylene collars.

In spite of shipping costs and a high proportion of breakages, the costs of constructing property drains using the smaller sized pitch fiber pipes are comparable with those of other types of sewer pipes.

Trunk and Branch Sewer Construction

It is proposed that trunk and branch sewers up to diameters of 9 inches (225 millimeters) shall be constructed using PVC pipes.

In Kenya today, concrete pipes are usually bedded and hunched or surrounded by concrete. However, there are several instances where rigid-jointed concrete sewers have been laid directly upon the bed of the trench; this practice has resulted in many fractured, leaking sewer hence not recommended.
There is rarely any attempt to reconcile the strength of the sewer pipe and its bedding with the external load the sewer has to withstand during its life-time. This load can be estimated from the formula:-

$$W = CwB^2$$

Where;

W = the load on the pipes in pounds per linear feet of trench,
B = the width of the trench just below the crown of the pipe, in feet,
C = the coefficient of the soil which varies with the depth/width ratio (see table 4.5),
w = the weight of the trench filling in pounds per cubic meter foot (see table 4.6).

This formula assumes that the trench width has been kept to a minimum that is roughly in accordance with formula:-
Trench width = 1.5x the internal diameter of the sewer in inches 12 inches.
It is usual to allow a safety factor of between 1.5 and 2 which calculating external loads, to allow for trench widths being cut wider than minimum during construction and for errors in categorizing the type of soil.

By using a suitable bedding, usually of granular subsoil or concrete, the strengths of pipes is relative to external loading may be increased by a "bedding" factor. Figure No. 4.4 gives details of typical pipe beds, together with notes as to their limitations and bedding factors. It should be noted that the range of possible pipe beds is very limited if the pipes have rigid joints.

Currently, concrete pipes are supplied in a wide range of strengths and hence flexible joints can be used; flexible joints have additional important advantage they can be made using only semi-skilled labor.

PVC sewers pipes are slightly flexible and are able to carry high external loads only by deriving additional supporting strength from literal restraint provided by the soil at the sides of the pipe. These pipes should therefore always be backfilled by a free-draining, incompressible granular material (for example, three quarter inch (20 millimeters) graded gravel at least to the top of the pipe. It is normally desirable to bed the pipe on similar material, so as to give even support and to prevent any isolated stones or tree roots in the trench bed fro damaging the pipes.
Where sewers are so shallow as to be in danger from foundations or from excavations for other public utilities, they should be protected by concrete arches or surrounds.

Table 7.5 Values of C for use in Formula $W=CwB^2$

Ratio of Depth to trench width	Sand and Damp topsoil	Saturated topsoil	Damp clay	Saturated clay
0.5	0.46	0.46	0.47	0.47
1.0	0.85	0.86	0.88	0.90
1.5	1.18	1.21	1.24	1.28
2.0	1.46	1.50	1.56	1.62
2.5	1.70	1.76	1.84	1.92
3.0	1.90	1.98	2.08	2.20
3.5	2.08	2.17	2.30	2.44
4.0	2.22	2.33	2.49	2.66
4.5	2.34	2.47	2.65	2.87
5.0	2.45	2.59	2.80	3.03
5.5	2.54	2.69	2.93	3.19
6.0	2.61	2.78	3.04	3.33
6.5	2.68	2.86	3.14	3.46
7.0	2.73	2.93	3.22	3.57
7.5	2.78	2.98	3.30	3.67
8.0	2.81	3.03	3.37	3.76
8.5	2.85	3.07	3.42	3.85
9.0	2.88	3.11	3.48	3.92
9.5	2.90	3.14	3.52	3.98
10.0	2.92	3.17	3.56	4.04
11.0	2.95	3.21	3.63	4.14
12.0	2.97	3.24	3.68	4.22
13.0	2.99	3.27	3.72	4.29
14.0	3.00	3.28	3.75	4.34
15.0	3.01	3.30	3.77	4.38
Very great	3.03	3.33	3.85	4.55

Source: Iowa State University Eng. Sta. Bull. 47

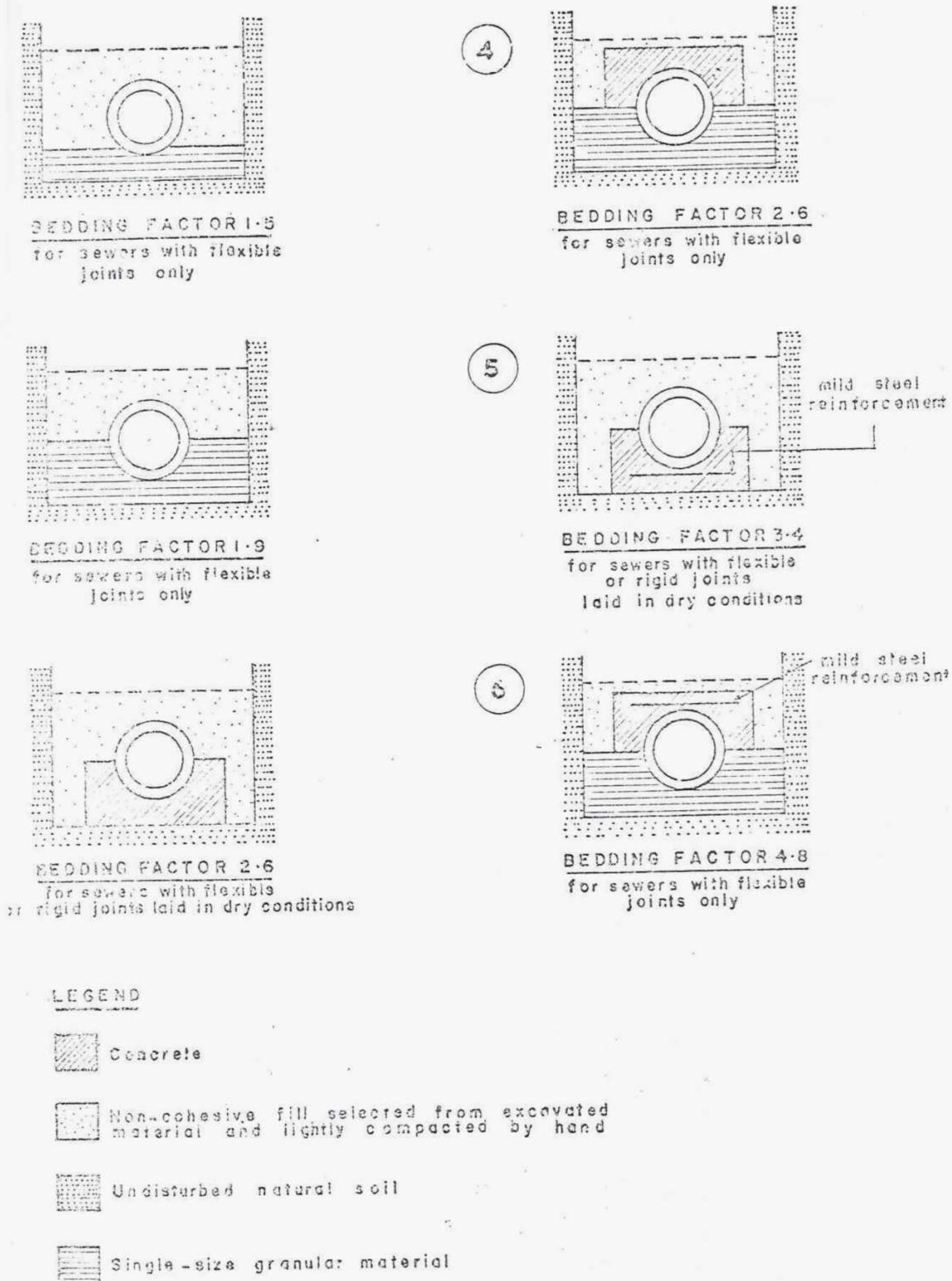


Figure 7.4 **Details of Typical Bedding to Concrete Sewers**

Table 7.6 weights of Trench Filling Material

Material	Lbs per Cubic Foot
Dry Sand	100
Ordinary (damp) sand	115
Wet Sand	120
Damp Clay	120
Saturated Clay	130
Saturated Topsoil	115
Sand and Damp Topsoil	100

Property Drains

It is proposed that property drains should also be constructed using either concrete or PVC pipes except that pitch fiber pipes may also be used within private property boundaries, but not in public roads.

Property drains should be the same standards as public sewers; beddings and protection should be used where necessary. Pitch fiber pipes should be bedded and backfilled with granular material as if they were PVC pipes. Property drains should connect into sewerage systems into the sewers for the purpose. Junctions should be tilted so that property drains “fall” into the sewer; the minimum allowable fall should be such that the soffits (that is, the inside of the roof of each pipe) of the property drain and the sewer are level adjacent to the junction.

Blanked junctions for property drains should be left in convenient locations at the time the main sewers are constructed; generally, more junctions then believed necessary should be provided. When, exceptionally, an additional junction is required on an existing sewer, it should be carefully constructed using purposed-made “saddle” provided by the sewer pipe manufactures.

Manholes

Manholes permit the inspection and cleaning of sewers and removal of blockages. They should be provided on sewers at all changes of direction, sewer level or gradient, at every junction and generally throughout the sewerage systems at intervals sufficiently close to simplify sewer cleaning.
The recommended spacing of manholes is:-

Table 7.7 Recommended Manhole Spacing

Sewer diameter		**Manhole spacing**	
Inches	Millimeters	Feet	Meters
4 to 8	100 to 200	130	40
9 to 18	225 to 450	200	60
21 to 33	525 to 825	300	90
36 & above	900	400	120

Manholes should not leak and they should have sufficient strength to prevent collapse; they should preferably be closed by heavy cast iron covers which have holes to allow some ventilation; this means that manholes should be carefully sited to ensure that flood water does not drain through their covers during heavy rain.

In Kenya today, four types of manholes and inspection chambers, which are manholes and inspection chambers, which are manholes on property drains, are normally constructed:-

i) Concrete block work with rendering;
ii) Brickwork with rendering;
iii) In-situ mass concrete;
iv) Precast concrete rings, with or without in-situ mass concrete surrounds.

Notwithstanding their rendering, it is very difficult to construct a water-tight blockwork or brick manhole and it is proposed that, in future, such manholes are only used on property drains in dry ground; that is, all manholes on public sewers should be made using either in-situ or precast concrete. Manholes constructed using precast concrete rings, which can be provided by most of the concrete pipe manufactures in Kenya, are normally much cheaper than in-situ concrete manholes.

To construct this type of manhole, preacast chamber rings are set vertically upon an in-situ manhole base, which has low walls constructed around sewers. In firm dry ground a concrete surround is not required; however, where the strength of the ground is suspect and certainly where the ground is waterlogged, precast concrete manholes should always have a 6 inch (0.15 meters) thick surround of in-situ concrete.

Miscellaneous

Sewer Sizes

When a sewer laid at shallow gradient runs into a steeper sewer, it is often theoretically possible to decrease the diameter of the downstream sewer; however, this is not good practice and it is recommended that, except under very exceptional circumstances, no sewer should discharge into a sewer of smaller diameter than itself.

Sewers should not run surcharged; therefore, where sewers of differing diameters meet at a manhole, they should be laid so that their soffits (that is the inside roof of each pipe) are level.

Maximum Depths of Sewers

The depths of sewers must generally be sufficient to take the gravity flows of sewage from adjacent domestic and industrial premises.
It is difficult to make any generalization about the maximum economic depth to which a sewer should be constructed; this depends upon ground conditions, the depth of water table, local topography, the size of sewer and its proximity to buildings; however, sewers should rarely be deeper than 26 feet (8 meters).

Duplication of Sewers

Where a long design period is used for new sewers, and if it is anticipated that the sewage flows will increase only slowly, it is sometimes possible to achieve some economy by sizing a sewer initially for a shorter design period, and allowing for its subsequent duplication. When a decision to duplicate at a later date made, it must be certain that there is sufficient space in roadways and between other services to make this possible.
As a general rule, duplication should not be considered when the design period is less than 20 years and then only in the case of sewers 36 inches (900 millimeters) in danger of larger.

The Ventilation of Sewers

It is important that sewers should be ventilated, both to keep them safe as possible for the sewer maintenance workers and also to keep the sewage aerobic in order to reduce the evolution of hydrogen sulphide.

It is recommend that ventilation is achieved partly by the provision of ventilating columns at pumping stations and at manholes where pumping mains discharge, and also by ventilation stacks attached to each property which drains into sewer. The primary function of the ventilation holes in the manhole covers is to stop excessive pressure building up in the manholes, should surcharging of the sewers take place.

Sewer Design Procedure

Combined sewerage is an exception in Kenya and totally separate sewers are not considered suitable; therefore, the sewers discussed in this Section are those which form part of partially separate sewerage systems.

The waters in a partially separate sewerage system may or may not include domestic sewage, industrial effluent, storm water, or ground water. Assuming all these components are present, a sewer on partially separate system should, by varying its size and gradient, be designed so as to confirm with each one of the following three criteria:-

1. Capacity to deal with Peak Flows

A sewer should be capable of dealing with the peak flows which it receives; these will normally occur during wet weather and will be equivalent to the sum of:-

The average daily design flow of domestic sewage multiplied by a factor (2A or 2B or 2C),

Plus

The average daily design flow of industrial effluent multiplied by a factor (2D or 2E or 2F etc),

Plus

The maximum rate of inflow of storm water,

Plus

The maximum anticipates rate of inflow of groundwater.

(The factors A, B, C, and D etc are explained in sub-section 6.5 of this report).

2. Sewer Design Procedure

Having computed the flow of sewage along a pipe length, the sewer is designed to ensure that the following conditions are satisfied:

- **Minimum self cleansing velocities are attained. Ensuring that minimum gradients are provided automatically ensures that minimum self cleansing velocities are attained**
- **Depths to which pipes are laid are minimized within permissible limits**
- **Adequate pipe flow capacity is available to carry the maximum peak future design flow**
- **The depth of flow at peak flow is within the recommended limits.**

- **Minimum sewer diameters are provided**

Design Computation

Application of Hydraulic Computations

For sanitary sewer systems, hydraulic computations are used in planning, design, and interpretation of flow mearements. In planning or preliminary design, the computations are usually concerned with conduit capacity and maximum or minimum slopes. Typical design computations involve capacity, slope, and significant water surface elevations at important locations in the conduit, such as upstream and downstream of junctions or changes in conduit size. It is often possible to assume that the water surface profile between these locations is a straight line connecting the significant elevations. Occasionally, design requires more detailed computation of actual water surface profiles, such as drawdown and backwater curves.

Capacity and Flow Estimates

Full Pipes

Many design computations begin with an estimate of the capacity of a pipe of a given size and roughness flowing full at a given slope, or with the selection of the size and slope of a pipe to carry a given discharge. Either the Manning Equation:-

$V=1/n\ (R^{2/3}S^{1/2})$

OR

Alignment chart Fig. 7.1

Are usually employed, and the solution for a full pipe is obtained. An example of the use of the alignment chart is shown in Fig. 7.1.

Partly Full Pipes

In order to consider the range of discharge that will occur in a partly full pipe it is often necessary to determine the depth and velocity of flow at discharges less than capacity. The partly full flow is assumed to be steady and uniform, and the depth and velocity are determined from a hydraulic element chart (Refer Fig. 7.2). For convenience, the velocity or flow rate in the partly full pipe can be expressed as a fraction of the corresponding value in the full pipe. Let a symbol without a subscript represent the value of a variable when the conduit is partly full and let the subscript "f" indicate values for the full conduit.

The flow ratio can be expressed as:

$$Q/Qf \qquad =AV/(A_fV_f) \qquad =(nf/n)(A/A_f)(R/R_f)^{2/3}$$

The ratios Q/Qf, V/Vf, A/Af, R/Rf and n/nf, are called the hydraulic elements of the conduit. They are usually determined as functions of the relative depth d/df and are often presented graphically as a hydraulic element chart or graph.

From fig. 7.2, two sets of curves are shown for the velocity and discharge ratios. The broken lines show the values based on constant n values. For all d/D values, the value of n is equal to nf. The solid lines show the values that result if n varies as indicted by the curve for n/nf.

The decision to use constant or variable n is left to the Engineer.

The method of using the hydraulic elements graph is illustrated below:-

Example 7.1

A 300mm (12 inch) sanitary sewer is laid on a slope of 3 units per 1000 units. Find the velocity and discharge if the sewer is flowing 0.4 full (120mm -4.8inch-depth). Assume Manning's n_f=0.013

a) From figure 4.1, $Q_f=0.0566m^3/sec.$, $V_f=0.76m/sec$

b) From figure 4.2, for d/d=0.4

i. n=constant:

$Q/Q_f=0.33$; $V/V_f=0.90$

$Q=0.27x0.0566 \qquad =0.0187m^3/sec$ or 18.7 L/sec.

$V=0.90x0.76 \qquad =0.68m/sec.$

ii. n = variable:

$Q/Q_f=0.27$; $V/V_f=0.71$

$Q=0.27x0.0566 \quad =0.0153m^3/sec$ or 15.3 L/sec.

$V=0.71x0.76 \quad =0.54m/sec.$

Many Engineers prefer a general tabulated version of the curve for Q/Q_f. With the Manning equation, the ratio for circular conduits can be written as:

$$Q/Q_f=\frac{Qn_f\ (4)^{5/3}}{1.486\Omega D^{8/3}S^{1/2}}$$

It follows that:-

$$\frac{Qn_f}{D^{8/3}S^{1/2}} \quad =0.4632\ Q/Q_f$$

The numerical value of the right hand side can be tabulated as a function d/D as has been done. With such tables, Q can be found for any value of d/D if D, S and n_f are known.

Organization of Computations

The first step in the hydraulic design of a sewer system is to prepare a map showing the locations of all the required sanitary sewers and from which the area tributary to each point can be measured. Preliminary profiles of the ground surface along each line also are needed.. These should show the critical elevations which will establish the sewer pipe grades, such as the basements of low lying houses and other buildings and existing sanitary sewers which must be intercepted. Topographic maps are useful at this stage of the design.

Several trial designs may be required to determine which one will properly distribute tentatively by graphical means on profile paper before selecting final grades and computing the sewer pipe invert elevations.

Design computations being repetions, may best be done on tabular forms permitting both wastewater quantity and the sanitary sewer design calculations to be placed on the same form. The form shown in table 7.3 is fairly comprehensive and can be adapted to the particular needs of the designer. It is convenient in using this form to record the data on transition losses and invert drops.

In using forms of this type, it is assumed that uniform flow exists in all reaches.

The use of Table 8.8 for sanitary sewer design requires supplementary charts, graphs, or tables for calculating wastewater flows and hydraulic data.

Table 7.8 Typical Computation Form for Design of Sanitary Sewers

		Manhole No.			Area		Average Flow		Peak Flow		
Line No.	Location	From	To	Length (m)	Increment(Acre)	Total (Acre)	Infiltration (m3/sec)	Sewage M3/sec.	Infiltration (m3/sec)	Sewage M3/sec.	Slope of sewer %
1	2	3	4	5	6	7	8	9	10	11	12

								Sewer Invert level (m)		Ground Level (m)	
Peak Factor	Sewer Dia. Mm	Capacity Full m3/sec	Velocity (full) M/sec	Min Velocity m/sec	Max Velocity m/sec	Manhole Invert (m)	Fall in Sewer m	Head	tail	Head	Tail
13	14	15	16	17	18	19	20	21	22	23	24

7.6 Sewer Construction Drawings

Mostly, Construction of Sewers requires Construction drawings

7.6.1 Purpose

The purpose of the drawings (plans) is to convey graphically the work to be done, first to the owner, then to project reviewers, to the bidder and later to the construction observer's inspector's engineer and the contractor. All information which can best be conveyed graphically, including configurations, dimensions and notes, should be shown on the drawings. Lengthy words description are best included only in the specification, and need not be repeated on the drawings

7.6.2 Field Data

A survey and investigation of the route of the sewer is required to obtain information as to the existing topography, underground utilities and property boundaries to be shown on the drawings. The route may be mapped from data obtained by conventional ground surveys or by aerial photogrammetry and should be checked against any existing plans and records if they are available.

7.6.3 Preparation

The plan view is usually drawn on the top half of the sheet and the profile is plotted directly beneath it on the bottom half, facilitating coordination of the two views.

Topography for the sewer plan is plotted either from the ground survey field notes or from the aerial photography, resulting in a strip map showing relevant existing facilities and ground surface features along the path of the proposed sewer. This topography may be transferred directly to the sewer plan/profile sheets from which prints can be made for use in sewer design; when aerial photogrammetry is used, the topography may be plotted directly on the aerial photographs and the resulting photomap may be reproduced in the configuration of a strip plan for use in design. The plotted topography and record data obtained from utility companies as to underground utilities are plotted on the plan. The plan showing existing conditions is completed by plotting property and easement boundaries. Relevant basement elevations, inverts of existing utilities or other structures that may affect the work may be shown.

The proposed sewer alignment is developed on prints of the plan/profile sheets or photomaps showing existing conditions. A profile of the ground surface along the proposed sewer alignment is drawn and proposed sewer invert elevations are determined and plotted on the profile. The horizontal and vertical alignments of the proposed sewer are transferred from the work prints to reproducible plan/profile sheets. Sections and details of the proposed sewer and appurtenances are drawn and traced on reproducible sheets for incorporation into the drawings. When applicable, proposed temporary and permanent easement boundaries are determined and drawn on the sewer plans. Contract limit lines are added to complete the preparation of the sewer plans.

The scale of the sewer plans should be large enough to show all of the necessary surface and subsurface information without excessive crowding. A horizontal scale of 1:500 to 1:1000 is suitable for most sewer plans; in extremely crowded urban areas 1:250 should be considered. In such areas, large scale plans of street intersections are quite useful. Generally, a satisfactory vertical scale for the profile is 1:100 or 1:50 for relatively flat terrain. Larger scales are used for sections and details.

When plans and profiles are drawn, each station will be 100m. Bench mark elevations will be given to 1mm(i.e. three places of decimals). However, it will generally be appropriate to show sewer inverts, etc, only to two decimal places. Nominal sewer sizes should always be shown in millimeter.

Lettering on drawings falls into three general categories: Labelling and dimensioning, notes, and titles. Notes should be lettered in the size used for labels and dimensions for proposed facilities. Labels and dimensions associated with proposed facilities should stand out from those for existing facilities. Titles should consist of larger letters than those for labeling and dimensioning notes.

7.6.4 Contents

7.6.4.1 Arrangements

The most logical arrangement for a set of drawings develops the project from general views to more specific view, and finally to more minute details. The subsections below are arranged to follow this generally accepted order of plan presentation.

7.6.4.2 Title Sheet

The title sheet should identify the project by presenting the following information:

- ❖ Project name
- ❖ Contract number
- ❖ Federal or state agency project number (if applicable)
- ❖ Owner's name
- ❖ Owner's officials, key people or dignitaries
- ❖ Design Engineer's name
- ❖ Engineer's project number
- ❖ Plan set number (for distribution records)
- ❖ Professional engineer's seal and signatures

7.6.4.3 Title Blocks

Each sheet except the title sheet should have a title block containing the following:

- ❖ Sheet title
- ❖ Project name
- ❖ Federal or state agency project number (if applicable)
- ❖ Owner's name
- ❖ Owner's officials, key people or dignitaries
- ❖ Design Engineer's name
- ❖ Sheet number
- ❖ Engineer's project number
- ❖ Scale
- ❖ Date
- ❖ Designer, drafter and checker identification
- ❖ Revisions block
- ❖ Sign off by owner's chief engineer or district superintendent

7.6.4.4 Index/Legend

Contract drawings should contain an index which lists all the drawings in the set by title and drawing number in order of presentation. It also is useful to include a sheet index drawn on the general plan map to identify the sheets which show the details for each length of proposed sewer shown on the general plan. These indices should be located on the drawing following the title sheet. A legend showing a set of symbols for elements of topographic abbreviations and the various items of the sewage works indicated in the sewer plans should be included on the index sheet.

7.6.4.5 Location Map

There should be a general location map showing the location of all work in the contract in relation to the community, either on the title sheet or on the index/legend sheet. This location map also may be used as an index map as outlined in the preceding paragraph.

7.6.4.6 General Notes

Notes which pertain to more than one drawing should be presented on the index/legend sheet. An example is a note warning that the location and sizes of existing underground utilities shown on the contract drawings are only approximate and that it is the responsibility of the contractor to confirm or locate all underground utilities in the area of his work. Each sheet should contain a note warning against the unauthorized alteration or addition to the documents.

7.6.4.7 Subsoil Information

Whether or not the locations of soil borings made during the design phase of a project and the boring logs should be included in the contract documents is a decision that should be made only after proper legal advice and consideration. In any event, whatever subsurface information has been obtained should be made available to bidders.

Drawings and specifications should indicate where special construction is required because of known unfavourable subsoil conditions. Neither the owner nor the engineer should guarantee subsoil conditions as a known element of the contract agreement.

7.6.4.8 Survey Control and Data

Survey control information may be shown on the sewer plan/profile sheets or on a general plan on a separate sheet. Baseline bearings and distances should be included with reference ties to permanent physical features. Vertical control points or benchmarks should be indicated, and the datum plane used for determining these elevations must be defined. A note indicating the dates of the ground survey and aerial photography should be included.

7.6.4.9 Sewer Plans

A continuous strip map, drawn directly above the profile, to indicate the plan locations of all work in relation to surface topography and existing facilities is an integral part of each sewer plan and profile drawing. Underground and overhead utilities along, across or near the proposed construction route also should be shown.

Sewer plans always should be oriented so that the flow in the sewer is from right to left on the sheet. Each sewer plan should include a north arrow. Stationing should be upgraded from left to right, generally along the sewer centerline. Survey baseline stationing also may be given on the plans, but it should not be substituted for stationing along the centerline of the sewer

7.6.4.10 Sewer Profile

Contract drawing should include a continuous profile of all sewer runs indicating centerline ground surface and sewer elevations and grades. The profile is also a convenient place to show the size, slope and type of pipe, the limits of each size, pipe strength or type, the locations of special structures and appurtenances, and crossing utilities and drainage pipes. The profile should be located immediately under the plan for ready reference. Stationing shown on the plan should be repeated on the profile.

7.6.4.11 Sewer Sections

When sewers consist of pipes of commonly known or specified dimensions, materials, or shapes, no sewer sections need to be shown. For cast in place concrete sections, complete dimensions with all reinforcement steel shown should be included in the drawings.

7.6.4.12 Sewer Details

Separate sheets of sewer details normally follow the plan/profile sheet.
The following details, when applicable, should be included:

- Trenching and backfilling -payment limits including those for rock excavation and types of backfill materials
- Pipe bedding and cushion -dimensions, material types and payment limits
- House lateral connections -type and arrangement of fitting and minimum pipe grade
- Special connections -type and configuration of fittings and dimensions
- Manholes -foundation, base, barrel, top slab, frame and cover, and invert details
- Sewer/water main crossings -separation requirements
- Stream, highway or railroad crossings - casing, inverted siphon, encasement or other related details

7.7 Operation and Maintenance

7.7.1 Health Effect of Sewage

Workers whose activities bring them into contact with sewage and sewage products are at risk of contracting a work-related illness. The majority of illnesses are relatively mild cases of gastroenteritis, but potentially fatal diseases, such as leptospirosis (Weil's disease) and hepatitis have been reported.

How to protect Yourself

- Make sure that you understand the risks to health and the ways in which you can pick up infections.
- Use safe systems of work and wear the protective equipment that is provided.
- Reports damaged equipment and get it replaced.
- Avoid becoming contaminated with sewage.
- Avoid breathing in sewage dust or spray. Do not touch your face or smoke, eat or drink, unless you have washed your hands and face thoroughly with soap and water. Cleanse all exposed wounds, however small, and cover with a sterile waterproof dressing.
- Change out of contaminated clothing before eating, drinking or smoking.
- If you suffer from a skin problem, seek medical advice before working with sewage.
- Clean contaminated equipment on site. Do not take contaminated clothing home for washing. Your employer should deal with this.

If you become ill

- Consult your doctor in the event of flu-like illness or fever, particularly where associated with severe headache and skin infections. Show your pocket card to the doctor.
- Seek medical advice if there are persistent chest symptoms, particularly if consistent with asthma or alveolitis (inflammation of the lung).
- Report any of the above illnesses to your employer, who should investigate any Work-related link.

7.7.2 Confined Spaces

Sewers and manholes are confined spaces.
In view of the above, there is lack of oxygen. This occurs due to poisonous gas, fume or vapour emission from sewage or induction through pumping equipments. Some of the poisonous gas is flammable.

Some of the above conditions may already be present in the confined space. However, some may arise through the work being carried out, or because of ineffective isolation of plant nearby, eg leakage from a pipe connected to the confined space. The enclosure and working space may increase other dangers arising through the work being carried out, for example:

- machinery being used may require special precautions, such as provision ofdust extraction for a portable grinder, or special precautions against electric shock;
- gas, fume or vapour can arise from welding, or by use of volatile and often flammable solvents, adhesives etc;
- if access to the space is through a restricted entrance, such as a manhole, escape or rescue in an emergency will be more difficult

If your assessment identifies risks of serious injury from work in confined spaces, contain the following key duties:

- avoid entry to confined spaces, e.g. by doing the work from outside;
- if entry to a confined space is unavoidable, follow a safe system of work; and
- Put in place adequate emergency arrangements before the work starts.

You need to check if the work can be done another way so that entry or work in confined spaces is avoided. Better work-planning or a different approach can reduce the need for confined space working. Ask yourself if the intended work is really necessary, or could you:

- Modify the confined space itself so that entry is not necessary;
- Have the work done from outside, for example: Blockages can be cleared in silos by use of remotely operated rotating flail devices, vibrators or air purgers;
- Inspection, sampling and cleaning operations can often be done from outside the space using appropriate equipment and tools;
- Remote cameras can be used for internal inspection of vessels.

If you cannot avoid entry into a confined space make sure you have a safe system for working inside the space.

Use the results of your risk assessment to help identify the necessary precautions to reduce the risk of injury. These will depend on the nature of the confined space, the associated risk and the work involved. Make sure that the safe system of work, including the precautions identified, is
Developed and put into practice. Everyone involved will need to be properly trained and instructed to make sure they know what to do and how to do it safely.

The following checklist is not intended to be exhaustive but includes many of the essential elements to help prepare a safe system of work.

Appointment of a supervisor

Supervisors should be given responsibility to ensure that the necessary precautions are taken, to check safety at each stage and may need to remain present while work is underway.

Health and Safety
Executive

Are persons suitable for the work?

Do they have sufficient experience of the type of work to be carried out, and what training have they received? Where risk assessment highlights exceptional constraints as a result of the physical layout, are individuals of suitable build? The competent person may need to consider other factors, eg concerning claustrophobia or fitness to wear breathing apparatus, and medical advice on an individual’s suitability may be needed.

Isolation
Mechanical and electrical isolation of equipment is essential if it could otherwise operate, or be operated, inadvertently. If gas, fume or vapour could enter the confined space, physical isolation of pipework etc needs to be made. In all cases a check should be made to ensure isolation is effective.

Cleaning before entry
This may be necessary to ensure fumes do not develop from residues etc while the
Work is being done.

Check the size of the entrance
Is it big enough to allow workers wearing all the necessary equipment to climb in and out easily, and provide ready access and egress in an emergency? For example, the size of the opening may mean choosing air-line breathing apparatus in place of self-contained equipment which is more bulky and therefore likely to restrict ready passage.

Provision of ventilation
You may be able to increase the number of openings and therefore improve ventilation. Mechanical ventilation may be necessary to ensure an adequate supply of fresh air. This is essential where portable gas cylinders and diesel-fuelled equipment are used inside the space because of the dangers from build-up of engine exhaust. **Warning: carbon monoxide in the exhaust from petrol-fuelled engines is so dangerous that use of such equipment in confined spaces should never be allowed.**

Testing the air
This may be necessary to check that it is free from both toxic and flammable vapours and that it is fit to breathe. Testing should be carried out by a competent person using a suitable gas detector which is correctly calibrated. Where the risk assessment indicates that conditions may change, or as a further precaution, continuous monitoring of the air may be necessary.

Provision of special tools and lighting
Non-sparking tools and specially protected lighting are essential where flammable or potentially explosive atmospheres are likely. In certain confined spaces (eg inside metal tanks) suitable precautions to prevent electric shock include use of extra low voltage equipment (typically less than 25 V) and, where necessary, residual current devices.

Health and Safety
Executive
Provision of breathing apparatus

This is essential if the air inside the space cannot be made fit to breathe because of gas, fume or vapour present, or lack of oxygen. Never try to ‘sweeten’ the air in a confined space with oxygen as this can greatly increase the risk of a fire or explosion.

Preparation of emergency arrangements
This will need to cover the necessary equipment, training and practice drills.

Provision of rescue harnesses
Lifelines attached to harnesses should run back to a point outside the confined space.

Communications
An adequate communications system is needed to enable communication between people inside and outside the confined space and to summon help in an emergency.

Check how the alarm is raised
Is it necessary to station someone outside to keep watch and to communicate with anyone inside, raise the alarm quickly in an emergency, and take charge of the rescue procedures?

Is a 'permit-to-work' necessary?
A permit-to-work ensures a formal check is undertaken to ensure all the elements of a safe system of work are in place before people are allowed to enter or work in the confined space. It is also a means of communication between site management, supervisors, and those carrying out the hazardous work. Essential features of a permit-to-work are:

- clear identification of who may authorise particular jobs (and any limits to their authority) and who is responsible for specifying the necessary precautions (eg isolation, air testing, emergency arrangements etc);
- provision for ensuring that contractors engaged to carry out work are included;
- training and instruction in the issue of permits;
- Monitoring and auditing to ensure that the system works as intended.

Emergency procedures
When things go wrong, people may be exposed to serious and immediate danger. Effective arrangements for raising the alarm and carrying out rescue operations in an emergency are essential. Contingency plans will depend on the nature of the confined space, the risks identified and consequently the likely nature of an emergency rescue.

Health and Safety Executive
Emergency arrangements will depend on the risks. You should consider:

Communications
How can an emergency be communicated from inside the confined space to people outside so that rescue procedures can start? Don't forget night and shift work, weekends and times when the premises are closed, eg holidays. Also, consider what might happen and how the alarm can be raised.

Rescue and resuscitation equipment
Provision of suitable rescue and resuscitation equipment will depend on the likely emergencies identified. Where such equipment is provided for use by rescuers, training in correct operation is essential.

Capabilities of rescuers
They need to be properly trained people, sufficiently fit to carry out their task, ready at hand, and capable of using any equipment provided for rescue, eg breathing apparatus, lifelines and fire-fighting equipment. Rescuers also need to be protected against the cause of the emergency.

Shut down
It may be necessary to shut down adjacent plant before attempting emergency rescue.

First-aid procedures
Trained first aiders need to be available to make proper use of any necessary firstaid equipment provided.

Local emergency services
How the local emergency services (eg, fire brigade) are made aware of an incident?

What information about the particular dangers in the confined space is given to them on their arrival?

Relevant law Cap 532 (Public Health Safety)

- The Confined Spaces
- The Management of Health and Safety at Work
- The Control of Substances Hazardous to Health
- The Personal Protective Equipment at Work Regulations 1992 (as amended);

- The Provision and Use of Work Equipment Regulations 1998;
- Electricity at Work Regulations
- Workplace (Health, Safety and Welfare) Regulations Cap 532

Some of the above law is relevant because of the nature of the work to be carried out inside a confined space, eg where there are risks from machinery, electricity or from hazardous substances.

7.7.3 Sewer Maintenance Equipment

For proper maintenance of sewers, the following is a must:-

1) Portable Sewage Pump

HP and speed	3.2 at 3,500rpm
Fuel	Diesel
Consumption	2.4liters/hr
Starting	Rope
Cooling	Air
Construction	aluminium and cast iron
Air cleaner	replaceable element
Oil filter	replaceable element
Pump construction	Cast iron
Connection	2” B.S.P
Max solid size	10mm
Pressure limit	6kg/sq.cm
Seal	carbon ceramic
Unit weight	60 kilos nett

2) Sewer Mirrors

Dimensions	285x150mm^2
	380x250mm^2
	540x250mm^2
	420x190mm^2

Spare mirror glass of crystal glass for each of the mirrors above.
-Plastic box case
-Extension sticks of 4 parts
-Telescope extension sticks 3x1500mm

3) Search Lights

Smaller, practical and compact lights suitable for operation in difficult surroundings and well protected.

- Main light of 3.4W for 3 hours
- Economy light of 1.1W for 10hours
- Flash light 3.4W -6hours
  1.1W -20hours
- diameter of reflector, approx. 90mm
- dimensions 1xwxh =100x60x120mm
- inclusive two sets of spare lamps and rechargeable batteries

4) Battery Charger

Suitable for the above items

5) Manhole Cover Lifter

The cover lifter should be a simple and ingenious device for removing manhole covers without damage to the cover or strain to the operator. The equipment should be supplied as a kit of 2 bars, 2 sets of keys, 3jacks and 2 handles, adjustable to most shapes and sizes of manhole as well as providing ease of transportation. Maximum operating length of about 1,220mm

Spare Keys for Lifter

32x13x7mm (1 1/4x1/2x3/8 inch) rectangular keys 13mm (1/2 inch) dia. Shank.
38x15x7mm (1 1/2x5/8x1/4 inch) rectangular keys 19mm (3/4 inch)

6) Motorized rod rotating Machine

The machine shall be capable of cleaning sewer lines upto 80m in length using 10mm sectional rods with speed couplers. It shall be powered by a petrol engine of at least Biggs & Stratton 4.5HP-5HP with three forward spinning speeds, all in the range of 80-200rpm. These speeds may be further varied by throttle adjustment. The frame should be of steel welded construction mounted on four wheels (i.e. 2No. 200mm wheels and 2No. caster wheels). Overall dimensions of the machine not to exceed 1mx0.8mx0.75m.

Power transmission to be through an instant action clutch with automatic throttle adjustment. The clutch on drive shaft shall be protected against overloading and twisting of rods.

The machine is to be supplied with the following equipment:

- Rod holder
- Tool adaptor
- Drive pin
- Rod turner
- Torque adaptor
- Coupling pin key
- Operators gloves (left hand and right hand)

7) Sewer Cleaning Rods and Accessories

These shall be manufactured from treated steel suitable for heavy duty sewer cleaning application and which can also be used with motorized tools. Each shall be of a size stated in the Bill of Quantities.

8) Sewage Hose

The hose should be of super elastic PVC material with a shock resistant rigid PVC spiral. Minimum wall thickness to be 7.4mm and weight of at least 2.7kg/m

It should be able to withstand the required working pressure of 3 bar and vacuum at a temperature of $+18^0$c.

9) Perrot Coupling

The coupling shall be made of stainless steel with the hook and lever in galvanized steel.

10) Bucket Machines

A set of mechanically operated medium duty pull in machine equipped with 10HP engine (model 9H PPI) together with truck loader of corresponding construction and horsepower rating; This machine to be used for cleaning sewers from manhole to manhole removing accumulations of rods, sand, silt and grease. The chassis construction to include base frames, and uprights, heavy duty welded channel and I-beams. Units to be designed to bucket travel upto 125 feet per minute

7.7.4 Sewer Cleaning

There are times when the flow in the sewer lines is very low and debris, grit and grease may be deposited in the sewer lines. To remove these deposits, it is necessary to flush the sewer lines with plenty of water i.e. 1,000litters per flush whenever it is necessary:- Flushing is useful because it temporarily increases the velocity of flow and also the depth of the liquid, thus increasing the transportation power of the water

7.7.4.1 Inspection Practice

a) As built drawings are normally provided so that the location of sewer line, sizes, and gradients, manholes and direction of flow are easily identified. Using these plans, the trunk sewers are routinely inspected every three months, intercepting sewers inspected two times per month. During these inspections, cleaning and repairs as necessary are carried out and records of conditions found, the work done and the costs are kept. The cracked or crushed lengths of pipes must be replaced with new ones.

b) Unblocking sewers

In many cases, a blockage can be cleared by forcing a rod headed by a pointed tool through the obstruction and then allowing the velocity of the released sewage to scour the pipe. Large objects may require excavation and opening of the line

c) Manholes are inspected and any broken manhole covers replaced and sealed. During inspection of lines, several adjacent manholes are opened at the same time and left open for **about 30minutes before anybody enters them**. This is to enable poisonous gases to get out of the manholes and fresh air to enter the system.

d) First Aid Kit should always be available during these inspections. They should include apparatus that will test for poisonous gases and explosives gases. The tester should always wear a horse mask or safety belt with a rope attached and other workmen waiting outside the manhole ready to pull him out if he is overcome.

7.7.4.2 Flow Measurement

The incoming flow should be measured on hourly basis.

7.8 Costing

The Designer is referred to current cost of pipes prevailing in the market in deducing his costs.

However, for preliminary estimates Figure 4.5 can be used.

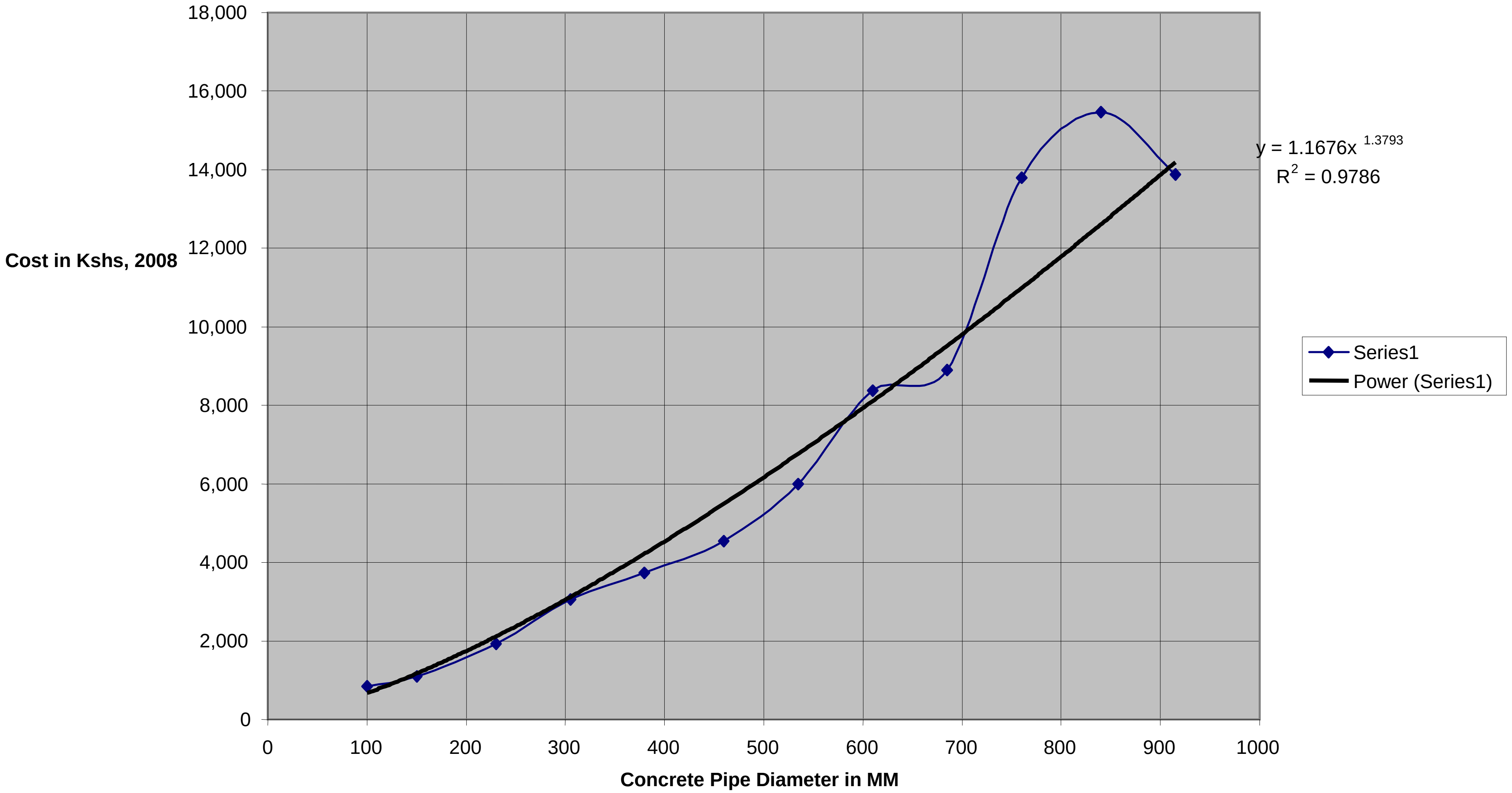
From the figure, it can be deduced that the pipe diameter is related to the construction cost of the pipe as follows:-

$Y=1.1676x^{1.3793}$

Where x=pipe diameter in mm

Y= Cost of construction of the sewer in Kshs.

Figure 7.5 Unit Costs for Construction of Sewer Reticulation Using Concrete Pipes



8.0 MANAGEMENT OF SEWERAGE SCHEMES

8.1 Listed Methods of O & M

Sewerage O & M comprises the following:-

1) General duties

General affairs, personnel, wages, budgeting and settlement of accounts, accounting, welfare and benefits, public relations, etc

2) Budgeting

- Purchasing and management of materials (fuel, chemicals, consumables)
- Construction contract
- Consignment agreement

3) Management of Assets

- Management of fixed assets (property management, etc)
- Building and repairs

4) Guidance, etc., for drainage and other facilities

- Installation, guidance for drainage facilities
- Inspections
- Surveys relating to drainage facilities

5) Direction and guidance, etc., for workplace drainage

- Installation of pre-treatment facilities
- Guidance on improvement
- Guidance of management and maintenance of pre-treatment facilities
- Execution of water quality inspections for factory, etc.
- Surveys relating to pre-treatment facilities (including drainage water quality control or QC tests)

6) O & M of pipe facilities

- Planning, design, execution and direction of inspections and surveys of pipe facilities
- Planning, design, execution and direction of cleaning and dredging of pipe facilities
- Planning, design, execution and direction of repairs and improvements
- Security and protection of pipe facilities

7) O & M of pumping stations and treatment plants

(1) Duties related to operation
- Planning of water and sludge treatment plants
- Operational planning of pumping station and treatment plant equipment
- Formation, design, direction and execution of plans for management duties of pumping stations and treatment plants
- Planning disposal of sand sediment and screen waste; design, direction and execution of transportation and disposal

- ❖ Design, direction and execution of cleaning and management of buildings, plants etc.
- ❖ Recording and reporting (daily, monthly and yearly reports) of operation and management of pump stations and treatment plants
- ❖ Directions and operation during malfunctions and emergencies

(2) Duties related to maintenance and inspections

- ❖ Planning of maintenance and inspections of mechanical and electrical pump station facilities; formulating outline procedures
- ❖ Design and direction of maintenance and inspections of mechanical and electrical pump station facilities
- ❖ Formulation and execution of maintenance and inspection duties for mechanical and electrical pump station facilities.

(3) Duties related to repairs and improvements

- ❖ Formulation, design, direction and execution of work plans for repairs.

8) Duties related to water quality control

- ❖ Formulations of plans for water quality tests, surveys, research, etc.
- ❖ Execution of water quality and sludge tests
- ❖ Execution of activated sludge tests
- ❖ Organization and analysis of data, and preparation of reports
- ❖ Formulation of operation management indicators
- ❖ Measures to handle problems and malfunctions
- ❖ Adjustment of water quality testing devices

9) Management of Ledgers

- ❖ Adjustment and storage of ledgers
- ❖ Correction and perusal of ledgers
- ❖ Management of maps and reference works (general plans, longitudinal plans, maps of drainage area, maps of electrical and pipe systems, etc.)

10) Duties related to environmental protection

- ❖ Formulation of plans for atmospheric measurements
- ❖ Execution of atmospheric measurements
- ❖ Formulation of plans for noise and vibration measurements
- ❖ Execution of noise and vibration measurements
- ❖ Formulation of plans for unpleasant odor measurements
- ❖ Execution of unpleasant odor measurements
- ❖ Formulation of plans for water quality measurements in river for discharge
- ❖ Execution of water quality measurements in river used for discharge.

11) Other Duties

- ❖ Reporting to supervisory office
- ❖ Understanding and improving safety and hygiene conditions
- ❖ Construction authorization, etc. by someone other than manager
- ❖ Survey and research related to sewerage system
- ❖ Visitor guidance
- ❖ Publicity and educational activities
- ❖ Other

Personnel allocated to the above O &M activities will vary according to the size of the cities and towns, the degree of population density, the terrain, the size and configuration of the sewerage system, the age of the systems, the conditions of the system, etc. Thus, it is not possible to use a general standard for required number of personnel for O &M. Nevetheless, personnel must be placed rationally and appropriately.

8.2 Office Requirements for O & M

8.2.1 Sewerage Ledger

8.2.1.1 Basic Items

The sewerage Ledger (the sewerage record plans) is fundamental to the improving of the O &M of the sewerage system, and of a good understanding of the installed sewerage facilities. It can also allow those operating the sewerage system to view its data, and if necessary for providing documentation to other related authorities such as the road management agency. Therefore, it must be updated regularly so that it correctly indicates the existence and current condition of all the sewerage facilities, also the ledger must be stored safely because of its value.

Also, while the sewerage ledger (SL) is comprised of written revisions and diagrams, it is not merely a collection of basic material relating to technical O &M, but is also useful for collecting and providing information concerning complaints by members of general public and corrective actions. SL is also useful for discussions with other public agencies and related institutions as well as for collection and provision of information necessary in the event of natural disasters, etc. Therefore, maintaining a sewerage ledger is an indispensable part of the O &M of a sewerage system. This objective must not be lost from the view of O &M. A record of the sewerage ledger must be kept in a rational and appropriate manner.

Based on the above, the following points must be considered when making adjustments to the ledger.

1) General diagrams for SL for sewers should utilize street maps (white diagrams) based on street planning and cover the entire planned sewerage area.
2) Supplementary diagrams and documentation should be prepared.
3) At least 2 sets of all diagrams should be prepared, and each set should be stored separately for security to avoid the loss of the basic information in the event of a fire, etc.
4) To enable record keeping of the SL to be carried out easily, from the time of construction, diagrams should be made in such a away that completed work diagrams can be converted to standard diagrams for record keeping.
5) When alterations or improvements are made in sewerage facilities, the personnel in charge of keeping the SL should be able to note down their progress at suitable intervals, so that the current situation is always clearly recorded, and corrections can be made quickly.

In addition, whenever alterations or improvements are made to the sewerage facilities, the ledger must be updated immediately. In order that the most up to date information recorded therein is available whenever necessary, correct storage management of the ledger becomes very important.

8.2.1.2 Contents of Sewerage Ledger

(a) Updates

Written records must include at least the following items:-

- ❖ The area and population of the drainage area and place names within the drainage area
- ❖ The area and population of the seaward area and place names within the sewered area

- ❖ The commencement date for collection and the date that sewage treatment commenced at the treatment plant
- ❖ Location of the outfall and name of places where sewage is discharged at averflows and outfalls
- ❖ Length of sewers and numbers of manholes, house basins, and rainwater basins
- ❖ Location, floor area, construction and capacity of treatment facilities
- ❖ Location, floor area, construction and capacity of pumping facilities.

(b) Diagrams

Diagrams refer to general drawings and standard diagrams of facilities, and must include at least the following items.

1) General drawings

- ✓ Town/village names and borders
- ✓ Border of planned sewered area, and the borders between other sewered areas and drainage areas, and their names
- ✓ Border of drainage area and sewered area
- ✓ Location of sewers and outfalls, and name of place where sewage is discharged at overflows and outfalls
- ✓ Locations and names of treatment facilities and pumping facilities
- ✓ Bearing, contraction scale, legend and date of any updates.

2) Plane diagrams of facilities

- ✓ Items 1 to 3 and 6 of (1) above
- ✓ Location, form, internal dimensions, slope, block distance and invert levels of sewers, and direction of sewage flow
- ✓ Location, form and internal dimensions of branch pipes
- ✓ Location, type and internal dimensions of manholes
- ✓ Location and type of house basins and rainwater basins
- ✓ Location of outfalls and names of discharge places and the high water, low water and ordinary water levels, etc of receiving water bodies.
- ✓ Location, form, internal dimensions and names of points of diversion of road drains into public sewers
- ✓ Names and floor boundaries of treatment and pumping facilities
- ✓ Locations, forms, water levels and names of main equipment within treatment and pumping facilities
- ✓ Location of nearby roads, rivers, train lines, etc

8.3 Sewage Charge

8.3.1 Individual Factors for O &M Expenses

O &M costs for the sewerage system are inevitable incurred for maintaining good water quality in the sewage effluent discharge to the receiving water bodies. They may be categorized as follows:-

1) Repairs -The cost of repairs to facilities, plant and equipment, etc.
2) Facility replacement -Necessary in cases when facilities are severly deteriorated and no longer repairable (Depreciation)
3) Equipment Replacements -Necessary in cases when facilities are severly deteriorated and no longer repairable (Depreciation)

4) Wear and Tear Expenses -Cost of purchasing equipment, replacements for plant and equipment, and consumables etc., necessary for running the treatment facilities
5) Water Bills -Cost of using water

6) Electricity Bills -Since the majority of the plant and equipment (pumps, sludge collectors, etc.). run on electricity, the electricity bill is one of the most important operational expenses.
7) Chemicals -Cost of chemicals used in testing water quality, in disinfection, etc. Treated water is made epidemiologically safe and released into public waters, so that the costs of disinfectant chemicals are important
8) Sludge disposal Costs -This is the costs involved in final disposal of sludge. After the sludge is taken from the settling tank and dried, it is transported to its ultimate disposal site.
9) Personnel Costs -Salary and upkeep costs
10) Consignment costs -Necessary when other organizations are commissioned to carryout certain O &M duties.

8.3.2 Method of Calculating a Sewage Charge

A method of calculating sewage charges should only be decided upon after careful discussion.

Basically, those elements related to rainwater are considered public expenses, while those elements dealing with wastewater are considered personal and are then covered by sewage charges based on the polluter pay principle (PPP).

8.3.2.1 Sewage Charges

Sewage charges are collected on the basis of the following considerations.

1) Basic principles of sewage charges

- ✓ Must be appropriate to the volume and quality of the sewage discharged by the user
- ✓ Must not exceed the cost required to conduct efficient management
- ✓ Must be clearly specified by a fixed rate or a fixed amount. However, domestic wastewater and industrial wastewater are considered separately
- ✓ Must not segregate specific users by setting up an unreasonable sewage charge system.

2) Concept of Sewage Charges

Sewage charges must take into account economic realities and comply with the general thinking on the way of bearing O &M costs and the basic principles of sewage charges. They should be within a cost range allowing for efficient management of the system. The cost of wastewater treatment is high in the initial stages of development of the sewage works, and the cost declines as the business develops. So, in actually calculating sewage charges, a variable calculation method (stage slide method) is used which can change the costs in accordance with the current stage of the business. It is expected that this will achieve a balance in revenue and expenditure over the long term.

The stage slide method is shown below:-

Stage 1: Because the cost of wastewater treatment is high, not even the O &M costs (OD) can be covered, so efforts are made to levy the entire O &M costs from users.

Stage 2: DShift from levying all O &M costs to also including the personal expense component of the capital cost

Stage 3; Add costs which can not be levied in stages 1 & 2.

3) Cost related charges for domestic wastewater

Domestic wastewater is wastewater other than the wastewater discharge by specific generators such as industrial plants. O &M costs relating to wastewater are costs required for the running of sewage facilities.

Cost related charges for domestic wastewater refer to the entire O &M costs relating to wastewater .

4) Costs related charges fir specific wastewater (Non domestic)

Charges with regards to specific wastewater generation (i.e. sewage above a specified volume discharged by factories, businesses, etc, as a result of business activities)

5) Sewage charge revision times

Sewage charges should be revised at suitable times every every two to three years to adjust for market fluctuations, the operational stage of the business, the economic condition of the particular region etc.

6) Sewage Charge System

a) With regard to the division between domestic wastewater released by household, etc., and specific wastewater discharged by factories and businesses, the range of cost charge varies, and distinguishing criteria have been decided based upon the condition of domestic wastewater normally discharged by households, and the actual situation of business related to daily life etc.

b) High volume discharge can be a factor which increases capital cost, so, under a suitable progression rate corresponding to the wastewater treatment cost, a cumulative charge system in accordance with the actual condition of the city or town concerned is used.

c) Inorder to assure that all users are treated fairly, as well as to prevent drainage which does not conform to a specified water quality, the actual drainage situation of local public bodies and the treatment capacity of the wastewater treatment plantr are considered. If necessary, a water quality surcharge is levied on top of the usual sewage charge. Note also that in such cases it goes without saying that the discharge from specific users must set the limits of water qualiity which will not severly obstruct the functioning of the sewerage system, or damage the facilities, and can respect the water quality standards for the effluent from the wastewater discharged from the specific users, the lower limit is usually defined to be the average water quality of domestic wastewater.

9.0 SEWAGE PUMPING STATIONS

The standardization of pumping stations and their equipment is very desirable. It simplifies design, maintenance and repair, and the training of operatives; it also reduces considerably the amount of spare parts which must be kept in store against breakdowns.

9.1 Duties of Sewage Pumping Stations

There are two basic types of sewage pumping stations, “lift” stations and stations which discharge into pumping mains.
In the lift station, sewage is merely raised from a low to a higher level, for subsequent gravity flow.

9.1.1 Sewerage Pumping Stations

As their name implies, such stations are usually located on sewerage systems, although they may also be at the entrance to sewage treatment works.

The function of a particular station may be to:-

a) Deal with sewage from low-lying parts of a drainage area;
b) Avoid excessive depths of sewer by periodically raising the sewage, in flat areas;
c) Overcome some obstacle which makes gravity flow impracticable; for example, to pump sewage over a hill or across a stream;
d) Cater for new building development in areas too low to low to allow gravitational discharge into an existing sewerage system;
e) Lift incoming sewage to the head of a treatment works;
f) Discharge sewage into sea

Except in the case of examples a) and d), sewerage pumping stations are not usually “lift” stations.

9.1.2 Treatment Works’ Pumping Stations

Such stations may be required:-

a) To pump raw digested, or perhaps activated, sludge;
b) For pumping sludge liquor to the head of the treatment works;
c) For emptying tanks and other units;
d) To reticulate effluent;
e) To pump the works’ effluent to disposal.

9.2 Design criteria: Pumping Stations

In order of size, the following types of pumps are considered most suitable for sewage, in this country:-

i) Solids diverters (flows of 80 gallons (360 litres) per minute or less).
ii) Submersible pumpsets incorporating centrifugal pumps (flows between 100 gallons (450 litres) per minute and 550 gallons (2 500 litres) per minute.

iii) Centrifugal pumps (flows between 550 gallons (2 500 litres) per minute and 4 000 (18 200 litres) per minute).

iv) Mixed-flow pumps (flows above 4 000 gallons (18 200 litres) per minute).

However, where the public can be excluded, screw pumps are considered suitable for sewage “lift” stations.

Wherever electricity is available, it is recommended that pumps be driven by electric motors; elsewhere, diesel engines are considered the better alternative type of prime mover.

For the pumping of sewage sludge, screw pumps, air lifts and reciprocating pumps are all considered applicable.

The design of pumping stations buildings is usually dictated by the type of pumping equipment which they will contain. It is suggested that, although submersible pumps may be installed in a simple sewerage wells; other roto-dynamic pumps should always be housed in “dry well” stations. Screw pumps are satisfactory in the open air, provided that their motors are switchgear are protected by small buildings.

Apart from solids diverters, submersible pumpsets and screw pumps, it is recommended that all sewage and sludge pumps should be protected against blockage by screens; for the smallest pumps, 1½ inch (40 millilitres) clear opening screens are required, but 4 inch (100 millimetre) openings are suitable for the larger centrifugal and mixed-flow pumps.

It is recommended that no attempt be made to remove grit from sewage at any pumping stations located at intermediate points on a sewage system.

Table 9.1 Types of Sewage Pumping Plants Recommended for Kenya

Approximate Range of Flows		**Types of Pump**	**Optimum Applications**	**Limitations**
Liters per second	**Gallons per Minute**			
2 to 9	25 to 100	Solids diverter (centrifugal)	Raw Sewage	
8 to 40	100 to 550	Submersible (Centrifugal)	Raw Sewage	
40 to 150	550 to 2000	Centrifugal (Dry well station)	Will perform any duty but probably inferior to reciprocating pumps when dealing with sludge	The impeller should be adapted to the particular duty
300 and above	4000 and above	Mixed Flow	Treatment works effluents	Hot raw sewage
75 to 7500	1000 to 100000	Axial Flow	Treatment works effluents and digested sludge	Hot raw sewage
10 to 1300	130 to 18000	Archimedcan Screw	At inlets to treatment works and for sludge	Lift stations only, low heads
4 to 8	50 to 100	Reciprocating (The range given assumes sludge pumping)	Sludge	Screened and digested sludge only
		Air Lift	Activated Sludge	Low heads and effluents
		Sludge Wheels	Activated Sludge	Low heads and capacities

9.3 Pumps

Several types of pumps are available for pumping sewage, sludge and effluents, but they each have limitations and optimum applications as discussed below.

Table 9.1 above has the details.

9.3.1 Roto-Dynamic Pumps

The types of pumps are that they each have a rotating element. Pumps of this type can be placed in one of three categories:-

i) Centrifugal pumps
ii) Mixed-flow pumps
iii) Axial-flow pumps

The centrifugal pump has an impeller which rotates within a spiral chamber; it is the pump most commonly used for pumping sewage.

Centrifugal pumps are available in a wide range of capacities. Standard units vary from about 100 gallons per minute (450 liters per minute) to 2000 gallons per minute (9100 liters per minute); normally the maximum head is about 120 feet (37 meters) but larger pumps with capacities up to 10000 gallons per minute (45 000 liters per minute) operating at heads up to 180 feet (55 meters) can be manufactured to order. Centrifugal pumps with capacities smaller than 100 gallons per minute (450 liters per minute) are made, but have passages too narrow to deal with the solids normally contained in sewage.

Centrifugal pumps for sewage differs from those designed for water or treated sewage effluents by have simpler impellers and wide waterways so that they are not easily blocked by the fibrous and solid objects found in sewage.

Grit contained in sewage causes rapid wear in pumps; therefore sewage pumps should generally be provided with cast-iron impellers and renewable shaft sleeves through stuffing boxes.
Centrifugal pumps are sufficiently versatile to cover nearly all pumping duties entailed in the sewage disposal programme for Kenya recommended in World Health Organization Report No 8, except when flows are very low, and possibly in the few, largest towns. However, occasionally it may be more economical to use screw pumps in lift stations and preferable to have reciprocating or air-lifting pumps to deal with sludge.

Mixed-flow pumps have twisted impeller vanes which readily trap any stringy material in the sewage so that it builds up eventually causes blockages; these pumps are difficult to clear and are therefore not generally suitable for untreated sewage.
Mixed-flow pumps are made only in relatively large sizes, from about 4000 gallons per minute (18200 liters per minute) upwards; they are suitable for pumping heads in the range of 20 feet (6 meters) 60 feet (18 meters).

Axial-flow pumps have relatively small waterways; although they are therefore not suitable for sewage, they are useful where there are large volumes of effluents or digested sludge to pump. Standard pumps are available with capacities ranging from 1000 gallons per minute (4500 liters per minute) to 100 000 gallons per minute (45000 liters per minute) but are only suitable for relatively low heads up to about 45 feet (14 meters).

9.3.2 Screw Pumps

These operate on the principle of the “Archimedean Screw”, and comprise an inclined screw which rotates, relatively slowly, within a trough. The rotation of the screw slowly raises the sewage or water between the open “threads”, which are of sheet steel, and the trough floor and walls.

Screw pumps merely lift water and are therefore only suitable for lift pumping stations; they can handle virtually any untreated sewage and are also suitable for sludge and effluents.
Standard pumps cover tremendous range from 130 gallons per minute (600 liters per minute) to 18000 gallons per minute (82 000 liters per minute); a particularly useful characteristics of these pumps is that they can operate, without damage, at zero flow and therefore they do not need automatic control.

Screw pumps operate only at low heads; the maximum lift is dictated by the largest screw which can conveniently be made and, in practice, this limits the maximum lift to about 20 feet (6 meters) per unit. This limitation can be overcome by installing two or more screw pumps in series, one above the other.

A major advantage of screw pumps over roto-dynamic pumps is that they require only a shallow excavation and simple inlet as, running continuously, they do not require only a shallow excavation and simple inlet as, running continuously, and they do not require storage on the suction side. Provided that the prime mover is housed screw pumps can be installed in the open where public access is restricted, such as on a treatment works site.

9.3.3 Reciprocating Pumps

Such pumps many be considered obsolete on sewage treatment works, except for dealing with sewage sludge; in this application, their positive displacement mode of operations helps prevent the clogging of pumping mains.

Reciprocating sludge pumps are robust, slow moving and reliable. Except in the very largest installations, they are considered to be the most appropriate type of sludge pump for Kenya. To reduce the risk of jamming of the pump valves, sludge should be carefully screened and preferably digested before pumping.

Although the ranges of capacities and pumping heads for reciprocating pumps are very large, a typical sludge pumping set would have a capacity lying between 50 gallons per minute (225 liter per minute) and 100 gallons per minute (456 liters per minute) and would operate at a delivery head of about 50 feet (15 meters). A characteristic of reciprocating pump is that the quantity delivered is effectively independent of the pumping head.

9.3.4 Pneumatic Ejectors

Ejectors are not recommended for use in Kenya as it is considered that other types of pumping equipment are more suitable.

An ejector comprises a closed tank into which sewage flows by gravity. When a certain depth is reached, an automatic control introduces compressed air which forces the sewage up a pumping main; the cycle then repeats. The compressed air is either piped from a central compressor house or supplied by a compressor house supplied by a compressor until located in the building housing the ejector.

The capacities of ejectors are usually less than about 80 gallons per minute (360 liters per minute) and they operate at heads rarely exceeding 40 feet (12 meters).

9.3.5 Solids Diverters

A diverter performs similar duties to pneumatic ejector and is considered more suitable for Kenya.

A diverter is also essentially a closed tank into which sewage flows by gravity; however, in this case, the solids are screened out by grating and, when the automatic control operates, screened sewage only is pumped by a small centrifugal pump. As they do not have to deal with sewage solids, these pumps may be as small as 20 gallons per minute (90 liters per minute) without risk of blockage.

During the pumping cycle, the screened sewage passes backwards through the grating and carries along the pumping main the solids previously removed.

9.3.6 Air Lifts

Air lifts are very simple; however, they have relatively low efficiencies and only operate at low heads, 20 feet (6 meters) being the practical maximum.

They comprise a vertical tube with its lower end in the liquid's density and atmospheric pressure forces it to rise and overflow the top of the tube.
This arrangement, which is very difficult to choke, is suitable for use at treatment works, especially for lifting sludge.

9.3.7 Priming of Pumps

All roto-dynamic pumps require priming; they will not pump unless they are internally flooded before starting.

Patented automatic priming devices are available, but they are all easily blocked and therefore cannot be recommended for use with sewage.
The alternative is to ensure that the impeller or propeller of any rotating element pump is below the level of the liquid to be pumped before starting. With solids diverters, this is automatically achieved as they fill under gravity and pumps are located at the bottoms of the units.

9.4 Prime Movers

Stationery steam engines are very rarely used today as other types of prime mover are cheaper and easier to maintain.

9.4.1 Electric Motors

Pumps usually have this type of driver. A guaranteed power supply is obviously an important consideration; duplication of power lines to important pumping stations is very desirable.
Electric motors are clean, convenient, reliable and relatively cheap. They land themselves easily to automatic control and are therefore particularly suitable for unattended pumping stations.

The rate of inflow to a sewerage pumping station usually fluctuates considerably throughout the day. If the part of the pumping head, for example when pumping sewage to sea, then the increase in the pumping head at maximum flow may present difficulties.
One solution to this problem is to have reliable speed electric motors; however, the necessary wiring and switch gear is expensive and complicated and therefore variable speed pump sets are not recommended for use in Kenya. The alternative solution in such circumstances is to install a range of pumps, some to pump low flows against and others to pump larger flows against higher heads.

The following types of electric motor are considered most suitable for use in Kenya; they all operate on alternating current:-

i) <u>The constant speed squirrel-cage motor simple, robust and economical</u>: however, it requires a high starting current which may not be acceptable to local electricity supply authority; "starters" can reduce the starting current by almost half.
ii) <u>Slip-ring motors</u> are not so simple and care therefore slightly more expensive than squirrel-cage motors, but they have lower starting currents.

iii) Synchronous motors are more complicated as the rotor windings are fed by a separate direct current "exciter". However, such motors are very economical for large pumping installations, when the electricity at a power factor unity. These motors require a relatively high starting current and starters are required for all but the smallest units.

9.4.2 Internal Combustion Engines

Such prime movers are usually more expensive in capital cost than electric motors; also, they are generally dirtier and noisier and require constant attendance by skilled operators, and competent maintenance.

The starting of internal combustion engines can be difficult and so they are not suitable for frequent start/stop duties, nor can they be easily controlled automatically.
However, if electricity is not available, then internal combustion engines may be the only possible source of power. In general, diesel engines are suitable for sewage pumping than either petrol-driven or gas engines.

If the local electricity supply is unreliable, then standby diesel pumping sets are warranted in important pumping stations, where pump failure would cause unacceptable flooding.

At a sewage treatment works, the gas evolved from sludge digestion process may be collected and used to operate gas engines; instead for coupling these engines directly to pumps, it is generally considered preferable to have them drive alternators. The electricity so produced can be invaluable during emergencies.

However, gas engines running on sludge gas can greatly complicate and even dominate the running of sewage treatment works and they should be installed only after very careful consideration of the economies of such arrangements and the reasons for the decision.

9.5 Design Features of Sewage Pumping Stations

The design of a pumping station is, considerable extent, dictated by the type of plant. Thus a station for a screw pump simply houses the prime movers, and the buildings for ejectors or diverters are essentially partly-buried boxes giving access to the equipment and its control gear.
Roto-dynamic pumps require more sophisticated stations, which can be roughly categorized as either at well or dry well. Both types of station normally comprise a substructure below ground level and superstructure, containing equipment which could be damaged by flooding, above the ground surface.

9.5.1 Wet Well Stations

In such stations, the substructure or wet well contains sewage into which pumps suspended; this arrangement ensures that the pumps are always primed. Usually, the prime movers are located in the superstructure and the drive in via cased shafting.

In wet well installations, pump maintenance, and especially the removal of blockages, is a constant problem as the pumps usually have to be withdrawn to gain access. For this reason, new sewage pumping stations of this type are rarely constructed, with one exception.

During recent years, several manufactures have started to produce watertight, submersible, portable pumping sets suitable for sewage, each comprising a centrifugal pump sets suitable for sewage, each comprising a centrifugal pump and an electrical motor; although it is preferable to have the compact control equipment above ground level, the remainder of the unit is lowered into simple underground chamber, such as manhole. This system considerable reduces capital costs. Maintenance is also greatly simplified as within minutes, a standby unit can replace a faulty set, which can then be transported to a workshop for overhauling and repair.
It is considered that such installations are especially suitable for Kenya, when the required pumping capacity lies within the range 100 gallons per minute (450 liters per minute) to 550 gallons per minute (2 500 liters per minute).

9.5.2 Dry Well Stations

The substructure of such stations comprises two compartments, a dry well to house the pumps and a sewage sump to store the sewage, sludge or effluent to be pumped.

The capital costs of such stations are more expensive than wet well stations of similar pumping capacity, but it is considered that the case of maintenance provided by this arrangement more than compensates for the differences; it is recommended that all larger sewage pumping stations in Kenya should be of this type.

Dry well sewage pumping stations will usually house centrifugal pumps. Both horizontal and vertical centrifugal pump sets are manufactured. In general, horizontal pumps are cheaper than vertical and easier to maintain. However, vertical pump sets have the great advantage that the prime mover may be installed above ground level, so that it is protected from flooding caused by heavy rain or perhaps by a burst pipeline; in such installations, the prime mover and pump are normally connected by shafting with universal joints. It is recommended that, when centrifugal pumps are used, vertical sets become the rule in Kenya.

Reciprocating sludge pumping sets may also be installed in dry well stations. These small sets, which include the prime mover, are usually located on the floors of the dry wells in order to reduce the suction heads on the pump; otherwise the station resembles one housing a centrifugal pump.

9.5.3 Packaged Pumping Stations

These self-contained, factory-built units are recent development; they operate by electricity and are fully automated. Usually, a unit is installed underground and comprises pumping sets enclosed in a protected steel substructure. Most are designed as dry well stations except that electric motors are usually close-coupled to vertical pumps so that they are also at bottom of the dry well.

9.5.4 Sitting

The sewage system dictates the approximate locations of all pumping stations should preferably be constructed away from residential property; they should always be readily accessible.
Sewage pumping stations and sewage treatment works, with their attendant stations, are frequently sited in low-lying areas, where flooding may be a risk. The floors of superstructures of pumping stations should always be elevated above the highest recorded flood level.
Electrical supply and mechanical failures do occur and all sewage pumping stations should be so located that any resulting sewage overflow causes a minimum of danger to public health, or nuisance or damage to property. Where possible, a screened overflow pipe, for use only during emergencies, should lead from a high point in the pumping station well to an adjacent ditch or stream.

9.5.5 Miscellaneous

It is false economy to provide too small a pumping station building; the difficulties met during installation, and during subsequent operation and maintenance, rapidly outweigh any saving in capital costs.

Pumping stations should be well illuminated. Also, they should be heated as necessary and well ventilated to minimize corrosion due to condensation and odour nuisance which can result from sewage gas; the ventilation of enclosed wet wells and sewage sumps is especially important, in order to reduce the hazard of explosion.

Water leaking from pumping glands, or used for washing floors and wells, collects in dry wells; their floors should consequently slope towards a simple sump. Either separate small drainage pumps or auxiliary suction pipes connected to main pumps should be provided to remove drainage water as it collects.

The designs and layouts of wet wells and sewage sumps are critical. If they are not hydraulically satisfactory, air will be drawn into pumps and efficiencies will be drastically reduced; for larger stations containing several pump sets, well and sump designed should be based upon model tests. Wherever possible, non-vertical surfaces should be steeply sloped to reduce the formation of silt or sludge banks.

9.6 Capacities

9.6.1 Pumping and Station Capacity

When sewerage pumping station has roto-dynameic pumps, it is total pumping capacity should be compatible with peak flows in the sewerage system it serves; if the sewers are not operating at their design capacities, then the installed pumping capacity should be correspondingly reduced.
It is relatively simple and inexpensive to change or add pumping sets, and thereby increase the pumping capacity of a station, provided that the station building is sufficiently roomy. It is considered reasonable to install pump sets to design period of from 5 to 10 years, depending upon how rapidly the local community is growing, but station buildings and other structural features should be normally be designed for 20 years ahead.

A fairly small sewerage pumping station, with the total pumping capacity less than about 800 gallons per minute (3 600 liters per minute), it is more convenient and simplifies maintenance to have all pumping set the same size; larger pumping stations, however, should preferably have two or more sizes of pump, the aim being to keep the rate of pumped flow from the station as uniform as possible.

Sewerage pumping stations with screw pumps or diverters cannot be designed in the way, as once initial installation is complete, the pumping capacities can only be changed by duplicating the installation. Where such types of station are provided, it is considered reasonable to design them for either the maximum flow the sewerage system served can produce or 50 per cent in access of the peak wet weather flows anticipated, which ever is the smaller. If, in the case of diverters, this formula results in design flows of 100 gallons per minute (450 liters per minute) or more, then centrifugal pumps rather than diverters should be installed.

9.6.2 Stand by Units

In the smallest sewage pumping station, the pumping equipment should be duplicated and should be so sized that either one of the two pump sets, working alone, .is able to deal with the peak inflow to the station; that is, there should be 100 per cent standby.

The percentage of standby may be reduced as the number of pump sets installed in a station increases; for example, for a station which has to deal with a peak inflow of 400 gallons per minute (1800 liters per minute), it may well prove cheaper to have three pump sets each rated at 200 gallons per minute (900 liters per minute) rather than two sets each with a capacity of 400 gallons per minute (1800 liters per minute); in this case, standby is only 50 per cent.

It is recommended that the percentage standby never drops below 33 per cent; that is, the total number of pump sets in larger stations should be such that about three-quarters of pumps are capable of dealing with peak flows, with the remaining pumps held in reserve.

9.6.3 Capacities of Wet Wells and Sewage Sumps

The rate of inflow to sewerage pumping station normally varies throughout the day. As the installed pump sets will each have finite, rather than variable, capacities, a sewage sump providing storage is required to deal with the inflow fluctuations; in the case of wet well type of pumping station, the terms “wet well” and “Sewage sump” are synonymous.

Effectively, the capacity of sewage sump is the volume between the highest level at which the pumps start and the lowest level at which they stop. Usually, the highest level will be just below the invert of lowest incoming sewer, to help prevent surcharging of the sewerage system.

A sewage sump's capacity should always be related to the rate of sewage inflow and the pump capacities, the aim being to reduce wear on the mechanical and electrical equipment in the station by minimizing the number of pump starts. Each pump should be limited to about six starts during any hour; the maximum number of starts occurs when the station inflows is equal to half the pumping capacity of one pump. On the other hand, if sewage sumps are too large, sewage will tend to become anaerobic during its detention.

It is recommended that the capacity of the sewage sump in a sewerage pumping station should be calculated in accordance with the formula

V =300Q

Where; **V is the capacity of the sewage pump in litres**

Q is the maximum rate of sewage inflow during dry weather in litres per second.

The calculation leading to this formula is given below.

In order to facilitate cleaning of the wells and pipe work and repairs to pumps, sewage sumps should be divided into at least two compartments; these should be interconnected by holes through the dividing walls which can be closed by penstocks, when necessary, to isolate a particular compartment. The capacities given above for sewage sumps are for the sum of the capacities of the individual compartments.

Calculations to determine Optimum size of Sewage Sump in a dry Well sewerage pumping Station

We wish to determine the size of the sewage sump so the pumps start only 6 times per hour.

Let V =volume of sewage sump (in liters)
Q =maximum rate of inflow during dry weather (in liters per second)
P =Capacity of pump (in liters per second)
(Here we assume only one pump operating but the argument still holds for several duty pumps operating simultaneously).

T = the time between starts (equals 600 seconds if there are 6 starts per hour)
Te = the time to empty sewage sump (in seconds)
Tf =the time to fill the sewage sump 9in seconds)

$$Te = \frac{V}{P-Q} \quad \text{and} \quad Tf = V/Q$$

$$T = Te + Tf = \frac{V}{P-Q} + V/Q = \frac{VQ + VP - VQ}{PQ - Q^2} = \frac{VP}{PQ-Q^2}$$

Hence $VP = 300P^2 - 150P^2 = 150P^2$

Or $V=150P$

If the pumps are sized so that Q=1/2P,

Then V=300Q

9.7 Pump Controls

Automatic pump control is relatively so simple and reliable that virtually all pumps driven electrically are controlled in this way; when this is done, only the larger stations require continuous manning.

The duties performed by pump controls in sewage pumping stations include:-

i) Starting pumps in sequence as the level of sewage in the sump reaches predetermined points.
ii) Cutting out smaller pump sets when larger are brought into operation.
iii) Isolating any faulty pump set and starting up its standby; when this happens, warning light or alarm should be simultaneously activated.

It should be possible for the operator to select manually whichever pumps he wishes to have on duty and those he wants as standby units, to operate only if one or more of the duty pump sets fails to start.

The pump controls are normally activated as the level of sewage pump varies. The more usual types of control for sewage pumping stations are:-

i) Float-operated systems where the float, necessarily located in the sewage sump, rises and falls with sewage; such are simple and satisfactory, provided that the floats are protected from turbulence. Rod and lever operated controls are generally more reliable than those with chains.
ii) Pneumatic systems incorporating a small air compressor, which bubbles through the sewage in sump; the pressure, which is generated in the air pipe-work, and is proportional to the depth of sewage in the sump, activates pressure-sensitive switches.
iii) Electrical systems, which usually comprises pairs of insulated probes which are activated as they become submerged; sometimes, to avoid rags and debris fouling the probes, they are enclosed in rubber bladders.

The pumps in stations at sewage treatment works are often more conveniently controlled by timing devices; for example, sludge pumps connected directly to the open sludge-withdrawal pipe of a settlement tank may be started at prearranged times, thus automating the periodic desluding operation. Where time-switches are used, there should also be an overriding safety control, such as one of those described above, which ensures that pumps can never run dry.
Where pumping station contains several pumps, it is usual to stagger operating levels of the pump controls so that the various pumps sets cut in at sewage levels not less than six inches apart.

This arrangement prevents the sudden electric power demands which would occur if several sets started simultaneously, and also gives sufficient time for the pump to accelerate to its full capacity before the next duty pump, or standby, starts, The first pump to start is normally the last to stop.

9.8 Pre-Treatment of Sewage before Pumping

9.8.1 Screening

Screw pipes and diverters will normally handle any solids, plastics or rags which are likely to be buried in Kenya's sewage. Roto-dynamic and reciprocating pumps are more susceptible to blockage and, when they are dealing with sewage or sludges, should be protected by coarse screens.

It is more convenient to install these screens in separate chamber, which can also be used to distribute the inflow between the various sump compartments. Bar-screens protecting smaller installations should have openings not exceeding 1½ inch (40 millimeters), but 4 inch (100 millimeter) openings are adequate for 12 inch (0.3 meter nominal size pumps (which can deal with over 2500 gallons per minute (11500 liters per minute).

At smaller stations, the bar-screens should preferably be inclined at about 45 degrees to the horizontal to facilitate handshaking. At larger stations, double, removal screens, located one behind the other, are simpler to keep clean;

these screens should have trays on their upstream bottom edges, to catch large objects, and should be counter-weighed so that they can be simply hoisted out of their chamber, one at a time, for cleaning.
Screenings should be drained and then quickly buried.

9.8.2 Grit Removal

The abrasive action of grit carried in sewage tends to wear pumps and their glands. However, grit removal before pumping is an expensive, impractical task except at a sewage treatment works, where the grit must be removed as part of treatment process. It is recommended that grit should only be removed from sewage at sewerage from sewage at sewerage pumping stations in Kenya in very exceptional circumstances, for example when it becomes economically viable as a means of increasing the life of an expensive submarine pipeline discharging sewage to sea.

9.9 Pumping Mains

The types of pipes used for water mains are also suitable for sewage pumping mains; pipes currently used for water supply in Kenya are described in Ministry of Water and Irrigation Practice Manual, 2005.

Sewage and sludge pumping mains should be less than four inches in diameter to reduce the risk of blockage; even this size is only acceptable if the sewage or sludge has been screened before pumping.

Pumping mains should be sized so as not to be less than four inches in diameter to reduce the risk of blockage; even this is only acceptable if the sewage or sludge has been screened before pumping.

Pumping mains should be sized so as to maintain a velocity of between 2 feet per second (0.6 meters per second) and 4 feet per second (1.2 meters per second) at the normal pumping rate; velocities should not exceed 6 feet per second (1.8 meters per second). Sewage and sludge often give off gas which can cause “air-locked” in a pipeline and therefore, whenever possible, pumping mains should rise steadily from the pumps to discharge point; if a peak in the main cannot be avoided, special air valves suitable for sewage should be used.
It should be possible to empty a pumping main in emergencies; this is preferably done by means of a wash out pipe leading into the sewage sump of the pumping station.

The normal hydraulic formulae used for water main design are applicable to sewage, but when pumping sludge the relatively high viscosity of this liquid must be taken into account.

9.10 Maintenance of Pumping Station

a) The area around the pumping station must be kept clean.
b) The solids, 9grit) settled in the settling tank must be removed every day
c) The inlet screens must be cleaned every day and the grit chamber should be cleaned every month
d) Grit and any solids which might have settled in the pumping sump must be removed regularly
e) All drainage channels in and around the pumping station must be kept clean
f) Keep all records of pump performance i.e. rate of pumping and quantity pumped.
g) Remove any scam forming in the settling tank and sump
h) Watch out for any leakages developing and stop them immediately
i) If any pump breaks down, it should be repaired within three days.
j) For operations and maintenance of the pumps sets, refer to the attached manual
k) Keep records of electrical performance, i.e. consumption, breakdowns etc.

9.11 Retention Pond

The retention pond will be used when both pumps have failed. The sewage will be diverted to the retention pond and after the pumps have been repaired then the stored sewage will be pumped out into the system.

10.0 PURPOSE AND RESULTS OF SEWAGE TREATMENT

Sewage is treated in order to reduce its undesirable characteristic, usually its BOD and suspended solids' concentration, to the degree necessary to ensure that it does not pollute or contaminate to an unacceptable degree any water resources into which it is discharged.

Sewage treatment should be carried out efficiently and economically, without nuisance or offense, or danger to the health of either the public or the plant operators.

Pathogens contained in sewage are reduced in numbers during treatment, but their removal is rather incidental unless specific disinfection techniques, incorporating chemicals, radiation, sand filtration or long retention maturation ponds, form part of the treatment processes.

The treatment of sewage normally results in the production of set solids, or sludge, which are offensive and dangerous to health in varying degrees; the disposal of sewage sludge can be a difficult and expensive problem.

Before deciding upon the methods and degree of treatment for any particular sewage, it is necessary to know what standard of final effluent will be acceptable.

Sewage treatment takes place naturally when a relatively small amount of sewage is introduced into a watercourse or lake or into the sea.

Natural treatment may be divided into two distinct processes, the sedimentation of the more dense suspended matter and the biological degradation of the organics still remaining in the sewage after settlement.

Two basic types of sewage treatment are discussed in chapters 12 and 13 of this manual:-

i) Treatment in waste stabilization ponds; this is usually, in effect, "natural" sewage treatment under controlled, optimum conditions.
ii) "Conventional" treatment where, as it were, the natural processes are speeded up.

As the subsequent chapters of this manual will show, the division between these two types of treatment is very imprecise; overlapping, and combinations of the two types, that frequently occur. .

10.1 Sewage Treatment Processes

For convenience, sewage treatment is usually divided into four categories:-

- Preliminary
- Primary
- Secondary and
- Tertiary

10.1.1 Preliminary Treatment Processes

Preliminary treatment comprises two distinct processes 'screening" and "grit removal

Screening removes coarse solids, such as rags, timbers, pineapple pieces and maize cobs. The grit removed is normally mineral matter, but it is contaminated by sewage. If preliminary treatment is not provided, the result could be blockages or damage to subsequent pipes and treatment equipment.

Preliminary treatment has a negligible effect on the sewage quality, as measured by BOD or suspended solids' concentrations; however, the removal of recognizable floatable by screening may be all that is required before the discharge of sewage into the open sea.

10.1.2 Primary Treatment

Primary Treatment removes easily settleable solids, for example those which would settle out in 30 minutes under quiescent conditions.

Purpose made "sedimentation" tanks are usually provided at conventional treatment works. In the case of waste stabilization ponds, sedimentation takes place in the vicinity of their inlets, as the velocity of flow of the sewage rapidly falls.

Sometimes, as in the case of septic tanks and waste stabilization ponds, the sludge which settles undergoes some biological breakdown in the primary treatment unit; more usually at a conventional works, effectively "raw" (that is undergraded) sludge is removed from the sedimentation units.

Primary treatment can be expected to achieve about 60% removal of suspended solids and approximately 40% reduction in BOD: BOD removal of about up to 60% may be achieved in a conventional sedimentation tank if chemical coagulation techniques are used; however, in this case, the resulting sludge is very voluminous, very difficult to de-water and virtually useless as a soil conditioner or fertilizer.

10.1.3 Secondary Treatment

The purpose of secondary treatment is biochemical to convert dissolved and colloidal organic matter into solids, which then settle out as sludge; the treatment is thus directed at reducing the BOD of the effluents from primary treatment units.

At "conventional" works, secondary treatment requires a biological unit followed by a "secondary" settlement tank. In the case of a waste stabilization pond, the entire process can take place in a single unit, the secondary sludge mingling with the "primary" sludge on the pond floor.
The degree of BOD removal by secondary treatment varies; so called "high rate" processes may achieve as little as 50% removal, conventional treatment about 90% and waste stabilization ponds up to 95%.

10.1.4 Tertiary Treatment

This is provided where it is necessary to "polish" an effluent from secondary treatment units; that is, to improve its quality, usually by the reduction or removal of one or more unacceptable characteristics.

Tertiary treatment of the effluent may, for example, be required in order to;-

- Reduce the BOD and/or suspended solids' concentration/s;
- Kill pathogens;
- Remove nutrient (phosphorus and nitrogen) or other undesirable dissolved salts;
- Destroy tastes and odors.

It should be noted that tertiary treatment is a supplement to preliminary plus primary plus secondary treatment; it is not intended to replace them.

10.1.5 Reductions in the Numbers of Pathogens during Sewage Treatment

The isolation, identification and counting of individual species of pathogens is very laborious, and it is customary to utilize coliform bacteria as presumptive evidence of the presence of pathogens in sewage or water.

The removal efficiencies of bacteria by sewage treatment processes are given as follows:-

Table 10.1 Removal Efficiencies of Bacteria by Sewage Treatment Processes

Type of Treatment	Percentage Removal of Bacteria
Conventional –Primary Only	25-35
Conventional –Primary plus secondary (percolating filters)	90-95
Conventional –Primary plus secondary (activated sludge)	90-98
Waste Stabilization Ponds	90-98

Although the populations of coliform bacteria are thus considerably reduced during sewage treatment, the numbers initially in the raw sewage are usually so large that after treatment there are still very many left.

Human faeces contain about $1x10^{12}$ coliforms per head per day. 100 mm of untreated sewage may contain 100 million coliforms; therefore, even if treatment reduces the numbers present in the sewage by 98%, two million coliforms will still survive in the treated effluent.

The rates of removal of other organisms are generally less than for coliform; for example, as against a reduction of up to 98% for bacteria during "complete" treatment including an activated sludge plant, the corresponding reduction for viruses is only about 90%; the effects of conventional treatment on ascaris and hookworms is much lower.

The methods of tertiary treatment usually employed to reduce the numbers of pathogens still further are:

(i) Slow sand filtration followed by chlorination, or
(ii) Maturation ponds, sometimes with chlorination of the effluent

11.0 CONVENTIONAL SEWAGE TREATMENT

This type of treatment is appropriate where is economically justified, perhaps because of scarcity or high price of land suitable for waste stabilization ponds. The better solution of sewage treatment in a particular case may well be combination of conventional units, providing preliminary and primary treatment, followed by "unconventional" waste stabilization ponds, which provide secondary.

When compared with waste stabilization ponds, the operation and maintenance of conventional treatment units are complicated. At the time a conventional works is proposed, its subsequent running should be thoroughly planned, and suitable personnel either recruited or trained well in advance of the commissioning of the installation.

The design of any conventional works for Kenya should comply with certain general rules. Any mechanical plants comply with certain general rules. Any mechanical plant proposed plant should be as simple as possible, and automation (as against automatic alarm systems which should certainly be used) should be kept to minimum. Except on very small works, treatment units should always be at least duplicated in parallel, so that the entire works is not put out of commission during the maintenance or repair of a single unit.

11.1 Design Criteria

The usual methods of conventional sewage treatment are shown diagrammatically on Figure 11.1. The methods considered more suitable for Kenya, for reasons explained later in this Section, are underlined; the recommended design criteria for these are summarized in Table 11.1 and 11.2.

Table 11.1: Summary of recommended Design criteria for preliminary and primary conventional sewage treatment in Kenya

Treatment	Design Criteria
<u>Preliminary – Screening</u> Trash screens (Occasionally used to protect bar screens) Bar screens	4 inch (100 millimetres clear opening 1 inch (25 millimetres) Clear opening. Maximum velocity of flow through opening – 2.5 feet (0.75 meters) per second
<u>Preliminary – Grit Removal</u> Constant velocity grit channels (controlled by proportional weir or parshall flume)	Constant velocity – 0.9 feet (0.27 metres) per second
<u>Primary – Sedimentation tanks</u>	<u>Dry weather</u> Surface loading (where there is no recirculation through tanks) 600 gallons per square foot per day (29 cubic metres per square metre) based upon average dry weather in-flow. <u>Wet weather</u> Surface loading: - 1 500 gallons per square foot per day (73.5 cubic metres per square metre) based upon the average wet weather inflow
<u>Primary – Septic Tanks</u>	Capacity = the average daily wet weather inflow + 100 gallons for each pound (1 000 litres for each kilogram) of suspended solids in the sewage.

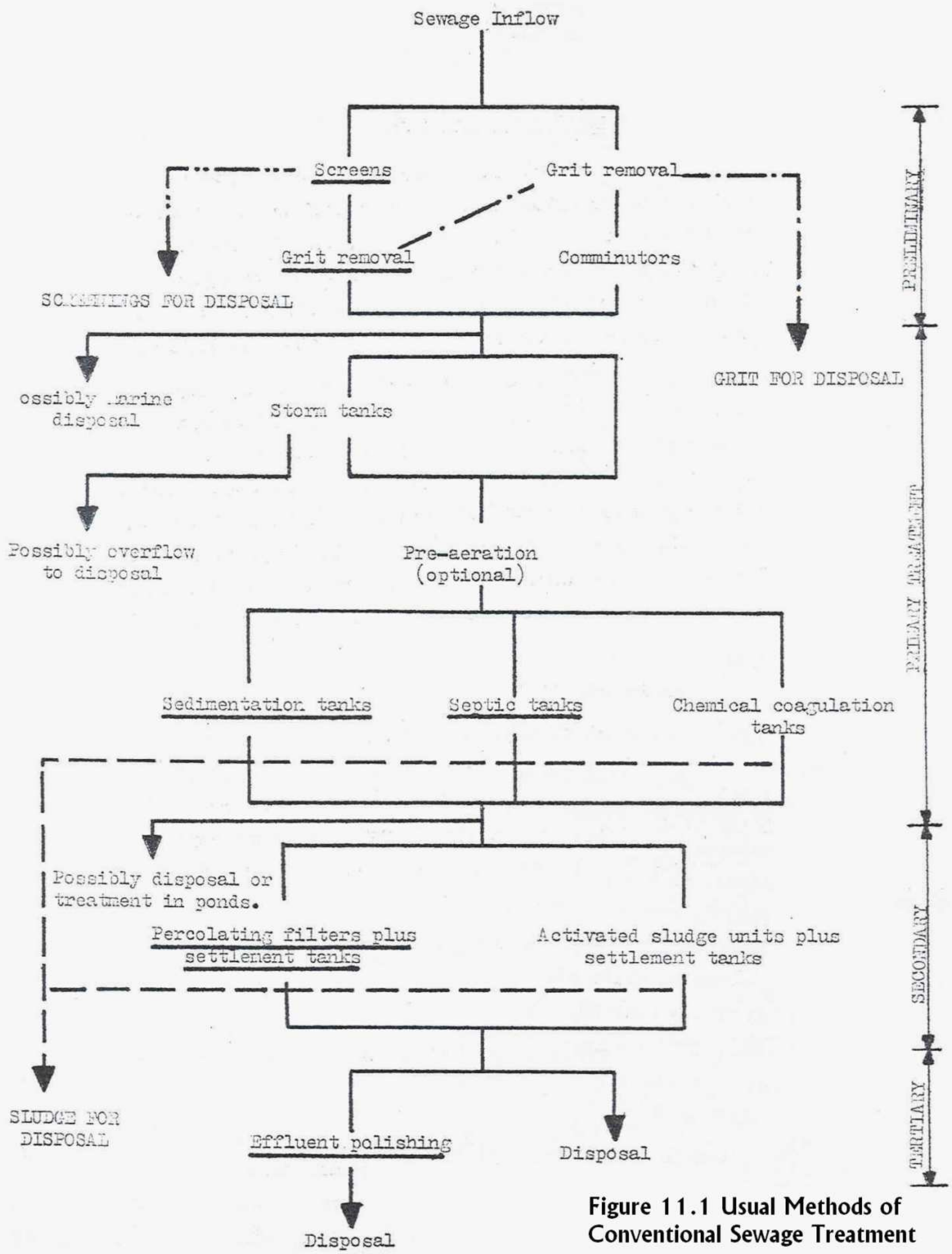


Figure 11.1 Usual Methods of Conventional Sewage Treatment

Note: Methods shown underlined are considered more suitable for Kenya

Table 11.2 Summary of Recommended design criteria for secondary and tertiary conventional sewage treatment in Kenya

Treatment	Design Criteria
Secondary – percolating filters	Single filtration 0.18 pounds of BOD per cubic yard (3 kilogram's per cubic metre) of filter media. With recirculation 0.30 pounds of BOD per cubic yard (5 kilogram's per cubic metre) of filter media.
Secondary - settlement tanks (after percolating filters)	Surface loading:- 1 500 gallons per square foot per day (73.5 cubic metres per square) based upon peak flows
Tertiary Land treatment	700 gallons of effluent per 100 square feet (340 litres per square metre) per hour.
Slow sand filters	250 gallons of effluent per 100 square feet (120 litres per square metre) per hour.
Upward flow clarifiers	1 500 gallons of effluent per 100 square feet (720 litres per square metre) per hour.

11.1.1 Preliminary Treatment-Screening

The types of screens here described are often referred to as "coarse", as against fine mesh screens which are sometimes used in industrial processes and for water supply intakes.

The purpose of screens at treatment works is to remove from sewage gross which could interfere with it subsequent treatment. When used for this purpose, the effectiveness of screens is measured largely by reduced maintenance costs for the entire works, not by the quantity of screening removed form the sewage.

For the treatment of sewage, vertical or inclined bar screens with one inch (25 millimeter) clear openings are most suitable; however, if the treatment works is at the end of a large diameter gravity sewer and large objects are expected in the sewage, it is often advisable to protect a one inch (25 millimeter) screen by a "trash" screen with 4 inch (100 millimeter) clear openings located immediately upstream.

The bars of screens should be tapered to reduce the risk of solids wedging in the spaces. Large bar screens normally have raking mechanisms to keep the openings clear. However, mechanically raked screens are recommended only for the largest municipalities in Kenya; elsewhere, hand-raked screens considered suitable.

Where mechanically cleaned screens are used, they should preferably be vertical with "grab" type raking mechanisms; such screens have the great advantage that they have very few submerged moving parts and therefore corrosion is minimized and maintenance simplified. It is convenient to control the raking mechanisms by time-switches, with overriding safety controls should be sewage head on the upstream side rise above a pre-determined height, thus indicating that the screens are becoming chocked.

Where mechanically cleaned screens are installed, hand-raked screens should be provided also to act as by-passes should a raking mechanism fail during the time of maximum flow.
Hand-raked screens should be placed at an angle of between 30 degrees and 45 degrees from horizontal. It is usual to have a perforated tray placed across the channel at the top of the screen to drain and store the screening before they are taken away for disposal. The total area of screens required is such as to ensure that the maximum velocity of flow through the openings does not exceed 2.5 feet (0.75 meters) per second, at this velocity; it is unlikely that screenings will be forced through the screens.

Screen chambers should be designed so as to minimize eddies which tend to trap screenings and grit in the corners of the chambers; at all times, the velocity of flow through a screening chamber should be sufficient to keep in suspension al sewage solids, including grit, if there is no grit removal unit upstream of screens.

The total volume of screenings removed by one inch (25 millimeters) clear opening screen, from typical Kenyan sewage during dry weather, is likely to be of order of 7 cubic meters) of sewage per day; the organic content of these screenings will be about 70 per cent.

On many works in developed countries, screenings are macerated and then returned to sewage flow. However, it is recommended that the screenings in Kenya are either buried, or incinerated after air drying.

A comminutor is a coarse screen, either rectangular or in the shape of a circular drum, that has mechanism which cuts materials retained on the screen to a size that permits them to pass through the screen openings; this eliminates the problem of disposing of screenings. The waer on the cutters comminutors can be very high and, as it is very likely that such equipment would have to be important, it is considered that for Kenya the disadvantages of using comminutors outweigh the advantages.

When screens are used, it is suggested that grit removal takes place after screening; however, grit removal units should be located before comminutors, to prolong the life of cutters.

11.1.2 Preliminary Treatment – Grit Removal

The grit in sewage is relatively inert mineral matter; its removal is necessary to reduce wear on machinery and attrition of pipeline linings to facilitate sludge treatment and handling and to prevent excessive accumulations of grit in tanks, paperwork and channels.

As the settlement velocities of grit are substantially greater than those of organic solids, grit can be removed preferentially, by selective deposition.

There are several patented type of grit removal unit on the market, all based upon this principle. The simplest type of unit is the constant velocity channel, which is not patented.

If the constant velocity channel is rectangular in section, then the flow should be controlled by installing a proportional weir at its downstream end; this type of weir has characteristic that the head on the weir is proportional to the flow, and so, regardless of the depth of flow sewage in the channel, its velocity is always the same.

More usually, and especially at larger treatment works where multiple channels are installed, in parallel, the constant flow channel has a “vee-shaped” cross section; the “vee” of the cross section should approximate to shape of a parabola. Constant velocity at varying depths of flow is achieved by installation of a Parshall (or standing-wave) flume at downstream end; this same Parshall flume can be used to meter the works’ sewage.

Grit removal units should be designed to remove grit of specific gravity 2.0 or above, 0.2 millimeters or more size. It is anticipated that where these criteria are used, 80 per cent of the grit contained in the sewage during the dry seasons will be removed; the daily volume of grit produced by the unit is likely to be of the

order of 32 cubic feet per million gallons (2 cubic meters per 10000 cubic liters) of sewage, for a new sewage system.
Although the Reynolds' Number will exceed 1, Strikes' Law gives a reasonable approximation of settling velocity of grit. For example, at a temperature of 26.3^0 Centigrade, which is the average of the "minimum" temperatures for Mombasa, based upon Stokes' Law the settling velocity of spherical grit particles is approximately 0.51 inches (13 millimeters) per second.

For design purposes, a constant velocity of flow in grit channels of 0.9 feet (0.27 meters) per second is considered suitable. A margin of approximately 30 per cent should be allowed when calculating the lengths of channels required, compensating for inlet and outlet defects, and also for the irregular shapes of grit particles.
Grit storage should be provided below the theoretical channel sections; the storage compartments should be shaped so as to simplify grit removal.

It is proposed that normally grit will manually removed form channels, using spades and wheelbarrows. However, mechanical dredgers fixed to travelling bridges may be used to larger works, but only where 24 hours supervision by component operators is assured; all mechanisms should be controlled by manual switches.

The grit which is removed should be used as landfill on dumping grounds from which the public is strictly excluded. For health reasons, grit should be handled by bare hands and, as the organic matter it contains is likely to attract insects, birds and rodents, the dumped grit should by soil each day.

11.1.3 Storm Tanks

Where storm water is allowed to enter the sewerage system, it may be that during heavy rain the works' inflow will be so great as to endanger the operation of the primary treatment units; in such cases, storm tanks are required.

It is recommended that in Kenya, as storm water ingress to sewers will be strictly limited, storm tanks should be identical to, and should be considered to be standby primary sedimentation tanks; they may then be used not only during storms, but also whilst the regular sedimentation tanks are out of commission, for repair or routine maintenance.

Storm tanks should be empty at a start of each storm. When they have filled, provided that the discharge standards imposed on the works' effluent allow storm water to overflow in this way during heavy rain, their effluent may be discharged directly into the local water resource; otherwise, the effluent must pass to secondary treatment units. When the storm has subsidized, both the dilute sewage and sludge held in the storm tanks should be conveyed to inlet to primary settlement tanks for treatment.

The criteria for the design of storm tanks should be the same as those used for sedimentation tanks.

11.1.4 Primary Treatment -Sedimentation Tanks

Primary settlement or sedimentation removes suspended matter from sewage by providing relatively quiescent conditions under which particles (other than colloids) which have higher densities than water can settle out as sludge. At the same time, substances such as oil and grease, with relative density less than unity, rise to surface as a scum. Sedimentation or primary settlement tanks are so designed that the sludge and scum can be removed separately from the clarified effluent.

Although the process is primarily one simple settlement under the influence of gravity, sedimentation, especially of smaller particles, is assisted by flocculation which takes place in the tank.

If sewage has traveled a long distance or if it contains a high proportion of wastes from food factories, there is a danger that it will be septic as it enters the settlement tanks, thus producing a tendency for settled sludge to gasify and rise again.

To guard against this, pre-aeration tanks may be used; the function of such units is to increase the dissolved oxygen content of the sewage, thus countering any septic, before it enters the sedimentation tanks. It is suggested that, where it is considered that pre-aeration tanks may be required, they should not be installed initially, but a part of treatment works' site should be set should the subsequently prove to be necessary.

There are several alternatives designs of primary settlement tanks. The type of tank selected has little effect upon the degree of purification achieved; however, certain types involve simpler operation and maintenance.

All types of sedimentation tanks have some features in common. Baffles are required at the point of inlet of the sewage, in order to dissipate its kinetic energy, to diffuse the flow equally and prevent short-circuiting and local eddies. The clarified sewage leaves the settlement tanks via overflow weirs, with baffles to prevent scum from passing over with the effluent.

Two types of settlement tanks (for both primary sedimentation and secondary settlement) are recommended for use in Kenya. For smaller works, "Dortmund" or hopper tanks with walls sloping at 60 degrees to horizontal are proposed; the square sides of hopper tank should not exceed about 24 feet (7.25 meters), in order to avoid excessive excavation. For larger works, relatively shallow, circular radial-flow type tanks, with central inlets and peripheral overflow weirs are recommended; they should have raking mechanisms which collect the scum and sludge produced. Scum should be removed from the tanks immediately, but sludge may be stored in central, integral hoppers, pending periodic removal.

The maximum size of each unit should be of the order of 120 feet (36.5 meters) in diameter, with an 8 feet (2.4 meters) side-wall depth; however, units up to 150 feet (45.5 meters) in diameter, with a minimum depth of 9 feet (2.7 meters), may be used at very large treatment works. It is suggested that the floors of each tank should slope at 1 in 12 to central sludge hopper which has walls sloping at 60 degrees to the horizontal. Notched peripheral weirs are recommended to reduce the effects of wind action.

It is considered that the sludge-raking mechanisms most suitable for Kenya are the type which is supported from rotating half-bridges, pivoted about the tank centers. The electric peripherals drive units normally run on rails on the tank walls. Each sludge-rake, or scraper, should have linked blades, attached to its access bridge or hinged anchor tubes.

Scum removal is usually by radial arms connected to and rotating with the scrappers.

Regardless of whether the sedimentation tanks are hopper-shaped, or circular, it is preferable to remove sludge hydrostatically, using telescopic draw-off valves.

The most important criterion for design of primary settlement tanks is the surface loading or overflow rate, which is equivalent to average upward velocity. For Kenya, the following surface loadings are considered suitable:-

<u>During dry Weather</u>

Without recirculation:

600 gallons per square foot (29 cubic meters per square meter) of sedimentation tank surface area per day, based upon the average dry weather flow.

With recirculation of secondary effluent through the sedimentation tanks

900 gallons per square foot (44 cubic meters per square meter) per day, based upon the maximum dry weather flow.

<u>During wet weather</u>

1500 gallons per square foot (73.5 cubic meters per square meter) per day based upon the average wet weather inflow, which may be taken as equivalent to 75 per cent of the peak flow calculated as described in sub-section 7.2 of this Report. (The choice of factors A or B or C etc should be the same as for the largest sewer carrying sewage into the treatment works).

The other important factor for sedimentation is the detention period, which is the volume of a settlement tank divided by the quantity of sewage flowing through it in one day; although, in the high temperatures which often prevail in Kenya, the detention period should be short to minimize the risk of sewage becoming septic, it should not be so brief that the efficiency of the process is reduced; a minimum detention period of approximately 1 hour, based upon the peak dry weather flow rate, is considered to be suitable.

If the sewage in a sedimentation tank is allowed to turn septic, or if the accumulated sludge is allowed to undergo anaerobic decomposition, then the efficiency of the settlement process will be very much impaired.

To keep conditions aerobic, operational visits at least every other day to the simplest hopper type sedimentation tank are required; tanks containing machinery require almost continual supervision during daylight hours.

Where such a high degree of attendance is not possible, either "Imhoff" tanks or septic tanks should be used. Imhoff tanks are two storey sedimentation tanks; the upper part acts as a conventional, aerobic sedimentation tank and the bottom half is a sludge compartment, where sludge "digestion" takes place.

The cross section of an Imhoff tank is complicated, requiring expensive concrete formwork during construction when compared with the normal type of sedimentation tank.

11.1.5 Primary Treatment-Chemical coagulation

The settlement of small particles may be improved by the technique of chemical coagulation, which is also called "precipitation"; iron or aluminium salts are normally used. Because of the great sludge disposal problems which result from chemical coagulation, it is recommended that this process should not be used in Kenya, except as a temporary measure for improving the efficiency of primary treatment in an overloaded existing works.

11.1.6 Primary Treatment-Septic Tanks

Septic tanks are primary sedimentation tanks where anaerobic conditions prevail.

The gassing which takes place in a septic tank results in a relatively poor efficiency of sedimentation; however, effluents from such units, although foul, contain little settle able matter. The septic tank requires very little maintenance, apart from occasional de-sludging, and has application where there is a lack of competent supervisory labour and also a relatively poor efficiency of primary treatment is acceptable.

A septic tank should, as a sedimentation tank, provide conditions as quiescent as possible. It has been found that, within reason, the shape of a septic tank is unimportant provided that the liquid depth varies between 0.9m for smaller tanks to 1.8m for larger. Two compartment tanks, where the first compartment contains between half and two thirds the capacity of the entire unit, have been found to provide more efficient treatment; little additional efficiency is achieved by having more than two compartments.

When in operation, scum floats on the surface of the liquid in a septic tank: the floor is covered by a layer of sludge; the "clarified" liquid is contained between the scum and the sludge. The amount of scum and sludge which accumulate depends upon the frequency of cleaning the particular septic tank.

Septic tanks should always be roofed. Because the process does not require oxygen, ventilation as such is not required, although an outlet should be provided to allow gas to escape as the tank is filled.

11.1.7 Secondary Treatment

The purpose of secondary treatment is biochemically to convert dissolved and colloidal organic matter in the effluent from the primary units into settleable solids, which are then removed.

The two methods of secondary treatment normally used at conventional sewage treatment works are:-

(i) Percolating filters,
(ii) The "activated sludge" process.

In each case, the biological process is followed by settlement.

A percolating filter comprises a tank containing inert media which provides a large surface for the growth of the sessile organisms mainly responsible for treatment. Aerobic conditions are usually provided by natural ventilation, which limits the depth of media in a filter bed to about 2m. Very occasionally, deeper filters ventilated by forced draught are used, but such units suffer badly from corrosion and rarely compare economically with other arrangements.

Primary effluent is distributed evenly over the upper surface of the media. As this settled sewage flows past, the organisms attached to the surface of the media remove any organic material, which are digested. The sewage leaves the filter bed via drains laid across its normally sloping floor; air to ventilate the bed also flows through these drains.

The effluent from a filter bed contains the excreta and dead bodies of the filter organisms; this organic material settles out when the filter effluent is passed through a secondary settlement tank (often called a humus tank) which resembles a primary sedimentation tank and similarly produces sludge.

The activated sludge process depends upon "seeding" the primary effluent with a very large population of degrading organisms suspended in the" activated sludge". The mixture is aerated and the sludge kept in suspension by pneumatic or mechanical means. When biological activity is complete, the sludge is removed in a secondary settlement tank, which also resembles a primary sedimentation tank.

Percolating filtration, depending as it does upon natural ventilation, is a less intensive process then activated sludge; also, the depth of filter beds is limited. Consequently, excluding the preliminary and primary treatment units and the final settlement tanks, all of which are common to both processes, the land requirements for percolating filters may be seven times greater than those for activated sludge units.

As sewage gravitates through a percolating filter, it suffers a loss of head which may be 2.4m or more, compared with as little as 0.3min a typical activated sludge unit. There, the use of percolating filters could result in expensive pumping costs, should the site of the treatment works be on relatively level, rather than sloping ground.

Even where pumping is not necessary, as a rule the construction costs of percolating filters are higher than those of equivalent activated sludge tanks, although the operating costs of the latter process are normally much higher.

Percolating filters have the major advantage that they are extremely simple to operate and maintain, whereas the activated sludge needs constant, competent supervision. Therefore, percolating filters, rather than activated sludge units, are generally recommended for use in Kenya.

Design Details of Percolating Filters

The simplest method of spreading the primary treated effluent over the surface of a percolating filter is by means of rotating distribution; although these can be driven electrically, the thrust generated by the jets of sewage which discharge from the arms are sufficient to rotate a properly balanced distributor. It is proposed that all filter beds in Kenya could have rotating distributors and consequently should be circular in plan; all the existing filter beds in Kenya, outside Nairobi, are in fact constructed in this way.

If the primary sedimentation tanks ha may be taken as designed in accordance with the criteria given elsewhere in this section, it may be assumed that they will achieve a reduction of approximately 40% in the BOD load of the sewage; therefore, the leading on the secondary treatment units being equivalent to 60%ve of the BOD of raw sewage.

If, however, primary treatment is by means of septic tanks, in view of the poor maintenance which such a treatment works is likely to receive, it is preferable to assume that the septic tank efforts no reduction at all in the sewage BOD.

It is possible to achieve very great reductions in BOD (measured in weight of FC) removed rather than as the percentage reduction by utilizing what are known as "high rate" filters . However, the costs and the complications of running such filters can be high and, for Kenya, "low rate" filters only are recommended.

Assuming a filter is 2m deep, then a suitable loading is 0.1kg/m^3 BOD of filter media per 24hrs.

This loading may be increased to 0.17kg/m^3 if the technique of recirculation is adopted. In this context, recirculation means returning some of the effluent from the secondary settlement tanks to the inlet to the filter beds, so that the flow through the filter beds is always equivalent to the peak dry weather flow; this means that, when the actual sewage inflow to the works equals the peak flow, recirculation temporarily ceases. Often, the recirculated effluent plus secondary sludge is discharged into the inlet of the primary sedimentation tanks.

Design Details of Activated sludge units

The primary treatment effluent contains organic matter mainly in colloidal and dissolved states. If this settled sewage is aerated, the organic matter will be consumed and "activated sludge", comprising zoogleal flocs will be produced; this activated sludge will settle out under quiescent conditions.

The principle of the activated sludge process is that the primary effluent is "seeded" as it enters the aeration unit with activated sludge previously formed and settled.

The characteristics of a properly designed and operated activated sludge plant are aerobic conditions throughout and activated sludge which will settle; a minimum dissolved oxygen concentration in the aeration tank liquor of the order of 1.5 milligram's per litre should be aimed at.

The process variables, which are inter-related include:-

(i) The solids' concentration within the aeration tank, which depends upon the solids' concentration and the volume, relative to the settled sewage inflow, of the activated sludge returned from the settlement tank.

(ii) The ratio between the applied BOD load and the total weight of solids carried in the aeration tank.

(iii) The capacity of the aeration tank, which may be described either in terms of the detention period, based upon the average settled sewage inflow, or as a volume related to the applied BOD loadings.

Suggested design for Kenya are the maintenance of a suspended solids concentration of 4000milligrames per litre in the mixed liquor in the aeration tanks, and applied loadings of 22.5kg BOD for every 100kg of sludge solids carried in these units.

It should be noted, however, that as the climate throughout Kenya varies so greatly, the optimum design criteria in one location may produce inefficient results in another. Probably more than any other sewage treatment process, activated sludge units should be constructed so as to be very flexible in operation. The optimum conditions for a particular plant may then be found by trial and error, by altering the controls, the ratio of sludge return and the solids concentration while monitoring the volumes and qualities of unit inflow and effluent.

The efficiency of the activated sludge treatment process is assessed by measuring the percentage of the applied BOD load which is removed-the removal of suspended solids is incidental to the process –and also by the weight of applied BOD removed per unit of electricity consumed by the aeration systems.

In an activated sludge unit, the dissolved oxygen content of the mixed liquor in the aeration tank may be maintained by bubbling through pressurized air; alternatively, aeration may be provided by mechanical aerators which, by vigorously agitating the liquid, allow it to trap and dissolve atmospheric oxygen.

Mechanical aerators are considered more suitable for Kenya because of their simplicity and reliability. It is preferable to have several rather than a single mechanical aerator; by these means, breakdowns of a single unit are not too important. If the mechanical aerators are provided with individual drives, then a complete assembly may be simply replaced should a failure occur.

Normally, a unit will be in the form of a channel containing severalplus activa aerators acting in series; short circuiting of the treatment process is thus minimized. Each channel will operate in parallel with other identical channels.

Flexibility in plant control can be achieved. The simplest way is to make use of the "step aeration" principles; that is, instead of all the settled sewage entering at the beginning of each aeration channel, the inflow is introduced at various points along the unit.
Activated sludge plants are often supplied as factory made units with integral settlement tanks; the walls are made of steel protected against corrosion. Often provision is made in the unit to aerate, and thereby aerobically degrade, the surplus activated sludge. Such units are often termed 'extended aeration plants". The applied loadings on an extended aeration plant should not exceed 10kg BOD per 100kg of sludge solids carried (compared with 22.5kg BOD per 100kg of sludge solids recommended for conventional activated sludge plants).

Fabricated activated sludge plants are suitable for certail applications, but it must be remembered that they require frequent, competent, supervision and although they may be simple and relatively inexpensive to install, their running costs may be very high.

Design Details of Secondary Settlement Tanks

The function of secondary settlement tanks is similar to that of primary sedimentation tanks; however, because secondary sludge are less dense than primary sludge, the design criteria are slightly different. For Kenya, the following maximum surface loadings (viz. based upon peak flows) are considered suitable:-

Table 11.3 Recommended Maximum Surface Loadings for Secondary Settlement Tanks

Type of Treatment	**Surface Loading**	
	M^3/M^2/day	Gallons/ft^2/day
Percolating filters	44	900
Activated sludge units	29	600

In each case, these are to be based upon the peak flows through the units; however, in the case of settlement tanks following activated sludge units, the volume of sludge returned to the aeration units may be ignored in the calculations.

Another important factor in the design of secondary settlement tanks is the weir loading. In order to avoid scouring and carrying over sludge, the weir loading, based upon the peak flow rate, should not exceed $300m^3/m$ of weir (20, 000 gallons per foot length) per day.

In the case of activated sludge, in order to keep the organisms viable, continuous sludge withdrawal is required; excessive retention of sludge in these tanks, for example to increase the solids' concentration, merely results in the consumption of oxygen and decreases the activity of the sludge. Activated sludge required by the process should be returned to the aeration tanks immediately.

Unwanted sludge resulting from percolating filter treatment, and also surplus activated sludge, is usually returned to the head of the works for settlement and concentration in the primary sedimentation tanks, along with the primary sludge.

11.1.8 Tertiary Treatment

Should the effluents from a "conventional" sewage treatment works be of too poor a quality, then tertiary treatment is required.

Chemical engineering processes, such as absorption, electrodialysis, reverse osmosis, ion exchange, foam separation and distillation, can be used to eliminate specific, undesirable, constituents; these techniques are likely to be required only rarely in Kenya during the next few years.

More usually, it will only be necessary to effect a decrease in the BOD and/or the suspended solids concentration in the effluent; four ways of achieving this are considered particularly suitable for Kenya:-

If the elimination of pathogens from the effluent is necessary, then this may be most economically achieved by the chlorination of effluents from tertiary treatments; effluents from slow sand filters and maturation ponds are most suitable for chlorination, and are likely to require doses of about 2milligrams of chlorine per litre.

11.1.9 Land Treatment using grass plots

In this method, effluent is run on to grassland through a system of channels; after it has flowed over the surface, it is collected by a second series of drains. The gradient of the field should not exceed about 1 in 60; otherwise, there is a tendency for the effluent to cut its own channel.

340 liters of effluent per square meter (700 gallons per 100 square feet) of land per hour is a typical rate of treatment; this should achieve reductions in BOD and suspended solids' concentrations of approximately 50% and 70% respectively.

11.1.10 Soil as a wastewater treatment system

Land treatment is defined as "the controlled application of partially treated wastewater onto land to achieve treatment and disposal goals in a cost-effective manner" (Crites et al., 2000). Land application is the oldest practice used to manage wastewater and control environmental pollution. Historical evidence of wastewater spreading to the soil for crop irrigation and sanitation purposes goes back to the ancient cities and palaces of Minoan Civilization (Angelakis and Spyridakis, 1996; Angelakis et al., 2005). With the progress of time land treatment has gone through different stages of development but the basic principles regarding the planning, operation and management practices were developed after runoff; and rapid infiltration (RI); the major difference from SR systems is the lack of vegetation and the higher application rates (Reed et al., 1995).

Nowadays, land treatment is recognized as an alternative or supplement to conventional
Wastewater treatment plants that can be both environmentally sound and economically
Viable. Land treatment systems have been currently expanded for the treatment of various types of effluents including landfill leachates, dairy effluents, and meat processing wastewater, olive oil mill wastewater, agricultural drainage, and contaminated groundwater (Paranychianakis et al., 2006 and references there in). In addition, the direction of land treatment systems toward biomass production and bioenergy observed in recent years may contribute to climate change mitigation. Currently, the scientific interest focuses on the adoption of appropriate operation and management strategies that could alleviate the potential adverse environmental impacts, understanding the cycling of nutrients and their removal, the fate of pathogenic organisms and the factors affecting their survival, and the fate of toxic organic compounds.

A brief overview of historical evolution of land treatment concept is discussed with emphasis on the most important technological developments and on the sanitation requirements. Furthermore, the current trends and the future prospects of land treatment concept are presented in terms of management of wastewater and other polluted water sources.

11.2 Chronological development of land

In 1850's when the "sewage farms" were expanded in Europe and USA in an effort to control pollution and protect public health (U.S. EPA, 1979) ,The development of conventional wastewater treatment technologies in the turn of 19^{th} century resulted in a decline of land treatment systems (Reed et al., 1995), but the interest was renewed after the passage of Clean Water Act in 1972 and particularly the last two decades. This is mainly due to the low construction, operation and maintenance costs, making this technology suitable for small communities or decentralized clusters of homes, institutions and isolated industrial units (Crites and Tchobanoglous, 1998; Angelakis, 2001). Different types of land treatment systems were developed through the passage of time depending on the rate of applied hydraulic load, the presence or absence of vegetation, the needs for reapplication treatment and the intended level of treatment.
These include: slow rate systems (SR): these systems utilize soil matrix for treatment and
The applied load is based on vegetation water requirements; overland flow (OF): they
Utilize soil surface and vegetation for treatment.

11.2.1 Ecological and Health Risk

Ecological and heath risk issues arising for wastewater application to the land Waste has played a significant role in history. Bubonic plague, cholera and typhoid fever were diseases that altered the populations of Europe and influenced monarchies toward to waste and wastewater management. Greek ancient civilizations and Romans were aware of the benefits of sanitation and developed innovative hydraulic technologies and practices to protect public health; however issues of environmental protection were not considered at all during this period. Athens, in 500 BC, had the first municipal dump in the western world. Regulations required waste to be dumped at least a mile from the city limits. In following years until the early 1800s the sanitation concept was totally lost (US EPA, 1979). A renewed concern about sanitation appeared during Renaissance while the great epidemics of cholera and typhoid fever in the early 1800s stressed the need for sanitation and the protection of water sources. Although the germ theory had not been developed yet, polluted water sources were considered responsible for the occurrence of diseases. These epidemics motivated the responsible agencies to establish the sanitation requirements by constructing sewerage systems, wastewater treatment units and drinking water treatment and environmental protection policies.

In 1865 the Commission on Towns Sewage Disposal stated that land application was the only way to avoid river pollution and to make a profit through the higher yields by irrigated crops. Furthermore, the Sewage Utilization Acts of 1865 and 1867 prevented the construction of sewers which discharge directly into rivers and ocean. These laws in fact encouraged the adoption of land treatment by municipalities for the management of wastewater. In 1872, the first standards of effluent discharge to rivers were published in England and information was provided about land application. A second report was published in 1884 by a Royal Commission which also encouraged cities to apply land treatment practice. From that period until the beginning of the 20th century many demonstration and full scale projects were established in USA and Britain to investigate the treatment potential of land in order to prevent adverse health effects, and to protect the environment.

The Royal Commission on Sewage Disposal adopted standards for effluent discharge in 1912 that included limits of 20 mg/L for BOD and suspended solids (US EPA, 1979).
Few years later in 1914 standards for drinking water quality were suggested in USA which formed the basis for the national standards that adopted in 1974. The first standards for effluent reuse were adopted in 1918 by the California State Board of Health (1918) and they have been continually revised until today to cover new uses and to meet increasing environmental requirements. In 1989 WHO published guidelines for the safe use of wastewater in agriculture (WHO, 2006).

Despite the increasing use of land treatment systems for wastewater management Regulations/guidelines that govern the operation of all types of these systems have not set yet. It is somewhat surprising since it has been shown by several studies that effluent application to the land may result in significant ecological and public health risks (Bouwer, 2000; Paranychianakis et al., 2006). It can be considered however that limits of pollutants and pathogens in wastewater effluent which are applied to the soil must not exceed the limits suggested by the existing regulations or guidelines for effluent reuse. Principal factors that affect the requirements of preapplication treatment are:

a) the degree of public access to the site;
b) the degree of process control the application area;
c) the end-use of the irrigated crop; and
d) the treatment object (e.g. removal of organic carbon, nitrogen, or pathogens)

Thus, primary treatment should be acceptable for isolated sites with restricted public access when irrigated crops are not intended for direct human consumption or when effluent application is implemented by subsurface techniques and the underground part of the irrigated crops is not consumed raw. Biological treatment using lagoons or other processes, and strict control of pathogens should be practised in locations with public access or for crops to be eaten raw.

11.2.2 Conclusions

Land application of wastewater known as “sewage farming” has been practiced for Centuries as a mean to manage wastewater, to control pollution and to eliminate risks for Public health. Information provided over the years suggest that land application systems, with prudent management, can be compatible with the current high public health and Environmental standards adopted by environmental agencies and international organizations and with the sustainable use of land. Land-based wastewater treatment systems appear to be an ideal practice, particularly in arid or semi-arid regions where effluents can be used efficiently for increasing irrigated areas. The use of land-based systems to treat municipal and other types of wastewater is expected to further expand in the future. This expanded use is in response to the high construction and maintenance costs of Complex tertiary treatment processes, as well as the need to eliminate disposal of effluents into streams and lakes.

11.3 Slow Sand Filters

Slow sand filters comprise shallow, under drained beds containing up to 0.4m of selected sand. Effluent is applied gently to the filter surface, and percolates through to the under drains whence it is collected.

A rate of application of about 120L/m^2/hr is likely to achieve a reduction in suspended solids of about 60%, and a corresponding reduction in BOD of about 40%.

11.4 Maturation Ponds

These units, which are particularly suitable for treating effluents from activated sludge processes, are discussed in chapter 8 of this manual.

11.5 The Treatment and Disposal of Sludge from a Conventional Works

Sewage sludge is of three types:-

1) Primary sludge from primary sedimentation tanks.
2) Secondary sludge, from secondary treatment units
3) Chemical sludge, from coagulation or precipitation tanks (these are likely to be very unusual in Kenya).

Scum from primary sedimentation tanks is usually disposed of mixed with the primary sludge. As a general rule for Kenya, it is recommended that secondary sludge are returned to the inlet to primary sedimentation tanks so that the primary and secondary sludge are settled and dealt with together; secondary sludge usually have very high moisture contents and, by utilizing this technique, a relatively dense mixed sludge is obtained. In this country, sewage sludge is all potentially dangerous to health, because of the likely pressure of pathogens, and therefore great care must be exercised during their disposal. Secondary and chemical sludge are less dangerous than primary.

Disposing of sludge which is produced during conventional sewerage treatment is a major problem. Usual sludge disposal methods are summarized on Figure 7.2; as shown; pre-treatment of the sludge is often required.

11.5.1 Sewage Sludge Quantities

Where secondary treatment is by percolating filters, the total weight of sludge produced at conventional treatment works is likely to be of the order of 0.17 lbs (75 grammes) measured as dry solids per person per day; this figure excludes the contribution made by industry and commerce.

Secondary treatment using activated sludge increases this daily per cent production to about 0.21 lbs (95 grammes), again based on dry solids.

In themselves, these figures do not demonstrate the extent of sludge disposal problem. The difficulties arise because of the high moisture contents of sludge, as shown in table 11.4, which is based upon the daily sludge contribution of 1000 persons:

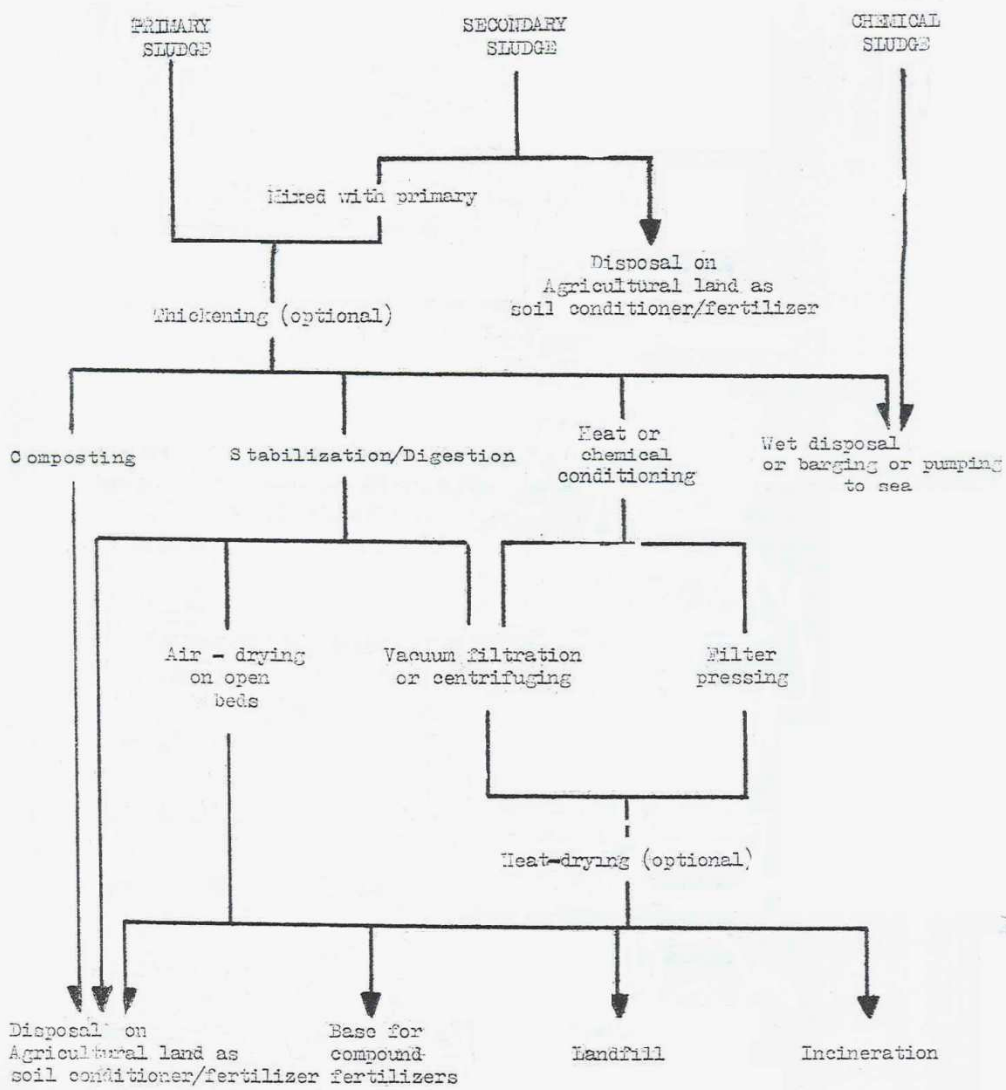


Figure 11.2 The Treatment and Disposal of Sewage Sludges

Table 11.4 Daily Sludge Contribution of 1000 persons

Type of Sludge	Weight of Dry Solids	Moisture Content (per cent)	Wet Sludge	
			Weight (Kilogrammes)	Volume (litres)
Mixed primary & secondary (from percolating filters)	75000	96	1880	1880
Mixed Primary & Secondary (from activated	95000	97	3140	3140

Sludge resulting from chemical coagulation is much more voluminous.

11.5.2 Simple Disposal, without pre-Treatment

Sewage sludge rapidly decomposes, and are consequently offensive; as previously explained, they are also dangerous. Therefore, any method for disposal of raw (that is untreated) sludge should quickly take it out of the reach, sight and smell of humans, and make it inaccessible to disease vectors.
There are two such methods:-

i) Disposal into the deep sea, which is of course only appropriate for relatively few Kenyan countries on seashore or conveniently located on creeks; either barges or long submarine pipelines may be used.

ii) Disposal by burying; the wet sludge is dumped into a hole in the ground and immediately covered by at least 2 feet (0.6 meters) of soil, to avoid nuisance and health hazard and also to prevent exposure by borrowing animals. Care must be taken that the drainage from the sludge cannot contaminate underground sources of water supplies.

Often, in other countries, wet secondary sludge, which have a relatively high content of nitrogen, are spread upon grasslands; because of hazard to health, this is considered a risky procedure not suitable for Kenya.

11.5.3 Disposal Proceeded by Treatment

Sewage sludge has some agricultural value, provided that it has not been contaminated by poisonous industrial chemicals.
The raw sludge daily by treating the sewage from 1000 persons may be expected to contain:-

66 lbs (30 kilogrammes) of carbon,
1.8 lbs (0.8 kilogrammes) of nitrogen,
0.6 lbs (0.28 kilogrammes) of phosphorous (as P),
0.36 lbs (0.16 kilogrammes) of potash (as K)

Rather than as a fertilizer, sewage sludge is probably more valuable to agriculturalists as a soil conditioner which, by virtue of its content of undecomposed organics, can improve both the structure and water retaining characteristics of soil.

In addition to pathogens, sewage sludge also contains oils and soaps which can make fertile soil “sick”. Therefore sludge should be treated, not only to reduce the health hazard but also to “condition” it, before applying to agricultural land. Treatment also reduces the bulk of the sludge, thus simplifying handling and transport.

Sludge treatment is expensive and, if it is decided to utilize sewage sludge as an agricultural aid, the decision should always be economically justified; it may prove difficult to do this in Kenya at present time. Also, at the very least, there should be guaranteed market with a demand for sludge not appreciably vary seasonally.

From the point of view of the community, a good arrangement would be to sell the treated sludge to a factory to be used as a base for compound fertilizers; such dry fertilizers are usually specific for a particular crop and are made by adding calculated amounts of inorganic nutrients and trace elements to an organic base.

Treated sludge may be used as landfill, without being deeply buried; it may be convenient to use it at the local tip to cover the community’s solid wastes, or refuse. However, the treatment of sewage solely to make it suitable for dumping on the ground surface is likely to prove very uneconomical.

11.5.4 Incineration

Partially-dried sewage may be burned in a purpose-made incinerator; a small amount of innocuous non-combustible ash afterwards remains for disposal.

An incinerator is complicated and expensive, both in capital and operating costs. Its economical justification would be very difficult to demonstrate in Kenya, unless incineration proves to be the optimum solution for the disposal of refuse from particular, exceptionally large, community; in this case, sewage sludge could be burned along with reuse, at little additional cost.

11.5.5 Composting

Aerobic fermentation, or composting, can convert sewage sludge into an inoffensive and safe material, suitable for agricultural of nitrogen for efficient composting and it is therefore better to mix it with carbonaceous materials such as refuse, or straw, or green vegetation harvested from a maturation pond; these additions will, of course, increase the quantity of compost produced.

Composting may be carried out mechanically or manually, the later system being particularly simple and cheap. The climate of Kenya is well suited for composting and, provided that there is a market for the potentially valuable compost, this process would be very suitable for Kenya.

11.5.6 Other Methods of Sludge Treatment

Sludge can be treated in order to:-

i) Thicken (that is to reduce the bulk of) the sludge, as a preliminary to further treatment or in order to reduce transport and handling costs.
ii) Remove unpleasant or dangerous characteristics of the sludge
iii) To de-water sludge; it is rarely necessary for de-watering to be absolute; for example, a sludge with a moisture content as high as 70 per cent can be lifted with a fork, and may be transported in an open wagon rather than by tanker.

Before describing the techniques of sludge treatment, the point that sludge treatment should always be economically justified is repeated, for emphasis.

11.5.7 Thickening

This is normally carried out in circular tanks which have submerged, slowly-rotating stirrers; these may be of the "picket fence" type or may encourage thickening by bubbling air through the sludge, After stirring, the sludge is left under quiescent conditions and the liquids which eventually rise to the sludge surface are decanted. Thickening reduces the size of sludge by half.

Thickening tanks tend to be very foul units and are generally not considered necessary or appropriate for Kenya, except possibly as pre-treatment where mechanical de-watering is used.

11.5.8 Anaerobic Digestion

During this process, sewage sludge is allowed to putrefy (that is to decompose anaerobically) under controlled conditions.

Digestion produces a much less offensive sludge, but one which still carried some health risk. During treatment, possibly 50 per cent of the organic matter in the sludge is destroyed. Decanting of water which collects in layers, if the sludge is left under quiescent conditions when digestion is complete, can result in the sludge volume decreasing to one quarter of that of the raw sludge.

Anaerobic digestion can be inhibited by substances often found in sewage, including synthetic detergents; certain factory wastes produce similar results.

In temperate countries, sludge digestion tanks are normally heated. Kenya's climate makes heating unnecessary and it is therefore not recommended.

A by-product of the digestion process is sludge gas, with the approximate analysis methane 64 per cent, carbon dioxide 33 per cent, nitrogen 2.5 per cent an hydrogen 0.5 per cent; sludge gas can be used for cooking, or for operating gas engines or as a base from complex organic compounds can be synthesized.

The collection and utilization of sludge gas brings with it many problems and it is strongly suggested that, unless the sewage treatment works is located close to a large industrial area where there is a definite market for sludge gas, digestion tanks should be left open so that the gas wastes to atmosphere; in the latter case, careful sitting of the digesters to ensure that the prevailing wind does not carry the foul digestion gases into the town or centre becomes important.

Digestion tanks are normally large concrete tanks, with hopper bottoms. Both primary and secondary tanks should be provided. Digestion takes place in the primary tanks, which should incorporate some method of stirring the sludge to achieve through mixing and thereby speed the process, and also to prevent the accumulation of thick scum on the sludge surface. The purpose of secondary digestion tanks is provided quiescent conditions, to assist dewatering, and also to provide storage of digested sludge during periods when, perhaps because of wet weather, it cannot be further dried, or distributed.

The design of primary digestion tanks may be based upon table 11.5; the volumes given apply to each 2.2 lbs (viz each kilogram) of suspended solids contained in the sewage inflow to the treatment works:-

Table 11.5 Suspended Solids for design of Primary Digestion Tanks

Type of Sludge	Required Capacity of Primary Digestion Tanks	
	(Cubic Feet)	Cubic Meters)
Mixed primary and secondary (from percolating filters)	75	2.1
Mixed primary and secondary (from activated sludge units)	90	2.5

The values given in this table are approximately equivalent to 6 cubic meters (0.17 cubic meters) per head of contributing pollution and 7 cubic feet (0.2 cubic meters) per head, respectively. Secondary digestion tanks should have at least 50 per cent of the capacity of primary tanks and possibly much more, depending upon storage requirements.

Digested sludge may be dumped in its wet state without causing undue offence, although it is still a potential health hazard; therefore, if it is applied to agricultural land, it should be used only on crops which are not eaten raw. It may used directly as landfill, or dumped on the community refuse tip.

Alternatively, provide that the local climate is favorable, digested sludge may be de-watered on open drying beds. Digestion of sludge as pre-treatment to mechanical de-watering is rarely warranted.

11.5.9 De-Watering on Open Drying Beds

The effects of de-watering sludge on drying beds, or indeed by any other method is to convert a wet sludge into relatively dry material, very much reduced in both weight and volume; also, drier sludge is usually less pleasant and less hazardous to health.

The relationship between the weight and volume of sludge and its moisture content is shown in table 11.6:-

Table 11.6 Relationship between the weight and volume of sludge and its moisture content

Weight of dry solids in the sludge (grammes)	Moisture Content (per cent)	Weight of wet sludge (grammes)	Volume of wet sludge (litres)
1000	97	33300	33.3
1000	95	20000	20
1000	90	10000	10
1000	70	3300	3.3
1000	50	2000	2

Digested sludge but not raw sludge which is too offensive, may be de-watered on open drying beds. In such units, de-watering is achieved by a combination of drainage and evaporation and thus this method of treatment is unsuitable where there is daily rainfall or elsewhere during a prolonged rainy season.

Sludge drying beds are shallow, under drained tanks, containing a layer of media of order of 12 inches (0.3 maters) thick; this media is normally graded, with coarse materials on the bottom and finer media such as sand above.

During use, wet sludge is run into each bed to form a layer about 15 inches (0.4 meters) deep; this is allowed to stand until it has dried sufficiently to be removed by fork (that is when moisture content has dropped to about 70 per cent). During dry weather, this will normally take about one week.

Sludge drying beds have disadvantages; they occupy large areas of land and sludge lifting is always a problem; mechanized lifting equipment is available, but is very expensive and usual method, manual lifting, depends upon possibly unreliable labour. Open sludge drying beds should be sized as shown in table 11.7:-

Table 11.7 Sizing of Sludge Drying Beds

Type of Digested Sludge	Loading of open sludge drying beds measured as dry solids per day	
	Lbs per square foot	Kilogrammes per square metre
Mixed primary and secondary (from percolating filters)	35	170
Mixed primary and secondary (from activated sludge units)	30	146

11.5.10 Mechanical De-Watering

Raw sewage sludge, after "conditioning" by heat or more usually by the addition of chemicals, may be de-watered by mechanical means; such are complex and expensive, both in capital and operating costs, but they take up little space and can operate under all weather conditions.

During heat conditioning, sludge is heated under pressure to temperatures of the order of 2000 Centigrade. The plant required is complicated, and is prone to blockage; also during the process very foul liquor, virtually untreatable by conventional sewage treatment methods, is produced. This method of sludge conditioning is not considered suitable for Kenya.

Chemical condition involves the mixing of wet sludge with suitable chemicals, in special mixing tanks which resemble sludge thickeners. Chemicals normally used include aluminum chlorohydrate, lime plus iron salts and polyelectrolytes (which are organic polymers).

The more usual methods of mechanical de-watering are centrifuging, or on vacuum filters or in filters or in filter presses; although mechanical methods of de-watering sludge are not generally recommended for use in Kenya, if it is decided to use them, then filter pressing is considered to be to be the most suitable process.

A filter press consists of a number of recessed parallel cast iron plates suspended vertically from an overhead steel bean, each plate being separated from its neighbors by two filter cloths. The process is intermittent, rather than continuous.

At the start of processing cycle, the plates are clamped together by a screw and wet sludge is introduced under pressure between each pair of filter cloths. The cloths are forced back into the recesses of the cast iron plates and chambers are formed. The feed pump, which provides the necessary pressure, continues to pump sludge into the chambers with the result that liquid in sludge is strained through the filter cloths and leaves the press along drainage grooves in the cast iron plates.

The process continues until the chambers will accept no more sludge; the feed pump is then stopped, the screw clamp is released and , as the cast iron plates are pulled apart, the filter "cakes" drop through the floor into a waiting container.

During the process, the moisture content of the sludge is reduced to about 65 per cent. If the “Cakes” are stacked in well-ventilated heaps, protected from heavy rain, further drying will take place due to the exothenic biological activity of aerobic bacteria within the sludge; the final result is powdery, apparently-dry sludge virtually free from any health risk.

11.5.11 Heat Drying

Semi-dried sludge can be further de-watered by applying external heat, usually in flash-drying or similar units. Heat drying is a very expensive process, unless it can be subsidized by using flue gases from boilers or incinerators. Heat dried sludge is effectively sterile and can be crushed, bagged and spread on to agricultural land in the form of granules or powder. It is considered most unlikely that heat drying of sludge will prove economical in Kenya.

11.5.12 Sludge Liquors

Unless sludge loses its moisture by evaporation, as it does during composting or heat drying, polluting drainage liquor is produced. This liquor should receive treatment as if it were sewage and, when estimating the costs of nay sludge treatment process which has such a by-product, costs increasing the capacity of sewage product units to deal with liquor should be taken into account.

12.0 WASTE STABILISATION PONDS

12.1 Introduction

Waste Stabilization Ponds (WSPs) are large, shallow basins in which raw sewage is treated entirely by natural processes involving both algae and bacteria. They are used for sewage treatment in temperate and tropical climates, and represent one of the most cost-effective, reliable and easily-operated methods for treating domestic and industrial wastewater. Waste stabilization ponds are very effective in the removal of faecal coliform bacteria. Sunlight energy is the only requirement for its operation. Further, it requires minimum supervision for daily operation, by simply cleaning the outlets and inlet works. The temperature and duration of sunlight in tropical countries offer an excellent opportunity for high efficiency and satisfactory performance for this type of water-cleaning system. They are well-suited for low-income tropical countries where conventional wastewater treatment cannot be achieved due to the lack of a reliable energy source. Further, the advantage of these systems, in terms of removal of pathogens, is one of the most important reasons for its use.

Waste Stabilization Ponds (WSP) or lagoons, are holding basins used for secondary wastewater (sewage effluents) treatment where decomposition of organic matter is processed naturally, i.e. biologically. The activity in the WSP is a complex symbiosis of bacteria and algae, which stabilizes the waste and reduces pathogens. The result of this biological process is to convert the organic content of the effluent to more stable and less offensive forms. WSP are used to treat a variety of wastewaters, from domestic wastewater to complex industrial waters, and they function under a wide range of weather conditions, i.e. tropical to arctic. They can be used alone or in combination with treatment processes.

A WSP is a relatively shallow body of wastewater contained in an earthen man-made basin into which wastewater flows and from which, after certain retention time (time which takes the effluent to flow from the inlet to the outlet) a well-treated effluent is discharged. Many characteristics make WSP substantially different from other wastewater treatment. This includes design, construction and operation simplicity, cost effectiveness, low maintenance requirements, low energy requirements, easily adaptive for upgrading and high efficiency.Photo 12.1on page 267 has a bird's view of WSP System.

12.1.1 Usage

Waste Stabilization Ponds (WSP) is now regarded as the method of first choice for the treatment of wastewater in many parts of the world. In Europe, for example, WSP are very widely used for small rural communities (approximately up to 2000 population but larger systems exist in Mediterranean France and also in Spain and Portugal) (Boutin et al., 1987; Bucksteeg, 1987). In the United States one third of all wastewater treatment plants are WSP, usually serving populations up to 5000 (EPA, 1983). However in warmer climates (the Middle East, Africa, Asia and Latin America) ponds are commonly used for large populations (up to around 1 million). In developing countries and especially in the tropical and equatorial regions sewage treatment by WSPs has been considered an ideal way of using natural processes to improve sewage effluents.

In Kenya, Waste Stabilization Ponds are used in 25 out of 38 existing sewage treatment works. These generally operate problem free with the exception that most of the older ponds are filling with sludge and vegetation, which generally reduces retention times and thus treatment efficiency.

The remaining treatment works, use conventional processes such as biological attached growth filters, oxidation ditches araerated lagoons etc.

12.1.2 Advantages of waste stabilisation ponds

Waste stabilisation ponds (WSP) are shallow man-made basins into which wastewater flows and from which, after a retention time of many days (rather than several hours in conventional treatment processes), a well treated effluent is discharged. WSP systems comprise a series of ponds - anaerobic, faculties and several maturation. The advantages of WSP systems, which can be summarized as simplicity, low cost and high efficiency, are as follows:

Simplicity

WSP are simple to construct: earthmoving is the principal activity, other civil works are minimal - preliminary treatment, inlets and outlets, pond embankment protection and, if necessary and pond lining. They are also simple to operate and maintain: routine tasks comprise cutting the embankment grass, removing scum and any floating vegetation from the pond surface, keeping the inlets and outlets clear, and repairing any damage to the embankments. Only unskilled, but carefully supervised, labour is needed for pond O & M.

Low cost

Because of their simplicity, WSP are much cheaper than other wastewater treatment processes. There is no need for expensive, imported electromechanical equipment (with its attendant problems of foreign exchange and spare parts), nor for a high annual consumption of electrical energy. The latter point is well illustrated by the data in table 8.1 from the United States (where one third of all wastewater treatment plants are WSP systems) for a flow of 1 million US gallons per day (3780m^3/d) (*Middlebrooks et al., 1982*)

Table 12.1 Energy Consumption Data from USA

Treatment process	***Energy Consumption (kWh/yr)***
Activated sludge	1,000,000
Aerated lagoons	800,000
Biodiscs	120,000
Waste stabilization ponds	Nil

The cost of advantages of WSP are analysed and compares four treatment processes - trickling filters, aerated lagoons, oxidation ditches and WSP, all designed to produce the same quality of final effluent. Summary details are given below:-

<u>Wastewater Treatment Costs</u>

A World Bank report Authur (1983) gives a detailed economic comparison of waste stabilisation ponds, aerated lagoons, oxidation ditches and biological filters. The data for this cost comparison were taken from the city of Sana'a in the Yemen Arab Republic, but are equally applicable in principle to countries in the region. Certain assumptions were made, for example the use of maturation ponds to follow the aerated lagoon, and the chlorination of the oxidation ditch and biological filter effluents, in order that the four processes would have an affluent of similar bacteriological quality so that fish farming and effluent reuse for irrigation were feasible. The design is based on a population of 250,000; a per caput flow and BOD contribution of 120 litres/day and 40 g/d respectively; influent and required effluent faecal coliform concentrations of 2 x10 and 1 x 10 per 100 ml, respectively; and a required effluent BOD, of 25 mg/litre. The calculated land area requirements and total net present cost of each system (assuming an opporutinity cost of capital of 12 per cent and land values of US$ 5/m) are shown in the Table 12.2 below. Waste stabilisation ponds are clearly the cheapest option.

The cost of chlorination accounts for US$0.22 million per year of the operational costs of the last two options.

Clearly the preferred solution is very sensitive to the price of land, and the above cost of US$5 per m represents a reasonable value of low-cost housing estates in developing countries.

If the cost of land is allowed to vary, then the net present cost of each process varies as shown in Gigre A, for a discount rate (opportunity cost of capital of 12 percent. Ponds are the cheapest option up to a land cost of US$7.8 per m, above which oxidation ditches become the cheapest. In fact, for discount rates between 5 and 15 percent, the choice is always between WSP and oxidation ditches: the other two processes are always more expensive. The variation with discount rate of the land cost below which WSP are cheapest varies between US$ 5 and 15 per m (US$ 50,000 and 150,000 per ha).

Table 12.2 below has the details.

Table 12.2 Comparison of Costs of Various Technological Innovative

Description	WSP System	Aerated Lagoon System	Oxidation Ditch system	Conventional Treatment (Biofilters)
Costs (million US $)				
Capital	5.68	6.98	4.80	7.77
Operational	0.21	1.28	1.49	0.86
Benefits (million US $				
Irrigation Income	0.43	0.43	0.43	0.43
Pisciculture income	0.30	0.30	-	-
Net Present cost (million US $)	5.16	7.53	5.86	8.20
Land Area (ha)	46	50	20	25

The most important conclusion is that WSP systems are the cheapest treatment process at land costs of US$ 50,000-150,000 (1983 $) per hectare, depending on the discount rate (opportunity cost of capital; range 5-15 percent). These figures are much higher than most land costs in the region, and so land costs are unlikely to be a factor operating against the selection of WSP for wastewater treatment (but land availability may be).

The above economic methodology strongly recommends for use at the feasibility stage of all wastewater treatment projects in which a choice between different treatment projects has to be made. This should include, if necessary, the extra cost of conveying the wastewater to an area of low-cost land.

The Dandora Phase II WSP at Nairobi, Kenya (6 series, each comprising a primary facultative pond (367,500 m3) and 3 equal maturation ponds (@54,000m3) had a capital civil works cost of US$ 25 millions, plus US$ 4.5 millions for electrical and mechanical works and US$1.8 millions for 55 houses for the resident work force. The civil works cost is equivalent to nearly US$ 8 per m^3 of pond volume (1992 rate).

High efficiency

BOD removals >90 percent are readily obtained in a series of well-designed ponds. The removal of suspended solids is less, due to the presence of algae in the final effluent (but, since algae are very different to the suspended solids in conventional secondary effluents, this is not cause for alarm. Total nitrogen removal is 70-90 percent and total phosphorus removal 30-45 percent.

WSP are particularly efficient in removing excreted pathogens, whereas in contrast all other treatment processes are very inefficient in this and require a tertiary treatment process, such as chlorination (with all its inherent operational and environmental problems), to achieve the destruction of faecal bacteria.

Activated sludge plants may, if operating very well, achieve a 99 percent removal of faecal coliform bacteria: this might, at first inspection, appear very impressive, but in fact it only represents a reduction from 102 per 100 ml to 102 per 100 m. (that is, almost nothing). A properly designed series of WSP, on the other hand, can easily reduce faecal coliform numbers from 102 per 100 ml to <103 per 100ml (the WHO guideline value for unrestricted irrigation ;), which is a removal of 99.999 percent for 5 log units).

A general comparison between WSP and conventional treatment processes for the removal of excreted pathogens is shown in Table 12.3; detailed information is given in Feachem et al. (1983).

Table 12.3 Removals of excreted pathogens achieved by waste stabilization ponds and conventional treatment processes

Excreted Pathogen	Removal in WSP	Removal in Conventional Treatment
Bacteria	Up to 6 log units a/	1-2 log units
Viruses	Up to 4 log units	1-2 log units
Protozoan cysts	100%	90-99%
Helminth eggs	100%	90-99%

A/ 1 log unit =90% removal; 2=99%; 3=99.9% and so on.

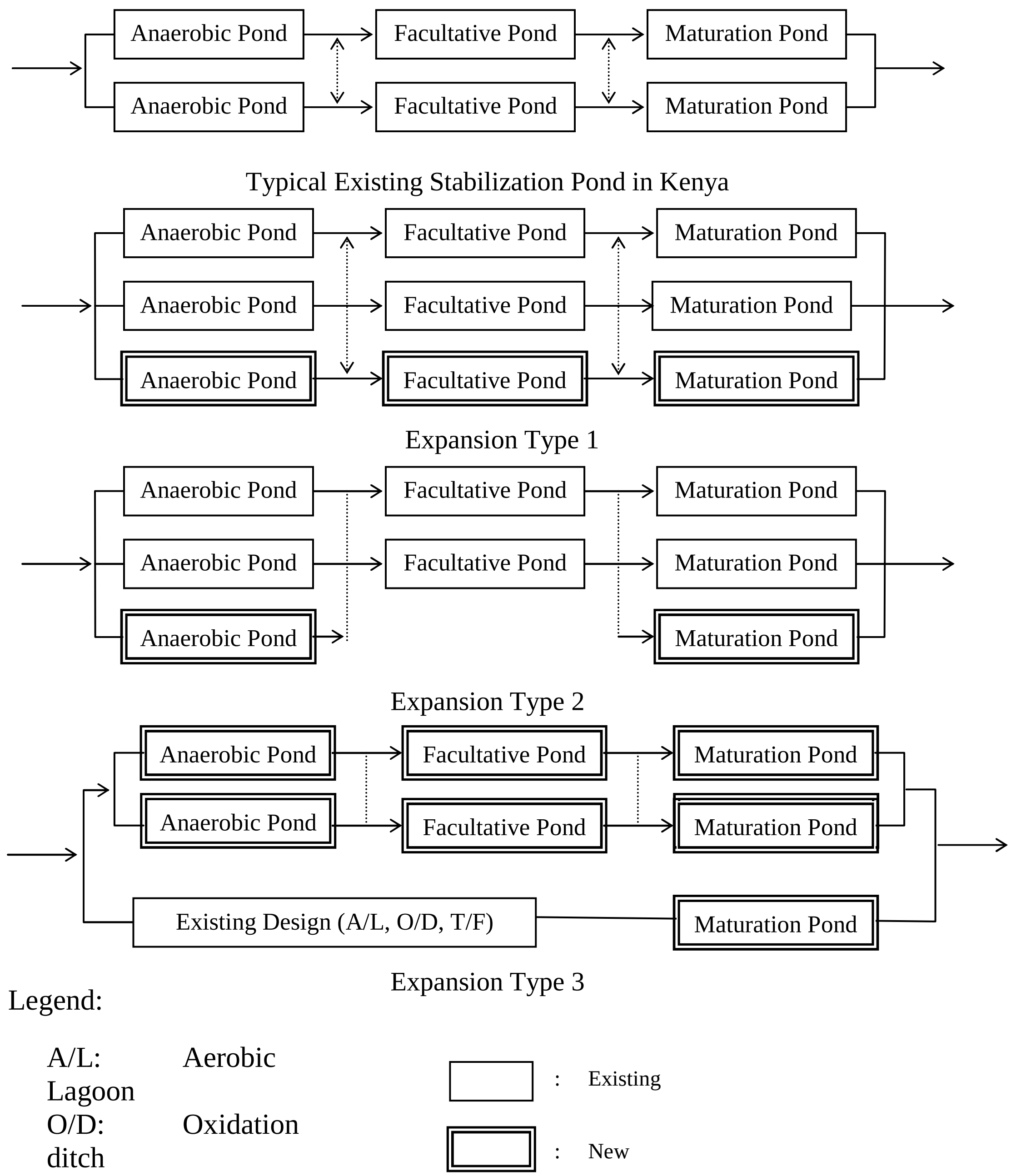
WSP are also extremely robust due to their long hydraulic retention time, they can withstand both organic and hydraulic shock loads. They can also cope with high levels of heavy metals, up to 60 mg/l (Moshe et al., 1972), so they can treat a wide variety of industrial wastewater that would be too toxic for other treatment processes. Strong wastewaters from agro-industrial processes (for example, abattoirs, food canneries, dairies) are easily treated in WSP. Finally, WSP are the only secondary treatment process that can readily and reliably produce effluents safe for reuse in agriculture and aquaculture.

The **principal requirements** for **WSP** are that **sufficient land is available** and that the **soil** should preferably have a coefficient of **permeability** less than **10^{-7} m/s (to avoid the need for pond lining)**. The investment made by the sewerage authority in land for ponds can always be realized later. For example, the city of Concorde in California purchased land for ponds in 1955 at US$ 375,000 per ha (Oswald, 1976). Inflation during this 20 year period was exactly 100 percent, so the land increased in real value by 375 percent (or 6.8 percent per year).

12.1.3 Layouts

There are four different Typical Layouts. Figure 12.1 has the detailed.

Figure 12.1 Typical Layout of WSP



12.2 Wastewater Treatment in WSP

12.2.1 Types of WSP and their function

WSP systems comprise a single series of anaerobic, facultative and several maturation ponds, or several such series in parallel. In essence, anaerobic and facultative ponds are designed for BOD removal and maturation ponds for pathogen removal, although some BOD removal occurs in maturation ponds and some pathogen removal in anaerobic and facultative ponds. The final effluent quality depends largely on the size and number of maturation ponds.

Designers should not be afraid of including anaerobic ponds. Their principal perceived disadvantage - odour released can be eliminated at the design stage), and they are so efficient at removing BOD that their inclusion substantively reduce the land area required (see design example)

There are a few special pond types - septage and night soil ponds, partially aerated facultative ponds, effluent storage reservoirs, microphyte ponds and high rate algal ponds and these are discussed in subsequent chapters. The last two types in particular are more complicate, expensive and more problematic than anaerobic, facultative and maturation ponds, recommended, although there may occasionally be special local circumstances which justify their use.

12.2.1.1 Anaerobic ponds

Anaerobic ponds are 2 to.5 m deep and receive such a high organic loading (usually >100 g BOD/m^3 d, equivalent to > 3000 kg/ha d for a depth 3 m) that they contain no dissolved oxygen and no algae (occasionally a thin film of mainly *Chlamydomonas* can be seen at the surface). They function much like open septic tanks, and their primary function is BOD removal. They work extremely well in warm climates; a properly designed and not significantly underlaoded anaerobic pond will achieve around 60 percent BOD removal at 20°C and as much as 75 percent at 25 °C. Retention times are short.

The process of anaerobic digestion is more intense at temperatures above 15^o C. The anaerobic bacteria are usually sensitive to pH <6.2. Thus, acidic wastewater must be neutralized prior to its treatment in anaerobic ponds. A shorter retention time of 1.0 - 1.5 days is commonly used.

Designers have in the past been too afraid to incorporate anaerobic ponds in case they cause odour. Hydrogen sulphide, formed mainly by the anaerobic reduction of sulpahte by sulphate-reducing bacteria such as Desulfovibrio, is the principa potential source of odour. However in aqueours soloution hydrogen sulphide is present as either dissolved hydrogen sulphide gas (HS) or the bisulphide ion (HS), with the sulphide ion (S)only really being formed in significant quantities at high pH. Figure 12.1 shows how the distribution of HS, HS and S^2changes withpH. At pH values normally found in well designed anaerobic ponds (around 7.5), most of the sulphide is present as the odourless bisulphide ion. Odour is only caused by escaping hydrogen sulphide molecules as they seek to achieve a partial pressure in the air above the pond which is in equilibrium with their concentration in it (Henry's law). Thus for any given total sulphide concentration, the greater the proportion of sulphide present as HS, the lower the release of H_2S. Odour is not a problem if the recommended design loadings are not exceeded and if the sulphate concentration in the raw wastewater is less than 500 mg SO/l (Gloyna andEspino, 1969). A small amount of sulphide is beneficial as it reacts with heavy metals to form insluble metal sulphides which form insoluble metal sulphides which precipitate out, but concentrations of 50 -150 mg/l can inhibit methanogenesis (Pfgfffer, 1970).

12.2.1.2Facultative ponds

Facultative ponds (1-2 m deep) are of two types: primary facultative ponds which receive raw wastewater, and secondary facultative ponds which receive settled wastewater (usually from anaerobic ponds, septic tanks, primary facultative ponds, and shallow sewerage systems). They are designed for BOD removal on the basis of a relatively low surface loading (100-400 kg BOD/ha,d) to permit the development of a healthy algal population as the oxygen for BOD removal by the pond bacteria is mostly generated by algal photosynthesis. Due to the algae, facultative ponds are coloured dark green, although they may occasionally appear red or pink (especially when slightly overloaded) due to the presence of anaerobic purple sulphide oxidising photosynthetic bacteria. The algae that tend to predominate in the torcid waters of facultative ponds (see Table 12.4 and figure 12.2) are the motile genera (such as *Chlamydomonas*, *Pyrobortys* and *Euglena*) as these can optimize their position in the pond in relation to incident light intentisty and temperatute more easily than non-motile forms (such as *Chlorella*, although this is fairly common in facultative ponds). The concentration of algae in a healthy facultative pond depends on loading and temperature but is usually in the range 500-2000 mg chlorophyll a per litre.

Table 12.4 Examples of algal genera present in facultative and maturation ponds

Algal genius	Facultative Ponds	Maturation Ponds
Euglenophyta		
❖ Euglena	+	+
❖ Phacus	+	+
Chlorophyta		
o Chlamydomonas	+	+
o Chlorogonium	+	+
o Eudorina	+	+
o Pandorina	+	+
o Pyrobotrys	+	+
o Ankistrodesmus	_	+
o Chlorella	+	+
o Micractinium	_	+
o Scenedesmus	_	+
o Selenastrum	_	+
o Carrteria	+	+
o Coelastrum	_	+
o Dictyosphaerium	_	+
o Oocystis	_	+
o Rhodomonas	_	+
o Volvox	+	_
Chrysophyta		
• Navicula	+	+
• Cyclotella	_	+
Cyanophyta		
• Ocsillatoria	+	+
• Anabaena	+	+

Note:

+ =Present -- = absent

As a result of the photosynthetic activities of the pond algae, there is a diurnal variation in the concentration of dissolved oxygen. After sunrise, the dissolved oxygen level gradually rises to a maximum in the mid-afternoon, after which it falls to a minimum during the night. The position of the oxypause (the depth at which the dissolved oxygen concentration reaches zero) similarly changes, as does the pH since at peak algal activity carbonate and bicarbonate ions react to provide more carbon dioxide for the algae, so leaving an excess of hydroxyl ions with the result that the pH can rise to above 9 which kills faecal bacteria.

The wind has an important effect on the behavior of facultative ponds, as it induces vertical mixing of the pond liquid. Good mixing ensures a more uniform distribution of BOD, dissolved oxygen, bacteria and algae and hence a better degree of waste stabilization. In the absence f wind-induced mixing, the algal population tends to stratify in a narrow ban, some 20 cm thick, during daylight hours. This concentrated band of algae moves up and down through the top 50 cm of the pond in response to changes in incident light intensity, and causes large fluctuations in effluent quality (BOD and suspended solids) if the effluent take-off point is within this zone .

The processes in anaerobic and secondary facultative ponds occur simultaneously in primary facultative ponds, as shown in Figure 12.2. It is estimated that about 30% of the influent BOD leaves the primary facultative pond in the form of methane (Marais 1970). A high proportion of the BOD that does not leave the pond as methane ends up in algae. This process requires more time, more land area, and possibly 2 -3 weeks water retention time, rather than 2 -3 days in the anaerobic pond. In the secondary facultative pond (and the upper layers of primary facultative ponds), sewage BOD is converted into "Algal BOD," and has implications for effluent quality requirements. About 70 – 90% of the BOD of the final effluent from a series of well-designed WSPs is related to the algae they contain.

In secondary facultative ponds that receive particle-free sewage (anaerobic effluent), the remaining non-settleable BOD is oxidised by heterotrophic bacteria (*Pseudomonas, Flavobacterium, Archromobacter and Alcaligenes* spp). The oxygen required for oxidation of BOD is obtained from photosynthetic activity of the micro-algae that grow naturally and profusely in facultative ponds

The dissolved oxygen concentration in the water gradually rises after sunrise, in response to photosynthetic activity, to a maximum level in the mid-afternoon, after which it falls to a minimum during the night, when photosynthesis ceases and respiratory activities consume oxygen. At peak algal activity, carbonate and bicarbonate ions react to provide more carbon dioxide for the algae, leaving an excess of hydroxyl ions. As a result, the pH of the water can rise to above 9, which can kill faecal coliform. Good water mixing, which is usually facilitated by wind within the upper water layer, ensures a uniform distribution of BOD, dissolved oxygen, bacteria and algae, thereby leading to a better degree of waste stabilization.

12.2.1.3Maturation ponds

A series of maturation ponds (1-1.5 m deep) receives the effluent from a facultative pond, and the size and number of maturation ponds is governed mainly by the required bacteriological quality of the final effluent. Their primary function is to remove excreted pathogens. Although maturation ponds achieve only a small degree of BOD removal, their contribution to nutrient removal also can be significant
Maturation ponds usually show less vertical biological and physicochemical stratification and are well oxygenated throughout the day.

Their algal population is thus much more diverse than that of facultative ponds (Table 12.4) with non-motile genera tending to be more common; algal diversity increases from pond to pond along the series.

The primary function of maturation ponds is the removal of excreted pathogens, and this is extremely efficient in a properly designed series of ponds (Table 12.5). Maturation ponds achieve only a small removal of BOD, but their contribution to nutrient (nitrogen and phosphorus) removal can be significant.

Photo 12.1 Birds Eye view of WSP System

Table 12.5 Geometric mean bacterial and viral numbers a/ and percentage removals in raw wastewater and the effluents of five waste stabilization ponds in series (P1-P5) b/ at a mean mid depth pond temperature of 26^{0}c

Organism	Raw Water	P1	P2	P3	P4	P5	Percentage Removal
Faecal Coliform	2x 10^7	4x10^6	8x10^5	2x10^5	3x10^4	7x10^3	99.97
Faecal streptococci	3x10^6	9x10^5	1x10^5	1x10^4	2x10^3	300	99.99
Clostridium perfringens	51x10^4	2x10^4	6x10^3	2x10^3	1x10^3	300	99.40
Campylobacters	70	20	0.2	0	0	0	100.00
Salmonellae	20	8	0.1	0.02	0.01	0	100.00
Enteroviruses	1x10^4	6x10^3	1x10^3	400	50	9	99.91
Rotaviruses	800	200	70	30	10	3	99.63
BOD (mg/L)	215	36	41	21	21	18	92

a/ bacteria numbers per 100ml, viral numbers per 10 litres

b/ P1 was an anaerobic pond with a mean hydraulic retention time of 1day; P2 and P3-P5 were secondary facultative and maturation ponds respectively, each with a retention time of 5 days. Pond depths were 2.8-3.2m.

Source: Oragui et al. (1987)

12.3 BOD removal

In anaerobic ponds BOD removal is achieved (as in septic tanks) by sedimentation of settleable solids and subsequent anaerobic digestion in the resulting sludge layer: this is particularly intense at temperatures above 15°C when the pond surface literally bubbles with the release of biogas (around 70 percent methane and 30 percent carbon dioxide); methane production increases sevenfold for every 5 °C rise in temperature (Marais, 1970). The bacterial groups involved are the same as those in any anaerobic reactor - the anaerobic acidogens and the methanogens, and those in anaerobic ponds are equally sensitive tot he same toxicants, one of which is low pH (<6.2). Acidic wastewaters thus require neutralizing prior to treatment in anaerobic ponds.

In secondary facultative ponds that receive settled wastewater (anaerobic pond effluent "small-bore" sewerage), the remaining non-settleable BOD is oxidized by the normal heterotrophic bacteria of wastewater treatment (Pseudomonas, Flavobacterium, Archromobacter and Alcaligenes spp.), but with one important difference: these bacteria obtain the oxygen they need not from mechanical aeration (aas they do in aerated lagoons, oxidation ditches and activated sludge tanks), but from the photosynthetic activities of micro-algae which grow naturally and profusely in facultative ponds, giving them their characteristic dark green colour. The algae, in turn, depend largely on the bacteria for the carbon dioxide which they photosynthetically convert into sugars:

$$6CO_2 + 12H_2O \longrightarrow C_6H_{12}O_6 + 6H_2O + 6O_2$$

So there exists a mutualistic relationship between the pond algae and the pond bacteria: the algae provide the bacteria with oxygen and the bacteria provide the algae with carbon dioxide (Figure 12.2). Of course some oxygen and carbon dioxide comes from the atmosphere by mass transfer, but the bulk is supplied by algal-bacterial mutualism.

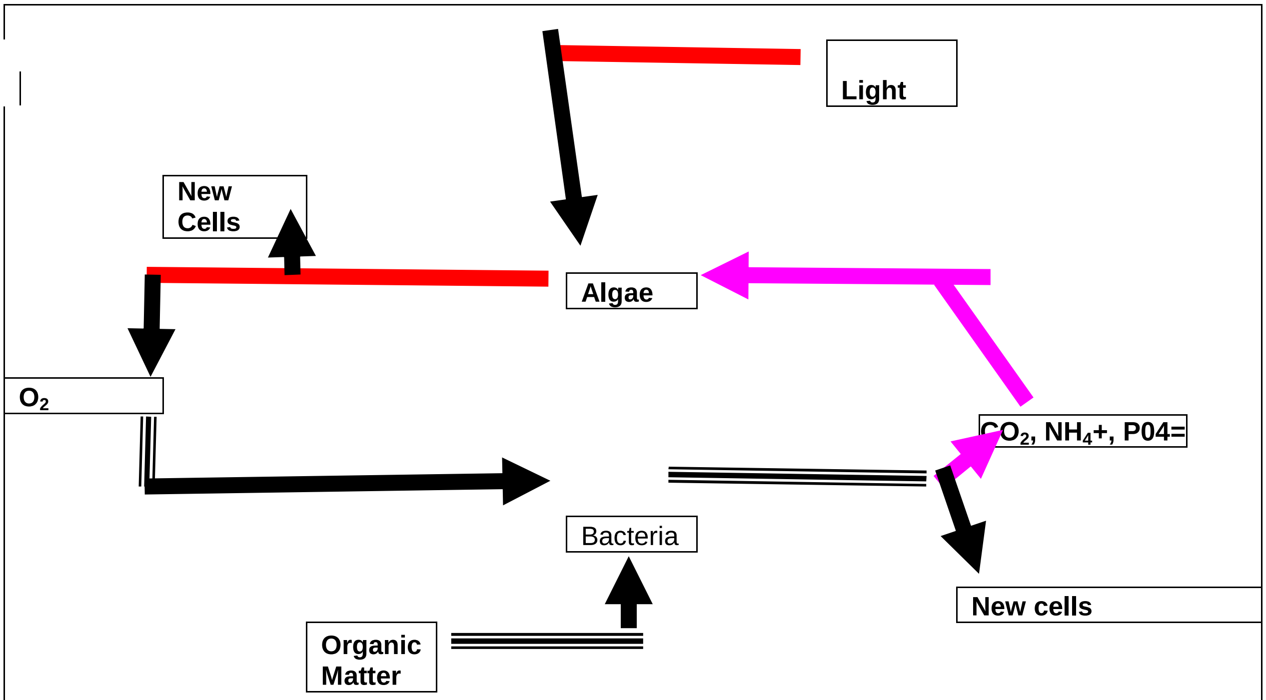


Figure 12.2 Mutualistic relationship between pond algae and pond heterotrophic bacteria

In primary facultative ponds (those that receive raw wastewater) the above functions of anaerobic and secondary facultative ponds are combined, as shown in Figure 12.2 Around 30 percent of the influent BOD leaves a primary facultative pond in the form of methane (Marais, 1970).

As a result of these algal-bacterial activities, a large proportion of the BOD that does not leave as methane ends up as algal cells. Thus in secondary facultative ponds (and in the upper layers of primary facultative ponds) "sewerage BOD" is converted into "algal BOD" and this has important implications for effluent requirements (.

In maturation ponds only a small amount of BOD removal occurs, principally as a result of lower alagal concentrations (and hence lower "algal BOD") which, in turn, result from a decreased supply of nutrients and predation by protozoa and micro-invertebrates such as Daphnia or by fish such as tilapia if these are present. Around 70-90 percent of the BOD of a maturation pond effluent is due to the algae it contains.

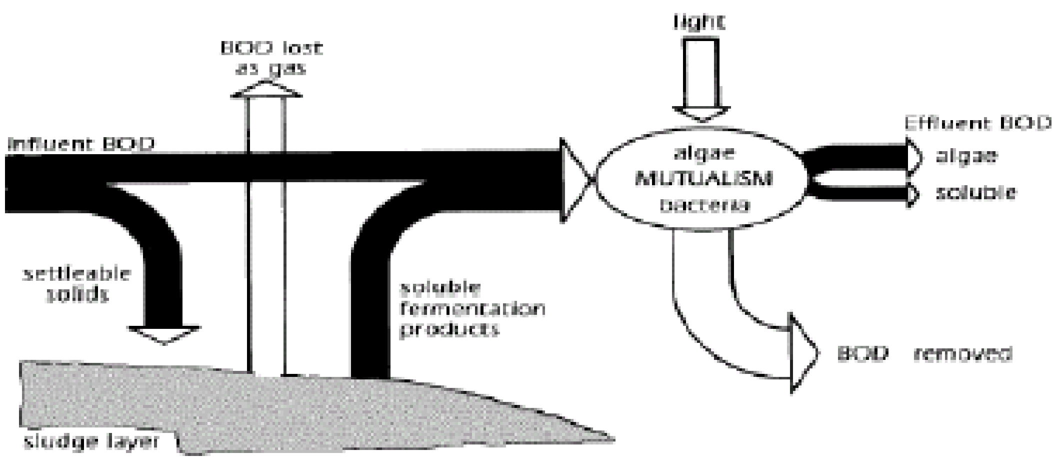


Fig. 12.3 Pathways of BOD removal in primary facultative ponds (After Marais, 1970)

12.4 Pathogen removal

12.4.1 Bacteria

Faecal bacteria are mainly removed in facultative and especially maturation ponds whose size and number determine the numbers of faecal bacteria (usually modeled in terms of faecal coliforms) in the final effluent (Section 12.2), although there is some removal of anaerobic ponds principally by sedimentation of solids-associated bacteria (see table 12.5).

The principal mechanisms for faecal bacterial removal in facultative and maturation ponds are now known to be:

(a) time and temperature
(b) high pH (> 9), and
(c) high light intensity

Time and temperature are the two principal parameters used in maturation pond design. Faecal bacterial die-off in ponds increases with both time and temperature (Feachem et al., 1983). High pH values above 9 occur in ponds due to rapid photosynthesis by the pond alga which consumes CO^2 faster than it can be replaced by bacterial respiration; as a result carbonate and bicarbonate ions dissociate:

$2HCO_2 = CO^2_3 + H_2O + CO_2$

$CO^2_3 + H_2O \quad 2OH^- + CO_2$

The resulting CO_2 is fixed by the algae and the Hydroxyl ions accumulate so raising the pH, often to above 10. Faecal bacteria (with the notable exception of Vibro cholerae) die very quickly (within minutes) at pH > 9 (Pearson et a., 1987).

The role of high light intensity has recently been elucidated (Curtis et al, 1992). Light of wavelengths 425 - 100 mm can damage faecal bacteria by being absorbent by the humic substances ubiquitous in wastewater: these then enter an excited state for long enough to damage the cell. Light -inadiated die-off is completely dependent on the presence of high pH. The sun thus plays a threefold role in promoting faecal bacterial removal in WSP: directly, by increasing the pond temperature; and more indirectly, by providing the energy for rapid algal photosynthesis which not only raises the pond pH above 9 but also results in high dissolved oxygen concentrations which are necessary for its third role, that in promoting photo-oxidative damage.

12.4.2 Viruses

Little is definitely known about the mechanisms of viral removal in WSP, but it is generally considered that it occurs by absorption on to settleable solids (including the pond algae and consequent sedimentation.

12.4.3 Parasites

Protozoan cysts and helminth eggs are removed by sedimentation. Their settling velocities are quite high (for example, 3.4 x 10^4 /s in the case of Ascaris lumbricoides), and consequently most removal takes place in the anaerobic and facultative ponds.

It has recently become possilbe to design WSP for helminth egg removal (Ayres et al, 1992) see design example no. 4) this is necessary if the effluent is to be used for restricted crop irrigation.

Grimason et al., (1993) studied the removal of Cryptosporldium oocysts and Giardia cysts in 12 Kenyan WSP systems with overall retention times in the range 15-62 days. The concentrations in the raw wastewaters were 0-73 oocysts/l and 0-6200 cysts/l; no oocysts or cysts were detected in the final effluents (except in one case, Eldoret, which had certain operational pecullarities).

12.5 Nutrient Removal

12.5.1 Nitrogen

In WSP systems the nitrogen cycle is at work, with the probable exception of nitrification and denitrification. In anaerobic ponds organic nitrogen is hydrolysed to ammonia, so its concentration in anaerobic pond effluents is generally higher than in the raw wastewater (unless the time of travel in the sewer is so long that all the area has been converted maturation ponds, ammonia is incorporated into new algal biomass. Eventually, the algae become moribund and settle to the bottom of the pond; around 20 percent of the algal cell mass is non-biodegradable and the nitrogen associated with this fraction remains immobilized in the pond sediment. That associated with the biodegradable fraction eventually diffuses back into the pond liquid and is recycled into algal cells to start the process again. At high pH, some of the ammonia will leave the pond by volatilization.

There is little evidence for nitrification (and hence denitrification, unless the wastewater is high in nitrates). The populations of nitrifying bacteria are very low in WSP due primarily to the absence of physical attachment sites in the aerobic zone, although inhibition by the pond algae may also occur.

12.5.2 Phosphorus

The efficiency of total phosphorus removal in WSP depends on how much leaves the pond water column and enters the pond sediments - this occurs due to sedimentation as organic P in the algal biomass and precipitation as inorganic P (principally as hydroxyapatite at pH levels above 9.5) - compared to the quantity that returns through mineralization and resolubilization. As with nitrogen, the phosphorus associated with the non-biodegradable fraction of the algal cells remains in the sediments. Thus the best way of increasing phosphorus removal in WSP is to increase the number of maturation ponds so that progressively more and more phosphorus becomes immobilized in the sediments. A first order plug-flow model for phosphorus removal has been developed (Huang and Gloyna, 1984) but it is not in a form useful for design. The model shows, however, that if the BOD removal is 90 percent, then phosphorus removal is around 45 percent.

12.6 Environmental Impact of WSP Systems

Adverse environmental impacts resulting from the installation of a waste stabilization pond system should normally be minimal, and the positive impacts, such as alleviation of water pollution, should greatly outweigh any potential negative impacts such as odour nuisance or mosquito breeding. However, environmental impact assessments (EIA) are now recognized with an essential component in any development project and as an important decision making goal and the appropriate procedures should be followed.

12.7 Process Design of WSP

12.7.1 Effluent quality requirements

Effluent requirements are generally expressed by regulatory agencies in terms of, for example:

- Organic matter (commonly as BOD but increasingly also as COD)
- Suspended solids
- Nitrogen (total N, ammonia, oxidized N)
- Total phosphorus
- Numbers of faecal coliform bacteria
- Numbers of human intestinal nematode eggs (Ascaris lumbricodes, Trichuris trichiura and the human hookworms)
- Numbers of human trematode eggs (Schistosoma spp.)

The last three microbiological requirements are particularly appropriate if the effluent is to be used for crop irrigation or fishpond fertilization.

In many countries in eastern Africa, especially those that were former British colonies, effluent requirements are all too frequently based on the UK Royal Commission 20/30 standard (that is, 20 mg BOD/l and 30 mg SS/l), despite the fact that such a standard is rarely (if ever) suited to local circumstances (see, for example Mara 1976). In some countries, however, an attempt has been made to promulgate more appropriate requirements which depend on the receiving water. For example, in Botswana different standards exist for discharge into perennial and ephemeral rivers (the latter not being used for potable supply):

Table 12.6 Botswana Standards for discharge into perennial and ephemeral Rivers

	Perennial	Ephemeral
BOD (mg/l)	20	30
Ammonia (mg N/l)	1.5	-
Nitrate (mg N/l)	2.0	-
Total phosphorus (mg P/l)	1.0	10
Faecal coliforms (per 100 ml)	100	500

In Zimbabwe receiving water are classified into two zones. Zone I contains 15 river catchments with sensitive receiving waters and stricter requirements; Zone II includes all other catchments. The effluent discharge requirements are:

Table 12.7 Effluent Discharge requirements in Zimbabwe

Zone I		**Zone II**
COD (mg/l)		
		30
	60	
Suspended solids (mg/l)		
10		25
Total nitrogen (mg N/l)		
10		10
Ammonia (mg N/l)		
	0.5	
0.5		
Total phosphorus (mg P/l)		
1.0		1.0

(These Zimbabwean standards are, in fact, extremely strict and they are unlikely to be appropriate elsewhere).

However, it must be remembered that 70-90 percent of the BOD of the final effluent from a series of well designed WSP is due to the algae it contains, and "algal BOD" is different in nature to "sewerage BOD". Thus many countries permit a higher BOD in WSP effluents than they do in effluents from other types of treatment plant, or they make some other allowance for WSP effluents. In the United States pond effluents can have a BOD up to 45 mg/l (EPA, 1977); in France up to 40 mg/l (but on filtered samples – see below) (Circulaire Interministerielle, 1980), and in Germany allowance is made for the algal BOD on the basis that 100 µg chlorophyll a is equivalent to 3 mg BOD (Bucksteeg, 1987), so that the requirement is relaxed as follows:

$BOD_R > BOD_e - 0.003\ (Chl)$

Where BOD_R = required BOD (mg/l)

BOD_e = actual unfiltered BOD (mg/l)

Chl = chlorophyll a concentration (µg /l)

In the European Community pond effluents have to meet the same requirement as other effluents (<25 mg/l)but with one very important difference: filtered samples are used to determine the BOD, which is therefore the residual non-algal BOD (Council of the European Communities, 1991). This recognizes the distinction between algal BOD and sewage BOD. The algae in WSP effluents readily disperse and are consumed by zooplankton in receiving waters, so they have little chance to exert their BOD, and during daylight hours they, of course, produce oxygen. In agricultural re-use schemes pond algae are very beneficial: they act as slow release fertilizers and increase the soil organic matter, so improving its water-holding capacity. It is recommended, therefore, that WSP effluents be assessed on the basis of filtered samples, together with an appropriate requirement for suspended solids (which include the algae): in Europe, for example, this is 150 mg/l (Council of the European Communities, 1991), but this may be too lax for some rivers in the Region.

12.7.2 Design Parameters

The five most important design parameters are temperature, net evaporation, flow, BOD and faecal coliform numbers. Helminth eggs are also important if the final effluent is to be used in agriculture or aquaculture.

12.7.2.1Temperature and net evaporation

The usual design temperature is the mean air temperature in the coolest month (or quarter). This provides a small margin of satiety as pond temperatures are 2-3ºC warmer than air temperatures in the cool season). Other design temperatures used are the air temperatures in the coolest period of the irrigation season and that in the coolest month of the tourist season. Net evaporation (= evaporation – rainfall) has to be taken into account in the design of facultative and maturation ponds, as these generally have a scum layer which effectively prevents significant evaporation. The net evaporation rates in the months used for selection of the design temperatures are used; additionally a hydraulic balance should be done in the hottest month.

Kenya has very good meteorological records, and temperature and evaporation data are readily available from national meteorological offices. Reference may also be made to Meteorological Office.

12.7.2.2 Flow

The mean daily flow should be measured if the wastewater exists. If it does not, it must be estimated very carefully, since the size of the ponds and hence their cost, is directly proportional to the flow. The wastewater flow should not be based on the design water consumption per caput, as this is unduly high since it contains an allowance for losses in the distribution system. A suitable design value is 85 percent of the in-house water consumption, and this can be readily determined from records of water meter readings. If these do not exist, the design wastewater flow should be based on local experience in sewered communities of similar socioeconomic status and water use practice.

12.7.2.3BOD

If the wastewater exists, its BOD may be measured using 24-hour flow-weighed composite samples. If it does not, it may be estimated from the following equation:

$$L_i = 1000\ B/q \qquad \textbf{(12.1)}$$

Where L_i = wastewater BOD, mg/l
B = BOD contribution, g/caput d
Q = wastewater flow, l/caput d

Values of B vary between 30 and 70 g per caput per day, and a suitable design value for Kenya is 40 g per caput per day.

12.7.2.4Faecal coliforms

Grab samples of the wastewater may be used to measure the faecal coliform concentration if the wastewater exists. The usual range is 10-10 faecal coliforms per 100 ml, and a suitable design value is 1 x 10 per 100 ml.

12.7.2.5Helminth eggs

If the wastewater exists, grab samples may be used to count the number of human intestinal nematodes eggs. The usual range is 100 – 1000 eggs per liter, with the latter serving as a conservative value for design.

12.7.3 Anaerobic ponds

Anaerobic ponds are designed on the basis of volumetric BOD loading (λv, g/m³d), which is given by:

$$\lambda v = L_i Q/V_a \qquad \textbf{(12.2)}$$

Where L_i = effluent BOD, mg/l (= g/m³)
Q= flow, m³/d
V_a = anaerobic pond volume, m³

The permissible design value of λv increases with temperature, but here are too few reliable data to permit the development of a suitable design equation. Nonetheless the results of the Kenyan pond study (Mara e al., 1990) indicate that the general recommendations made by Mara and Pearson (1986), which are given in Table 12.1, may be safely used for design purposes in Kenya.

These recommendations were based on those of Meiring et al. (1968) that λv should lie between 100 and 400 g/m³d, the former in order to maintain anaerobic conditions and the latter to avoid odour release. However, in Table 12.8 the upper limit for design is set at 300 g/m³d in order to provide an adequate margin of safety with respect to odour. This is appropriate for normal domestic or municipal wastewaters which contain less than 500 mg SO/l.

Table 12.8 Design values of permissible volumetric loading on and percentage BOD removal in anaerobic ponds at various temperatures

Temperature (^{0}C)	Volumetric Loading (g/m^{3}d)	BOD removal (%)
<10	100	40
10-20	20T-100	2T+20
>20	300	60a/

a/ Higher values may be used if local experience indicates that this is appropriate
T=temperature, ^{0}C
Source: Mara and Pearson (1986)

Once a value of λv has been selected, the anaerobic pond volume is then calculated from equation 8.2, the mean hydraulic retention time in the pond ($ø_a$., d) is determined from:

$ø_a = V_a/Q$ **(12.3)**

The performance of anaerobic ponds increases significantly with temperature, and the design assumptions given in Table 8.8 can be confidently adopted. At temperatures much in excess of 20ºC Table 12.8 is somewhat conservative: for example, in northern Brazil Mara et al. (1983) reported a mean annual BOD removal of 75 percent in an optimally loaded pond with a 0.8 d retention time at 25 - 27ºC. However, unless there is local experience of higher performance, the values given in Table 12.8 should be used for design.

12.7.4 Facultative ponds

Although there are several methods available for designing facultative ponds (Mara, 1976), it is recommended that they be designed on the basis of surface BOD loading (λv, kg/ha d), which is given by:

$\lambda_s = 10\ L_iQ/A_f$ **(12.4)**

Where A_f = facultative pond area, m²

The permissible design value of λ_s increases with temperature (T, ºC). The earliest relationship between λ_s and T is that given by *McGarry and Pescod* (1970), but their value of λ_s is the maximum that can be applied to a facultative pond before it fails (that is, becomes anaerobic).

Their relationship which is therefore an envelope of failure is:

$\lambda_s = 60\ (1.099)^T$ **(12.5)**

Arthur (1983) recommended the following equation for design:

$\lambda_s = 20T – 60$ **(12.6)**

However, the Kenyan pond study (mara et al., 1990) found that facultative ponds (mainly secondary facultative ponds) which were receiving loads higher than those permitted by either equation 12.5 or equation 12.6 were operating reasonably well in terms of BOD removal. Regression analysis of data from facultative ponds in Kenya ponds operating at loadings below those given by the McGarry and Pescod equation yielded the relationship:

$$\lambda_s = 26T - 160 \quad \textbf{(12.7)}$$

However since equations 12.6 and 12.7 require confirmation from elsewhere in the Region before that can be used for design, it is recommended that the global design equation given by Mara (1987), which gives values of λ_s slightly less than those given by equation 12.7 be used. This is (cf. Table 12.9):

$$\lambda_s = 350\,(1.107 - 0.002T)^{T-25} \quad \textbf{(12.8)}$$

Table 12.9 Values of the permissible BOD loading on facultative ponds at various temperatures (calculated from equation 12.8)

T (^{0}C)	λ_s(kg/ha d)	T (^{0}C)	λ_s(kg/ha d)
11	112	21	272
12	124	22	291
13	137	23	311
14	152	24	331
15	167	25	350
16	183	26	369
17	199	27	389
18	217	28	406
19	235	29	424
20	253	30	440

Once a suitable value of λ_s has been selected, the pond area is calculated from equation 12.8 and its retention time (0, d) from:

$$Ø_f = A_f\, D/Q_m \quad \textbf{(12.9)}$$

Where; D = pond depth, m (see Section 8.8.1)
Q_m = mean flow, m³/day

The mean flow is the mean of the influent and effluent flows (Q_i and Q_e), the latter being the former less evaporation and seepage. If seepage is negligible, then equation 8.9 becomes:

$$Ø_f = A_f\, D\, (l½\, (Q_i + Q_e) \quad \textbf{(12.10)}$$

Since $Q_e = Q_i - 0.001\, A_f e$ (where e is the evaporation rate, mm/d), equation 4.10 becomes:

$$Ø_f = 2A_f\, D/(2Q_f - 0.001A_f e) \quad \textbf{(12.11)}$$

The BOD removal in primary facultative ponds is usually in the range 70-80 percent based on unfiltered samples (that is, including the BOD exerted by the algae), and usually above 90 percent based on filtered samples. In secondary facultative ponds the removal is less, but the combined performance of anaerobic and secondary facultative ponds generally approximates (or is slightly better than) that achieved by primary facultative ponds.

12.7.5 Maturation ponds

12.7.5.1Faecal coliform removal

The method of Marais (1974) is generally used to design a pond series for faecal coliform removal. This assumes that faecal coliform removal can be modeled by first order kinetics in a completely mixed reactor. The resulting equation for a single pond is thus:

$N_e = N_i / (1 + K_T Ø)$ **(12.12)**

Where N_e = number of FC per 100 ml of effluent
N_i = number of FC per 100 ml of influent
K_T= first order rate constant for FC removal, d
Ø= retention time, d

For a series of anaerobic, facultative and maturation ponds, equation 12.12 becomes:

$N_e = N_i / (1 + K_T Ø_a) (1 + K_T Ø_f) (1 + K_T Ø_m)^n$ **(12.13)**

Where N_e and N_i now refer to the numbers of FC per 100 ml of the final effluent and raw wastewater respectively; the sub-scripts a, f and m refer to the anaerobic, facultative and maturation ponds; and n is the number of maturation ponds.
It is assumed in equation 12.13 that all the maturation ponds are equally sized: this is the most efficient configuration (Marais, 1974), but may not be topographically possible (in which case the last term of the denominator in equation 12.13 is replaced by $(1 + K_T Ø)$)

The value of K_T is highly temperature dependent. Marais (1974) found that:

$K_T = 2.6 (1.19)^{T-20}$ **(12.14)**

Thus K_T changes by 19 percent for every change in temperature of 1 deg C (see Table 12.10)

Table 12.10 Values of the first order rate constant for faecal coliform removal at various temperatures (calculated from equation 12.14)

T (^{0}C)	K_T (day)	T (^{0}C)	K_T (day)
11	0.54	21	3.09
12	0.65	22	3.68
13	0.77	23	4.38
14	0.92	24	5.21
15	1.09	25	6.20
16	1.30	26	7.38
17	1.54	27	8.77
18	1.84	28	10.46
19	2.18	29	12.44
20	2.60	30	14.81

Maturation ponds require careful design to ensure that their FC removal follows that given by equations 12.13 and 12.14. If they are sub-optimally loaded, then their FC removal performance may be correspondingly suboptimal, as was found in Kenya (Mara et al. 1990; Mills et al., 1992). However, the FC removal performance of anaerobic and facultative ponds in Kenya closely followed equation 12.13.

Examination of equation 12.13 shows that it contains two unknowns, $Ø_m$ and n, since by this stage of the design process $Ø_a$ and $Ø_f$ will have been calculated, N1 measured or estimated (section 12.7.2), N_e set (at, for example, 1000 per 100 ml for unrestricted irrigation; see chapter on waste water reuse) and K_T calculated from equation 12.14. The best approach to solving equation 12.13 is to calculate the values of $Ø_m$ corresponding to n = 1, 2, 3, etc and then adopt the following rules to select the most appropriate combination of $Ø_m$ and n:

(a) $Ø_m > Ø_f$
(b) $Ø_m < Ø_m^{min}$

Where; $Ø_m^{min}$ is the minimum acceptable retention time in a maturation pond. This is introduced to minimize hydraulic short-circuiting and Marais (1974) recommends a value for it of 3 days.

The remaining pairs of $Ø_m$ and n, together with the pair $Ø_m^{min}$ and n where n is the first value of n for which $Ø_m$ is less than $Ø_m^{min}$, are then compared and the one with the least land area requirements. A check must be made on the BOD loading on the first maturation pond: this must not be higher than that on the preceding facultative pond, and it is preferable that it is significantly lower. In this Manual the maximum permissible BOD loading on the first maturation pond is taken as 75 percent of that on the preceding facultative pond. (It is not necessary to check the BOD loadings on subsequent maturation ponds as the non-algal BOD contribution to the load on them is very low).

The loading on the first maturation pond is calculated on the assumption that 70 percent of the BOD has been removed in the preceding pond(s). Thus:

$$\lambda_s\,(ml) = 10\,(0.3\,L_i)\,Q/A\,(ml) \qquad \textbf{(12.15)}$$

Or since $Q\,Ø_m\,(ml) = A\,(ml)\,D$

$$\lambda s = 10\,(0.3\,L_i)\,D/\,Ø_m\,(ml) \qquad \textbf{(12.16)}$$

The maturation pond area is calculated from a rearrangement of equation 8.11:

$$A_m = 2Q\,Ø_m\,/\,(2D + 0.001e\,Ø_m) \qquad \textbf{(12.17)}$$

The end of the chapter contains several examples showing how maturation ponds are designed according to the recommendations given above.

12.7.5.2Helminth egg removal

Helminth eggs are removed by sedimentation and thus most egg removal occurs in anaerobic or primary facultative ponds. However, if the final effluent is to be used for restricted irrigation (see chapter on wastewater reuse), then it is necessary to ensure that it contains no more than one egg per litre. Depending on the number of helminth eggs present in the raw wastewater and the retention times in the anaerobic and facultative ponds, it may be necessary to incorporate a maturation pond to ensure that the final effluent contains at most only one egg per litre. Analysis of egg removal data from ponds in Brazil, India and Kenya (Ayres et al., 1992a) has yielded the following relationship, which is equally valid for anaerobic, facultative and maturation ponds:

$$R = 100\,[1 – 0.14\,\exp\,[-0.38ø] \qquad \textbf{(12.18)}$$

Where; R = percentage egg removal
ø = retention time, d

The equation corresponding to the lower 95 percent confidence limit of equation 8.18 is

$$R = 100\,[1 – 0.41\,\exp\,[(-0.49\,ø + 0.0085\,ø^2)] \qquad \textbf{(12.19)}$$

Equation 12.19 is recommended for use in design (or Table 12.11 which is based on it); it is applied sequentially to each pond in the series, so that the number of eggs in the final how it is used for restricted irrigation. See design example No. 4 at end of the chapter.

Table 12.11 Design values of percentage Helmith egg removal (R) in individual anaerobic, facultative or maturation ponds for hydraulic retention times (ø) in the range 1 – 20 days (calculated from equation 12.19)

ø	R	ø	R	ø	R
1.0	74.67	4.0	93.38	9.0	99.01
1.2	76.95	4.2	93.66	9.5	99.16
1.4	79.01	4.4	93.40		
1.6	80.87	4.6	94.85	10	99.29
1.8	82.55	4.8	95.25	10.5	99.39
2.0	84.08	5.0	95.62	11	99.48
2.2	85.46	5.5	96.42	12	99.61
2.4	87.72			13	99.70
2.6	87.85	6.0	97.06	14	99.77
2.8	88.89	6.5	97.57	15	99.82
3.0	89.82	7.0	97.99	16	99.86
3.2	90.68	7.5	98.32	17	99.88
3.4	91.45			18	99.90
3.6	92.16	8.0	98.60	19	99.92
3.8	92.80	8.5	98.82	20	99.93

12.7.5.3BOD removal

Maturation ponds are not normally designed for BOD removal, yet it is often necessary to be able to estimate the BOD of the final effluent. Results from the Kenyan pond study (Mara et al., 1990) were equivocal and it was not possible to correlate BOD removal in maturation ponds with temperature or retention time. BOD removal in maturation ponds is very much slower than in anaerobic and facultative ponds, and it is therefore appropriate to estimate the filtered BOD of the final effluent on the assumption of 90 percent cumulative removal in the anaerobic and facultative ponds and then 25 percent in each maturation pond (Mara and Pearson, 1987).

12.7.5.4Nutrient removal

There are very few data from the region on nitrogen and phosphorus removal in WSP. For design recourse has to be made to equations developed in North America and designers should realize that these equations may not accurately predict performance elsewhere.

(a) Nitrogen

Pano and Middlebrooks (1982) present equations for ammonical nitrogen (NH_3 + NH_4^+) removal in individual facultative and maturation ponds. Their equation for temperatures below 20°C is:

$$C_e = C_i / \{1 + [(A/Q)\ (0.0038 + 0.000134T)\ \exp\ (1.041 + 0.044T)\ (pH-6.6)\} \qquad \textbf{(12.20)}$$

and for temperatures above 20°C:

$$C_e = C_i / \{1 + [1 +]\ 5.035 \times 10^3\ (A/Q]\ x\ \exp\ (1.540 \times (pH-6.6))] \qquad \textbf{(12.21)}$$

Where; C_e = ammoniacal nitrogen concentration in pond effluent, mg N/1
C_i= ammoniacal nitrogen concentration in pond influent, mg ‘N/1
A = pond area, m²
Q = influent flowrate, m³/d

Reed (1985) presents an equation for the removal of total nitrogen in individual facultative and maturation ponds:

$$C_e = C_i \exp/\{[-0.0064\ ]\exp(1.039)^{T-20})\ x\ (ø+60.6(pH-6.6)] \qquad \textbf{(12.22)}$$

Where C_e = total nitrogen concentration in pond effluent, mg N/1

C_i= total nitrogen concentration in pond influent, mg ‘N/1
T = temperature, ºC (range: 1-28ºC)
ø = retention time, d (range 5- 231 d)

The pH value used in equations 12.20 – 12.22 may be estimated from:
pH = 7.3 exp(0.0005A) **(12.23)**
Where; A = influent alkalinity, mg CaCO/l

Equations 12.20 – 12.22 are applied sequentially to individual facultative and maturation ponds in the series, so that concentrations in the effluent can be determined.

(b) Phosphorus

There are no equations for phosphorus removal in WSP. Huang and Gloyna (1984) indicate that, if BOD removal in a pond system in 90 percent, the removal of total phosphorus is around 45 percent. Effluent total P is around two-thirds inorganic and one third organic.

12.8 Physical Design of WSP

The process design prepared as described in Section 12.7 must be translated into a physical design. Actual pond dimensions, consistent with the available site, must be calculated; embankments and pond inlet and outlet structures must be designed and decisions taken regarding preliminary treatment, parallel pond systems and whether or not to line the ponds. By-pass pipes, security fencing and notices are generally required, and operator facilities must be provided.

The physical design of WSP must be carefully done: it is at least as important as process design and can significantly affect treatment efficiency.

12.8.1 Pond Location

Ponds should be located at least **200m** (preferably 500 m) downwind from the community they serve and away from any likely area of future expansion. This is mainly to discourage people from visiting the ponds (see section 12.8.10). Odour release, even from anaerobic ponds, is most unlikely to be a problem in a well designed and properly maintained system, but the public may need assurance about this at the planning stage, and a minimum distance of **200 m normally allays any fears.**

There should be vehicular access to the ponds and, so as to minimize earthworks the site should be flat or gently sloping. The soil must also be suitable (see section 12.8.2). Ponds should not be located within 2 km of airports, as any birds attracted to the ponds may constitute a risk to air navigation.

12.8.2 Geotechnical considerations

Geotechnical aspects of WSP design are very important. In France, for example, half of the WSP systems that malfunction do so because of geotechnical problems which could have been avoided at the design stage.

The principal objectives of a geotechnical investigation are to ensure correct embankment design and to determine whether the soil is sufficiently permeable to require the pond to be lined. The maximum height of the groundwater table should be determined and the following properties of the soil at the proposed pond location must be measured:

(a) particle size distribution;
(b) maximum dry density and optimum moisture content (modified Proctor test);
(c) Alterberg limits;
(d) organic content; and
(e) coefficient of permeability

At least four soil samples should be taken per hectare, and they should be as undisturbed as possible. The samples should be representative of the soil profile to a depth 1 m greater than the envisaged pond depth.

Organic, for example peaty, and plastic soils and medium-to-coarse sands, are not suitable for embankment construction. If there is no suitable local soil with which at least a stable and impermeable embankment core can be formed, it must be brought to the site at extra cost and the local soil, if suitable, used for the embankment slopes. Black cotton soils are impermeable and very suitable for ponds, but red coffee soils are too permeable and the ponds will require lining (see Section 12.8.3).

Ideally, embankments should be constructed from the soil excavated from the site, and there should be a balance between cut and fill, although it is worth noting that ponds constructed completely in cut may be a cheaper alternative, especially if embankment construction costs are high. The soil used for embankment construction should be compacted in 150 – 250 mm layers to 90% of the maximum dry density as determined by modified Proctor test. Shrinkage of the soil occurs during compaction (10-30 percent) and excavation estimates must take this into account. After, compaction, the soil should have a coefficient of permeability, as determined in situ, of **$<10^{-7}$ m/s** (see Section 12.8.3). Wherever possible and particularly at large pond installations, embankment design should allow for vehicle access to facilitate maintenance.

Embankment slopes are commonly 1 to 3 internally and 1.5-2 externally. Steeper slopes may be used if the soil is suitable; slopes maybe used if the slope stability should be ascertained according to standard soil mechanics procedures for small earth dams. Embankments should be planted with grass to increase stability: a slow –growing rhizomatous species should be used to minimize maintenance (see Section12.9.2).

External embankments should be protected from storm water erosion by providing adequate drainage. Internal embankments require protection against erosion by wave action, and this is best achieved by precast concrete slabs or stone rip-rap at top water level. Such protection also prevents vegetation from growing down the embankment into the pond, so preventing the development of a suitable habitat for mosquito or snail breeding.

12.8.3 Geotechnical Checklist

Experiences with WSP suggest that geotechnical engineering can correct many problems that prevent these cleverly designed systems from achieving their full potential. Here below is a ‘checklist’ for preventing problems related to water leakage, geological faults, and slope failures during excavation and construction as well as uncontrolled settlements after construction and a guideline to identify problematic soils (expansive, collapsible, dispersive, or highly compressible) with suggestions for their stabilization or improvement.

12.8.3.1Geotechnical deficiencies or failure mechanisms

Geotechnical deficiencies or potential failure mechanisms in WSP systems can be classified in three categories:

- flow-induced, instability, and deformation
- *Flow-induced deficiency or failure*
- Overtopping: Water flowing over the embankment crest causes a washout or total destruction of the structure.
- Piping: Seepage through the soil leads to transport of soil particles by internal erosion until a pipe forms (Fig 1).
- Leaks: Water leaks may occur through the pond bottom and embankment. They cause escape of water and diffusion of contaminants into surface waters and groundwater.
- Scour: Removal of soil particles from the soil-water interface by current or wave-induced shear forces possibly in combination with hydraulic gradient forces or because of rainfall run-off above the water line.
- Erosion: Precipitation may cause local concentrated erosion on the dry side of the embankment. Wave erosion will damage the lagoon side of the embankment.

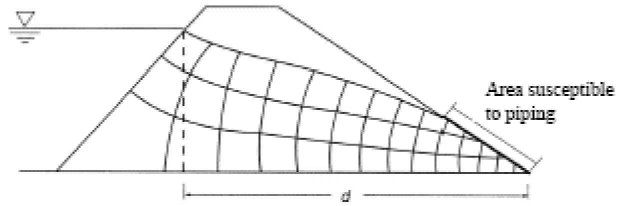


Figure 12.4 Area susceptible to initiation of pipeline.

Instability

- Geological fault: When a pond is constructed over an active geological fault not discovered during design, the embankment and liner system may be disrupted due to fault opening.
- Slides: Slip circles and deep sliding are frequent failure modes. Local shear failure parallel to the slope is possible either within the soil mass or at the soil-revetment interface. Local bearing failure may also be present. Many factors can cause slides and local failures: rapid drawdown, pre-existing slip planes within soil foundation, lenses and bands of weaker material, embankment crest overloading, inadequate slope design, etc.
- Liquefaction: Complete loss of grain-to-grain contact because of an increase in pore water pressure or shock loading of a loosely compacted granular soil. Consequent loss of effective stress results in a nearly zero shear strength and the soil behaving as a liquid.

Deformation

- Settlement: Deformation due to reduction in soil volume; may be caused by consolidation, compression, and shrinkage.
- Heave: Deformation due to increase in soil volume; may be caused by frost- or swelling-susceptible soils, especially during excavation.
- Cracking: Cracks may be transverse (perpendicular to the embankment axis) or longitudinal (parallel to the embankment axis), external or internal, shallow, or deep. They appear due to differential settlement or soil shrinkage.

Many of these mechanisms or failure modes are interrelated and should be anticipated and analyzed during geotechnical investigations. Piping and overtopping are of particular concern because they are responsible for most serious incidents and failures reported in the literature.

12.8.3.2Site investigation

Site investigation involves determining the nature and behavior of all aspects of a construction site and its environment that could significantly influence or be influenced by a project. Some of the most important aspects to be considered are:
• Groundwater effects: seepage, pumping, infiltration, and percolation.
• Environmental considerations: leachate contamination, health and safety issues on surrounding properties, and inhabitants.
• Excavation: short- and long-term stability, groundwater drawdown and possible damage to top clay layer.
• Ground improvement: barrier system and compaction.
• Placement of fill: slope stability, settlements, or heave.

The soil stratigraphy should be determined precisely, including soil composition, layer thickness, soil in homogeneity in horizontal extension, and water table location. It is also important to discover sand lenses, weak layers, and other relevant minor geological features. Investigation is not limited to the construction area: it should also cover widespread geological risks such as landslide, regional subsidence, and mining activity, among others. In seismically active regions, earthquakes may cause severe damage to embankments, barrier systems, and other WSP systems facilities. It is important to consider the seismicity of the site in the design.

Reconnaissance investigation is the first phase of site characterization. Borehole drilling and sampling are necessary for more detailed subsurface investigations. Soil samples should be tested in the laboratory to determine their physical, hydraulic, and mechanical properties. Several compatibility tests should be carried out to analyze the effect of some contaminants, salts, or temperature changes on the mechanical or index properties of the soil that forms the pond systems. Severe changes in the permeability and Atterberg limits in compacted clays under KCl solutions, for instance, have been observed. (Silva, [7])
In situ tests of subsurface soils often provide better information at lower cost than sampling and laboratory testing.

The field permeability and standard penetration tests are the most frequently used *in situ* tests. By measuring the drawdown in piezometers at certain distances from a pumped well, the field permeability test determines the permeability of pervious layers below the water table. Various types of borehole permeability tests made by pumping water either into or out of the boring are also used to determine the coefficient of permeability. On the other hand, the standard penetration test allows correlations of the number of blows with shear strength and other soil parameters.

12.8.3.3Problem soils

Many so-called ‘problem’ soils cause major damage to structures and embankments, and cannot be easily identified using routine soil testing procedures.

Expansive soils are widely distributed in Mexico. These soils, frequently found in nonsaturated residual soil deposits of fine material that contain clay minerals of the montmorillonite or illite type, are very sensitive to changes in humidity. Water adsorption in the active clay minerals causes them to expand when decompressed in the presence of water; they also shrink considerably when subjected to drying. If the embankment is constructed of or founded on

Expansive soils, evaporation, rainfall, temporal humidity variation, and change in water level of a pond may cause soil swelling and shrinkage; consequently, differential settlements and embankment fissures might occur. The shear strength of expansive soils also changes with variations in humidity, and a stability problem may arise in embankments. Visual inspection and determination of physical and mechanical properties can identify expansive soils. Oedometer testing can determine volume change under flooding.

Collapsible soils are fine sediments transported and deposited by wind. They range from sand dunes to loess deposits whose particles are predominantly of silt size with a certain amount of fine sand and aggregated clay particles. Although of low density, the naturally collapsible soils have a fairly high strength because of the clay binder, frequently calcium carbonate. However, they are easily eroded when flooded or rained on. Soil structure collapse causes large settlements. Oedometer testing can identify these soils.

Highly compressible clays of volcanic or organic origin may cause serious damage to barrier systems and embankments in WSP systems. One of the most notable examples of compressible clays is Mexico City clay, with water content as high as 350% to 400%.
These soils, usually high in adsorbed sodium, disperse or deflocculate easily and rapidly in water of low salt content.

The higher the percentage of sodium ions, the higher the susceptibility to dispersion. Such clays generally have high shrink-swell potential, low resistance to erosion, and low permeability in an intact state. Dispersive clay soils are identified by running a pinhole test, the Crum test, or the SCS dispersion test. Chemical tests are also frequently used to determine exchangeable sodium percentage, sodium adsorption ratio, percent sodium, and total dissolved salts. Dispersive soils are problematic for embankment stability, causing piping and rainfall erosion on slopes and channels.

A simple way to improve a problem soil site is to remove the problem soils and replace them with higher quality materials. Removal and replacement is most common for organic deposits and highly compressible soils, but has also been used for collapsible and expansive soils, for which treatment often consists of mixing soils of high

swelling potential (collapsibility) with those of low swelling potential. The use of chemical additives is another effective technique to improve problem soils. If special equipment is used, chemical stabilization can involve mixing and compaction of near-surface soils as well as deep soils. Chemical substances such as lime and Portland cement can be used alone or in combination to reduce the swelling and dispersion potentials of soils. Collapsible soils might be improved by pre-saturation and pre-loading procedures. Various other soil improvement techniques are also available and can be used if their cost is justifiable.
These problematic soils are described in detail, together with methods to identify and treat them to improve their mechanical behavior in geotechnical publications; see for instance Van Impe, [8] and Schaefer, [9].

12.8.3.4Design features

Apart from site investigations and geotechnical analysis (water flow, chemical diffusion, stability, and settlement studies), a number of provisions—as table 12.12 illustrates— should be considered to avoid or minimize deleterious effects from different environment actions.

Table 12.12 Provisions to be considered in Geotechnical Investigation

Excavation	The topsoil layer is generally not suitable for pond construction because it is frequently clayey, organic, weathered, or fissured. To use the barrier capacity of the lower part of the layer, the removal of the top clay layer must not be excessive. The remaining thickness of the clay layer after excavation depends upon local geologic conditions, grade and materials balance, heave potential, and thickness required to provide adequate contaminant attenuation. The excavation depth should be determined based on all these factors.
Liner	The pond retains contaminated water for long periods of time, so a barrier system should be designed to avoid excessive filtration. The barrier system would not only limit the physical escape of the liquid to either surface waters or groundwater, but also avoid the chemical migration by diffusion whereby the contaminants migrate from a point of high concentration to points of lower concentration. Clay liners and geomembranes are two of the most frequently used barrier systems. Soils classified as CL (clay of low plasticity), CH (clay of high plasticity) or SC (sandy clay) in the unified classification system are commonly used to construct clay liners expected to have a hydraulic conductivity less than $1x10^{-9}$ m/s. Although low hydraulic conductivity is the main design criteria for clay liners, other factors like cracking, strength, compatibility, diffusion, and temperature cannot be underrated. Geomembranes may be used alone or in combination with low permeability soils.
Revetment	The wet side face of the embankment should be protected from wind and wave action by riprapping the face with rock blocks. Riprap should extend at least one meter below the anticipated low water level. Reinforcing the dry side of the embankment with grass prevents gullying.
Freeboad	Pond overtopping should be avoided because it endangers embankment security, contaminates land and property beyond pond boundaries, requires excessively expensive repairs, and might cause serious public alarms about pond system safety and performance. A freeboard must be provided between the maximum water level and the minimum crest elevation to prevent overtopping. The freeboard is determined by considering the wind effect and long-term embankment and foundation settlement. The freeboard allowance for wind-induced wave action is the height of run-up of the significant wave as computed from adopted wind criteria. The freeboard allowance for settlement should be estimated from settlement analysis and seismic consideration for the embankment.
Compaction	Compaction procedures should be designed for materials to be used for construction of clay liners and embankments. Generally accepted compaction criteria, in which maximum dry density and optimum water content define the required compaction energy, are adequate for embankment construction. However, if expansive soils are used in the embankment construction, strongly compacted soils will have high heave potential. The preferred practice calls for placing expansive soils compacted with low compaction energy on the surface layer of the embankment. For clay liner, the compaction procedures should be very well controlled to obtain low permeability. It is generally believed that water contents higher than optimum help achieve lower permeability and adequate shear strength.
Earthquake defensive measures	If the pond is constructed in seismic zones, additional defensive measures should be taken. These might be: provision of more gentle slopes, allowance of ample freeboard for settlement and slumping, design of crest details for preventing erosion in the event of overtopping, and allowance of wide crest to mitigate propagation of transverse cracks that may be generated during shocks, etc. These measures are mentioned assuming that the embankment has a homogeneous clay section. Construction of filter and drainage systems is very effective in reducing earthquake damage.

12.8.3.5 Conclusion

Soil mechanics plays an important role in assuring the success of WSP systems. Overtopping and other Geotechnical-related damages may cause excessive repair costs and serious public alarm about security and safety. Efforts directed at identifying potential geotechnical problems before they materialize will generally result in lower Overall project costs.

Since the most important parameter for the correct function of these water treatment systems is the permeability of the liners and the *in situ* soils, special supervision and permeability tests must be part of both design and Construction.

It is recommended integrating the geotechnical investigation, analysis, and design from the beginning of project planning through construction and first operation stages. The geotechnical engineer's inspection is important during Excavation in order to identify new geological features and verify the accuracy of design assumptions; quality Control during embankment compaction must also be carried out, and design specifications met; the first filling is One of the most critical conditions, partly because piping or high pore pressures could develop for the first time; and,

Finally, frequent inspection is necessary during operations but more detailed inspections are required after extreme events such as heavy rainfall, rapid drawdown and earthquakes.

12.8.4 Hydraulic balance

To maintain the liquid level in the ponds, the inflow must be at least greater than net evaporation and seepage at all times. Thus:

$$Q \geq 0.001A\,(e + s) \qquad \textbf{(12.24)}$$

Where; Q = inflow to first pond, m/d

A = total area of pond series, m²

e = net evaporation (i.e. evaporation less rainfall), mm/d

s = seepage, mm/d

Seepage losses must be at least smaller than the inflow less net evaporation so as to maintain the water level in the pond. The maximum permissible permeability of the soil layer making up to the pond base can be determined from D' Arcy's law:

$$k = [Q_e/\,(86{,}400A)]\,(\Lambda 1/\,\Lambda 4) \qquad \textbf{(12.25)}$$

Where k = maximum permissible permeability, m/s

Q_e = maximum permissible seepage flow (=Q – 0.001Ae)m³

A = base area of pond, m²

Λ1= depth of soil layer below pond base to aquifer or more permeable stratum, m

Λ4 = hydraulic head (= pond depth + Λ1),m

If the permeability of the soil is more than the maximum permissible, the pond must be lined. A variety of lining materials is available and local costs dictate which should be used. Satisfactory lining has been achieved with ordinary Portland cement (8 kg/m²), plastic membranes and 150-300 mm layers of low-permeability soil (Figure 12.5). As a general guide, the following interpretations may be placed on values obtained for the in situ coefficient of permeability:

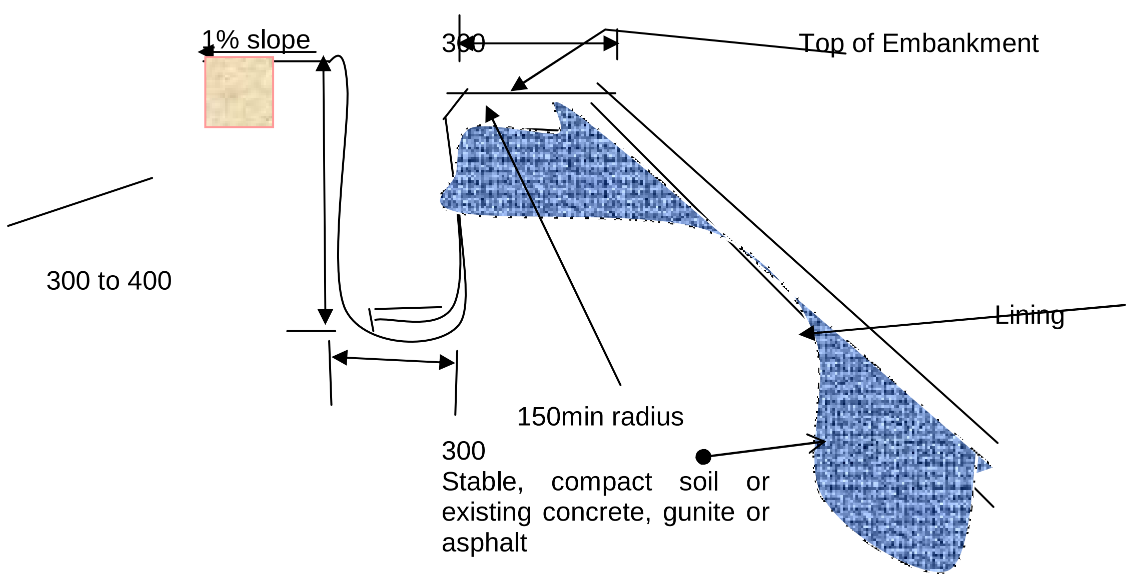
K $>10^{-6}$ m/s: the soil is too permeable and the ponds must be lined;

k$>10^{-7}$ m/s: some seepage may occur but not sufficiently to prevent the ponds from filling;

k$<10^{-8}$ m/s: the ponds will seal naturally;

k$< 10^{-9}$ m/s: there is no risk of groundwater contamination (if k$>10^{-9}$ m/s and the groundwater is used for potable supplies, further detailed hydrogeological studies may be required).

Figure 12.5 Anchoring of Pond Lining at Embankment Top



12.8.5 Preliminary treatment

Adequate screening and grit removal facilities must be installed at all but very small systems (those serving <1000 people). Design should follow standard procedures (For example, IWEM, 1992; Marais, 1971; Metcalf and Eddy, Inc., 1991), with particular attention being given to the high grit loads common in domestic wastewater in the Region (up to 0.17 m^3 per 1000 m^3 of wastewater, with an average of 0.075; Meiring et al., 1968). Adequate disposal of screenings and grit burial in trenches is usually the most appropriate method.

Wastewater flows up to 6 times dry weather flow should be subjected to screening and grit removal. Any flows in excess of 6 DWF should be discharged via a stormwater overflow to a receiving water course. Anaerobic ponds should not receive more than 3 DWF, in order to prevent washout of acidogens and methanogens; so excess flows between 3 and 6 DWF are diverted via an overflow weir to the facultative ponds.

After screening and grit removal and, if installed, the >6 DWF overflow weir, the wastewater flow should be measured in a standard Venturi or Parshall flume. This is essential in order to assess pond performance. Flow-recording devices may be installed, but these require careful calibration and regular maintenance. Often it is better to read the upstream channel depth from a calibrated brass rule and then calculate the flow from a standard flume formula (see IWEM, 1992; Metcalf & Eddy, Inc., 1991).

12.8.6 Pond geometry

There has been little rigorous work done on determining optimal pond shapes. The most common shape is rectangular, although there is much variation in the length-to-breadth ratio. Clearly, the optimal pond geometry, which includes not only the shape of the pond but also the relative positions of its inlet and outlet, is that which minimizes hydraulic short-circuiting.

In general, anaerobic and primary facultative ponds should be rectangular, with length-to-breadth ratios of 2-3 to 1 so as to avoid sludge banks forming near the inlet. Secondary facultative and maturation ponds should, wherever possible, have higher length-to-breadth ratios (up to 10 to 1) so that they better approximate plug flow conditions. Ponds do not need to be strictly rectangular, but may be gently curved if necessary or if desired for aesthetic reasons. A single inlet and outlet are usually sufficient, and these should be located in diagonally opposite corners of the pond. The use of complicated multi-inlet and multi-outlet designs is unnecessary and not recommended.

To facilitate wind-inducted mixing of the pond surface layers, the pond should be located so that its longest dimension (diagonal) lies in the direction of the prevailing wind. If this is seasonally variable, the wind direction in the hot season should be used as this is when thermal stratification is at its greatest. To minimize hydraulic short-circuiting, the inlet should be located such that the wastewater flows in the pond against the wind.

The areas calculated by the process design procedure described in Section 12.7 are mid-depth areas, and the dimensions calculated from them are thus mid-depth dimensions. These need to be corrected for the slope of the embankment, as shown in Figure 12.6.

A more precise method is advisable for anaerobic ponds, as these are relatively small. The following formula is used (EPA, 1983):

$V = (LB) + (L\text{-}2sD)\ (B\text{-}2sD) + 4(L - sD)\ (B\text{-}sD)]\ [D/6]$

Where V = anaerobic pond volume, m^3

L = pond length at TWL, m

B = pond breadth at TWL. M

S = horizontal slope factor (i.e a slope of 1 in s)

D = pond liquid depth, m

With the substitution of L as nB, based on a length to breadth ratio of n to 1, equation 12.26 becomes a simple quadratic in B. The dimensions and levels that the contractor needs to know are those of the base and the top of the embankment; the latter includes the effect of the freeboard. The minimum freeboard that should be provided is decided on the basis of preventing waves, induced by the wind form overtopping the embankment. For small ponds (under 1 ha in area) 0.5 m freeboard should be provided; for ponds between 1 ha and 3 ha, the freeboard should be 0.5 – m, depending on site considerations. For larger ponds, the freeboard may be calculated from the equation (Oswald, 1975):

$$F = (\log A)^{1/2} - 1 \qquad \textbf{(12.26)}$$

Where F = freeboard, m

A = pond area at TWL, m^2

Pond liquid depths are commonly in the following ranges:

Anaerobic ponds:	2-5 m
Facultative ponds:	1-2 m
Maturation ponds:	1-1.5 m

The depth chosen for any particular pond depends on site considerations (presence of shallow rock, minimization of earthworks). The depth of facultative and maturation ponds should not be less than 1 m so as to avoid vegetation growing up from the pond base, with the consequent hazard of mosquito and snail breeding.

At WSP systems serving more than around 10,000 people, it is often sensible (so as to increase operational flexibility) to have two or more series of ponds in parallel. The available site topography may in any case necessitate such a subdivision, even for smaller systems. Usually the series are equal, that is to say they receive the same flow, and arrangements for splitting the raw wastewater flow into the equal parts after preliminary treatment must be made (see *Stalzer and von der Emde, 1972*). This is best done by providing weir penstocks ahead of each series.

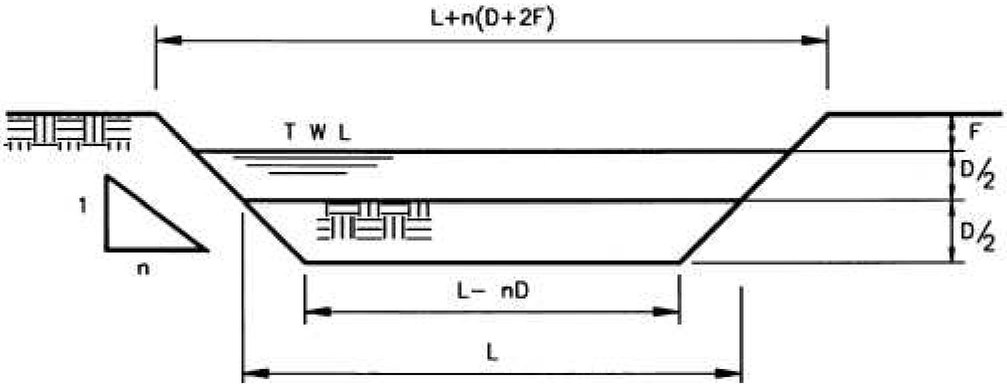


Figure 12.6 Calculation of top and bottom pond dimensions from those based on mid depth

12.8.7 Inlet and outlet structures

There is a wide variety of designs for inlet and outlet structures, and provided they follow certain basic concepts, their precise design is relatively unimportant. Firstly, they should be simple and inexpensive; while this should be self-evident, it is all too common to see unnecessarily complex and expensive structures. Secondly, they should permit samples of the pond effluent to be taken with ease. The inlet to anaerobic and primary facultative ponds should discharge well below the liquid level so as to minimize short-circuiting (especially in deep anaerobic ponds) and thus reduce the quantity of scum (which is important in facultative and maturation ponds should also discharge below the liquid level, preferably at mid-depth in order to reduce the possibility of short-circuiting. Some simple inlet designs are shown in Figures 12.9 and 12.10.

The outlet of all ponds should be protected against the discharge of scum by the provision of a scum guard. The take-off level for the guard depth is important as it has a significant influence on effluent quality. In facultative ponds, the scum guard should extend just below the maximum depth of the algal band when the pond is stratified so as to minimize the daily quantity of algae and hence BOD, leaving the pond. In aerobic and maturation ponds, where algal banding is irrelevant, the take-off should be nearer the surface: in anaerobic ponds it should be well above the maximum depth of sludge but below any surface crust, and in maturation ponds it should be at the level that gives the best possible microbiological quality. The following effluent take-off levies are recommended:

Anaerobic ponds:	300 mm
Facultative ponds:	600 mm
Maturation ponds:	50 mm

The installation of a variable height scum guard is recommended, since it permits the optimal take-off level to be set once the pond is operating.

Some simple designs for outlet structures are shown in Figures 12.7 and 12.8. If a weir is used in the outlet structures, the following formula should be used to determine the head over the weir and so, knowing the pond depth, one can calculate the required height of the weir above the pond base:

$$Q = 0.0567\ H^{3/2} \qquad \textbf{(12.27)}$$

Where Q = flow per metre length of weir, l/s
H = head of water above weir, mm

The outlet from the final pond in a series should discharge into a simple flow-measung device such as a triangular or rectangular notch. Since the flow into the first pond is also measure, this permits the rate of evaporation and seepage to be calculated or, if evaporation is measured separately, the rate of seepage.

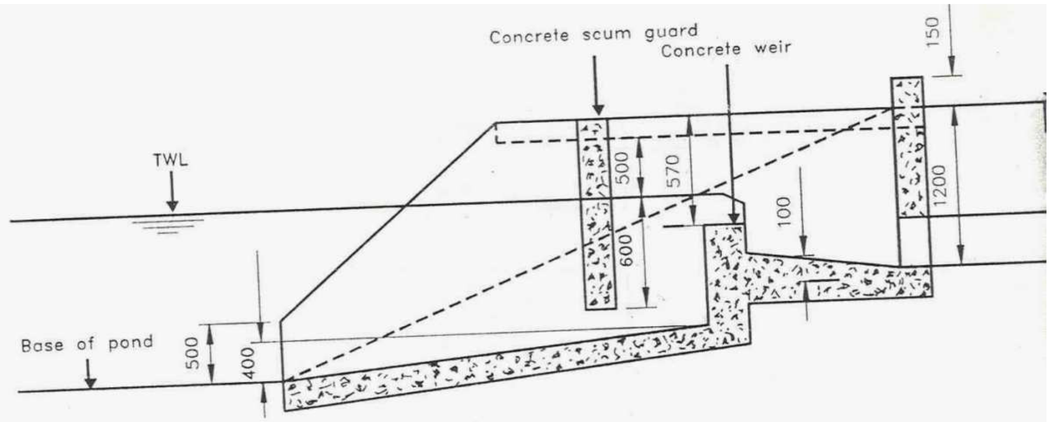


Figure 12.7 Outlet Weir Structure. The weir ir length from equation 12.27. The discharge pipe would connect with the inlet structure shown in 12.9

The concrets scum guard depth should be as described in section 12.8.7.

As alternative variable depth wooden scum guard may be used.

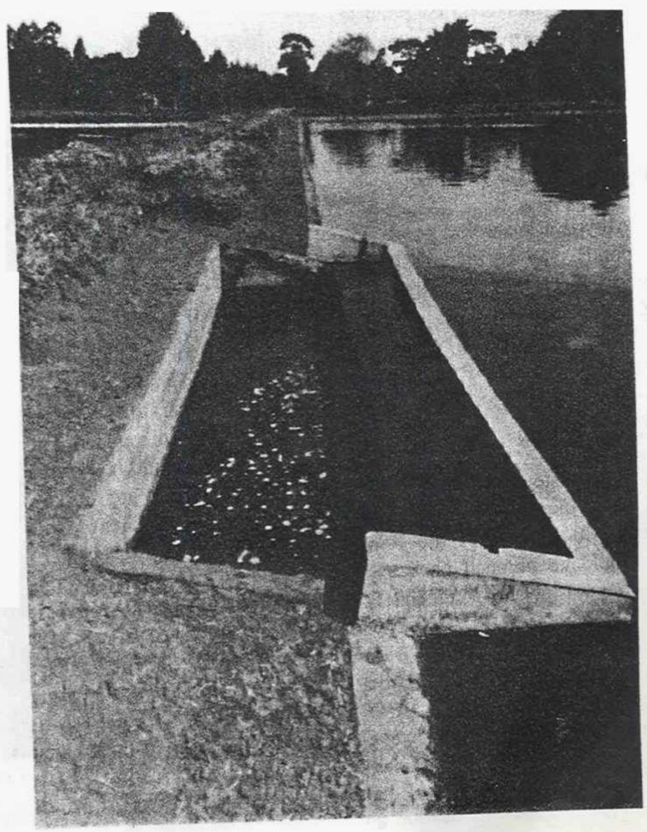


Figure 12.8 Outlet Weir structure at Karatina, Kenya

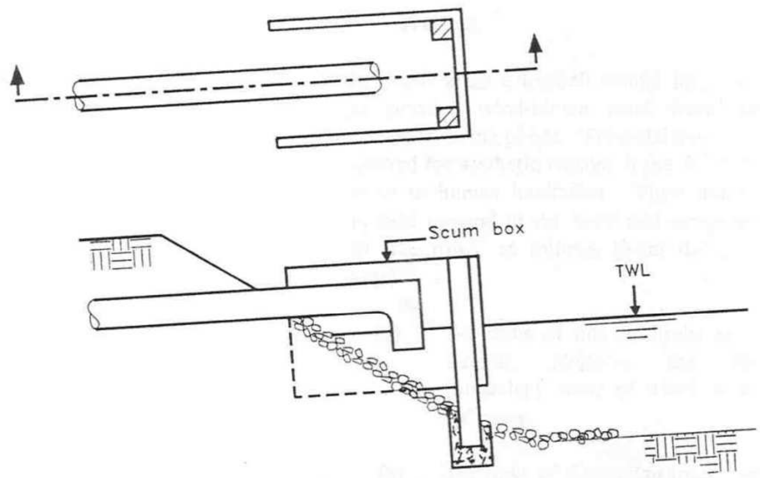


Figure 12.9 Inlet structure for Anaerobic and Primary Ponds. The scum box retains most of the floating solids, so easing Pond Maintenance

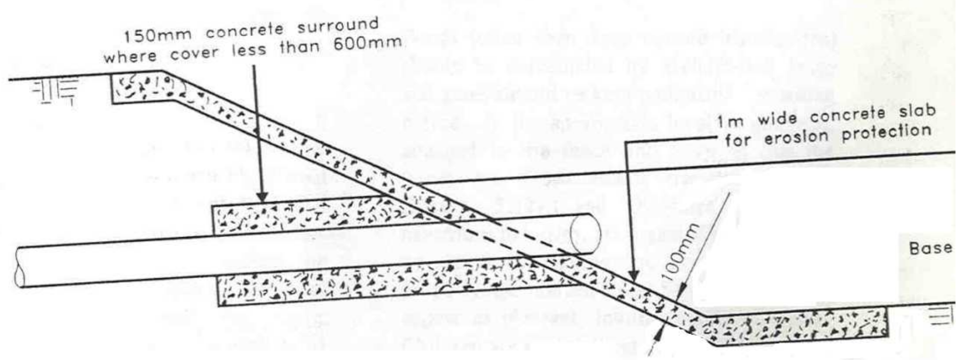


Figure 12.10 Inlet structure for Secondary Facultative and Maturation discharge from the outlet structure shown in Figure 12.8

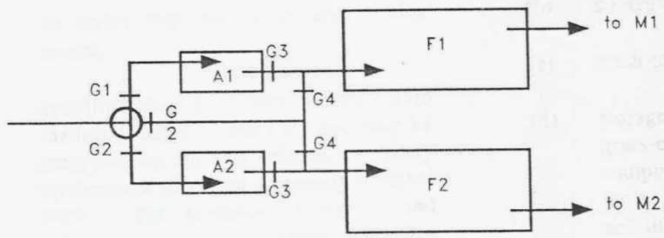


Figure 12.11 By-pass pipework for anaerobic ponds. During normal operation gates G1 and G3 are open and gates G2 and G4 closed. To bypass one (or both) anaerobic ponds gates G1 and G3 are closed and gate G2 and one (or both) of gates G4 open.

Figure 12.12 Chain-link fence and warning notice at the Karen ponds, Nairobi, Kenya.

12.8.8 By-pass pipe work

It is necessary to bypass anaerobic ponds so that facultative ponds may be commissioned first (see section 12.9) and also during desludging operations. Figure 12.12 shows schematically a by-pass arrangement for two series of WSP in parallel.

12.8.9 Recirculation

If the incoming raw wastewater is septic, it may be necessary to achieve odour control by recirculating up to 50 percent of the final effluent. This should be pumbed back and mixed with the raw wastewater immediately after preliminary treatment (i.e before the wastewater enters the first pond). The final effluent acts to oxygenate the septic wastewater, and it may help to increase BOD removal. The process design of the ponds has to be altered to allow for the recirculated flow and clearly recirculation, with its attendant problems of pump O & M, should only be considered as a measure of the last resort.

12.8.10 'Tree belt'

In desert areas a tree belt should be provided to prevent wind-blown sand from being deposited in the ponds. Tree belts may also be desired for aesthetic reasons if the WSP site is close to human habitation. They should be planted upwind of the WSP and comprise up to five rows, as follows (from the upwind side):

(a) 1-2 rows of mixed shrubs such as *Latana, Hibiscus and Nereo* (oleander), none of which is eaten by goats;
(b) 1-2 rows of Casuarina trees; and
(c) 1 row of a mixture of taller trees such as *Poinciana regia* (flame trees), *Tipuana tipu, Khaya senegalensis and Albizia lebbech.*

Local botanists will be able to advice on which species are most appropriate; those given above are suitable for semi-arid and arid zones. Such a tree belt is around 40-60 m wide.

12.8.11 Security

Ponds (other than very remote installations) should be surrounded by a chain-link fence and gates should be kept padlocked. Warning notices, in the appropriate local language(s), attached to the fence and advising that the ponds are a wastewater treatment facility (Figure 12.12), and therefore potentially hazardous to health, are essential to discourage people from visiting the ponds, which if properly maintained (see Section 8.9) should appear as pleasant, inviting bodies of water. Children are especially at risk, as they may be tempted to swim in the ponds. Birdwatchers and hunters are also attracted to ponds by the often rich variety of wildlife, and they may not be aware that the ponds are treating wastewater.

An electrified fence (12 V DC), located within the chain-link fence if there is one may be necessary to keep out wild animals, especially hippopotamuses which at commonly attracted to WSP. The presence of hippos and crocodiles in a pond makes sampling hazardous, and can be fatal: a pond operator at the Dandora WSP in Nairobi was killed by a hippo in 1989.

12.8.12 Operator facilities

The facilities to be provided for the team of pond operators depend partly on their number (see Section 12.3), but would normally include the following:

(a) first-aid kit (which should include a snake bite kit);
(b) strategically placed lifebuoys;
(c) wash-basin and toilet; and
(d) Storage space for protective clothing, grass-cutting and scum-removal equipment, screen rakes and other tools, sampling boat (if provided) and life-jackets.

With the exception of the lifebuoys, these can be accommodated in a simple building. This can also house, if required, sample bottles and a refrigerator for sample storage. Laboratory facilities, offices and a telephone may also be provided at large installations. Adequate space for car parking should be provided.

At very large WSP sites, such as at Dandora in Nairobi, consideration should also be given to providing housing for the large number of operators employed (59 in the case of Dandora ;).

12.9 Operation and Maintenance

12.9.1 Start-up procedures

Pond systems should preferably be commissioned at the beginning of the hot season so as to establish as quickly as possible the necessary microbial populations to effect waste stabilization. Prior to commissioning, all ponds must be free from vegetation. Facultative ponds should be commissioned before anaerobic ponds: this avoids odour release when anaerobic pond effluent discharges into an empty facultative pond. It is best to fill facultative and maturation ponds with freshwater (from a river, lake or well; mains water is not necessary so at to permit the gradual development of the algal and heterotrophic bacterial populations. Primary facultative ponds may advantageously be seeded in the same way as anaerobic ponds (see below). If freshwater is unavailable, facultative ponds should be filled with raw sewage and left for three to four weeks to allow the microbial population to develop; a small amount of odour release is inevitable during this period.

Anaerobic ponds should be filled with raw sewage and seeded, where possible, with digesting sludge from, for example, an anaerobic digester at a conventional sewage treatment works or with sludge from local septic tanks. The ponds should then be gradually loaded up to the design loading rate over the following week (or month if the ponds are not seeded). Care should be taken to maintain the pond pH above 7 to permit the development of methanogenic bacteria, and it may be necessary during the first month or so to dose the pond with lime or soda ash. If due to an initially low rate of sewer connections in newly sewered towns the sewage is weak or its flow low, it is best to by-pass the anaerobic ponds until the sewage strength and flow is such that a loading of at least 100 g/m^3 d can by-pass an anaerobic pond whilst it is being desludged (section 12.9.4), so the by-pass should be a permanent facility: (see section 12.8.7).

12.9.2 Routine Maintenance

The maintenance requirements of ponds are very simple, but they must be carried out regularly. Otherwise, there will be serious odour, fly and mosquito nuisance. Maintenance requirements and responsibilities must therefore be clearly defined at the design stage so as to avoid problems later. Routine maintenance tasks are as follows:

(a) removal of screenings and grit from the inlet works;
(b) cutting the grass on the embankments and removing it so that it does not fall into the pond (this is necessary to prevent the formation of mosquito-breeding habitats; the use of slow-growing grasses minimizes this tasks – see Section 12.8.2)
(c) Removal of floating scum and floating macrophytes, eg. *Lemna*, from the surface of facultative and maturation ponds (this is required to maximize photosynthesis and surface re-aeration and obviate fly breeding);
(d) spraying the scum on anaerobic ponds (which should not be removed as it aids the treatment process), as necessary, with clean water or pond effluent to prevent fly breeding;
(e) removal of any accumulated solids in the inlets and outlets;
(f) repair of any damage to the embankments caused by rodents, rabbits or other animals; and
(g) Repair of any damage to external fences and gates.

The operators must be given precise instructions on the frequency at which these tasks should be done, and their work must be constantly supervised. The supervisor/foreman should be required to complete at weekly intervals a pond maintenance record sheet, an example of which is given in Figure 12.13. The operators may also be required to take samples and some routine measurements (see section 12.10).

Figure 12.13 Pond Maintenance Record Sheet

Pond location							
Date and Time				Air Temperature ^{0}C			
Weather Conditions							
Pumping Station(If there is one)							
Elapsed time meter reading			No. 1		No. 2		
Electricity Meter Reading							
Observations (Flooding)							
Access road: State (Vegetation, damage) maintenance Carried out							
Pond Site: State; maintenance carried out							
Pretreatment works: state; maintenance carried out							
Screen (s)							
Other (grit, grease removal)							
Visual Inspection of Ponds							
Pond Number							
Colour of water (green, brown/grey, pink/red, milky/clear)							
Odour							
Scum/foam							
Rooted macrophytes							
State of Embankment (Erosion, rodent, damage, vegetation)							
Inlet and outlet (Blockages)							
Water Level(High, normal, Low)							
General Observation, other maintenance carried out							

12.9.3 Staffing Levels

In order that the routine O & M tasks can be properly done, WSP installations must be adequately staffed. The level of staffing depends on the type of inlet works (for example, mechanically raked screens and proprietary grit removal units require an electromechanical technician, but manually raked screens and manually cleaned grit channels do not), whether there are on-site laboratory facilities and how the grass is cut (manually or by mechanical mowers). Recommended staffing levels are given in Table 12.14 for WSP systems serving populations up to 250,000; for larger systems the number of staff should be increased pro rata. At the Dandora WSP in Nairobi, which serve a design population of around 800,000 (80,000 m³/d) the following staffs are employed:

Superintendent (Manager)	1
Assistant Manage	1
E & M Technician Engineers	3
Works Foreman	6
Laboratory Chemist	1
Laboratory Asst. & Technicians	4
Electricians	2
Artisans	2
Clerk	1
Assistant Clerk	1
Drivers	4
Labourers	28
Watchmen	5

12.9.4 Desludging and sludge disposal

Anaerobic ponds require desludging when they are one third full of sludge (by volume). This occurs every n year.

$$N = V/3Ps \qquad \textbf{(12.28)}$$

Where; V = volume of anaerobic pond, m³
P = population served
S = sludge accumulation rate, m³/caput year

The usual design value of s is 0.04 m³/caput/year Thus, for temperature above 20^0c (λ_v = 300 g/ m³d) and a BOD contribution of 40 g/person /day, desludging would be required annually (n = 1.11 years). The precise requirement for desludging can be determined by the "white towel" test (section 12.10.2), but it should be borne in mind that a task to be done annually has more chance of being done on time than one to be done at less regular intervals.

Sludge removal can be readily achieved by using a raft-mounted sludge pump. These are commercially available (e.g. Brain Industries Ltd., Kilgetty, Dyfed SA68 0UJ, UK), or they can be assembled locally. The sludge is discharged into either an adjacent sludge lagoon or tankers that transport it to a landfill location. Although pond sludge has a better microbiological quality than that form conventional treatment works, its disposal must be carried out in accordance with the relevant local regulations, if any, governing sludge disposal.

Table 12.14 Recommended staffing levels for WSP systems

Population Served	10,000	25,000	50,000	100,000	250,000
Foreman/Supervisor	-	-	1	1	1
Mechanical Engineer a/	-	-	-	1	1
Laboratory Technician b/	-	1	1	1	2
Assistant Foreman	-	1	2	2	2
Labourers	1	2	4	6	10
Driver c/	-	1	1	1	2
Watchman d/	1	1	3	5	
Total	**2**	**6**	**10**	**15**	**23**

a) Dependent upon amount of mechanical equipment used
b) Dependent upon existence of laboratory facilities
c) Dependent upon use of vehicle towed lawn mowers, etc.
d) Dependent upon location and amount of equipment used

12.10 Monitoring and Evaluation

Once a WSP system has been commissioned, a routine monitoring and evaluation programme should be established so that its performance can be verified and the actual quality of its effluent established.

Routine monitoring of the final effluent quality of a pond system permits a regular assessment to be made of whether the effluent is complying with the local discharge or reuse standards. This information may be required by the local regulatory river or health authority. Moreover, should a pond system suddenly fail or its effluent start to deteriorate, often give some insight into the cause of the problem and may indicate what remedial action is required.

The evaluation of pond performance and behavior, although a much more complex procedure than the routine monitoring of effluent quality, is nonetheless extremely useful as it provides information on how under loaded or overloaded the system is, and thus by how much, if any, the loading on the system can be safely increased as the community it serves expands, or whether further ponds (in parallel or in series) are required (see section 12.11.2). It also indicates how the design of future pond installations in the region might be improved to take account of local conditions.

12.10.1 Effluent quality monitoring

Effluent quality monitoring programmes should be simple, but should none the less provide reliable data. Two levels of effluent monitoring are recommended (reference should also be made to the routine maintenance record sheets to be completed by the pond supervisor – see Section12.9.2 and Figure 12.9.1):

(a) Level 1: representative samples of the final effluent should be taken at least monthly, although preferably weekly, intervals; they should be analyzed for those parameters for which effluent discharge or reuse requirements exist;
(b) Level 2: when level 1 monitoring shows that a pond effluent is failing to meet its discharge or reuse quality, a more detailed study is necessary. Table 12.14 gives a list of parameters whose values are required, together with directions on how they should be obtained.

Since pond effluent quality shows a significant diurnal variation (although this is less pronounced in maturation ponds than in facultative ponds), 24 hour flow-weighted composite samples are preferable for most parameters, although grab samples are necessary for some (pH, temperature and faecal coliforms). Composite samples should be collected in one of the following ways:

(a) in an automatic sampler, which takes grab samples every one or two hours, with subsequent annual fewn weighting if this is not done automatically by the sampler;
(b) by taking grab samples every one to three hours (depending on labour availability), with subsequent manual flow weighting; or
(c) by taking a column sample (See Section 12.2) near the outlet or the final pond; this can be done at any time of day and gives a good approximation to the mean daily effluent quality (*Pearson et al., 1987b*).

Table 12.15 Parameters to be determined in a "Level 2" effluent quality monitoring programme

Parameter	Type of Sample a/	Remarks
Flow	-	Measure both raw wastewater and final effluent flows
BOD	C	Unfiltered samples b/
COD	C	Unfiltered samples b/
Suspended Solids	C	
Ammonia	C	
Faecal Coliforms	G	Take sample between 08.00 and 10.00hr
pH	G)	Take two samples, one at 08.00-10.00h and the other at 14.00-16.00hours
Temperature	G)	
Total Nitrogen	C)	Only when effluent being used (For being assessed for use) for crop irrigation. Ca, Mg and Na are required to calculate the sodium absorption ratio d/
Total Phosphorus	C)	
Chloride	C)	
Electrical conductivity	C)	
Ca, Mg, Na	C)	
Boron	C)	
Helminth Eggs	C)	

a) C=24 hour flow weighted composite sample; G=grab sample

b) Also on filtered samples if the discharge requirements are so expressed

c) Ascaris lumbricoides, Trichuris trichiura, Ancylostoma duodenale and Necator americanus

d) SAR $=(0.044\ Na)/\{0.5(0.050Ca + 0.082\ Mg)\}^{1/2}$ where Na, Ca and Mg are the concentrations in mg/l

12.10.2 Evaluation of pond performance

A full evaluation of the performance of a WSP system is a time-consuming and expensive process, and it requires experienced personnel to interpret the data obtained. It is in many ways close to research, but it is the only means by which pond designs can be optimized for local conditions. It is often therefore a highly cost-effective exercise. The recommendations given below constitute a level 3 monitoring programme, and they are based on the guidelines for minimum evaluation of pond performance given in Pearson et al., (1987a), which should be consulted for further details.

It is not intended that all pond installations be studied in this way, but only one or two representatives systems in each major climatic region of a country. This level of investigation is most likely to be beyond the capabilities of local organizations, and it would need to be carried out by a regional or national body, or by a university under contract to such a body. This type of study is also necessary when it is required to know how much additional loading a particular system can receive before it is necessary to extend it.

Samples should be taken and analyzed on at least five days over a five-week period at both the hottest and coldest times of the year. Samples are required of the raw wastewater and of the effluent of each pond in the series and, so as to take into account most of the weekly variation in influent and effluent quality, samples should be collected on Monday in the first week, Tuesday in the second week and so on (local factors, such as a high influx of visitors at weekends, may influence the choice of days on which samples are collected). Table 12.15 lists the parameters whose values are required. Generally the analytical techniques described in the current edition of *Standard Methods (APHA, 1992*) are recommended, although the procedures detailed in Annex III should be followed for faecal coliforms, helminth eggs, chlorophyll and sulphide. The algal genera present in facultative and maturation pond samples should also be determined.

Composite samples, collected as described in Section 12.10.1, are necessary for most parameters; grab samples are required for pH and faecal coliforms; and samples of the entire pond water column should be taken for algological analyses (chlorophyll a and algal genera determination), using the pond column sampler. Pond column samples should be taken from a boat or from a simple sampling platform. Data on at least maximum and minimum air temperatures, rainfall and evaporation should be obtained from the nearest meteorological station.

On each day that samples are taken, the mean mid-depth temperature of each pond, which closely approximates the mean daily pond temperature, should be determined by suspending a maximum-and-minimum tehrmomenter at mid-depth of the pond at 08.00-09.00 h and reading it 24 hours later.

On one day during each sampling period, the depth of sludge in the anaerobic and facultative ponds should be determined, using the "white towel" test of Malan (1964). White toweling material is wrapped along one third of a sufficiently long pole, which is then lowered vertically into the pond until it reaches the pond bottom; it is then slowly withdrawn. The depth of the sludge layer is clearly visible since some sludge particles will have been entrapped in the toweling material. The sludge depth should be measured at least five ponds in the pond, away from the embankment base, and the mean depth calculated.

It is also useful to measure on at least three occasions during each sampling season the diurnal variation in the vertical distribution of pH, dissolved oxygen and temperature. Profiles should be obtained at 08.00, 12.00 and 16.00 h. If submersible electrodes are not available, samples should be taken manually every 20 cm.

12.11 Rehabilitation and Upgrading

12.11.1 Rehabilitation

Some WSP systems do not function well. This may simply be due to overloading (in which case the WSP system needs extending – see Section 12.11.2), but it can often be the result of:

(a) improper process and/or physical design;
(b) poor design and/or operation of the inlet works; and/or
(c) Inadequate maintenance of the ponds.

The effects can be quite serious: odour release from both anaerobic and facultative ponds; fly breeding in anaerobic ponds; floating macrophytes or emergent vegetation in facultative and maturation ponds leading to mosquito breeding – in extreme cases the ponds can slit up and completely "disappear".

Rehabilitation is achieved by a combination of the following:

(a) a complete overhaul (or redesign) of the inlet works replacing any units that cannot be satisfactorily repaired;
(b) repairing or replacing any flow measuring devices;
(c) ensuring that any flow into the required proportions;
(d) desludging the anaerobic or primary facultative ponds, and any subsequent ponds if necessary;
(e) unblocking, repairing or replacing pond inlets and outlets;
(f) repositioning any improperly located inlets and/or outlets, so that they are in diagonally opposite corners of each pond;
(g) repairing, replacing or providing effluent scum guards;
(h) preventing "surface streaming" of the flow when the pond is stratified by discharging the influent at mid-depth (or by installing a baffled inlet to achieve the same effect);
(i) removing scum and floating or emergent vegetation from the facultative and maturation ponds;
(j) checking embankment stability, and repairing, replacing or installing embankment protection;
(k) checking for excessive seepage (>10 percent of inflow) and lining the ponds if necessary;
(l) cutting the embankment grass; and
(m) Repairing or replacing any external fences and gates; fences may need to be electrified to keep out wild and domestic animals.

As rehabilitation can be expensive, good routine maintenance is much more cost-effective.

12.11.2 Upgrading and extending existing WSP

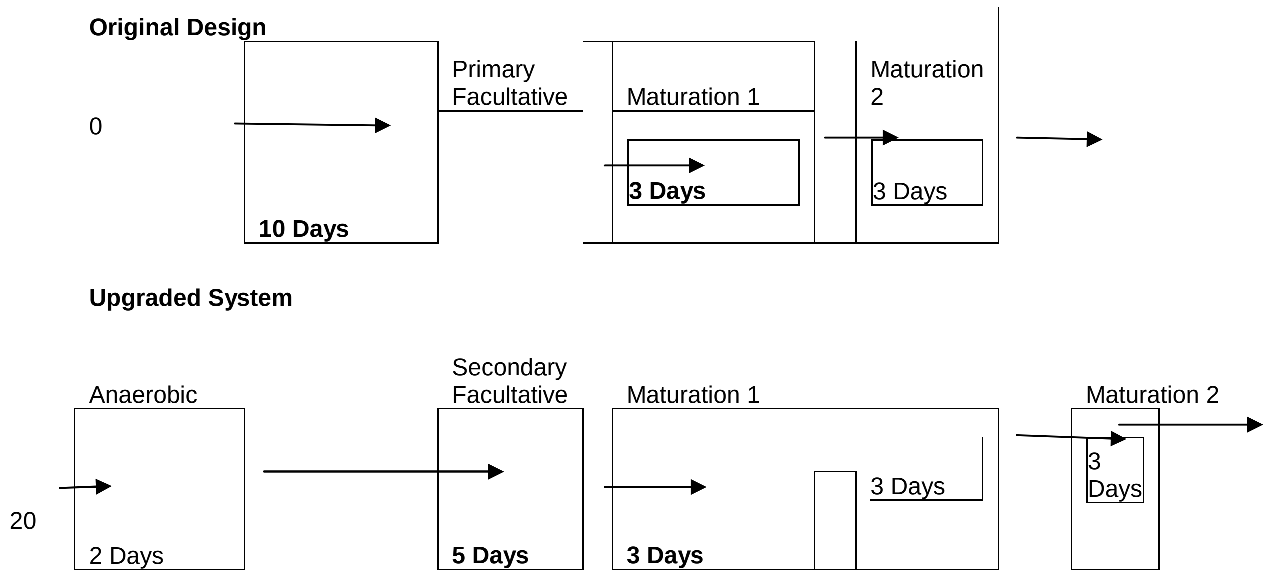
Prior to upgrading or extending a WSP system its performance should be evaluated as described in Section 12.10.2, as this will generally permit the correct decision about how to upgrade and/or extend the system to be made.

Figure 12.14 Upgrading a WSP series to treat twice the original flow. The embankment between the original maturation ponds becomes a baffle in the upgraded first maturation pond. The total retention time is reduced from 16 to 13 days and the improvement in microbiological quality can be illustrated as follows, by using equation 12.13 with N_i = 10 per 100 ml and k = 2.6 d (i.e. for 20ºC):

Original system: $N_e = 10/ [(1 + (2.6 \times 10) (1 + (2.6 \times 3)]$
$= 4.8 \times 10^4$ per 100 ml

Upgraded system: $N = 10 / [(1 + (2.6 \times 2) (1 + 2.6 \times 5)] (1 + (2.6 \times 3)]$
$= 1.5 \times 10^4$ per 100 ml

Figure 12.14 Upgrading a WSP series to treat twice the original flow



A number of strategies can be used to upgrade and extend WSP systems. In addition to any rehabilitation measures needed (Section 12.11.1) these includes:

(a) provision of anaerobic ponds;
(b) provision of additional maturation ponds;
(c) provision of one or more additional series of ponds;
(d) Alteration of pond sizes and configuration – for example, removal of an embankment between tow ponds to create a larger one.

Only when al these possibilities have been considered should thought be given to the installation of surface aerators, thereby creating a partially aerated facultative pond. These can do more harm than good, especially if they are not continuously operated (as is frequently the case to save energy costs), since sulphide-rich water form the pond bottom is brought to the surface and aerated with consequent odour release.

Figure 12.14 shows how (a) and (d) above can be combined to upgrade a single series of WSP to receive twice its original design flow at a lower overall retention time, and with the production of a higher quality effluent.

12.11.3 Algal removal techniques

The algal in a WSP effluent contributes to both its suspended solids content and BOD. If local regulatory agencies do not make allowance for the inherent difference between algal SS and BOD and "ordinary" effluent SS and BOD it may be necessary to incorporate an algal removal technique to "polish" the WSP effluent. Such techniques can be "high tech" or "low tech" flocculation (with alum, 40-60 mg/l at pH 6-7, or with lime, 100-200 mg/l at pH 10-11) followed by sedimentation; coagulation and dissolved air flotation and intermittent sand filtration. Further details are given by Ellis (1983).

"Low tech" systems comprise of microstrainers, rock filters and grass plots, which are briefly described below, together with floating macrophyte ponds) and fish culture (See Effluent Reuse).

12.11.4 Microstrainers

Conventional microstrainers (for example those used to filter humus tank effluents)have not generally been successful with pond effluents as the algae settle out as the effluent flows through. The algae decompose releasing nutrients which are utilized by bacteria growing on the surface of the rocks. In addition to algal removal, significant ammonia removal may also take place through the activity of nitrifying bacteria growing on the surface of the filter medium.

Performance depends on loading rate, temperature and rock size and shape. Permissible loading increases with temperature, but in general an application rate of 1.0 m^3 of pond effluent per m^3 rock bed per day should be used. Rock size is important, as surface area for microbial film formation increases with decreasing rock size but, if the rocks are too small, then problems can occur with clogging. Rock size is normally 100 mm, with a bed depth of 1.5-2.0 m. The effluent should be introduced just below the surface layer because odour problems are sometimes encountered with cyanobacterial films developing on we surface rocks exposed to the light.

Construction costs are low and very little maintenance is required, although periodic cleaning to remove accumulated humus is necessary, but this can be carried out during the lowest. BOD and SS removals of 50 and 70 percent have been reported for maturation pond effluents in the USA (Middlebrooks, 1988).

12.11.5 Grass plots

These operate by allowing the effluent to flow across gently sloping grass plots so that the algae become trapped in the soil where they decompose, so releasing nutrients that are then utilized by the grass. Application rates are of the order 2,000 – 5,000 m^3/ha d over plots with a uniform slope of between 1 in 60 and 1 in 100. Even distribution of wastewater is important and castellated weirs or perforated pipes are usually used. Porous soils provide better treatment, and in clayey soils it may be necessary to install under drains to prevent water logging. The size and shape of plots depend on land availability, but at least two should be used so that each plot can be rested at intervals to allow re-aeration of the soil. Plots can be separated from each other by small earthen embankments which should contain an impermeable core.

As with rock filters, construction costs are relatively low and, aside from periodic grass cutting and cleaning of the distribution channels, very little maintenance is required. Reductions of SS and BOD from over 100 mg/l or below can be achieved. Details on North American experience are given by Witherow and Bledsoe (1983); generally suspended solids reductions of 50-60 percent were obtained with WSP effluents. As with rock filters, some additional nutrient removal also occurs through grass plots.

12.12 Special Pond Types

12.12.1 Septage and nightsoil ponds

Several WSP systems in Kenya receive septage (septic tank sludge) in addition to municipal wastewater. However, WSP can also be designed to receive solely septage or nightsoil (Shaw, 1963).

Septage pond systems should comprise several primary facultative ponds in parallel which are designed such that, in the coldest month, evaporative losses equal the inflow with make-up water being added in the other months as necessary (anaerobic ponds are of no purpose as septage is already highly mineralized). Their process design follows that for ordinary WSP, with a value of around 5,000 mg/l for the BOD of the septage (EPA, 1980). Nightsoil ponds are similarly designed, on the basis of a BOD contribution of 22 g per caput per day (Mara, 1976).

Septage and nightsoil ponds also differ in their physical design and operation. The inlet to the facultative pond (for septage) and the anaerobic pond (for nightsoil) requires, on the upstream side, a chute into which the tankers can discharge their load. In several such chores are normally provided and these are used in rotation. It is also sensible to recirculate pond water (or add river water while the nightsoil or septage is being introduced to facilitate its dispersion throughout the pond.

Septage and nightsoil ponds should be located near a river, so that river water can be used to wash out tankers, wash down the concrete apron around the inlet chutes, and to provide sufficient flow to balance evaporative losses. Design example no. 6 illustrates how the required flow is determined.

12.12.2 Partially aerated facultative ponds

The installation of floating aerators on overloaded facultative ponds can be a useful temporary measure to supply additional oxygen for waste stabilization until such time as the WSP system can be properly upgraded (Section 12.11.1). Partially aerated facultative ponds are a variant of aerated lagoons. Indeed aerated lagoons were originally developed from facultative ponds in northern United States where in winter the rate of oxygen supply by photosynthesis was too low for waste stabilization. Floating surface aerators were placed on the facultative pond to make up the oxygen deficit. It was noticiable, however, that within two weeks or so the algae disappeared and the microbial flora resembled that of activated sludge systems.

There are now two types of aerated lagoon:

(a) completely mixed aerated lagoons; these are essentially activated sludge units which receive raw wastewater and in which there is no sludge recirculation; and
(b) partially mixed aerated lagoons: these have a lower power input than completely mixed aerated lagoons such that not all the solids are kept in suspension; some settle and are digested anaerobically

The design of completely mixed aerated lagoons is given in standard texts (for example: Mara, 1976; Rich, 1980; Metcalf & Eddy Inc., 1991). The design of partially mixed aerated lagoons is presented below in a modified form suitable for determining the power requirement for a partially aerated facultative pond.

The basic design equation is (EPA, 1983):

$$L_e = L_i / (1 + k_{pm\,(T)}\, ø) \qquad \textbf{(12.29)}$$

Where; L_e = effluent BOD, mg/l
L_i = effluent BOD, mg/l
ø = retention time, d
$k_{pm\,(T)}$= first-order rate constant for BOD removal in partially mixed aerated lagoon at TºC, d and given by:

$$k_{pm\,(T)} = 0.276\,(1.036)^{T-20}$$

BOD removal in the partially aerated facultative pond should be 70 percent (i.e. the same as in a non-overloaded primary facultative pond). Thus $L_e = 0.3\ L_i$ and equations 12.29 and 12.30 can be combined as follows:

$$ø = 8.45\ (1.036)^{20-T} \qquad \textbf{(12.31)}$$

This value of ø is compared with the actual retention time in the overloaded facultative pond (V/Q). For the partially aerated facultative pond to function properly, ø must not be greater than (V/Q). (If ø is greater than (V/Q) the BOD removal will be less than 70 percent and can be determined from equation 8.29 with ø = (V/Q). Equation 8.33 below will need modifying to reflect the actual BOD removal.)

The floating aerators should supply 1.5 kg O_2 per kg BOD removed. Thus the oxygen requirement (R, kg/h) is given by:

$$R = 1.5 \times 10^{-3}(L_i - L_e)\ Q/24 \qquad \textbf{(12.32)}$$

Since L = 0.3L, equation 8.32 becomes:

$$R = 4.4 \times 10^{-5}\ L_iQ \qquad \textbf{(12.33)}$$

The aerators must supply R kg O_2/h: in partially aerated ponds no consideration is given to the power requirement for mixing. The aerator power requirement (P, kW) is given by:

$$P = R/N_f \qquad \textbf{(12.34)}$$

Where N = the aerators oxygen transfer rate under field conditions kg O_2/kWh

N is given by the equation:

$$N_f = N_o[\alpha]\ [(1.024)^{T-20}]\ [\beta C - C_1)/C_{s\ (20,\ 0)}] \qquad \textbf{(12.35)}$$

Where N_0 = the aerators oxygen transfer rate under standard test conditions, kg O_2/kWh

The standard test conditions are tap water as the test liquid at a temperature of 20ºC and with an initial dissolved oxygen concentration of zero. Thus N (which is the value quoted by the manufacturers) is converted to N by multiplying it by the three correction factors shown in equation 12.35. The first is to allow for the nature of the liquid to be aerated:

α = ratio of O_2 transfer in the wastewater to that in tap water (for domestic wastewater, α = 0.7) is the second ratio temperature correction factor. The temperature T is the temperature of the partially aerated pond, and may be estimated from the equation (Rich, 1980):

$$T = T_e + (T_o - T_e)\ \beta \qquad \textbf{(12.36)}$$

Where T = mean air temperature in the coldest month ºC
T_e = mean temperature of influent wastewater in the coldest month ºC.

The third term allows for the difference in the dissolved oxygen concentration in the partially aerated pond (= 2 mg/l for design purposes) and that the adopted in the standard test (zero), as follows:

$C_{s\ (T,\ A)}$ = O2 saturation concentration (solubility) in distilled water at temperature T and altitude

A. The values of C at sea level (760 mm Hg) are given in Table 12.15 for T = 15 - 30ºC. the correction for altitude is made by considering the mean air pressure (P mm Hg) at that altitude:

C = O_2 saturation concentration in distilled water at 20ºC and sea level (= 9.08 mg/l)
DO concentration in partially aerated pond (2 mg/l for design) = ratio of O_2 saturation concentration in the waste to that in distilled water (typically for domestic sewage, β=0.9).

Maturation floating surface aerators are rated at 2.4 kg O_2/kWh.

Table 12.15 Oxygen saturation concentration in dissolved water at sea level at various measures

Temperature (^{0}C)	Sat. Concentration (mg/L)
15	10.07
16	9.86
17	9.65
18	9.46
19	9.27
20	9.08
21	8.91
22	8.74
23	8.57
24	8.42
25	8.26
26	8.12
27	7.97
28	7.84
29	7.70
30	7.57

Source: Montgomery et al (1964)

12.12.3 Effluent storage reservoirs

Effluent storage reservoirs (ESR) are especially useful in arid and semi-arid areas. They were developed in Israel (see Juanico and Shelef, 1991) to store the effluent from WSP systems during the period (8 months in Israel) when it is not rquired for irrigation. It is thus a method of conserving effluent so that, during the irrigation season, a greater area of land can be irrigated. Current Israeli practice is to treat the wastewater in an anaerobic pond and discharge its effluent into a single 5 - 10 m deep ESR with an 8 month retention time. This is perfectly satisfactory, as the ESR effluent is only used to irrigate cotton (i.e. restricted irrigation: see Chapter on Water Reuse) and so this usage complies with the WHO guidelines for restricted irrigation since any helminth eggs settle out in the anaerobic pond and ESR.

If the ESR effluent is to be used for unrestricted irrigation (i.e. of crops eaten raw), there it should contain < 1000 FC per100 ml which the above single ESR cannot achieve. Instead several sequential batch-fed ESR in parallel are required (Mara and Pearson, 1992). These are operated on a cycle of fill, rest and use, with FC die-off to < 1000 per periods. Strategies can be readily developed for irrigation seasons of different lengths.

12.12.4 Floating macrophyte ponds

Floating macrophyte ponds contain plants that float on the water with their aerial rosette of leaves close to the surface and their fibrous root systems hanging down into the pond water column to absorb nutrients. Several genera are used, including *Salvinia, Pistia, Lemna and Eichhornia*. Most shade out algae efficiently, but the larger species with their correspondingly larger root systems are considerably more efficient at nutrient stripping. *Elchhornia crassipes* (water hyacinth) has been studied in most detail, and it would seem that *Elchhornia* ponds receiving effluent from a facultative pond can be loaded at rates of up to 40 kg BOD/ha d; at higher loadings, odours develop at night.

The breeding of culicine mosquitoes is also a problem in floating macrophyte ponds, but this can be controlled by the introduction of larva-eating fish such as *Gumbusia* and *Peocelia*. Macrophytes such as water hyacinth, which trap clean water in their shaded leaf axils above the pond water surface, also encourage the breeding of clean water mosquitoes, and in this case the larvae are safe from fish predation.

There are other factors that require careful consideration when using floating macrophyte ponds. With some macrophytes, water loss via evaportranspiration from the leaf surfaces can be greater than evaporation from the free surface of an ordinary pond. Dissolved oxygen concentration in the pond water column during the day is also very much lower than in conventional ponds, because the photosynthetic oxygen produced by the macrophyte leaves is lost directly to the atmosphere. Furthermore, the use of nonindigenous macrophytes is totally unacceptable because, if they escape from the ponds, as has happened with Eichhornia, they may have a highly deleterious effect on the ecology and quality of the local environment.

No design for floating macrophyte ponds has been evaluated sufficiently to confirm its long-term efficiency of operation. Information to date suggests that floating macrophyte ponds require considerably more maintenance than conventional ponds; otherwise, effluent quality is poor. In particular, due to the lower pH and decreased light intensity, pathogen removal is very much less than in algal maturation ponds. Thus, floating macrophyte ponds should only be considered for use as a treatment technology when the waste is microbiologically safe, or as an additional process subsequent to conventional maturation ponds, and then only when a high degree of algal or nutrient removal is necessitated by the ecology of the receiving watercourse.

12.12.5 High-rate algal ponds

High-rate algal ponds (HRAP) are very specialized waste stabilisation ponds that are primarily designed to maximize the production of algae, using wastewater as a feed (substrate). It therefore follows that some economic use for the algal product is implicit in the decision to implement HRAP technology. Yields of up to 160,000 kg of algae (dry weight) per ha per year have been reported. This is equivalent to 80,000 kg of protein per ha per year, which is far in excess of that achieved by conventional agriculutre. This is one of the reasons for the apparent economic attractiveness of these ponds; yet even after more than 20 years of research few full scale systems are in operation.

An HRAP usually takes the form of a shallow channel 2-3 m wide with a water depth of 20 - 60 cm and arranged in a "race track" configuration. To prevent the algae settling out, the pond is mixed by stirring, either continuously or at regular intervals, with paddles located along its length. Detention times are between two and six days and are therefore frequently much sorter than those in conventional pond systems. The shallow depths of HRAP and their short retention times make them more sensitive to changes in environmental conditions and shock loads. Their short retention times may appear to offer a significant reduction in land area requirements, but this offset by their shallow depth and low removal of excreted pathogens, which may require the use of maturation ponds to produce a microbiologically satisfactory effluent.

The influent wastewater is pretreated by primary sedimentation or in an anaerobic pond to remove settleable solids. HRAP can be heavily loaded with wastewater, up to 350 kg BOD per ha per day in the tropics, and still produce an effluent with < 20 mg/l of filtered BOD. Since these ponds are designed to maximize algal biomass production, it follows that efficient harvesting of the algae is crucial to the economic viability of the system and to the economic viability of the system and to the quality of the final effluent. Harvesting techniques that have been used include centrifugation, mechanical filtration, autoflocculation and chemical flocculation, followed by dissolved air floatation (Ellis 1983).

Reduced algal yields have been associated with predation by zooplankton, wuch as Daphnia and Moinia, and as a consequence of fungal infections. Correction of such problems is possible but requires a high degree of microbiological competence. Careful manipulation of the performance of HRAP may also be necessary in an attempt to control algal speciation in instances where a particular type of algal product is required.

In summary, HRAP are highly sensitive biological reactors that require very careful control and maintenance by skilled personnel. They are much more complicated to operate than activated sludge systems, and they cannot be considered as a simple alternative to conventional pond systems. Thus their use should only be contemplated when the necessarily well trained and experienced technical staffs are routinely available and when there is a definite market for the algal product.

12.13 WSP Design Examples

1. ***Constant Population***

Design a WSP system to treat 10,000 m³/d of a wastewater which has a BOD of 350mg/l and 1 x 10^8 FC per 100 ml. The effluent should contain no more than 1000 FC per 100 ml, and the design temperature and evaporation rate are 18ºC and 6 mm/d, respectively.

Solution

(a) Anaerobic ponds

From Table 12.8 the design loading is given by:

$\lambda_v = 20T - 100 = (20 \times 18) - 100 = 260 g/m^3d$

The pond volume is given by equation 12.2 as:

$V_a = L_iQ/ \lambda_v = 350 \times 10{,}000/260 = 13{,}462 \ m^3$

The retention time is given by equation 12.3 as:

$\theta a = V_a/Q = 13{,}462/10000 = 1.35$ d

The BOD removal is given in Table 8.8 as:

R = 2T + 20 = (2 x 18) + 20 = 56 percent

(b) Facultative ponds

The design loading is given by equation 12.8 as:

$\lambda_s = 350 (1.107 - 0.002T)^{T-25} = 350 [1.107 - (0.002 \times 18)]^{18-25} = 216$ kg/ha d

Thus the area is given by equation 12.4 as:

$A_f = 10L_1Q/ \lambda_s = 10 \times 0.44 \times 350 \times 10{,}000/216 = 71{,}300 \ m^2$

The retention time is given by equation 12.11 as:

$\theta_f = 2A_fD/(2Q - 0.001 A_fe)$

Taking a depth of 1.5m, this becomes:

$\theta_f = 2 \times 71{,}300 \times 1.5/[(2 \times 10{,}000) - (0.001 \times 71{,}300 \times 6)] = 10.9$ d

The effluent flow is given by:

$Q_e = Q_i - 0.001A_fe = 10.000 - (0.001 \times 71{,}300 \times 6) = 9572 \ m^3/d$

(c) Maturation ponds

For 18ºC the value of k_T is given by equation 8.14 as:

$k_T = 2.6 (1.19)^{T-20} = 2.6 (1.19)^{-2} = 1.84 \ d^{-1}$

Equation 12.13 can be rearranged as follows:

$Øm = \{[Ni/Ne\ (1 + k_T \cancel{Ø}_a)\ (1 + k_T Ø_f)^{1/n} - 1\}/k_T$

$= \{[10^8/10^3(1 + (1.84 \times 1.35)\ (1 + (1.84 \times 10.9)] - 1\}/1.84$

= 740 d for n = 1
= 19.5 d for n = 2
= 5.5 d for n = 3
= 2.8 d for n = 4

The first two combinations of $Ø_{10}$ and n are rejected as $0_m > 0_r$. The fourth combination is also rejected as $0_m < Ø_m^{min}$= (3 d). A comparison is made between the third combination and that of $Ø_m = Ø_m^{min}$ = 3 d and n = 4: the latter has a small product (12) than the former (16.5), and is therefore chosen.

Check the loading on the first maturation pond from equation 12.16:
$\lambda s\ (^{ml})$ = 10 x 0.3 x 350 x 1.5/3
= 525 kg/ha d

This value is higher than 75 percent of the load on the facultative pond (= 0.75 x 216, = (162 kg/ha d). Thus $\lambda s(^{ml})$ is taken as 162kg/ha d and $Ø_{ml}$ calculated from:

$Ø_{ml} = 10\ LiD/\ \lambda s_{ml}$ = 10 x 0.3 x 350 x 1.5/162 = 9.7 d

The retention times in the subsequent maturation ponds are calculated from:
$Ø_m = \{[Ni/Ne\ (1 + k_T\ Ø_a)\ (1 + k_T\ Ø_f)\ (1 + + k_T\ Ø_{ml})^{1/n} - 1\}/k_T$

$= \{[10^8/10^3(1 + (1.84 \times 1.35)\ (1 + (1.84 \times 10.9)][1.84 \times 9.7]\}^{1/n}/1.84$
= 39 d for n = 1
= 4.1 d for n = 2
= 1.7 d for n = 3

The second combination is chosen as its product is 8.2, which is less than that for $Ø_m = Ø_m^{min}$ = 3 d and n = 3.

For a depth of 1.5m, the area of the first maturation pond is given by equation 12.17 as:

For a depth of 1.5 m, the area of the first maturation pond is given by equation 12.17 as:

$A_{ml} = 2Q_1\ Ø_m/(2D + 0.001_e\ Ø_m)$ = 2 x 9572 x 9.7/[(2 x 1.5) + (0.001 x 6 x 9.7)] = 60,721 m^2

The effluent flow is given by

$Q_e = Q_i - 0.001A_{ml}e$ = 9572 - (0.001 x 60,721 x 6) = 9208 m^3/d

Similarly the area of the second maturation pond and its effluent flow are given by

A_{m2} = 2 x 9208 x 4.1/[(2 x 1.5) + (0.001 x 6 x 4.1)] = 24,964 m^2

$Q_{e\,=}$ 9208 - (0.001 x 24,964 x 6) = 9058 m^3/d

And for the third maturation pond:
A_{m3} = 2 x 9058 x 4.1/ [(2 x 1.5) + (0.001 x 6 x 4.1)] = 24,557 m^2

Q = 9058 - (0.001 x 24,557 x 6) = 8911 m^3/d

BOD Removal

Assuming that a cumulative removal of filtered BOD of 90 percent in the anaerobic and facultative ponds and 25 percent in each of the three maturation ponds, the final effluent will have a filtered (non-algal) BOD of:

350 x 0.1 x 0.75 x 0.75 = 15 mg/l, which is satisfactory

Summary

The design thus comprises:

Anaerobic pond(s)	: volume	13,462 m³
	Retention time	1.35 d
Facultative pond(s)	: area	71,300 m³
	Retention time	10.9d
First maturation pond(s)	: area	60,721 m²
	Retention time	9.7 d
Second maturation pond(s)	: area	24,964 m²
	Retention time	4.1 d
Third maturation pond(s)	: area	24,557 m²
	Retention time	4.1 d

The overall retention time is thus 30.14 days, and the removal filtered BOD and FC throughout the pond series is as follows:

	BOD (mg/l)	FC (per 100 ml)
Raw wastewater	350*	1.0×10^8
Anaerobic pond effluent	154*	2.9×10^7
Facultative pond effluent	35	1.4×10^6
1st maturation pond effluent	26	7.2×10^4
2nd maturation pond effluent	20	8.5×10^3
3rd maturation pond effluent	15	9.9×10^2

*unfiltered BOD

The effluent flow is 8911 m^3/d, so evaporative losses are 10.9 percent.

Note

If the above design were done without anaerobic ponds, the result would be a primary facultative pond and four maturation ponds, as follows:

Facultative pond(s)	: area	162,037 m²
	Retention time	25.5 d
First maturation pond(s)	: area	57,270 m²
	Retention time	9.7 d
Second maturation pond(s)	: area	17,263 m²

	Retention time	3 d
Third maturation pond(s)	: area	17,057 m²
	Retention time	3 d
Fourth maturation pond(s) : area		16,854 m²
	Retention time	3 d

The overall retention time is thus 44.2 days, which is 46.6 percent greater than when anaerobic ponds are included. Evaporative losses are 16.2 percent, which is 48.6 percent more than the design with anaerobic ponds. This clearly shows the advantages of including anaerobic ponds: they substantially reduce retention times, and thus land area requirements, and also losses due to evaporation (which is important if the effluent is to be used for crop irrigation).

2. Seasonally Varying Population

The design is to be the same as for design example No. 1 but, due to seasonal effects of tourism, the cool and hot season flows, temperatures and evaporation rates are as follows:-

Cool	:	$10{,}000 m^3/d$	18^0C	6mm/d
Hot Season	:	$30{,}000 m^3/d$	28^0C	11mm/d

(a) Anaerobic Ponds

Design the anaerobic pond for the hot season and compare this with the cool season design example no.1.

For 28^0C, $\lambda_v = 300 g/m^3 d$. Therefore:

V_a =350x30, 000/300=35,000m^3

This V_a is higher than that for 18^0c, and is therefore chosen

$Ø_a$ =35,000/30,000 =1.17d (hot season)

=35,000/10,000 =3.5d (cool season)

λ_v =350x10,000/35,000 =100g/m^3d (Cool Season) which is satisfactory.

(b) Facultative Ponds

For 28^0C, the permissible surface BOD loading is given by:

$\lambda_a = 350[1.107-(0.002x28)]^{28-25} = 406 kg/ha\ d$

The BOD removal in the anaerobic pond is 60 percent at 28^0C. Therefore

A_f =10x0.4x350x30, 000/406 =103,448m^2

This is greater than that required in the cool season, and is therefore chosen

$Ø_f$ =2x103, 448x1.5/ [(2x30, 000) - (0.001x103, 448x11)] =5.3 d (hot season)

=2x103, 448x1.5/ [(2x10, 000) – (0.001x103, 448x6)]=16.0 d (cool season)

Q_e = 30,000-(0.001x103, 448x11) =28,862 (hot Season)

=10,000 - (0.001x103, 448x6) =9,379 m^3/day (Cool Season)

c) Maturation ponds

These will be designed first for the hot season and then checked for performance in the cool season. At 28ºC, k_T is given by:

$k_T = 2.6\ (1.19)^{28-20} = 10.5d^{-1}$

$Ø_m = \{[10^8/10^3(1 + (10.5 \times 1.17)(1 + (10.5 \times 5.3)]^{l/n} - 1\}/10.5$

= 12.6 for n = 1
= 1.0 for n = 2

Choose 2 ponds each with a retention time of 3 d ($=Ø_m^{min}$) and calculate the loading on the first:

$\lambda_{(ml)}$ = 10 x 0.3 x 350 x 1.5/3 = 525 kg/ha d which is higher than (0.75 x 406), = 304 kg/ha d. Thus the retention time in the first maturation pond is:
$Ø_{ml}$ =10x0.3x350x1.5/304 =5.2 d

The retention time in the subsequent maturation pond (s) is:

$Ø_m = \{[10^8/10^3(1 + (10.5 \times 1.17)(1 + (10.5 \times 5.3)(1+(10.5 \times 5.2)))]^{l/n} - 1\}/10.5$

= 0.13 d for n = 1
So choose a single pond with a 3d retention time.

Check performance in the cool season (when the retention time in the maturation ponds will be three times greater):

$N = 10^8 / [(1 + (1.84 \times 3.5)(1 + (1.84 \times 16)(1 + (1.84 \times 15.6)(1 + (1.84 \times 9)]$

= 847 per 100 ml, which is satisfactory.

The maturation pond areas are now calculated:

A_{ml} = 2 x 28,862 x 5.2/ [(2 x 1.5) + (0.001 x 11 x 5.2)] = 98,183 m²

Q_e = 28,862 – (0.001 x 98,183 x 11) = 27,782 m²/d

A_{m2} = 2 x 27,782 x 3/[(2 x 1.5) + (0.001 x 11 x 3)] = 54,960 m²

Q_e = 27,782 – (0.001 x 54,960 x 11) = 27,177 m^3/d

Calculate actual retention times in the cool season:

$Ø_{ml}$ = 98,183 x 1.5/9,379 = 15.7 d

Q_e = 9,379 – (0.001 x 98,183 x 6) = 8790 m^3/d

$Ø_{m2}$ = 54,960 x 1.5/8790 = 9.4 d

Q_e = 8,790 – (0.001 x 54,960 x 6) = 8,460 m^3/d

These retention times are slightly greater than those assumed above in the check on the cool season performance. So N_e will be slightly less than 847 per 100 ml, but still of course satisfactory.

Summary

The design thus comprises:

Anaerobic pond(s)	: volume	35,000 m³
	Retention time	1.17 d (3.5 d)*

Facultative pond(s)	: area		103,448 m^2
	Retention time	5.3 d (16.0d)	
First maturation pond(s) : area		98,183 m^2	
	Retention time	5.2 d (15.7 d)	
Second maturation pond(s)	: area		54,960 m^2
	Retention time	3 d (9.4 d)	

- Cool season retention times given in parentheses.

Thus the overall retention time is 14.7 d in the hot season and 44.6 d in the cool season.

3. Unrestricted Irrigation

The design is to be the same as design example no. 1, except that the effluent is to be used for unrestricted irrigation only in those months in which the temperature is above 25ºC (when the evaporation rate is 9 mm/d).

Solution

(a) Anaerobic and facultative ponds

These must operate satisfactorily at all times and therefore they have to be designed for 18ºC; so they are as in design example no. 1. The retention time in the facultative pond when the evaporation rate is 9 mm/d is given by:

$Ø_f$ = 2 x 71,300 x 1.5/[(2 x 10,000) – (0.001 x 71,300 x 9)] = 11.0 d and the corresponding effluent flow from the facultative pond is:

Q_e = 10,000 – (0.001 x 71,300 x 9) = 9358 m^3/d

(b) Maturation ponds

The value of kT at 25ºC = 2.6 $(1.19)^{25-20}$ = 6.20 d^{-1}

The retention time is:

$Ø_m = \{[10^8/10^3(1 + (6.2 \times 1.35)(1 + (6.2 \times 11)]^{l/n} - 1\}/6.2$
= 24.7 d for n = 1
= 1.8 d for n = 2

Choose 2 ponds each with 0 each with $Ø_m$ = 3 (=$Ø_m^{min}$) and check loading on the first pond:

λ_{ml} = 10 x 0.3 x 350 x 1.5/3 = 1675 kg/ha d

This is greater than 75 percent of the permissible loading on the facultative, which at 25ºC is (0.75 x 350), = 262 kg/ha d. Thus the retention time in the first maturation pond is calculated from:

$Ø_{ml}$ =10x0.3x350x1.5/262 =6.0 d

The retention time in the subsequent maturation pond (s) is:
$Ø_{ml} = \{[10^8/10^3(1 + (6.2 \times 1.35)(1 + (6.2 \times 11)(1 + (6.2 \times 6)]^{l/n} - 1\}/6.2$

= 0.5 d for n = 1

So choose a single secondary maturation pond with a retention time of 3 d (=$Ø_m^{min}$)

The maturation pond areas are calculated as follows:

A_{ml} = 2 x 9,358 x 6/[(2 x 1.5) + (0.001 x 9 x 6)] = 36,770 m²
Q_e = 9,358 = (0.001 x 36,770 x 9) = 9,027 m^3/d

A_{m2} = 2 x 9,027 x 3/ [(2 x 1.5) + (0.001 x 9 x 3)] = 17,893 m²

Q_e = 9,027 – (0.001 x 17,893 x 9) = 8,866 m^3/d

Summary

The design thus comprises:

Anaerobic pond(s)	: volume	13,462 m³
	Retention time	1.35 d
Facultative pond(s)	: area	71,300 m^2
	Retention time	11 d
First maturation pond(s)	: area	36,770 m²
	Retention time	6 d
Second maturation pond(s)	: area	17,893 m²
	Retention time	3 d
Third maturation pond(s)	: area	54,960 m²
	Retention time	3 d (9.4 d)

Thus the overall retention time is 21.35 d, which is 30 percent less than in design example no. 1. This illustrates the advantage of designing pond systems to produce an effluent containing >1000 FC per 100 ml only in those months when it will be actually used for unrestricted crop irrigation.

4. Restricted Irrigation

The design is to be the same as design example no.3, except that the effluent is to be used for restricted irrigation. The raw wastewater contains 600 human intestinal nematode eggs per litre.

Solution

(a) Anaerobic pond

As in design example no.3, $Ø_a$, will be 1.35 d. For this retention time, the percentage egg removal is given by equation 8.19 as:

$R = 100[1 – 0.41 \exp (0.49Ø_a + 0.0085\ Ø_a^{2})]$

= 100[1 – 0.41 exp (-(0.49 x 1.35) + (0.0085 x 1.8225)] = 78.5 percent

So the number of eggs per litre of anaerobic pond effluent is (600 x 0.215), = 129

(b) Facultative pond

$Ø_f$ = 10.9 d, so R is given by:

R = 100[1 – 0.41 exp(-(0.49 x 10.9) + (0.0085 x 118.81)] = 99.4 percent

So the number of eggs per litre of facultative pond effluent is (0.006 x 129), = 0.8. Thus no further treatment is necessary as the facultative pond effluent complies with the WHO guideline value of > 1 egg per litre

Suppose, however, that the facultative pond effluent contained 10 eggs per litre. A maturation pond would then be required to reduce the number to 1. This is equivalent to an egg removal of 90 percent. From Table 12.8 this requires a retention time of 3.1 days, and so a single maturation pond of this size would be provided.

5. Partially aerated facultative pond

The WSP system designed in design example no 1 is now receiving twice its design flow. The anaerobic ponds are to be duplicated and floating aerators installed on the facultative pond. What is the required aerator power input? The temperature of the anaerobic pond effluent in the coldest month is 24ºC.

Solution

The extended anaerobic ponds will achieve the same BOD removal as before (56 percent). So the influent to the facultative ponds still has a BOD of 154 mg/l.

The current retention time in the facultative ponds is half that in the original design, e.g. 5.45 d.

The temperature T is the temperature of the partially aerated pond and is given by equation 12.36 as:

$T = T_a + (T_o – T_a)/3 = 18 + (24 – 18)/3 = 20ºC$.

Use equation 8.31 to calculate required retention time in the partially aerated facultative pond for 70 percent BOD removal:

$Ø_l = 8.45\ (1.036)^{20-T}$

For T - 20ºC, Ø = 8.45 d. This is greater than what is available (5.45 d) and so the BOD removal is given by equation 8.29 and 8.30:

$L_e = L_i\ (1 + k_{pm\ (T)}\ Ø^{)} = 154/\ [1 + (0.276 \times 5.45)] = 62$ mg/l

Thus the BOD removal is 60 percent and the oxygen requirement is given by equation 8.32 as:

$R = 1.5 \times 10^{-3}\ (L_i – L_e)\ Q/24 = 1.5 \times 10^{-3}\ (154 – 62)\ 20{,}000/24 = 115$ kg/hr

Assume that the manufacturer's rating for the aerators (no.) is 2 kg O_2/kWh. This is corrected for filed conditions by equation 8.35 (assuming the pond system is at sea level):

$N_f \quad = N_o[x]\ [(1.024)^{T-20}]\ [(BC_{s\ (T,\ A)} - C_1)/C_{s\ (20,\ 0)}$

$= 2 \times 0.7 \times 1\ \{[(0.9 \times 9.08) – 2]/9.08\} = 0.95$ kg 0_2 kWh

The aerator power requirement (P) is given by equation 12.34 as:

$P = R/N_f = 115/0.95 = 121$ kW

As the area of the facultative ponds is large (71,300 $m^{2;}$ say, for four ponds each of 17,825 m^2 or 77 m x 231 m), twelve 10 kW aerators would be used (three per pond).

6. Septage ponds

Design a WSP system to receive septage from fifty 5000 litre tankers a day. The septage BOD and FC count are 5000 mg/l and 1 x 10 per 100 ml respectively, and the design temperature and evaporation rate are 20ºC and 6 mm/d.

Solution

Facultative pond

The daily BOD load (L, kg/d) is calculated from:

$L = 10^{-3}L\,Q = 10^{-3} \times 5000 \times (50 \times 5000 \times 10^{-3})$

= 1250 kg/d

The permissible BOD surface loading at 20ºC is 350 kg/ha d, so the facultative pond area (A_f, ha) is given by:

$A_f = 1250/350 = 3.50$ ha.

This area is now compared with that calculated on the basis of evaporative losses equaling the septage inflow:

$0.001A_fe = Q$

So that:

$A_f = 250/(0.001 \times 6) = 41{,}700\ m^2$

This is greater than that based on the permissible loading and so adopted. In practice one would have say, four ponds in parallel. Make up water is added to each pond in those months when there is positive net evaporation (0, mm/d) at the following rate (Q, m^3/d):

$Q_m = (0.001 \times 10, 425 \times e) – (250/4)$

13.0 CONSTRUCTED WETLAND

13.1 Introduction

A **constructed wetland** is an artificial marsh or swamp, created for anthropogenic discharge such as wastewater, stormwater runoff or sewage treatment, and as habitat for wildlife, or for land reclamation after mining or other disturbance. Natural wetlands act as biofilters, removing sediments and pollutants such as heavy metals from the water, and constructed wetlands can be designed to emulate these features.

Photo 13.1 Vertical Flow Constructed Wetlands

Vegetation in a wetland provides a substrate (roots, stems, and leaves) upon which microorganisms can grow as they break down organic materials. This community of microorganisms is known as the periphyton. The periphyton and natural chemical processes are responsible for approximately 90 percent of pollutant removal and waste breakdown. The plants remove about seven to ten percent of pollutants, and act as a carbon source for the microbes when they decay. Different species of aquatic plants have different rates of heavy metal uptake, a consideration for plant selection in a constructed wetland used for water treatment.

Constructed wetlands are of two basic types: subsurface-flow and surface-flow wetlands. Subsurface-flow wetlands can be further classified as horizontal flow and vertical flow constructed wetlands. Subsurface-flow wetlands move effluent (agricultural or mining runoff, tannery or meat processing wastes, wastewater from sewage or storm drains, or other water to be cleansed) through a gravel or sand medium on which plants are rooted; surface-flow wetlands move effluent above the soil in a planted marsh or swamp, and thus can be supported by a wider variety of soil types including bay mud and other silty clays. In subsurface-flow systems, the effluent may move either horizontally, parallel to the surface, or vertically, from the planted layer down through the substrate and out. Subsurface horizontal-flow wetlands are less hospitable to mosquitoes, whose populations can be a problem in constructed wetlands (carnivorous plants have been used to address this problem). Subsurface-flow systems have the advantage of requiring less land area for water treatment, but are not generally as suitable for wildlife habitat as are surface-flow constructed wetlands.

Photo 13.2 Newly Planted Constructed Wetland

Plantings of reedbeds are popular in European constructed wetlands, and plants such as cattails (*Typha* spp.), sedges, and bulrushes are used worldwide. Recent research in use of constructed wetlands for subarctic regions has shown that buckbeans (*Menyanthes trifoliata*) and pendant grass (*Arctophila fulva*) are also useful for metals uptake.

In Kenya, there are four main emergent macrophytes types, which are used in constructed wetland viz:-

- Cyperus Papyrus
- Cyperus immensus
- Typha domingensis
- Typha domingensis and
- Phragmites mauritianus

Photo 13.3 The same constructed wetland two years later

13.1.1 General

Constructed wetlands (CWs) are planned systems designed and constructed to employ wetland vegetation to assist in treating wastewater in a more controlled environment than occurs in natural wetlands. Hammer (1990) defines constructed wetlands as a designed, manmade complex of saturated substrate, emergent and submerged vegetation, animal life, and water that simulate wetlands for human uses and benefits. Constructed wetlands are an "eco-friendly" alternative for secondary and tertiary municipal and industrial wastewater treatment. The pollutants removed by CW's include organic materials, suspended solids, nutrients, pathogens, heavy metals and other toxic or hazardous pollutants. In municipal applications, they can follow traditional sewage treatment processes. Different types of constructed wetlands can effectively treat primary, secondary or tertiary treated sewage. However wetlands should not be used to treat raw sewage and, in industrial situations, the wastes may need to be pre-treated so that the biological elements of the wetlands can function effectively with the effluent. CW's are practical alternatives to conventional treatment of domestic sewage, industrial and agricultural wastes, storm water runoff, and acid mining drainage.

13.1.2 Types of Constructed Wetlands

Constructed wetlands for wastewater treatment can be categorized as either Free Water Surface (FWS) or Subsurface Flow (SSF) systems. In FWS systems, the flow of water is above the ground, and plants are rooted in the sediment layer at the base of water column (Figure 13.1).

In SSF systems, water flows though a porous media such as gravels or aggregates, in which the plants are, rooted (Figure 9.2). Table 13.1 illustrates the type of wetlands, vegetation types and wetlands.

Fig. 13.1 Emergent macrophyte treatment system with horizontal sub-surface flow (Brix 1993)

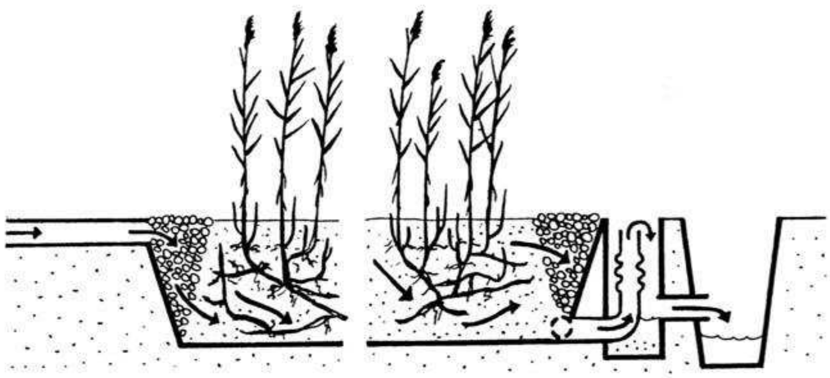


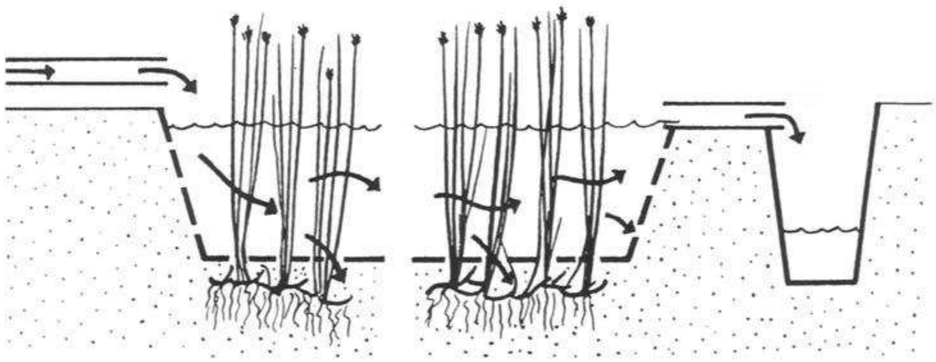
Table 13.1 Vegetation type and water column contact in constructed wetlands

Constructed wetland type	Type of vegetation	Section in contact with water column
Free water surface (FWS)	Emergent	Stem, limited leaf contact
	Floating	Root zone, some stem / tubers
	Submerged	Photosynthetic part, possibly root zone
Sub-surface flow (SSF)	Emergent	Rhizome and root zone

SSF systems are most appropriate for treating primary wastewater, because there is no direct contact between the water column and the atmosphere. There is no opportunity for vermin to breed, and the system is safer from a public health perspective. The system is particularly useful for treating septic tank effluent or grey water, landfill leachate and other wastes that require removal of high concentrations organic materials, suspended solids, nitrate, pathogens and other pollutants. The environment within the SSF bed is mostly either anoxic or anaerobic. Oxygen is supplied by the roots of the emergent plants and is used up in the Biofilm growing directly on the roots and rhizomes, being unlikely to penetrate very far into the water column itself. SSF systems are good for nitrate removal (denitrification), but not for ammonia oxidation (nitrification), since oxygen availability is the limiting step in nitrification.

There are two types of SSF systems: horizontal flow SSF (hSSF) and vertical flow SSF (vSSF). The most common problem with hSSF is blockage, particularly around the inlet zone, leading either to short circuiting, surface flow or both. This occurs because of poor hydraulic design, insufficient flow distribution at the inlet, and inappropriate choice of porous media for the inlet zone. Properly-designed SSF systems are very reliable.

Fig. 13.2 Emergent macrophytes treatment system with surface flow (Brix, 1993)



FWS systems are very appropriate for polishing secondary and tertiary effluents, and for providing habitat. The environment in the FWS systems is generally aerobic at, and near, the surface, tending toward anoxic conditions near the bottom sediment. The microbial film grows on all available plant surfaces, and is the main mechanism of pollutant removal. FWS usually exhibits more biodiversity than does SSF systems.

The objective of using CWs is to remove organic matter, suspended solids, pathogenic organisms, and nutrients such as ammonia and other forms of nitrogen and phosphorus. The growing interest in wetland system is due in part to recognition that natural systems offer advantages over conventional activated sludge and trickling filter systems. When the same biochemical and physical processes occur in a more natural environment, instead of reactor tanks and basins, the resulting system often consumes less energy, is more reliable, requires less operation and maintenance and, as a result costs less. They also are used for removing heavy Metals and toxic compounds. This manual is concerned with the design, operation and Maintenance of sub-surface flow constructed wetlands.

13.1.3 Advantages and disadvantages of constructed wetlands

Constructed wetlands are

(1.) relatively inexpensive to construct and operate,
(2.) easy to Maintain,
(3.) provide effective and reliable wastewater treatment,
(4.) relatively tolerant of fluctuating hydrologic and contaminant loading rates (optimal size for anticipated waste load), and
(5.) provide indirect benefits such as green space, wildlife habitats and recreational and Educational areas

The disadvantages are;

(1.) the land requirements (cost and availability of Suitable land),
(2.) current imprecise design and operation criteria,
(3.) biological and hydrological Complexity and our lack of understanding of important process dynamics,
(4.) the costs of gravel or other fills, and site grading during the construction period, and
(5.) possible problems with pests; mosquitoes and other pests could be a problem for an improperly designed and managed SSF

The system may be used for small communities and, therefore, may be located close to the users. The dependence of wetland community on hydrologic patterns is most obvious in the change in species composition resulting from alterations in water depths and flows.

13.1.4 Configuration, Zones and components of constructed wetlands

Influents to constructed wetland can range from raw wastewater to secondary effluents.

Most constructed wetlands have the following zones: inlet zone, macrophyte zone, and littoral zone and outlet zones. The components associated in each zones are as shown in Table 13.2 and can include substrates with various rates of hydraulic conductivity, plants, a water column, invertebrate and vertebrates, and an aerobic and anaerobic microbial population. The water flow is maintained approximately 15 – 30 cm below the bed surface. Plants in wastewater systems have been viewed as nutrient storage compartments where nutrient uptake is related to plant growth and production. Harvesting before senescence may permanently remove nutrients from the systems. Within the water column, the stems and roots of wetland plants significantly provide the surface area for the attachment of microbial population. Wetland plants have the ability to transport atmospheric oxygen and other gases down into the root to the water column. Most media used include crushed stones, gravels, and different soils, either alone or in combination. Most beds are underlain by impermeable materials to prevent water seepage and assure water level control. Wastewater flows laterally, being purified during contact with media surface and vegetation roots. The sub-surface zone is saturated and generally anaerobic, although excess DO conveyed through the plant root system supports aerobic micro sites adjacent to the root and rhizomes.

Table 13.2 Wetland zones and their associated components

Zones	Components	Functions
Inlet zone	Inlet structure, splitter box	Flow distribution across the full width at a minimum of 3 – 5 m interval
Macrophyte zone	Porous bed/substrate, open water, vegetation, island, mixing baffles, flow diversion	To provide the substrate with high hydraulic conductivity; to provide surface for the growth of Biofilm; to aid in the removal of fine particles by sedimentation or filtration; to provide suitable support for the development of extensive root and rhizome System for emergent plants.
Deep water zone	Usually deeper, non-vegetated	Reduce short circuiting by re-orienting flow path; reduce stagnant areas by allowing for mixing by wind; enable UV disinfections of bacteria and other Pathogens; provide habitat for waterfowl.
Littoral zone	Littoral area	Littoral vegetation protects embankment from erosion; littoral vegetation serves to break up wave Action.
Outlet zone	Collection devices, spillway, weir, outlet structures	Control the depth of the water in the wetland; collect the effluent water without creating of dead zones in the wetlands; provide access for sampling and flow monitoring.

13.2 Planning for Constructed Wetland

13.2.1 Viability

The following criteria dictate the feasibility of using a constructed wetland for stormwater and Tertiary treatment of Municipal Seffluent:

1) The type of wetland designed and its characteristics;
2) The hydrologic characteristics of the designed wetland;
3) The vegetation planted within the wetland (to utilize and lower nutrients and pollutants);
4) The type and volume of nutrients and pollutants entering the wetland prior to treatment; and
5) soil texture

13.2.2 Planning for Constructed Wetland

Before Construction of a Constructed Wetland, the following needs attention.

13.2.2.1Soil/Topography/Climate

Soil at the site proposed for a created wetland must be suitable to allow for sufficient water retention, infiltration and wetland plant growth. For wetland vegetation, soils must be suitable, from the ground surface to below the static water level. It may be necessary to stockpile topsoil during construction and later overlay it along the wetland bottom and side slopes.

The topography of the site proposed for a created wetland must also be considered. Steep side slopes surrounding the wetland should be avoided since they will deter the growth of wetland vegetation, which in turn increases problems with harvesting and maintenance problems (which may raise potential safety concerns). Minimal excavation is preferred to reduce constructions costs and to produce a more natural looking wetland.

It is also important to know the location of the water table. This information will aid in designing areas that will have standing water.

Climate may be a factor if the wetland will receive large amounts of storm water or effluents from Municipal sewerage works during the winter months. Shallow wetland zones may freeze solid in the northern temperate area, thereby decreasing the overall effectiveness of the wetland. The lack of vegetation during the winter will also lower the amount of nutrients and pollutants that can be assimilated into plant tissues. Without aquatic vegetation, the sediment may move through the wetland quickly unless the detention time is long enough for the particles to settle out. Therefore, if the wetland will receive large amounts of
Storm water during freezing weather, it may be necessary to provide deep pools that will not freeze solid.

13.2.2.2Where to Apply

Apply in areas where nutrients and sediment are the primary pollutants of concern. Pre-treatment of toxic contaminants must be assured.

There are some locations where wetlands were historically drained for agricultural and other purposes and may no longer meet the scientific and legal definition of a wetland; these sites may provide an excellent opportunity for the re-establishment of wetland habitat for storm water storage and treatment.

13.3 Specifications

13.3.1 Planning Considerations

Determine if the site selected for the constructed wetland:

- meets the soil/topography/climate and other conditions above. Prior to seeding/planting a wetland, test the soil to determine if the soil will support wetland vegetation, or if a soil enhancement plan should be developed.
- meets the legal definition of a wetland. An existing wetland cannot be destroyed to create another wetland for non point source control. The area must also not contain any threatened or endangered plant or animal species, as these will be impacted upon construction. If any of these conditions exist, the site is not appropriate for a constructed wetland. A qualified wetland scientist should perform the necessary wetland delineation and plant/animal survey prior to design.
- meets sizing requirements. The total surface area of the created wetland should be a minimum of 1% of the area draining into the wetland if it is to treat storm water.

Determine the need the wetland will fulfill.

This may include one or more of the following: hydrologic benefits, nutrient uptake, and sediment trapping. The design of the constructed wetland will differ depending on its intended use.

The construction of a wetland may require local, state and federal permits, depending on the specific circumstances. All relevant laws should be investigated prior to plans being developed to determine the legality of constructing a wetland for treatment of storm water or effluent from municipal sewage treatment plant to ensure that necessary Permits are obtained.

Again, it may be necessary to know the location of the water table.

It is essential to establish the emergent and upland plant communities as soon as possible following construction. This should be included in the construction sequence schedule.

13.3.2 Design Considerations of Wetland treating Storm Water

Several examples of constructed wetland design are shown in Figures 9.3 through 9.7 Please note that plant community "zones" can be used to describe constructed wetlands.

These zones are shown in Figure 11.7

Deep Marsh 18 to 72 inches in depth.
Low Marsh 6 to 18 inches in depth.
High Marsh 0 to 6 inches in depth.
Semi-wet 0 to 24 inches above the normal water level.

13.3.2.1 Wetland Configuration

The wetland should be irregular in shape, with a length to width ratio of at least 2:1 preferably 4:1.

Inlets and outlets must be placed far apart to avoid short circuiting (in other words, inlet water going directly into the outlet without receiving the treatment of the wetland).

The length to width ratio can be increased by using high marsh areas or islands to cause incoming water to meander back and forth on its way through the system. With the proper design characteristics these wetlands can have a natural appearance and still provide all the desired functions for storm water treatment.

All constructed wetlands should contain a fore bay at the inlet and micro pool at the outlet. The Fore bay at the inlet allows for sediment and other solids to settle out of the storm water before entering the wetland. This fore bay should be located in such a way that sediment can be removed with machinery as it fills up. The micro pool at the outlet allows for the collection of all the water in the system at one common point. It also provides for cooling of the water before discharge.

In some cases the “Pocket Wetland” design shown in figure 11.7 may not lend itself to the use of a properly designed fore bay. A smaller “cattail” fore bay may be useful at least to trap trash and oil. The following are guidelines for the size ratios in percent of total surface area of each plant community:

Deep Marsh (Fore bay) 20%-45%, Low Marsh 25%-40%, High Marsh 30%-40%, Semi-wet (the size of this area depends on the topography surrounding the wetland; steep slopes will produce less semi-wet habitat and shallow slopes will produce more semi-wet habitat). A variety of different depths must be present within the wetland to meet the growing requirements of diverse emergent wetland plants.

13.3.2.2Surface Area

Runoff

The total surface area of the created wetland should be a minimum of 1% of the area draining into the wetland. The pollutant removal capability of the wetland is increased as the surface area to volume ratio is increased.

This ratio can be increased by

a) Increasing the overall area of the wetland, or
b) Creating a complex microtopography within the wetland of various pools, shoals and Islands.

13.3.2.3Volume

Runoff

The wetland should be able to contain a treatment volume capable of capturing the runoff generated by 90% of the runoff-producing storms in the region on an annual basis. The forebay should have a minimum treatment volume of 10% of the total wetland treatment volume. The micropool should also have a minimum treatment volume of 10% of the total treatment volume.

13.3.2.4Water Depth

The normal water depth in the fore bay and micropool areas should be 3.0 to 6.0 feet. Be sure to allow sufficient capacity for 3 to 5 years of sediment accumulation in the fore bay. The depth of the standing water for the remaining surface area, where the wetland vegetation is installed, should vary between 6 to 18 inches. The depth/area allocation of the wetland should be designed to produce the desired plant communities at maturity.

If the wetland is also used for hydraulic detention, the temporary increase in water depth above the normal water level of the wetland should be no more than 3 feet and should not occur for more than 24 hours. Some wetland vegetation cannot survive inundation for extended periods of time.

A wetland specialist can provide detailed information about specific species.

13.3.2.5Side Slopes

Side slopes leading into the wetland should be not more than 3:1 and not less than 10:1. Shallower slopes will promote better establishment and growth of wetland plant species, and will produce a more natural wetland appearance. Shallower slopes also allow for easier mowing and maintenance activities. It is recommended to include in the design a vegetated ten-foot wide shelf, one foot deep, leading to any deeper waters (fore bay and micropool) to reduce the hazard potential.

13.3.2.6Outlets

The wetland outlet will control the release rate from the wetland. The outlet must maintain the desired water level in the wetland and provide the desired release rate for a range of storm events.

Wetlands which are designed for extended detention may need to use multiple outlets. Outlet design and flow routing through the wetland are complex procedures which should be done by Licensed professional engineers.

If an outlet pipe is used, it should be designed to draw water from one foot below the water surface. This will decrease clogging from floating vegetative material and will also draw cooler water from the bottom of the wetland.

The outlet should be designed so that trapped trash and debris can be easily removed. An additional valved outlet should be provided to drain the wetland for maintenance. A stabilized outlet structure must be used at the discharge of the fore bay and at the outlet from the wetland. This will prevent erosion within the wetland and at the discharge point. See the Stabilized

An example outlet structure is shown in Figure 13.7.

13.3.2.7Emergency Spillway

An emergency spillway must be provided to safely discharge from the wetland during storms which exceed design. A common design condition of an emergency spillway is the 50 to 100-year storm event. However, in wetland design the emergency spillway should be placed to limit the extended detention of storm water to a maximum of 3 feet or the 50 to 100-year design storm, whichever is less.

13.3.2.8Water Balance

An adequate dry weather water balance for the wetland must be maintained throughout the year.

This entails the measurement of the incoming base flow to the wetland as well as using soil borings to determine the elevation of the water table and soil permeability rates. This data can then be used to determine if the water inputs (runoff, precipitation, and groundwater) are greater than the water losses (discharge, infiltration, and evaporation). To maintain the water level during the dry season, it may be necessary to install a clay or plastic semi-permeable or impermeable liner. The need for a liner shall be determined by the examination of the preceding information. Some liners are discussed further in the Pond Sealing and Lining BMP (although this should not be treated as an exclusive source of information).

13.3.2.9Vegetation

A qualified wetland scientist should prepare the portion of the design that relates to vegetation (Plant species) selection, installation, and harvesting procedures. The wetland should contain a High diversity and density of wetland plant species. The plant communities should be designed by creating a functional ponds cape within and around the wetland. This planning will increase the wetland’s ability to remove nutrients and pollutants and will provide habitat diversity within the created wetland.

Establishing the emergent and upland plant communities as soon as possible following construction will allow the wetland to begin storm water treatment and will provide erosion control during the first growing season.

Periodic harvesting of the vegetation is essential in stimulating the growth of many plant species, thereby allowing them to remove more of the nutrients flowing into the wetland. Periodic harvesting also may remove accumulated nutrients and excess organic material and thereby extend the life of the constructed wetland.

13.3.2.10 Wildlife Enhancement

Measures to further enhance habitat for wildlife are encouraged. Wildlife enhancement, however, is a secondary concern. For the purposes of this BMP, pollutant removal and hydraulic detention are the primary concern. Additional wildlife elements may be added to increase the use of the wetland by wetland-dependent animal species. This becomes even more important in areas which are predominantly urban and have lost much of their natural habitat. For example, maximizing vegetation density around the wetland will attract numerous waterfowl and other species while discouraging the entry of domestic animals that would prey on wildlife. Wildlife use should not be encouraged if toxic or harmful pollutants are expected to accumulate within the water, soil or plants.

13.3.2.11 Construction Considerations

Wetlands contain areas of deep water and muck soils which may present a safety hazard for those persons working or playing in and around them. Depending on local regulations, wetland areas may need to be fenced during and after construction for increased safety. Special care should always be taken during the initial seeding/planting and at harvesting times to minimize potential problems.

The majority of the shallow marsh system is zero to eighteen inches deep, which creates favorable conditions for the growth of emergent wetland plants. A deeper fore bay is located at the major inlet, and a deep micropool is situated near the outlet.

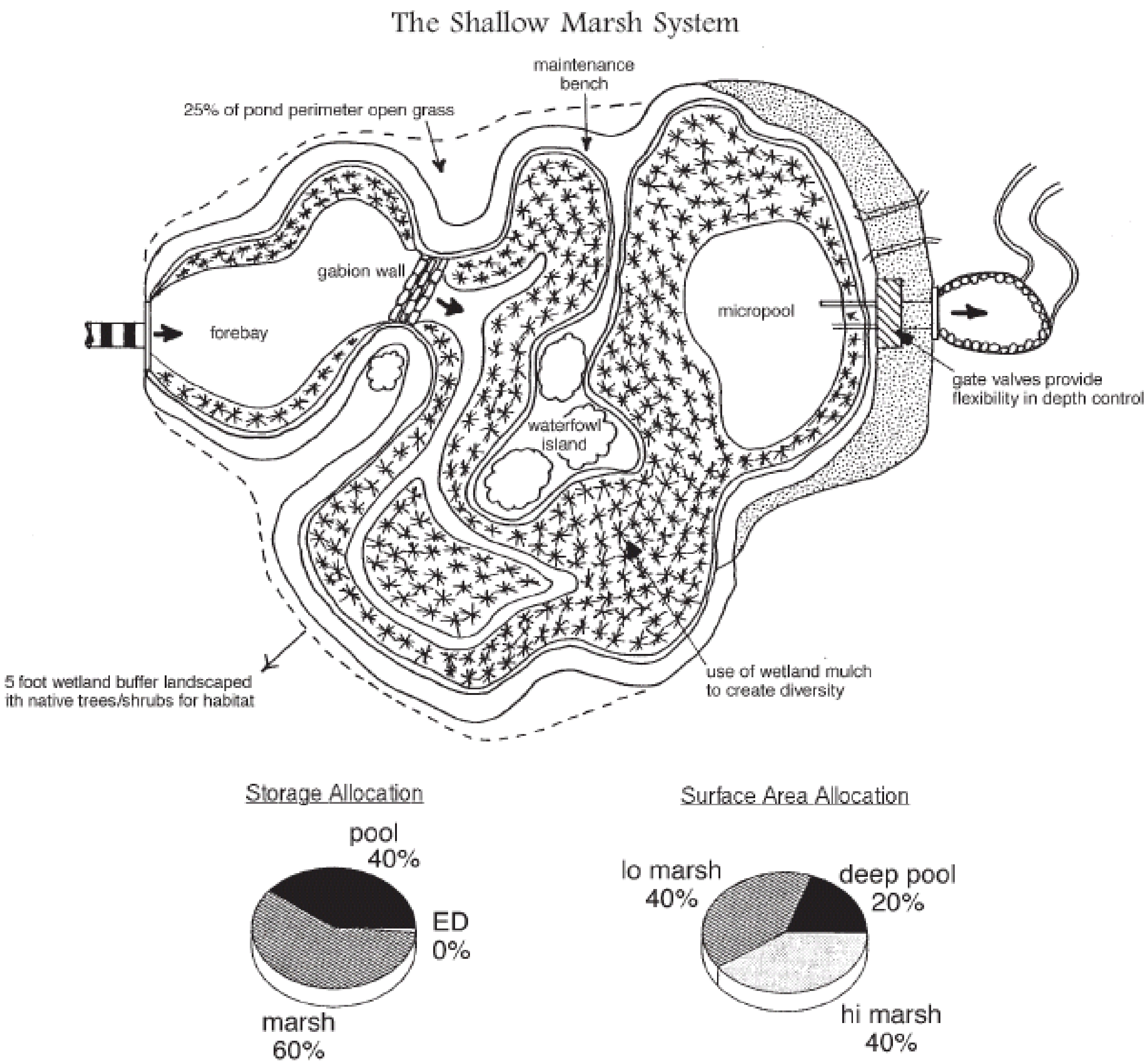
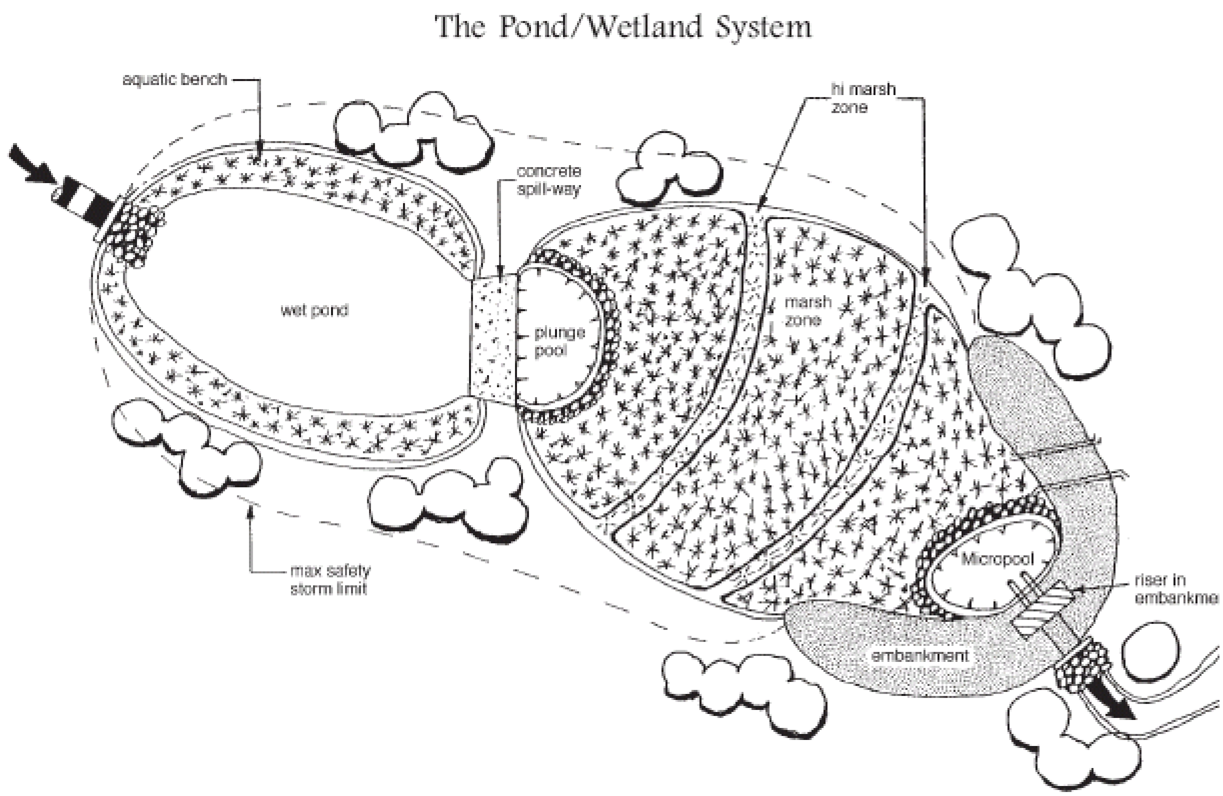


Figure 13.3 The Shallow Mash system



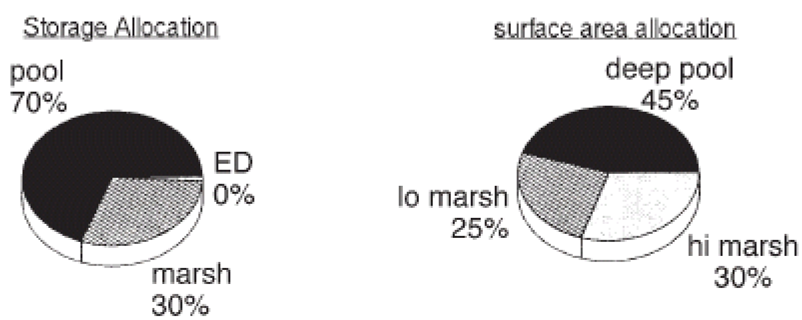


Figure 13.4 The Pond/Wetland System

The pond/wetland system consists of two separate cells .A deep pond leading to a shallow wetland. The pond removes pollutants, and reduces the space required for the system.

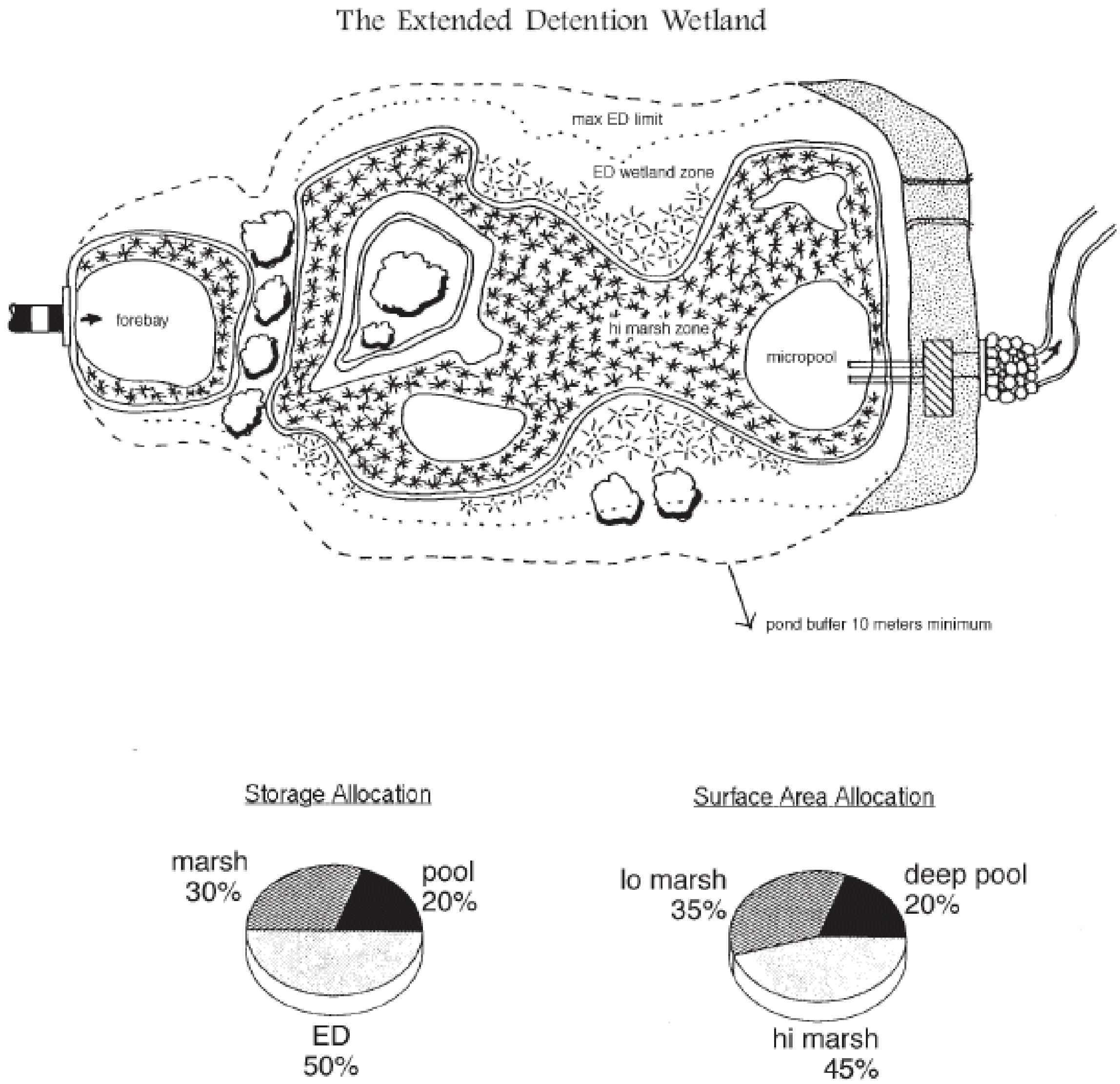


Figure 13.5 The Extended Detention Wetland System

The water level within an ED wetland can increase by as much as three feet after a storm event, and then returns to normal levels within 24 hours. As much as 50% of the total treatment volume can be provided as ED storage, which helps to protect downstream channels from erosion, and reduce the wetland's space requirement.

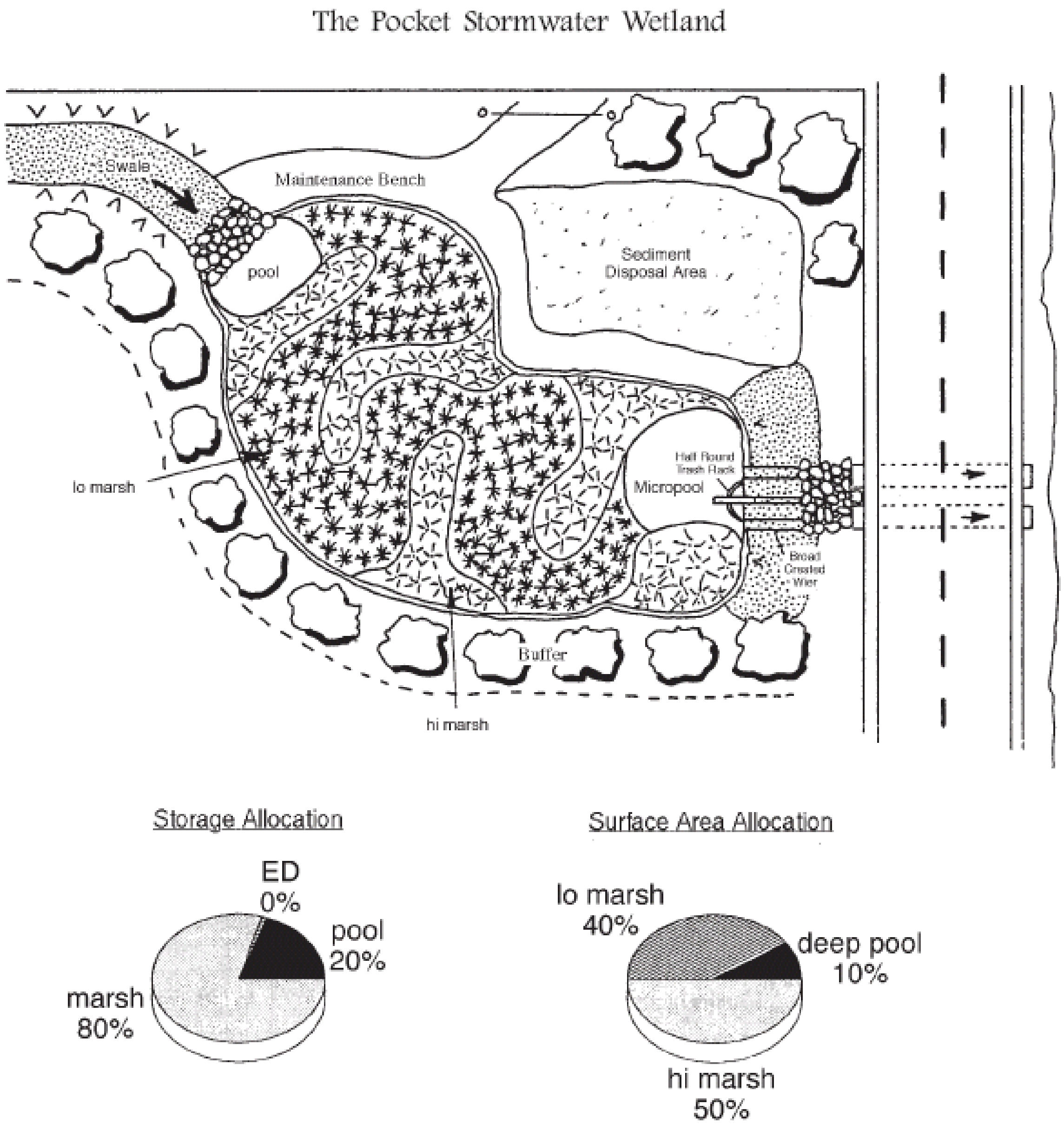


Figure 13.6 The Pocket Storm water Wetland

Pocket wetlands seldom are more than a tenth of an acre in size, and serve development sites of ten acres or less. Due to their size and unreliable water supply, pocket wetlands do not possess all of the benefits of other wetland designs.

Most pocket wetlands have no sediment fore bay. Despite many drawbacks, pocket wetlands may be attractive.

Extended Detention Wetland Outlet Structure

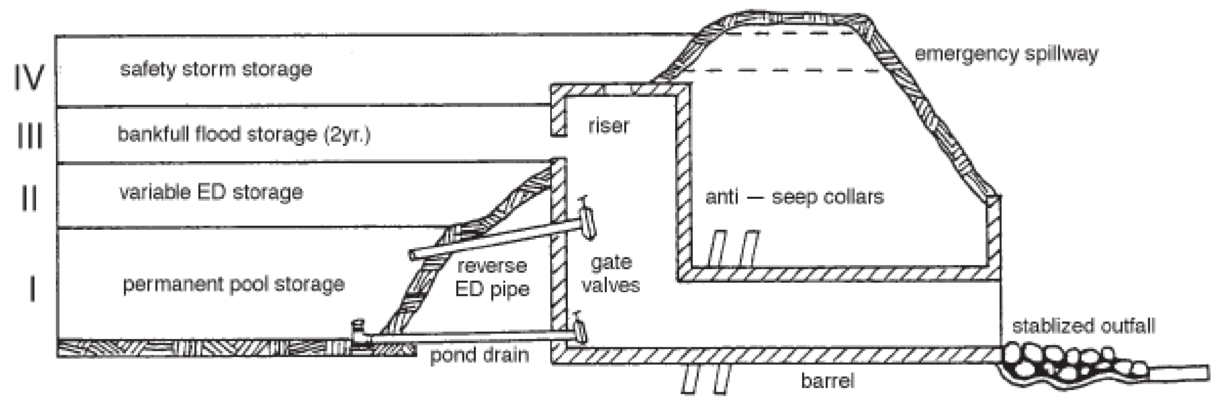


Figure 13.7 The Extended Detention Wetland Outlet Structure

The micropool of an ED wetland system is 4 to 6 feet deep, and helps protect the orifice of the reverse slope pipe extending from the riser. The pipe withdraws water within one foot of the normal pool, and is equipped with a gate valve to adjust detention times. The pond drain pipe is also equipped with a gate valve, and is used to drain the entire wetland for planting or sediment cleanout.

13.4 Design Criteria of Surface Water Flow (SWF)

13.4.1 Design Criteria

The following are design criteria for free water surface constructed wetland.

Table 13.3 Design Criteria for Surface Water Flow Constructed Wetland

Factor	**Design Value**
Detention time (day)	5 to 14
Max. COD or BOD loading Rate (Kg/ha.d)	80
Water Depth above media (Meters)	0.1 -0.5m
Hydraulic loading rate (mm/day)	7 to 60
Aspect ration (Length to width)	2:1 to 10:1
Mosquito Control	Required
Bed Bottom Slope (%)	0-2
Bed depth (substrate) (m)	0.3 -0.6

Source: Design Criteria and Practice for Constructed Wetlands by Ronald W. Crites, 1994

13.4.2 Hydraulic Retention Time

For free water surface unit designed to achieve COD removal, the relation between detention time and removal rate is estimated by the following expression:-

$C_e/C_0 = e^{-[kt\ x(86.68)xt-InF]}$ **(13.1)**

Where C_e =effluent BOD (mg/L)

C_0 =Influent BOD (mg/L)

Kt =temperature dependent rate constant, d^{-1}

T =average detention time

F =fraction of BOD that does not settle out within a few metres in the wetland

86.68 =constant accounting for surface area and void fraction

kt =$0.0057d^{-1}$ for 20^0C but for any other temperature, the relationship is as follows $kt=k20x1.06^{(t-20)}$

Where:-

1.06= α =temp. correction factor

t =average temperature of wastewater in ^0C.

F fraction ranges as follows:

0.52 =Primary effluents

0.75 =Secondary effluents

0.8 =Tertiary Effluent

From expression **(13.1)**

$Kt=0.0057 \times 1.06^{(23-20)}$ **(13.2)**

Where 23^0C is the average water temperature in the wetland

13.4.3 Hydraulic Loading Rate

Hydraulic Flow Rate is determined by the following expression:

$A=Qxt/ (d \times ø)$ **(13.3)**

Where :

A= Plan area of Wetland

Q= Average flow (daily) =hydraulic loading rate

t= Hydraulic detention time (days)

ø= Void ratio (typically 0.75)

d= depth of flow above the media

Applying the dimension to expression 13.3 above

13.4.4 COD Loading Rate

The removal mechanisms of both COD and BOD is the same

13.5 Hydraulic Design of Subsurface Water Flow (SSF) Wetland

13.5.1 Design Criteria

Table 13.4 has the design criteria of a subsurface water flow constructed wetland.

Table 13.4 Design Criteria for Subsurface Water Flow Constructed Wetland

Factor	Value
Bed depth (m)	0.3 – 0.9m
Hydraulic Retention time (Days)	5 - 10
Hydraulic Loading Rate (mm/day)	60 – 80 mm
COD loading rate (Kg/m^3.day)	0.08
Maximum Water Depth	Water level below surface (at least 50 mm below surface)
Aspect Ratio	1 : 1 to 1 : 2

Source: Organic Waste Recycling pp 298 (Aquatic weeds and their utilization) by C. Polprasert

13.5.2 Detention Time

The relationship between detention time and COD removal for a subsurface flow constructed wetland is expressed by the following expressions:-

$$Ce/Co = e^{-ktxT} \qquad (13.4)$$

Where:-
Ce =effluent COD (mg/L) =50mg/L (Max allowed)
Co =influent COD (mg/L) =462mg/L
Kt =temperature dependent first order reaction rate constant per day
T =hydraulic detention time days
Kt values ranges from 0.8 to 1.1 per day for sand to gravel

Using gravel as the bed media then kt= 1.1 per day

Others remain as before.

13.6 Processes in Sub-surface Flow Constructed Wetlands (SSFCW)

Wetland can effectively remove or convert large quantities of pollutants from point sources (municipal, industrial and agricultural wastewater) and non-point sources (mines, agriculture and urban runoff), including organic matter, suspended solids, metals and nutrients. The focus on wastewater treatment by constructed wetlands is to optimise the contact of microbial species with substrate, the final objective being the bioconversion to carbon dioxide, biomass and water. Wetlands are characterized by a range of properties that make them attractive for managing pollutants in water (Bavor and Adcock, 1994). These properties include high plant productivity, large adsorptive capacity of the sediments, high rates of oxidation by microflora associated with plant biomass, and a large buffering capacity for nutrients and pollutants. Table 13.5 provides an overview of pollutant removal mechanisms in constructed wetlands (Mitchell, 1996).

Table 13.5 Overview of pollutant removal mechanisms

Pollutant	Removal Process
Organic material (measured as BOD)	Biological degradation, sedimentation, microbial uptake
Organic contaminants (e.g., pesticides)	Adsorption, volatilization, photolysis, and biotic/abiotic degradation
Suspended solids	Sedimentation, filtration
Nitrogen	Sedimentation, nitrification/denitrification, microbial uptake, volatilization
Phosphorous	Sedimentation, filtration, adsorption, plant and microbial uptake
Pathogens	Natural die-off, sedimentation, filtration, predation, UV degradation, adsorption
Heavy metals	Sedimentation, adsorption, plant uptake

13.6.1 Biological processes

There are six major biological reactions involved in the performance of constructed wetlands, including photosynthesis, respiration, fermentation, nitrification, denitrification and microbial phosphorus removal (Mitchell, 1996b). Photosynthesis is performed by wetland plants and algae, with the process adding carbon and oxygen to the wetland. Both carbon and oxygen drive the nitrification process. Plants transfer oxygen to their roots, where it passes to the root zones (rhizosphere). Respiration is the oxidation of organic carbon, and is performed by all living organisms, leading to the formation of carbon dioxide and water. The common microorganisms in the CW are bacteria, fungi, algae and protozoa. The maintenance of optimal conditions in the system is required for the proper functioning of wetland organisms. Fermentation is the decomposition of organic carbon in the absence of oxygen, producing energy-rich compounds (e.g., methane, alcohol, volatile fatty acids). This process is often undertaken by microbial activity. Nitrogen removal by nitrification/denitrification is the process mediated by microorganisms. The physical process of volatilization also is important in nitrogen removal. Plants take up the dissolved nutrients and other pollutants from the water, using them to produce additional plant biomass. The nutrients and pollutants then move through the plant body to underground storage organs when the plants senesce, being deposited in the bottom sediments through litter and peat accretion when the plants die.

Wetland microorganisms, including bacteria and fungi, remove soluble organic matter, coagulate colloidal material, stabilize organic matter, and convert organic matter into various gases and new cell tissue (Mitchell, 1996a). Many of the microorganisms are the same as those occurring in conventional wastewater treatment systems. Different types of organisms, however, have specific tolerances and requirements for dissolved oxygen, temperature ranges and nutrients.

13.6.2 Chemical processes

Metals can precipitate from the water column as insoluble compounds. Exposure to light and atmospheric gases can break down organic pesticides, or kill disease-producing organisms (EPA, 1995). The pH of water and soils in wetlands exerts a strong influence on the direction of many reactions and processes, including biological transformation, partitioning of ionized and un-ionised forms of acids and bases, cation exchange, solid and gases solubility.

13.6.3 Physical processes

Sedimentation and filtration are the main physical processes leading to the removal of wastewater pollutants. The effectiveness of all processes (biological, chemical, physical) varies with the water residence time (i.e., the length of time the water stays in the wetland). Longer retention times accelerate the remove of more contaminants, although too-long retention times can have detrimental effects.

13.6.4 Limitations of wetland processes

13.6.4.1 Process rates

The chemical and biological processes occur at a rate dependent on environmental factors, including temperature, oxygen and pH. Metabolic activities are decreased by low temperature, reducing the effectiveness of pollutant uptake processes relying on biological activity. Low oxygen concentrations limit the processes involving aerobic respiration within the water column, and may enhance anaerobic processes, which can cause further degradation of water quality. Many metabolic activities are pH-dependent, being less effective if the pH is too high or low.

13.6.4.2 Hydrological limitations

The capacity of wetlands to treat wastewater is limited, both in terms of the quantity of water, and the total quantity of the pollutants. Hydraulic overloading occurs when the water flow exceeds the design capacity, causing a reduction in water retention time that affects the rate of pollutant removal. Pollutant overloading occurs when the pollutant input exceeds the process removal rates within the wetland (White *et al.*, 1996). Hydraulic overloading may be compensated for by using surcharge mechanisms, or the design may be based on a flush principle, whereby large water flows bypass the wetland when used for storm water treatment (White *et al.*, 1996). Inflow variations are typically less extreme for wetlands treating municipal wastewaters, with incoming pollutant loads also being more defined and uniform.

13.6.5 Wetland nitrogen processes

The most important nitrogen species in wetlands are dissolved ammonia (NH_4^+), nitrite (NO_2^-), and nitrate (NO_3^-). Other forms include nitrous oxide gas (N_2O), nitrogen gas (N_2), urea (organic), amino acids and amine (Kadlec & Knight, 1996). Total nitrogen in any system is referred to as the sum of organic nitrogen, ammonia, nitrate and nitrous gas (Organic-N + NH_4^+ + NO_3^- + N_2O). The various nitrogen forms are continually involved in transformations from inorganic to organic compounds, and vice-versa. Many of these transformations are biotic, being carried out by nitrobacter and nitrosomonas (Kadlec & Knight, 1996). As it undergoes its various transformations, nitrogen is taken up by wetland plants and microflora (preferentially as $NH_{4,}^+$ and NO_3^-), some is leached to the subsoil, some is liberated as gas to the atmosphere, and some flows out of the wetland, normally in a dissolved form. Organic nitrogen comprises a significant fraction of wetland biota, detritus, soils, sediments and dissolved solids (Kadlec and Knight, 1996). It is not readily assimilated by aquatic plants, and must be converted to $NH_{4,}^+$ or NO_3^- through multiple conversions requiring long reaction time (Kadlec & Knight, 1996). The process of biological nitrogen removal follows several sequences. Nitrification first takes place, generally in the rhizosphere and in biofilms (aerobic process). Denitrification may then follow, occurring in soils and below the oxidized microzone at the soil/water interface, as it is an anaerobic process (Broderick et al., 1989).
Nitrification is a two-step process catalysed by Nitrosomonas and nitrobacter bacteria. In the first step, ammonia is oxidized to nitrite in an aerobic reaction catalyzed by Nitrosomonas bacteria, as shown in Equation 9.5:

$$NH_{4,}^+ + O_2 \xrightarrow{+\text{Nitrosomonas}} H^+ + NO_2^- + H_2O \qquad \textbf{(13.5)}$$

The nitrite produced is oxidized aerobically by nitrobacter bacteria, forming nitrate (Equation 9.6), as follows:

$$NO_2^- + O_2 \xrightarrow{\text{Nitrobacteria}} NO_3^- \qquad \textbf{(13.6)}$$

The first reaction produces hydroxonium ions (acid pH), which react with natural carbonate to decrease the alkalinity (Mitchell, 1996a). In order to perform nitrification, the nitrosomonas must compete with heterotrophic bacteria for oxygen. The BOD of the water must be less than 20 mg/l before significant nitrification can occur (Reed et al., 1995). Temperatures and water retention times also may affect the rate of nitrification in the wetland. Denitrification is the process in which nitrate is reduced in anaerobic conditions by the benthos to a gaseous form. The reaction is catalyzed by the denitrifying bacteria Pseudomonas spp. and other bacteria, as follows:

$$NO_3^- + \text{Organic-C} \xrightarrow{\text{Denitrifying Bacteria}} N_2 \, (NO \,\&\, N_2O)_{(G)} + CO_{2(G)} + H_2O \qquad \textbf{(13.7)}$$

Denitrification requires nitrate, anoxic conditions and carbon sources (readily biodegradable). Nitrification must precede denitrification, since nitrate is one of the prerequisites. The process of denitrification is slower under acidic condition. At a pH between 5- 6, N_20 is produced. For a pH below 5, N_2 is the main nitrogenous product (Nuttall et al., 1995). NH_4^+ is the dominant form of ammonia-nitrogen at a pH of 7, while NH_3 (present as a dissolved gas) predominates at a pH of 12. Nitrogen cycling within, and removal from, the wetlands generally involves both the translocation and transformation of nitrogen in the wetlands, including sedimentation (resuspension), diffusion of the dissolved form, litter fall, adsorption/desorption of soluble nitrogen to soil particles, organism migration, assimilation by wetland biota, seed release, ammonification (mineralisation) (Orga-N – NH_4^+), ammonia volatilization (NH_4^+ – NH_3 (gas)), bacterially-mediated nitrification/denitrification reactions, nitrogen fixation (N_2, N_2O (gases – organic-N)), and nitrogen assimilation by wetland biota (NH_4^+, Nox organic – N, with NO_x usually as NO_3^-). Precipitation is not a significant process due to the high solubility of nitrogen, even in inorganic form. Organic nitrogen comprises a significant fraction of wetland biota, detritus, soils, sediments and dissolved solids (Kadlec and Knight, 1996).

13.6.6 Phosphorus removal

Phosphorus is an essential requirement for biological growth. An excess of phosphorus can have secondary effects by triggering eutrophication within a wetland, and leading to algal blooms and other water quality problems. Phosphorus may enter a wetland in dissolved and particulate forms. It exits wetlands in outflows, by leaching into the sub-soil, and by removal by plant and animals. Phosphorus removal in wetlands is based on the phosphorous cycle, and can involve a number of processes. Primary phosphorus removal mechanisms include adsorption, filtration and sedimentation. Other processes include complexation/precipitation and assimilation/uptake. Particulate phosphorus is removed by sedimentation, along with suspended solids. The configuration of constructed wetlands should provide extensive uptake by Biofilm and plant growth, as well as by sedimentation and filtration of suspended materials. Phosphorus is stored in the sediments, biota, (plants, Biofilm and fauna), detritus and in the water. The interactions between compartments depend on environmental conditions such as redox chemistry, pH and temperature. The redox status of the sediments (related to oxygen content) and litter/peat compartment is a major factor in determining which phosphorus cycling processes will occur. Under low oxygen conditions (low redox potential), phosphorus is liberated from the sediments and soils back into the water column, and can leave the wetland if the anaerobic condition is not reversed (Moss et al., 1986).

13.6.7 Suspended solids

Solids may be derived from outside a wetland (e.g., inflows and atmospheric inputs), and from within a wetland from plankton (zooplankton and phytoplankton), and plant and animal detritus. With low wetland water velocities and appropriate composition of influent solids, suspended solids will settle from the water column within the wetland. Sediment resuspension not only releases pollutants from the sediments, it increases the turbidity and reduces light penetration. The physical processes responsible for removing suspended solids include sedimentation, filtration, adsorption onto Biofilm and flocculation/precipitation. Wetland plants increase the area of substrate available for development of the Biofilm. The surface area of the plant stems also traps fine materials within its rough structure.

13.6.8 Pathogen removal

Pathogens are disease-causing organisms (e.g., bacteria, viruses, fungi, protozoa, and helminthes). Wetlands are very effective at removing pathogens, typically reducing pathogen number by up to five orders of magnitude from wetland inflows (Reed at al., 1995). The processes that may remove pathogens in wetlands include natural die-off, sedimentation, filtration, ultra-violet light ionization, unfavorable water chemistry, temperature effects, and predation by other organisms and pH (Kadlec & knight 1996). Kadlec and Knight (1996) showed that vegetated wetlands seem more effective in pathogen removal, since they allow a variety of microorganisms to grow which may be predators to pathogens.

13.6.9 Heavy metal removal

Heavy metals is a collective name given to all metals above calcium in the Periodic Table of Elements, which can be highly toxic, and which have densities greater that $5 g/cm^3$ (Skidmore and Firth, 1983). The main heavy metals of concern in freshwater include lead, copper, zinc, chromium, mercury, cadmium and arsenic. There are three main wetland processes that remove heavy metals; namely, binding to soils, sedimentation and particulate matter, precipitation as insoluble salts, and uptake by bacteria, algae and plants (Kadlec & Knight, 1996). These processes are very effective, with removal rates reported up to 99% (Reed et al., 1995). A range of heavy metals, pathogens, inorganic and organic compounds present in wetlands can be toxic to biota. The response of biota depends on the toxin concentration and the tolerance of organisms to a particular toxin. Wetlands have a buffering capacity for toxins, and various processes dilute and break down the toxins to some degree.

13.7 Abiotic Factors and their Influence on Wetlands

13.7.1 Oxygen

Oxygen in wetland systems is important for heterotrophic bacterial oxidation and growth. It is an essential component for many wetland pollutant removal processes, especially nitrification, decomposition of organic matter, and other biological mediated processes. It enters wetlands via water inflows or by diffusion on the water surface when the surface is turbulent. Oxygen also is produced photosynthetically by algae. Plants also release oxygen into the water by root exudation into the root zone of the sediments. Many emergent plants have hollow stems to allow for the passage of oxygen to their root tissues. The oxygen-demand processes in wetlands include sediment-litter oxygen demand (decomposition of detritus), respiration (plants/animals), dissolved carbonaceous BOD, and dissolved nitrogen that utilize oxygen through nitrification processes (Kadlec & Knight, 1996). The oxygen concentration decreases with depth and distance from the water inflow into the wetland. It is typically high at the surface, grading to very low in the sediment –water interface.

13.7.2 pH

The pH of wetlands is correlated with the calcium content of water (pH 7 = 20 mg Ca/L). Wetland waters usually have a pH of around 6-8 (Kadlec and Knight, 1996). The biota of wetlands especially can be impaired by sudden changes in pH.

13.7.3 Temperature

Temperature is a widely-fluctuating abiotic factor that can vary both diurnally and seasonally. Temperature exerts a strong influence on the rate of chemical and biological processes in wetlands, including BOD decomposition, nitrification and denitrification.

13.8 Operation and maintenance of constructed wetlands

13.8.1 Commissioning

Sometimes commissioning of the wetland is referred as the time from planting to the date where the wetland is considered operational. Operation during this period should ensure an adequate cover of the wetland vegetation. The water level within the wetland during this time needs to be controlled carefully, to prevent seedling from being desiccated or drowned. Once the plants are established, the water level may be raised to operational level. Plant loss may occur during the commissioning, therefore requiring transplanting.

13.8.2 Operation

The operation of a constructed wetland depends on the type of wetland, and the number of preliminary treatment units used for wastewater treatment. Constructed wetlands are designed to be passive and low maintenance, thereby not requiring continual upkeep. Constructed wetland, however, are dynamic ecosystems, with many variables that require managing. If not, problems may occur when the operator does not understand the needed operation and maintenance, the wetland is either hydraulically or organically overloaded, unavoidable disasters (e.g., flooding, drought) occur, the wetland is plagued by weed problems and/or if excessive quantities of sediments, litter and pollutants accumulate and are not removed from the wetland. The management of the constructed wetlands consists of four tasks, as outlined below in Table 13.6.

Not all constructed wetlands are the same, given that they can be designed for a range of objectives. These objectives will determine the kind of operation and management activities needed to be undertaken. Thus, the operation and management of a constructed wetland must be tailored to a particular constructed wetland, reflecting desired objectives and site-specific constraints, including local hydrology, salinity, climate and relevant aspects of public safety. The essential elements of the operation and maintenance of a constructed wetland must include:

- Description of the wetland and its objectives;
- List of tasks or management; and
- Monitoring activities, including inspection checklists.

Table 13.6 Management of constructed wetlands

Tasks	Example
Operational control	Varying water level
Monitoring	Water quality, habitat, flora and fauna
Inspection	Structures and Embankments
Maintenance	Repair damage to the structures and control weeds

The operation of a constructed wetland after commissioning must include:

- Maintaining the embankments;
- Removing litter and debris;
- Checking the water flow rate to a constructed wetland to determine if it is in accordance with the design;
- Removing any blockages in the inlet and outlet works;
- Replacing plants as required;
- Removing any unwanted weed species from the constructed wetland;

• Checking the plants for any sign of diseases;
• Protecting the deep open water;
• Correcting erosion and slumping; and
• Checking for any signs of over-flooding (for sub-surface flow constructed wetlands).

These tasks may be addressed in the form of a checklist to direct the required maintenance, and to identify who should be immediately contacted in the event of problems.

13.8.3 Monitoring

Monitoring selected performance parameters should provide sufficient information to measure performance in meeting wetland objectives. If water quality improvement is the primary objective, the performance indicator should either be presented as a concentration or load at the outlet, or a comparison of inflow and outflows, also in terms of concentrations or loads. If monitoring results indicate that the system is not working according to the objectives, corrective measures must be applied. Improvement of water quality may be assessed by monitoring a range of inflow and outflow water-quality parameters. Useful parameters for monitoring wetland performance include dissolved oxygen (DO), BOD, COD, total phosphorus, orthophosphorus, total nitrogen, total Kjeldhal nitrogen, ammonia nitrogen, oxidized nitrogen, faecal coliform, pH, suspended solids, electrical conductivity, and heavy metal concentrations. Water flow rates to and from the constructed wetland also must be measured. The sampling may be done using either an automatic or manual sampler. Samples within the wetland must sometimes be taken for the purpose of comparison.

13.8.4 Decommissioning and refitting

Decommissioning and refitting of a constructed wetland may take place if its design lifetime is over. At the end of its design life, a wetland will be either be refitted, or decommissioned if no longer required. Refitting may be required when the accumulation of wetland sediments is adversely affecting wetland performance, or when changing catchment conditions require modifications of the wetland. Major refits may include the removal of accumulated peat, including aquatic plants, and replacements of substrates. Decommissioning of a wetland may be required if the land supporting it is utilized for other purposes, or if the wetland functioning is unable to achieve the original design objectives.

14.0 THE MARINE DISPOSAL OF SEWAGE

Where a Kenyan community is located on the coast, the most economical method of disposing of its sewage will usually be to discharge it into the Indian Ocean.

Because of its large volume and well-oxygenated state, the Indian Ocean has a tremendous potential for self-purification. If sewage is discharged in such a way as to take advantage of this ability, the sea will act as a sewage treatment works.

To achieve this, sewage must be diluted by many times of its volume of sea-water. The natural characteristics of the sea assist in this. Migrating organisms and ocean tidal and wind-induced currents will tend to carry away and disperse any sewage discharges and the problem becomes one of sitting the discharge point so as to ensure that unacceptable local effects are avoided.

14.1 Characteristics of a Sewage Field

Sea water is denser than sewage unless it is at a very much higher temperature. Therefore, except under very unusual thermal conditions, sewage when discharged near a sea bed rises to the surface in a manner resembling plume of smoke leaving a chimney. Occasionally, temperature variations in the sea can result in sewage being trapped beneath a warmer, less dense layer of sea water, as denser layers in the atmosphere can cause smoke to flatten out rather than rise.

As the buoyant sewage rises, the turbulence created by its movement causes sea water to be drawn into the plume and so the rising discharge has an ever-increasing plan area. The sewage/sea-water mixture eventually forms a layer or sewage field, a metre or more thick, close to the surface of the sea.

The relative volume of sea-water which mixes with and dilutes the sewage is a function of the depth of the discharge pipe in the sea, of the differences in the densities of the sewage and sea-water, and of the velocity and direction of the sewage discharge; this initial dilution can be estimated by calculations. The sea currents have little effect on the initial dilution, although the plume will be elongated in accordance to the local currents.

The subsequent drifting of the sewage field is determined by tidal movements and the system of sea currents in the area. As it drifts, natural density currents and turbulence cause lateral and vertical erosion of the sewage field and further dilution of the sewage by sea-water takes place.

Any floating solids, grease and other insoluble with surface-active properties contained in the sewage will form a usually oily ‘slick’ on the surface of the sea; this slick tends to drift with the wind, irrespective of the direction of the sea currents. Sewage slicks, visible mainly because of their tendency to damp out ripples, are characteristics of marine sewage discharges. Eventually, inter-mixing with sea-water and oxidation cause the slick to disappear

14.2 Effects of Coastal Discharges

The undesirable effects of marine sewage discharges may be summarized as:-

1) Hazards to public health.
2) Aesthetic nuisance, resulting in loss of amenity.
3) Marine pollution, especially when it affects fisheries and tourist attractions such as the Kenya reefs.
4) The accelerated corrosion of marine structures.

14.2.1 Public Health

Sewage contains considerable numbers of pathogenic and parasitic organisms, which do not naturally occur in sea-water. To be acceptable, any arrangement for disposing of sewage into the sea must prevent these organisms from infecting humans.

In general, the marine disposal of sewage will only be a hazard to public health if:-

a) Sewage containing sufficient organisms to cause disease to humans reaches waters and beaches used for bathing;
b) Fish used for human food are contaminated by disease organisms.

The presence of disease organism in bathing water or on a beach does necessarily result in diseases. To it self, external bodily contact with disease organism is very unlikely to cause any harm, although fingers can easily transport pathogens from the skin into the mouth.

The number of organisms required to infect a person varies tremendously. However , it is reasonable to say that the risk of catching from sea-water , or from a beach, is proportional to the number of disease organisms present ;therefore, any marine sewage discharge must be sited so as to reduce as much as possible the number of pathogens and parasites reaching the inshore waters.

When considering the disease organisms in sewage, the effect of the sea-water environment upon the organism is important. The sea is, for them, most unfavorable environment and they begin to die off; unless protected by being enclosed within large pieces of organic wastes, the number of organism's decreases roughly logarithmically with time, as a result of a combination of factors including increasing dilution and salinity, sedimentation and the action of sunlight and predators.

Isolating, identifying and counting individual species of pathogens and parasites is tedious for marine sewage discharges. It is therefore usual to utilize the harmless coliforms bacteria as indicators of the possible presence of disease organisms.

The implied assumption is that the die-off rates of coliforms in the sea is similar to that of other micro-organisms .This does not apply especially in the case of viruses, spores, cysts and ovas.However, provided that it is always realized that they are merely indicators of the possible presence of disease organisms rather than proof of their absence.Coliform bacteria as indicators has many advantages.

Coliforms bacteria can be identified and counted relatively quickly; they are certainly more numerous in sewage than are individual species of disease organisms and therefore provide a sensitive method with, as it were, a safety factor. When compared to any non-living indicator, coliform bacteria, when discharged into the sea, share a similar fate with other micro-organisms, although the die off rate may vary. If their mortality is taken into account, the relative number of coliforms in any part of the sea gives a positive and reasonably accurate method of estimating the dispersion and dilution of a sewage field.

The disadvantage of using coliform bacteria is that many of the coliforms in the sewage are of terrestrial origin and their numbers tend to vary seasonally. The intestinal bacteria E.Coli Type 1, being of faecal origin, is more indicative of sewage pollution; however ,its culture is more tedious and it is more often used as a confirmatory test when the normal coliform test indicate the possibility of danger.

During daylight hours, probably 90 per cent of the coliform bacteria contained in any sewage discharges into the Indian Ocean will die within an hour.

Any disease organisms contained in fish are destroyed by proper cooking. This safeguard does not exist in shellfish, which are particularly suspectable to such contamination are eaten raw or lightly cooked. It is possible to remove disease organisms from shellfish, but the mixing of clean and unsafe fish is so simple that some risk is inevitable.

14.2.2 Aesthetic Nuisance

Most of the nuisances which allegedly arise from marine sewage discharges are psychological. If the public are aware of the presence of sewage discharge, they will often imagine odours and other nuisances; unfortunately, the sewage discharge point is frequently indicated by the unavoidable presence of sea-birds, seeking food and the inevitable ‘slick’.

For practical purposes, it is reasonable to assume that aesthetics nuisance will be prevented if a matter recognizable as being of sewage origin does not reach the inshore waters and beaches.

14.2.3 Marine Pollution

This is defined as the introduction into the marine environment of substances which result in harm to living resources and impair the natural qualities of the sea-water.

The more serious results of excessive marine pollution in the Indian Ocean are likely to be the effects on inshore fisheries. Fish could for example, be:-

a) Directly poisoned by toxic industrial effluents;
b) Asphyxiated because of the depletion of the dissolved oxygen in the water by decomposing organic matter in the sewage;
c) Reduced in numbers because of solids settling and spawning grounds;
d) Driven away to more favorable environments because their food is eliminated.

A particular danger arises from cumulative poisons, such as organic insecticides and heavy metals; organic mercury compounds, which can make apparently innocuous fish poisonous to man, are especially dangerous. In order to minimize the health risk, commercial fish taken from the neighborhood of any marine discharge of sewage, which may contain these substances, should be examined regularly to ensure that undue amounts of toxic materials are not being concentrated in their tissues.

Another undesirable effect of a sewage discharge could be an explosive increase in the amount of seaweed as a result of nutrient salts and bio-stimulants, which are complex compounds which promote the growth of vegetation by catalytic action, released as the sewage decomposes. Excessive growths of seaweed may hinder navigation and fishing, and may be washed up to spoil bathing beaches.

14.2.4 Acceleration Corrosion of Marine Structures

The presence of sewage containing sulphate-reducing bacteria can have a marked effect on the rate of corrosion of marine structures, such as steel piles and buoys, in tidal waters.

14.2.5 The Treatment of Sewage before Discharge

Most of the undesirable characteristics of sewage may be reduced by treatment before discharge into the sea; for example, recognizable sewage solids may be removed or chopped up, undesirable constituents of industrial effluents may be eliminated or neutralized.

Usually, the major task of the designer of an installation for the marine discharge of sewage is to strike an economic balance between providing pre-treatment or extending the length of the discharge pipe so as to carry the sewage further out to sea.

14.3 Recommended Criteria for the Marine Disposal of Sewage

The following standards are considered to be suitable for marine sewage discharges into the Indian Ocean; for the reasons explained, the standards must cover both the quality and the effects of discharges.

14.3.1 Quality of Discharges

1) All industrial effluents which are disposed of into the Indian Ocean shall be pre-treated and "balanced" volumetrically, to the extent necessary to ensure that the initial dilution of any toxic chemicals they contain is sufficient to ensure that their concentrations are always within acceptable limits.
2) The discharge of accumulative poisons shall be restricted so far as is practicable; in particular, sewage shall not contain organic mercury compounds.
3) Before discharge to sea, all sewage is to pass through a screen with clear openings not exceeding 20 millimetres (0.8 inches).

14.3.2 Effects of Discharges

1) Solid or floating matter, recognizable as of sewage origin, shall never be visible in bathing waters or upon beaches.
2) The discharge shall not adversely affect the ecology of the inshore waters; in particular, it shall not harm fish and crustacean taken locally for food and it shall not cause damage to the reef, or to the life which it supports.
3) The dissolved oxygen content of all samples of sea-water taken from one metre below the surface shall never be less than 2 milligrammes per litre.
4) The epidemiological experience of all bathing waters shall be satisfactory.

If standard 4) is satisfied, it is very unlikely that there will be any risk to public health. However, such visual standards have the disadvantage that they cannot be precisely measured. If it is felt that a quantitative standard for bathing water is necessary, then it could be:-

- The MPN of coli form bacteria shall not exceed 5 000 per 100 millilitres as an average of 30 random samples examined in a month, provided that not more than 20 per cent of the samples may exceed 6 000 coli forms per 100 millilitres.

If during the rainy season the MPN coli forms determinations regularly exceed these limits, then the test shall not be deemed to have failed if the MPN of E. Coli Type 1 bacteria does not exceed 2 500 per 100 millilitres, as an average of 30 random samples examined in a month. This alternative presumptive E. Coli Type 1 count makes allowance for terrestrial coliforms washed into the sea from estuaries and from water drains and ditches during storms.

The numbers of bacteria suggested have not been assigned on any logical basis, but are in accordance with standards which have proved satisfactory in other parts of the world. Imposing a bacterial standard implies compliance with it, and this is illogical in the case of marine sewage discharges: the results of coliform counts do no more than indicate the prevailing conditions; they do not measure the safety of bathing water.

A corollary to these standards is that any shellfish taken from the shores of the Indian Ocean within 4.5 kilometres (3 miles) of any sewage discharge point shall be assumed to be unfit for human consumption until examination proves them free from disease organisms and toxic compounds.

14.4 Determining Suitable Locations for Sewage Discharges

Sewage is usually discharged close to the sea-bed, in order to achieve maximum initial dilution. Sewage is conveyed to the discharge point through a submarine pipeline; as such pipelines, or outfalls, are expensive to construct, they should generally be as short as possible.

The optimum location for the sewage discharge point is normally determined by a hydrographic survey.

14.4.1 Hydrographic Surveys

Conditions in the sea vary with the tidal cycle and also usually with the seasons; therefore, all hydrographic tests and measurements should be repeated at different states of the tide, at representative seasons throughout the year.

A hydrographic survey should provide information on:-

a) tidal ranges;
b) wind velocities and directions;
c) wave heights and periods;
d) the magnitudes and directions of major sea currents;
e) local current patterns in the upper layers of the sea;
f) temperature and salinity, and salinity, and therefore the relative density, of the sea-water;
g) the depth, shape and slope of the sea-bed.

Details of the local tides, winds and waves provide vital background information; for example, the volume of sea-water available for diluting sewage is largely dependent upon the tidal range, and the strength and direction of the prevailing wind indicate the likelihood of floating sewage being blown.

Knowledge of the magnitude and direction of the major sea currents at various depths indicate the probable degree of mixing and diffusion of the sewage field with sea-water, and also where the deposition of the solids in the sewage will probably occur.

The sewage field is likely to follow the current patterns in the upper layers of the sea and therefore acknowledge of these will indicate how quickly a typical particle of sewage will be carried into bathing waters, and also whether the sewage field is likely to return to the vicinity of the discharge point after each complete tidal cycle, thus reducing the dilution provided by the sea.

It is necessary to know the relative density of sea-water, at various depths, in order to predict the initial dilution of the sewage field will form.

A knowledge of the depth of the sea t various points is required when estimating the initial dilution of the sewage, as this is proportional to the depth of the discharge point; general information about the shape and slope of the sea-bed gives an indication of the total volumes of sea-water available for diluting marine sewage discharges.

14.5 The Design of Submarine Pipelines

Sewage may gravitate, or be pumped, along the submarine pipeline; the decision generally depends upon the height of the land relative to high tide levels. Usually, pumping will result in a more even and rapid rate of flow, which will prevent deposition of solids in the submarine pipeline, reduce the tendency for marine vegetation to block the outlets and also increase the initial dilution of the sewage in the sea; however, pumping results in high running expenses.

14.5.1 Diffusers

The dispersion of a sewage field, once formed, depends only upon the prevailing natural conditions, which cannot be varied. The area covered by the sewage field and consequently the rate at which it will mix with the surrounding sea-water, is dependant upon the initial dilution. As previously described, the initial dilution depends upon several factors; however, once the sewage discharge point has been located, the only factors which the designer can vary are those associated with the pipeline's outlets.

Generally, the smaller and more numerous the outlets, or diffusers, the initial dilution, and so multiple diffusers are usual.

14.5.2 Methods of construction

Smaller size marine outfalls, up to about 2 meters in diameter, are usually constructed either:-

a) by pulling;
Or
b) Using a lay-barge.

The pulling method involves assembling the pipeline method involves assembling the pipeline in long strings on shore; the strings are successively pulled out to sea over shore-based launching rollers by means of a powerful winch located on a barge, anchored at sea on the route of the pipeline. Successive strings are joined together on shore and the sequence of pulling and jointing continues until the complete pipeline has been assembled and the seaward end has reached the proposed discharge point. Throughout the launching operation, the pipeline is kept sealed and full of air and, if required, additional buoyancy tanks are attached so that the submerged weight of the pipeline is such that it will just sinks.

In the lay-barge method, pipes are assembled and jointed on a barge. As the pipeline grows in length, it is progressively lowered to the sea-bed, down rollers attached to an inclined ladder, as the barge moves slowly seawards.

Large diameter submarine pipelines are usually laid from rigs, fitted with extensible legs so that they stand upon the sea-bed. Individual pipes or sections are carried to the rig in barges, lowered to the sea-bed in cradles and then positioned, aligned and jointed to the previously laid pipeline with the aid of divers.

14.5.3 Excavating for and Burying Submarine Pipeline

Submarine sewage outfalls are usually buried in the sea-bed. This can be carried out in a variety of ways, depending upon the nature of the sea-bed materials and the strengths of the local ocean currents.

If the sea-bed comprises bedrock, or other hard materials, it will be necessary to blast, usually employing shaped charges which, when fire, throw the loosened material out of the trench.

Where the subsoil is sand, silt or soft clay, much cheaper methods of excavation are available.

Dredging is suitable when it is certain that any trench excavated in the sea-bed will remain open sufficiently long to allow a pipeline to be laid in it. Self-propelled trailing suction hopper dredgers are usually most convenient and economical for preparing trenches for submarine pipelines; bucket and cutter/ suction dredgers, which are used in harbor work, are usually too expensive and cumbersome for use at sea. Dredgers are normally kept on line by electronic method, which are independent of visibility conditions.

A dredged trench usually has a minimum bottom width of about 5 metres (**16 feet**), which makes this method very expensive for small diameter submarine pipeline; in such cases, a clamshell excavator, working from an anchored barge is often cheaper.

Trenching equipment may be used to sink pre-fabricated pipelines into the sea-bed, in non-cohesive materials. During this operation, powerful air or water jets liquefy the bed material so that the pipeline slowly sinks. The equipment usually comprises a moveable frame which fits round the pipeline; the form is slowly pulled along the pipeline by means of a winch fixed to an anchored barge.

The same principle can be used to excavate a trench before the pipeline is constructed; in this case, submersible suction or air-lift pumps are used to remove the liquefied subsoil from the trench.

If submarine trenches are not likely to be refilled by natural deposition resulting from ocean currents, they are usually backfilled with sandy material, either dredged from other parts of the sea-bed and pipeline trench, or dug from shore-based borrow pits; the backfill is dropped over the trench from bottom-opening barges.

14.5.4 Design Details

A submarine pipeline must be strong enough to withstand all temporary and operating stresses and, after it has been laid, must have sufficient strength and stability to withstand any movement.

Temporary stresses, which are those which occur during construction, depend upon the methods of laying, jointing and burying employed. The magnitudes of operating stresses are affected by the flow velocities and the internal hydraulic pressure.

Movement may be caused by one or more of the following:

- i) Erosion of the sea-bed
- ii) Earthquakes
- iii) Current and wave-induced water forces
- iv) Foundation failure

14.5.5 Sea-bed Investigations

Although the hydrographic survey will be provided much important information, more detailed knowledge of the sea-bed is required before the optimum type of pipe and the method and depth of laying the pipeline can be decided.

The sea-bed investigations should normally provide additional information on:-

- i) The depth of bedrock, and other material requiring blasting
- ii) The engineering properties of the subsoil's, for example to ensure that they are capable of supporting the pipeline and any protection which may surround it;
- iii) The tendency of the bed to erode, or any evidence of deposition; these may be studied directly or by comparing sieve analyses of the subsoil with the measured local current close to the sea-bed.
- iv) Any corrosive properties of the subsoils
- v) Any submerged obstacles along the route of the pipeline

14.5.6 Materials for Submarine Pipelines

Steel, aluminum, concrete, plastics and fibreglass have all been used in the construction of submarine pipelines.

Usually, Marine outfalls laid by the pulling method are butt-welded steel pipes, with concrete surrounds to give negative buoyancy when empty. To prevent corrosion, steel pipelines have internal and external and external and external linings, often supplemented with cathodic protection.

Larger diameter outfalls are usually constructed using reinforced concrete pipes, jointed in situ.

Factors affecting the choice of material include;-

a) the relative costs and availability of alternative materials;
b) the diameter of the pipeline
c) simplicity of jointing
d) the strength of the material which includes, for example, the minim allowance radius of curvature;
e) the relative density of the material, which determines whether the pipeline must be weighted to make it sink;
f) the resistance of the material to biological and chemical corrosion;
g) the method of construction decided upon

15.0 THE RE-USE OF SEWAGE

Sewage contains over 99 per cent of water; water is often scarce in Kenya and there is thus a potential market for raw or treated sewage. Exploitation of this market could result in financial savings or even revenue for sewage authorities.

The re-use of sewage or treated sewage helps conserve natural water resources and also, by replacing sewage discharges, helps control water pollution.

The transport of sewage to a location where it may be re-used is technically simple; however, if the pumping is involved, it can be expensive.

Disadvantages inherent in the re-use of sewage are that it is most unpleasant in its raw state, and it is also a potential hazard to health. Treatment can virtually eliminate these characteristics. However, sewage also contains relatively high concentrations of dissolved inorganic salts, which remain even after the most extensive 'convectional' treatment, and these can remove only by prohibitively expensive further treatment.

Uses of raw or treated sewage considered appropriate for Kenya include:-

a) Irrigation of agricultural crops or forests.
b) Groundwater recharge.
c) Fish farming.
d) Industrial re-use.

15.1 Irrigation

This form of sewage re-use has been widely recognized and used internationally. In addition to being a virtually guaranteed permanent water source, sewage contains nutrients, especially nitrogen and trace elements, which give it some fertilizing value.

A disadvantage of irrigation as a method of sewage disposal is that irrigation water is not normally required during rainy seasons, and at such times an alternative method of disposing of sewage is required. There are two usual techniques for irrigation:-

a) Flood irrigation, where water is distributed by means of canals and ditches; this is often called the ridge and furrow technique.
b) Spray irrigation.

The first of these methods can encourage the spread of diseases such as bilharzia, or provide breeding grounds for insects, including mosquitoes. The second technique is probably more widely used; it is usual to subject the sewage to preliminary and primary treatment before spraying, to reduce aesthetic objections, to remove oils and greases which can clog soil and to prevent blockage of the spray nozzles.

Dissolved inorganic salts in sewage, such as sodium chloride, limit its use to those crops which have higher tolerances for these substances.However, the major restrictions on the use of sewage as irrigation water are related to health risks.

Several international bodies, including United States health departments, control crop irrigation by sewage in accordance with the following regulations:-

a) Irrigation is restricted to industrial or fodder crops and to crops which are not to be eaten raw.
b) Fruit trees may be irrigated using the ridge and furrow technique, so that the fruit is not contaminated.

c) Unrestricted irrigation of all crops with sewage effluent is permissible only when it meets the bacteriological standards of drinking water.

It is suggested that sewage irrigation in Kenya should be subjected to similar controls. The irrigation of forested areas appears to be an application particularly suited to Kenya.

15.1.1 Standard for Water for Irrigation Purpose

Any water which has to be used for irrigation has to be bacteriological and chemically fit.

The irrigation standard of which any irrigation water has to comply to be used for food which is to be used uncooked, the coliform count should be less than 1,000/100ml (MPN).

Further, it requires fruit to be picked off the ground and Overhead irrigation avoided.

Table 15.1 and 15.2 has the details.

Table 15.1 Physical Water Quality Guidelines for Irrigation water

Parameter	**Permissible Level**
pH	6.5-8.5
Aluminum (Al)	5 mg/L
Arsenic (Ar)	0.1 mg/L
Boron (Bo)	0.1 mg/L
Cadmium (Cd)	0.5 mg/L
Chloride (Cl)	0.01 mg/L
Chromium (Cr)	1.5 mg/L
Cobalt (Co)	0.1 mg/L
Copper (Cu)	0.05 mg/L
E. Coli	NIL/100 ml
Fluoride (F)	1.0 mg/L
Iron (Fe)	1.0 mg/L
Lead (Pb)	5 mg/L
Selenium (Se)	0.19 mg/L
Sodium Absorption Ratio (SAR)	6.0 mg/L
Total Dissolved Solids (TDS)	1200 mg/L
Zinc (Zn)	2 mg/L

Source: - The Environmental Management and Co-ordination (Water Quality) Regulations, 2006 Ninth Schedule page 558

Table 15.2 Microbiological Water Quality Guidelines for Irrigation water

Reuse Condition	**Exposed Group**	**Intestinal Nematodes (MPN/L)**	**Coliforms (MPN/100 ml)**
Unrestricted Irrigation (crops likely to be eaten uncooked)	Workers and Consumers	< 1	<1000
Restricted Irrigation (Cereal crops, industrial crops, fodder crops, and pasture	Workers and Consumers	< 1	No standard recommended

- Ascaris Lumbricoides, Trichuris and human hookworms

- A more stringent guideline (<200 coliform group of bacteria per 100ml) is appropriate for public lawns, such as hotel lawns, with which the public mast come into direct contact
- In the case of fruit trees, irrigation should cease two weeks before fruit is picked and fruit should be picked off the ground. Overhead irrigation should not be used.

Source: - The Environmental Management and Co-ordination (Water Quality) Regulations, 2006 Eight Schedule page 557

15.2 Ground water Recharge

This may be used to combat sea-water intrusion or to supplement underground water resources. In this latter case, the technique can be considered as an indirect re-use of sewage which takes advantage of the natural treatment and prolonged storage afforded by the ground; unfortunately, this hardly affect the concentration of dissolved inorganic salts in the sewage effluent.

To prevent clogging of the aquifer, only good quality sewage effluent may be utilized for recharge; in addition, when the underground water is used for public supply, tertiary treatment of the effluent, using ‘water supply treatment’ techniques such as coagulation, sand filtration and chlorination, should be carried out.

Recharge may be either by means of relatively shallow trenches or spreading basins or through deep boreholes, the choice depending upon the location of the aquifer.

15.3 Fish Farming

Sewage ponds are used internationally for breeding fish for food, and it is considered that secondary stabilization ponds (following primary ponds) and maturation ponds in Kenya could be utilized in this way.

Aesthetic objections from the public to such a form of food production may have to be overcome, and with some sewage there may be a problem of tainted flesh. Also, there is a potential health risk, but this hazard is acceptably low when the fish are properly cooked before eating.

15.4 Industrial Re-use

Some risks are inherent in the re-use of treated sewage by industry, for example because of the danger of cross-connecting sewage and water supply mains; re-cycling within a factory of its own treated effluents is generally safer. However, provided that care is taken to safeguard to public health, there are many profitable ways in which treated sewage may be utilized industrially-again, the guaranteed reliability of a sewage supply is a great asset.

Industry usually requires large volumes of low-grade water, for example for cooling purposes or for quenching during steel manufacture; effluent from a sewage treatment works is well-suited for such applications. Further processing of sewage effluent, by techniques ranging from ‘water supply treatment’ to deionization can produce virtually any higher grade water that industry is likely to need.

It is recommended that the re-use of water by industries should be nationally encouraged; however, this is not to suggest that it should be financially by sewerage authorities.

Although they should always be willing to let industry have sewage or sewage effluent, possibly at no charge sewerage authorities should not become involved in additional capital outlay or take an increase responsibilities, such as guaranteeing the quality of the sewage or sewage effluent they supply.

Industry it self should provide and operate any additional stations, distribution mains or treatment units required to bring the sewage or sewage effluent to the location and to the quality necessary. In this way, industry and not a sewerage authority will stand financial loss should the industrial demand for the sewage or effluent be less than anticipated.

It is suggested that sewage authorities should assure themselves that any transportation or use of sewage effluent by a particular factory will cause no health hazards; however. All risk pertaining to the correct use of these waters should be borne by industry.

15.5 Reuse of excrete

Human excrete should be regarded as a natural resource to be conserved and reused under careful control rather than being discarded. Excreta for reuse are derived from:

- nightsoil, including that collected by municipal systems or private contractors, and the nightsoil of individual households or groups of households and used on their own gardens or farms;
- solids from full pit latrines;
- sludge, scum and liquor from septic tanks, aqua-privies, vaults and cesspits; and
- raw and treated sewage and sludge from sewage treatment works (which are outside the scope of this book)

Solids from pit latrines are innocuous if the latrines have not been used for two years or so, as in alternating double pits. R‘1qq122`raw excrete from all other sources are likely to include recently excreted faeces and may therefore contain active pathogens.

There are three basic methods of using this resource: agriculture, aquaculture, and biogas production.

15.6 Use in agriculture

Human excrete are a rich source of nitrogen and other nutrients necessary for plant growth. The most common method of reuse is direct application to the soil as a fertilizer. Nightsoil contains about 0.6% nitrogen, 0.2% phosphorus and 0.3% potassium, all of which are valuable plant nutrients. The humus formed by decomposed faeces also contains trace elements which reduce the susceptibility of plants to parasites and diseases. Humus improves the soil structure, enhancing its water-retaining qualities and encouraging better root structure of plants. Soil containing humus is less subject to erosion by wind and water and is easier to cultivate.

Health risks

For centuries, untreated nightsoil has been widely used as a fertilizer in east and south Asia, although there is an increasing awareness of the public health dangers involved.
Pathogens of all kinds can remain viable in the soil and on crops. Death of pathogens on crops is usually caused by desiccation and direct sunlight, so pathogens are generally more persistent in humid cloudy climates than in arid areas.

It has been suggested that the use of raw excrete and the effluent from septic tanks is acceptable only if confined to industrial crops and foodstuffs that are cooked before being eaten. However, even with these crops, there is considerable risk of pathogen transmission to agricultural workers, to people involved in transporting crops, and to those processing industrial crops or preparing food for cooking. Therefore such use must be carefully planned with strict surveillance by the health authorities.

The risks arising from pathogen transmission from the use of untreated excrete or sludge on food crops may be greater for populations with high levels of hygiene and health (for example people in towns) than for agricultural workers living in areas where excretaderived diseases are endemic (Feachem et al., 1983).

Excreta on paddy fields

Fields with crops standing in water during part or all of the vegetation period are potential transmission sites for schistosomiasis if fresh excrete are used as fertilizer (Cross & Strauss, 1985).

Fertilization of trees

Treated or untreated sewage is sometimes used to irrigate trees. This practice is most common in arid climates, where trees are watered to control desertification, to provide shade and windbreaks, or to cultivate coconuts and some other food crops. The main health risk is to workers and members of the public who have access to the plantation.

Excreta on pasture

When excrete are applied to land on which cattle graze there is a danger of the spread of beef tapeworm, whose eggs may survive on soil or pasture for more than six months.

Composting

Excreta may be treated in various ways to eliminate the possibilities of disease transmission. Apart from storage in double-pit latrines, the most appropriate treatment for on-site sanitation is composting.

Composting consists of the biological breakdown of solid organic matter to produce a humic substance (compost) which is valuable as a fertilizer and soil conditioner. It has been practiced by farmers and gardeners throughout the world for many centuries. In China, the practice of composting human wastes with crop residues has enabled the soil to support high population densities without loss of fertility for more than 4000 years (McGarry & Stainforth, 1978).

Nightsoil or sludge may be composted with straw and other vegetable waste, or with mixed refuse from domestic, commercial or institutional premises. The process may be aerobic or anaerobic.

Aerobic bacteria combine some of the carbon in organic matter with oxygen in the air to produce carbon dioxide, releasing energy. Some energy is used by the bacteria to reproduce. The rest is converted to heat, often raising the temperature to more than 70°C. At high temperatures there is rapid destruction of pathogenic bacteria and protozoa, worm eggs and weed seeds. All faecal microorganisms, including enteric viruses and roundworm eggs, will die if the temperature exceeds 46 °C for one week. Fly eggs, larvae and pupae are also killed at these temperatures. No objectionable odour is given off if the material remains aerobic.

In the absence of oxygen, nitrogen in organic matter is converted to acids and then to Ammonia; carbon is reduced to methane and sulfur to hydrogen sulfide. There is severe odour nuisance. Complete elimination of pathogens is slow, taking up to twelve months for roundworm eggs, for example.

Practical composting

The traditional method of composting is to pile vegetable waste with animal manure and nightsoil or sludge on open ground. Aerobic conditions may be maintained by regular turning of the material, which also has the advantage of making the moisture content more uniform throughout the tip. Under aerobic conditions, rapid decomposition of organic matter takes place in the first 2- 4 weeks. The process is considerably shorter than under anaerobic conditions. Controlled composting in mechanized composting equipment shortens the process even more.

According to Flintoff (1984) there are five preconditions for successful composting:

- suitability of the wastes;
- marketability of the product;
- support of authorities, particularly those in agriculture;
- a price for the product that is acceptable to farmers; and
- a net cost (i.e., process costs less income from sale) that can be sustained by the operating authority.

Pretreatment

In developing countries most domestic refuse is vegetable matter, and there may be little paper, glass or metal. Where these materials are more common, paper can be composted and some glass is acceptable in compost if it is ground up at some stage of the composting process. Metals need to be removed. Textiles, plastics, leather and the like may be removed or they may be shredded and included in the compost. Dust and ash may also be included but, if they form too large a proportion of the refuse, the value of the compost is reduced.

Working over refuse heaps with forks to break down large lumps helps the composting process. Broken-down refuse has a greater surface area for air to enter and for bacteria to attack. It allows less penetration of rain and fly control is easier.

Control of composting

Too much moisture in a heap of composting material fills the spaces between particles, preventing air from getting in. On the other hand, bacteria do not flourish if the material is too dry. The optimum moisture content is 40-60%. Moisture content can be increased by spraying a compost heap with water, and can be decreased by adding dry straw or sawdust. Frequent turning allows a heap to dry naturally by evaporation.

For optimum value to plants, the ratio of available carbon to nitrogen in compost should be about 20. In the composting process carbon is used by the bacteria, so the best raw material for composting has a higher carbon: nitrogen ratio, say about 30. The carbon: nitrogen ratio of nightsoil is about 6, of fresh vegetable waste around 20, and of dry straw over 100. The ratio of mixed household refuse is often in the range 30 -50, but it may be higher if there is a high paper content. The desirable ratio of 30 can sometimes be obtained by judicious mixing of incoming waste, for example by adjusting the proportions of nightsoil or sludge and vegetable waste. It is rarely practical to determine the carbon: nitrogen ratio by chemical analysis; a good operator learns to judge what mix of materials will produce the best compost.

During composting the volume is reduced by 40-80% and the weight by 20-50%.

Windrows and pits

Unless expensive mechanical plant is used, aerobic composting of municipal refuse is usually carried out in long heaps called windrows. The best height for windrows is about 1.5 m. In heaps more than 1.8 m high, the material at the bottom becomes too compressed; in heaps less than I m high, too much of the heat generated by the bacteria is lost.

The width and length of windrows should be planned for the most efficient handling of materials and the best utilization of the area available. The initial width is often 2.5-3.5 m at the bottom. In dry weather the cross-section should be trapezoidal, as shown in Fig.
A1.1, but during the rainy season a more rounded shape prevents the material getting too wet.

For composting small quantities (for example, from a single village), refuse should be stored until there is enough to make a pile about 3 m in diameter and 1.5 m deep.

For composting nightsoil, a common method is to place alternate layers of nightsoil (about 50 mm thick) and vegetable waste (about 200 mm thick) in pits or windrows. Fig. 15.2(a) shows how a windrow can be formed to ensure destruction of faecal pathogens by high temperature. Vegetable matter below and at the edges provides some insulation. Fig. 15.2(b) shows an alternative method: after a windrow has been in use for two or three days and the temperature has risen, a trench or pocket is formed in the centre and nightsoil is poured in.

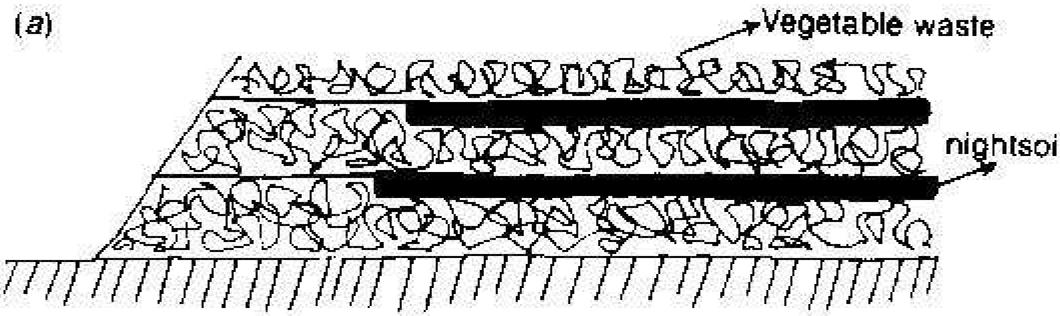


Fig. 15.1 A compost windrow

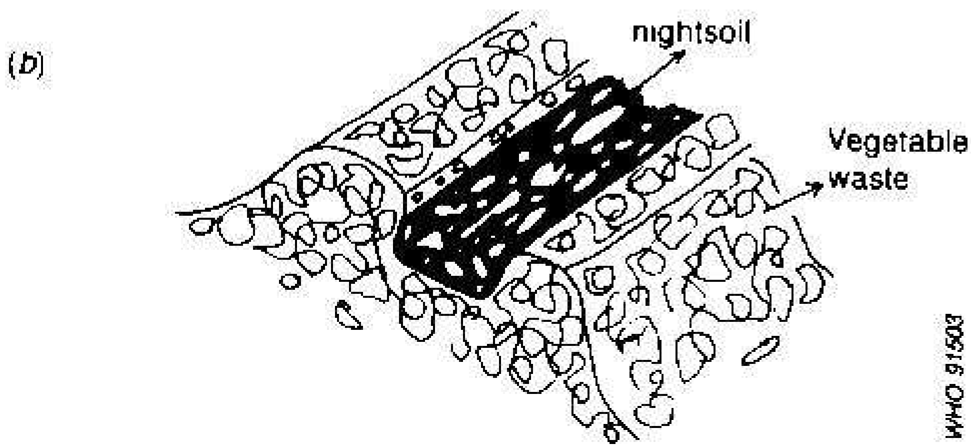


Fig. 15.2 Placing night soil in a compost windrow

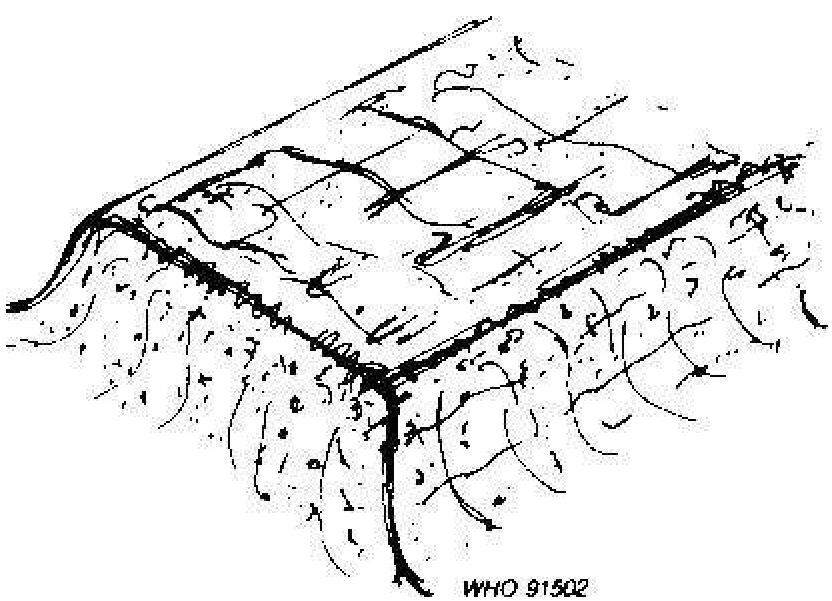
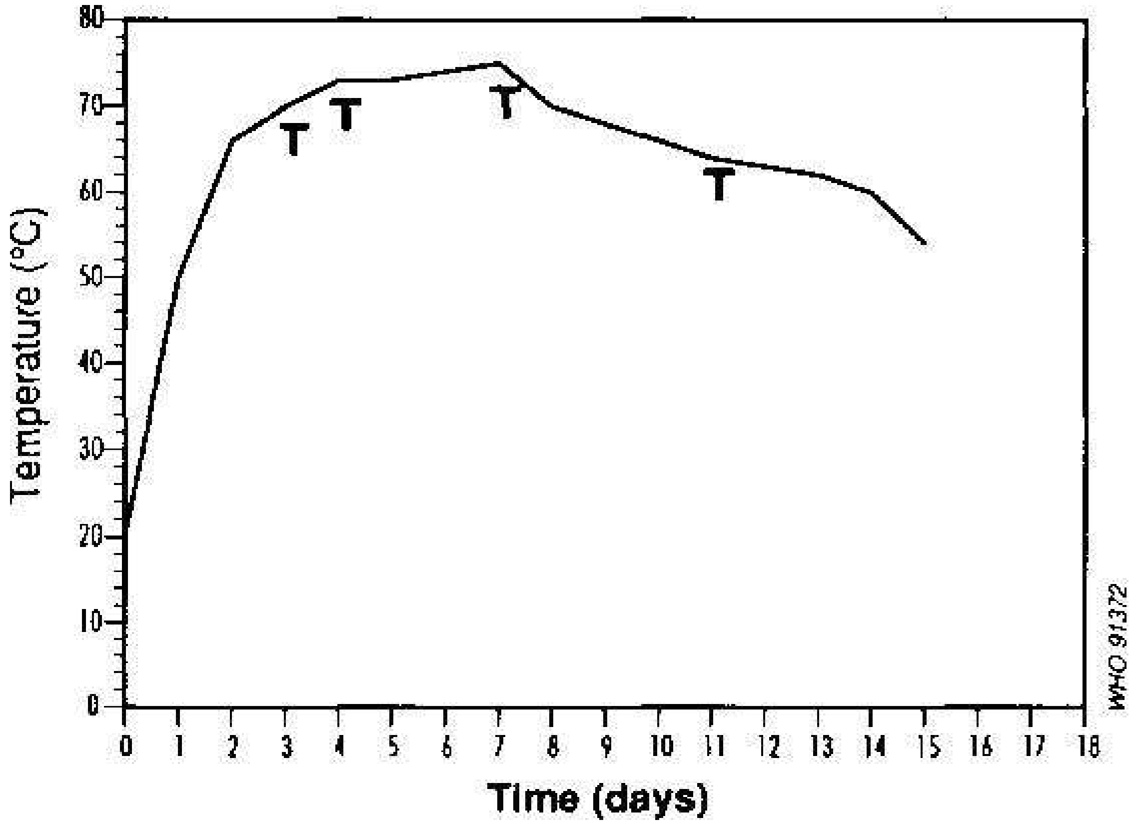


Fig. 15.3 Temperature variation during aerobic decomposition of mixed refuse (T=point at which material was turned for aeration)

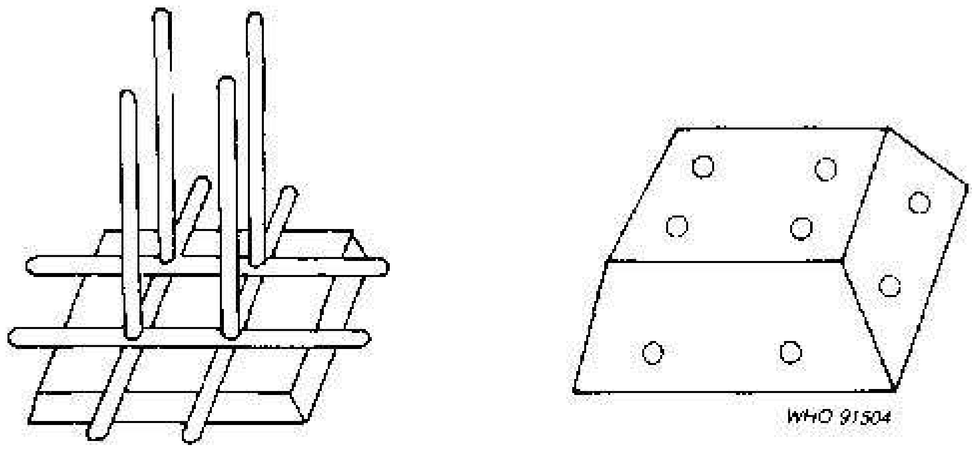


Temperature, aeration and turning

Providing that the material being composted remains aerobic the temperature may rise to 45-50 °C during the first 24 hours. A few days later it will reach 60-70-C, well above the lethal temperature for all pathogenic organisms. Fig. 15.3 shows the variation in temperature during aerobic composting of mixed municipal refuse; the points marked T indicate when the material was turned for aeration.

Various methods of aeration have been tried. For a small refuse heap (as in a village), refuse can be tipped over bamboo or timber poles which are removed when the heap is complete, leaving holes through which air reaches the refuse (see Fig. 15.4). Other approaches, including forced aeration (using compressed air blowers or suction) and use of porous floors, have not been very successful in keeping large masses of material aerated.

Fig. 15.4 Aeration of compost by placing around poles



Material in windrows can be turned by labourers using forks or by adapted earth-moving equipment. Turning should keep the heap aerobic. In addition, material at the outside should be moved to the centre, because the outer layers may:

- be too wet because of rain;
- be too dry owing to evaporation (especially on the side facing the wind);
- be unaffected by the temperature rise in the centre of the windrow;
- contain large numbers of flies, fly eggs and larvae

Some operators turn their windrows every two or three days. However, aerobic conditions can be maintained with less frequent turning after the first week or so. One suggested pattern is to turn the windrow after one day and then on the 3rd, 7th, 14th and 21st days. On the 28th day the material is put into a storage area to await removal.

Generally compost is only required at certain times of the year. If there is only one harvesting and sowing season a year, an area sufficient to store most of the year's production of compost may be required. During storage, compost continues to "mature", but high temperatures cannot be maintained. The time taken for stabilization depends on the initial carbon: nitrogen ratio, the moisture content, maintenance of aerobic conditions and the particle size. Unless precautions are taken, fly breeding may be a problem when compost is stored.

Condition and quality of compost

Tests of compost during and after stabilization show whether the process is going well and whether the finished product is suitable for agricultural use. Except in a large mechanical composting plant, the condition of the compost is gauged by simple methods. It is reasonable to assume that pathogenic organisms will be killed if the temperature rises above 65°C. This can be confirmed by poking an iron bar or wooden stick into the heap and pulling it out after about ten minutes. It should then be too hot to hold. The temperature falls when stabilization is complete. Absence of an unpleasant smell and absence of flies also indicate satisfactory aerobic composting (Flintoff, 1984). An experienced operator can check that all is well from the appearance of the composting material. It should look moist, but not so wet that liquid seeps out. While aerobic stabilization is progressing the appearance will change from day to day. Anaerobic conditions are shown by a pale green, slightly luminous appearance of material inside the heap.

Farmers and market gardeners may want to know the chemical composition of compost derived from nightsoil or sludge. The major plant nutrients (nitrogen, phosphorus pentoxide and potassium oxide) are likely to be about 3% by weight, three times the concentration in compost from municipal refuse.

Use in aquaculture

The practice of depositing excretes into fish ponds or a tank is common in many Asian countries. In some places, latrines are placed immediately over or alongside ponds; elsewhere nightsoil is tipped from carts, tankers or buckets. Nutrients in excrete result in a rich algal growth, which encourages aerobic conditions and provides food for certain fish.

Carp and tilapia are especially suitable for such ponds, but a variety of fish species may coexist, some feeding on large algae, some on small algae, some on zooplankton; some prefer the bottom layer, some the top. Fish are usually netted for human consumption, but in some places they are dried and ground up for feed for poultry or animals. Ducks may also be kept on the ponds.

There are three health risks associated with fish farming in ponds that receive excrete.

(1.) Pathogens may be transmitted on the body surfaces or in the intestines of the fish without causing overt disease in the fish; the pathogens may then be passed to people handling the fish.
(2.) Helminths, particularly flukes, may be transmitted to people who eat infected fish that has not been properly cooked.
(3.) Helminths with intermediate hosts (such as Schistosoma with water snails) may continue their life cycle in ponds.

The WHO publication, Guidelines for the safe use of wastewater and excreta in agriculture and aquaculture (Mare & Cairncross, 1989), gives further useful information.

Biogas production

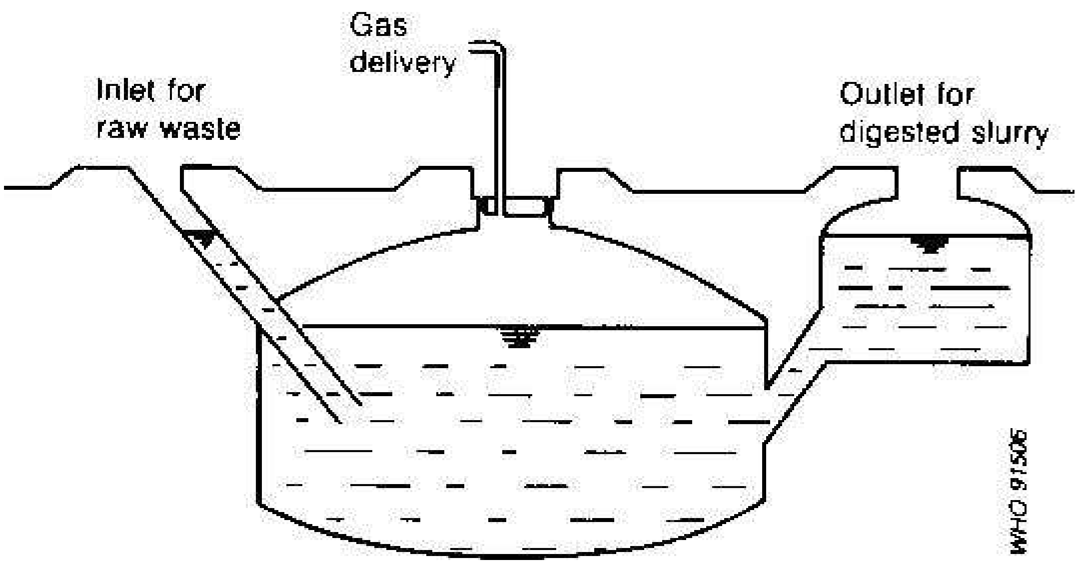
The search for alternative sources of energy has led to widespread use of organic waste to produce a combustible fuel which can be used for domestic cooking. Basically, a biogas plant consists of a chamber in which excrete are fermented, producing gas which contains about 60% methane. The biogas is collected at the top of the chamber, from which a pipe leads to domestic appliances or to flexible storage containers.

A few biogas plants operate entirely on human excrete. For example, in Patna, India a 24-seat pour-flush latrine serves several thousand people and generates sufficient energy to light a 4-km length of road. However, most plants, of which there are more than 7 million in China (Li, 1984), are dependent on animal excrete with which human excrete are processed. A medium-sized buffalo or cow provides about twenty times as much gas as a person. The minimum feed is that from one cow and a family of people, although it is more usual to add excrete from at least four cows. In China it is customary to produce biogas from the excrete of pigs.

Construction

Although there are many variations, the most common types of domestic plant have a floating or fixed dome under which the gas collects. The floating dome type, shown in Fig. 15.5, is widely used in India. In China, masonry or concrete fixed domes are usual, as shown in Fig. 15.6. They are generally cheaper than those with a floating roof. The daily gas output is approximately equal to one-third the volume of the digester.

Fig. 15.5 Biogas plant with floating dome



Operation

Excreta are often mixed with straw or other vegetable waste, such as that used for animal bedding, and equal quantities of water added to make slurry. This is fed to the inlet side of the chamber. Effluent slurry is removed after a retention time of 30-50 days. Biogas production is greater at higher temperatures. At 30^0C the rate of generation of gas is about twice that at 25^0C, and little gas is produced if the temperature is below 15^0C.

The effluent slurry is usually dried in the open and used as a fertilizer. On a dry solids basis, the nitrogen content is greater than in untreated excrete because of the loss of carbon in the gas. The nutrients in effluent slurry, whether dried or applied directly to land, are more readily taken up by plants.

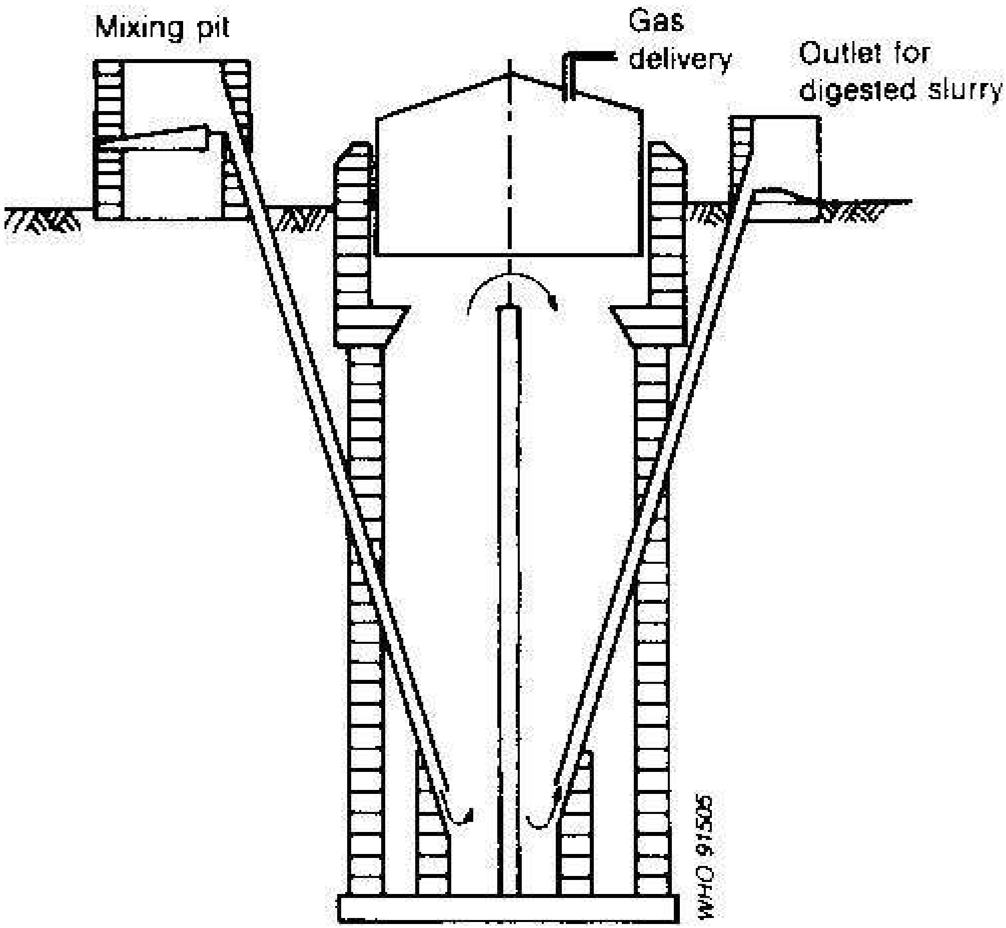


Fig. 15.6 Biogas plant with fixed dome

Health risks

Retention of excrete in biogas tanks results in the death of many pathogens, including Schistosoma eggs. A few hookworm eggs survive, and there is high survival of roundworm eggs.

16.0 COMPUTER AIDED DESIGN

16.1 Information Technology

Information technology is the technology, which supports activities involving the creation, storage, manipulation and communication of information together with their related methods, management and application

16.2 Computer

A computer is a device that works electronically under the control of stored programs, automatically accepting, storing and processing data to produce information that is the result of processing.

The forms in which data is accepted or produced by the computer vary enormously from simple words or numbers to signals sent from or received by other items of technology. The functions of the computer include: -

- **Input ----------** accepts data from outside for processing.
- **Storage--------** holds data internally before, during and after processing.
- **Processing----** performs operations on the data it holds within.
- **Output---------** produces data or information from within for external use.

16.3 Hardware

Hardware is the name given to all the physical devices found in a computer, including its associated facilities, the peripherals.

All electronic elements found in the computer e.g. the transistors; diodes, electronic paths/channels etc, and the input/output and storage devices are collectively described as the hardware.

Computer peripherals are the term used to describe all the elements connected to the computer apart from the computer itself. These are facilities connected to the computer to assist the computer in satisfying its users and include the input/output units that interface the computer from its users and the storage units that supplement the computer internal memory.

16.4 Software

Software is the general term used to describe all the various programs that may be used on a computer system together with associated documentation.

There are two classes of software: -

(i) **Applications software** – Software that is designed for specific use, for example, designed to solve sets of mathematical equations or controlling organizations budgetary allocations.

Many applications packages are designed in such a way that they can be used for a variety of similar problems. For example, engineering design packages are produced in forms that enable them to be set up and used by different disciplines, each having slightly different ways in which they need to produce their designs.

There is abundant selection of engineering packages that can be used on personal computers in the market.

(ii) **Systems software –** These are programs with associated documentation that control the way the computer operates or provide facilities that extend the general capabilities of the system.

Within systems software is operating system composed of a specific program or suite of programs that controls the performance of the computer by doing a variety of jobs to ensure the proper, orderly and efficient use of hardware and application programs. Examples of operating systems will include MICROSOFT WINDOWS AND MACNTOSH.

NOTE: Applications programs only work when used in conjunction with the operating system.

16.5 The Internet

Internet is the world's largest computer network, consisting of over 2 million computers supporting over 20 million users in almost 200 countries. Its growth is phenomenal – between 20 to 30% per month and hence any estimate is quickly out of date.

The value of the Internet lies precisely in its ability, easily and inexpensively, to connect so many diverse people from so many places all over the world. Anyone who has an Internet address can log on to a computer and peach virtually every other computer on the network, regardless of location, computer type, or operating system.

16.6 Functions

Internet carries many kinds of traffic and provides users with several functions that include:

- Communication through e-mail, Usenet, newsgroups, chatting and telnet.
- Information retrieval through Gopher, Archie, Wide Area Information Servers (WAIS), and the World Wide Web (The Web).

16.7 Using Internet

Most characteristics of Internet is how data if taken from one computer to the other. Data transfer can be summarized as follows: -

(i) Piece of data is broken into small pieces called PCK

(ii) A header is added. This header explains

a) Where the PCK comes from

b) Where it should end up

c) How it is fixed with the rest of the PCK

(iii) A PCK is sent from computer to computer until it finds its way to its destination

NB: A computer along the way decides where next to send the PCK depending on how busy other computers are at the time of PCK was RX.

(iv) At destination the PCK are examined. If a PCK is missing or damaged a message is sent asking for this PCK to be resent and this is procedure is repeated until all PCK have been received intact.

(v) Packets are re-assembled into their original form load and at computers must have TCP/IP responsible for receiving sending and checking Packets.

The Internet also consists of a number of regional exchanges, which are large computers called **servers**. These servers are connected by a large cable (of copper wire), which is quite separate to the telephone system. Undersea cables connect the Australian system to the American system, and thence to the rest of the world. Universities and government departments own these servers, and are thus able to pass information back and forth. They are thus permanently connected to the Internet.

Recently, private companies called **Internet Service Providers (ISPs)** have established servers on the Internet cable. Individual users from their home or office connect to the server of an ISP, and thereby gain access to the entire Internet system. This connection between a user and their ISP is made over a normal telephone line. The ISP has a number of incoming lines to their office (like an SP bookie), which enable a number of users to connect to their server.

16.8 Data & Information Transfer and Retrieval

Internet is a vast computer database with variety of computer resources and information. The only easy way of finding information on internet is by using URL which is specified on location or address bar.
Some of the problems encountered when searching information on the internet

(i) Incorrect URL or the URL has been changed
(ii) What if we do not know the URL
(iii) Solution to the above is use of search engines

Searching engine is a computer that goes around the internet to find documents that contain the word that you type. The search engine also incites or takes information provide web designer and put it in a database.

Search engines works in different ways:-

a) Some rely on people to maintain a catalogue of the web log
b) Some use S/W to search key information on sides across the internet

E.g. of search engines
-Google search
-Yahoo search
-Excite
-NL search

Services available on the Internet

Internet provides a variety of services ranging from accessing remote computers and using then for transferring files to and from the computers on the Internet. These services are:

16.9 The Web

- The web is an information retrieval tool similar to Groper, Archie and WAIS. However, it has had an enormous impact on the commercial use of the net owing to its being attractive and easy to use.
- While other information retrieval methods are text-based, the web combines text, hyper media, graphics and sounds, making information more appealing more informative and easier to grass.

16.10 Benefits of Internet Use

- Reducing communication costs
- Enhancing communication and Coordination
- Accelerating distribution of knowledge
- Improving customer service and satisfaction
- Facilitating marketing and sales.

16.11 Problems of Internet Use

- Lack of security
- Technology problems
- Legal issues
- Anti commercial culture.

16.12 Spreadsheet

16.12.1 Description

- A spreadsheet or worksheet is an application program that displays a working sheet consisting of rows and columns of cells; the rows are usually identified by numbers, and the columns by letters.
- Each cell can hold a numeric value, a text label or a formula that processes values contained in other cells.
- Many spreadsheets contain extensive libraries of built-in formulas and can even maintain links to other spreadsheets so that data entered in one spreadsheet updates entries in another.
- Extensive data presentation featured and standard in most spreadsheets, which include reports generation and powerful graphing capabilities.

16.13 GIS Database

At the frontier of the information age, geographic information management technology is emerging as a powerful means to manage voluminous geographic data, to help cope with the information explosion, and to provide a foundation for solving the problems that beset planet earth and its inhabitants.

Societies in general are becoming keenly aware of the need to manage information from a geographic perspective. This awareness has been brought about by the twentieth –century trends toward a global community and economy. At the same time, the often negative impact of advancing technology has shown the need for wise management of the earth's resources. Geographic information systems (GIS) provide the tools to help meet these challenges.

Geographic information systems (GIS) rely on the integration of three distinct aspects of computer technology:

a. Database management (of graphic and non-graphic data);
b. Routines for manipulating, displaying and plotting graphic representations of the data; and
c. Algorithms and techniques that facilitate spatial analysis.

16.13.1 Definitions

Data: A set of facts or figures, which have been gathered systematically, and form which conclusions may be drawn e.g. coordinates, pollution data etc.

Information: Data that have been processed to have meaning to a user.

Geographic Information Systems (GIS):
Francis Hanigan (1988) defines a GIS as "any information management system which can:

(a) Collect, store and retrieve information based on its spatial location;
(b) Identify locations within a targeted environment which meet specific criteria;
(c) Explore relationships among data sets within that environment;
(d) Analyze the related data spatially as an aid to making decisions about that environment.
(e) Facilitate selecting and passing data to application – specific analytical models capable of assessing the impact of alternatives on the chosen environment.
(f) Display the selected environment both graphically and numerically either before or after analysis".

GIS database: A pool of integrated and structured graphic and non-graphic data from which relevant facts may be retrieved and processed to provide information to users.

16.13.2 Digital Data for GIS

A G.I.S database can be divided into two basic types of data: graphic and non-graphic (attribute) data. Each of these types has specific characteristics and requirements for data storage, processing and display. In G.I.S, data consists of features or entities, which are represented by four geometric concepts i.e., points, lines, polygons and surfaces.

(a) Points: Points describe individual objects depicted by cartographic symbols e.g. Schools, hospitals, spot heights.
(b) Lines: Are used to describe networks e.g. roads, railways rivers canals, contours etc. Normally networks are assumed to have no area because their widths on maps are assumed to be zero.
(c) Polygons: Are used to describe area features such as zones of population density, administrative districts, lakes etc.
(d) Surfaces: Are a three-dimensional representation of features e.g. a Digital Terrain Model (DTM) of a hill.

16.13.3 Feature Definition in GIS

In the GIS database, any feature is defined by 3 parameters:

(a) Its position
(b) Its attributes; and
(c) Its relationships to other features.

Feature Position: Feature position consists of locational data defining a feature's spatial extent. It may be in either vector or raster formats. In vector format, locational data consists of Cartesian coordinates in some frame of reference e.g. UTM projection.
(a) Point: represented by a pair of coordinates
(b) Line: represented by series of coordinates
(c) Polygon: represented by a series of coordinates closing onto themselves
(d) Surface: represented by 3D coordinates.

In raster format:

(a) Point: represented by a single pixel whose value is different from the value of neighbouring pixels
(b) Line: represented by a series of contiguous pixels of the same value surrounded by pixels of different values.
(c) Polygon: represented by a 2-D extent of pixels of equal value
(d) Surface: represented y a 2-D extent of pixels of equal value, plus a height value at each pixel.

16.13.4 Feature Attributes

Feature attributes is data that specify the non-geometric characteristics of a feature. Attributes may be numeric (eg. size, slope, height etc) or semantic (eg. name, type, etc). Feature attributes in GIS are stored by means of feature codes. A feature code is a concise alphanumeric code which describes the type of feature represented by given coordinates according to some chosen coding scheme. A commonly used coding technique is that of generalization and specialization which arranges features hierarchically in a tree structure.

16.13.5 Feature Relationships

Spatial relationships i.e. neighborhood, connectivity and inclusively are not as apparent in digital data as in analogue data. Digital data cannot adequately represent the real world without relationships. There are three kinds of relationships:

(a) **Proximal-** Describe the closeness of non-adjacent features i.e. nearness or farness.
(b) **Hierarchical** – Describe the relationship of feature subclasses to their super classes. It is encoded by a tree structure.
(c) **Topological** – Describe the neighborhood, connectivity and inclusion properties of features. It is encoded via a topological data structure in a process called topology building.

16.13.6 Sources of Data for GIS Database

(a) Analogue maps and plans
(b) Digital remotely sensed imagery
(c) Surveying field notes
(d) Total Stations
(e) Photogrammetric work stations
(f) Aerial photographs
(g) GPS
(h) Tabular data e.g. Census, rainfall, soils etc. from various collecting agencies.
(i)Direct import from other GIS's

16.14 Computer Aided Design (CAD)

16.14.1 Introduction

- The use of a computer to produce designs is referred to "computer Aided Designs (CAD)" sometimes known as "Computer Aided Drafting and Design (CADD)".
- CAD programs present objects as wire frame outlines or more complex three- dimensional (3D) images.
- CAD reduces the time needed to create, edit, store and transmit drawings by using high performance computers and monitors, with inputs devices like scanners and graphics tablets, sending their output mostly to laser printers or to pen plotters.

16.14.2 CAD capabilities

The capabilities of modern CAD systems include:

- Reuse of design components
- Ease of design modification and versioning
- Automatic generation of standard components of the design
- Validation/verification of designs against specifications and design rules
- Simulation of designs without building a physical prototype
- Automated design of assemblies, which are collections of parts and/or other assemblies
- Output of engineering documentation, such as manufacturing drawings, and Bill of Materials
- Output of design directly to manufacturing facilities
- Output directly to or Rapid Manufacture Machine for industrial prototypes

16.14.3 Tools and Methods

Development in CAD has resulted in the following tools and methods:

- Wire frames
- Solid modeling
- Intelligent wiring diagrams and production linked database systems
- Graphically represented system or plant diagrams and databases
- Parametric design models
- Real-time process simulation
- Computer Numerically Controlled (CNC)) load files (tool path instructions)
- Finite Element Analysis (FEA)
- Rapid prototyping

Many CAD drawings are created from scratch using the application software using design sketches and other inputs. Other CAD drawings are created from pre-existing electronic CAD files by copying all or part of another CAD file, making changes, then saving it as a new file. Drawings that only exist in physical form (blueprints, plots of lost files, etc.) can be converted into CAD files using a procedure called "Paper-to-CAD conversion", drawing conversion, digitization, or vectorization.

16.14.4 CAD application in Sanitation and Sewerage Services

All personnel involved in the water supply need to have knowledge of data processing through computer. In more recent years, the technological developments associated with the creation, manipulation, storage and communication of data and information, principally computing, telecommunications and electronics is now carried out through computers. These developments have given rise not only to the rapid evolution of data processing techniques but also to a greater integration of the data processing techniques with other activities in an organization. This calls for technical personnel in the water supply industry to have knowledge of data processing and its related information technology.

In a Sanitation system, the computer-aided designs are based on computer model network analysis that is widely used to identify the causes of deficiencies in a proposed system, thereby developing the most effective components. The network analysis of water systems will include a number of interdependent variables: -

- Water sources (Quality and quantity)
- Water demand (Consumption)
- Intakes (Capacity and type)
- Pipelines (Diameters, material and pressures)
- Treatment (Quality, quantity and methodology)
- Storage (Capacity, duration and type)

System efficiency and effectiveness, and hence system performance can be affected by altering various combinations of these variables. Comprehensive steady state analysis of the water supply systems should perform for a wide range of water demand conditions. However, if the Sanitation System can operate satisfactorily under the most severe demand conditions, it will operate satisfactorily for all conditions. For this reason, the demand condition most limiting to the performance of the Sanitation System variables will be established, and the computer model runs will represent the system operations at these most limiting demand conditions.

16.14.5 Hydraulics Mathematical Models Applications

16.14.6 Models and Modeling Systems

The Use or numerical models as tools in engine {e ring practice has increased rapidly over the last decades}. In fact, the major hydraulics laboratories of Western Europe now do more than 80% of their modeling work with mathematical models.

Modeling has nearly always been a part of engineering practice. Solving a particular river problem mainly requires construction of a model; this model could be a physical (scale) model or mathematical (numerical) model.

16.14.7 Applications of mathematical models

With the introduction of the digital computer it became possible to analyze and solve whole sets of governing equations using solution methods which, because of their large number and long Complex Calculations had been previously considered too difficult and unreliable due to the possibility or making (simple) mistakes in hand calculations.

- The application of these solution methods to the analysis and design of engineering problems is referred to as *numerical (mathematical) modeling.*
- Models canbe applied to an extremely wide range of problems. However, in order to keep the models as Mathematical simple and efficient as possible, particular fields of application can be identified for which different types of models should be used; e.g.- free surface flows
- Pressurized flows
- Transport and dispersion of pollutants or heat
- Groundwater flows
- Morphological computations – Short waves, etc...

16.14.8 Development of Mathematical Model

1. Objective

The choice of the objectives of the model determines the level of complexity of the system and the main parameters that are involved.

2. Schematization

The schematization selected depends upon the complexity of the processes involved and the availability of models, which can be used at expenses that can be justified by the value of the answers given. Consideration must be given to the variations in time (t) and space (x, y, z). The variables t, x, y, and z are referred to as independent variables. All real processes vary with variations in one or more of the independent variables. However, by carefully examining the particular process, assumptions can be made about the dimensionality of the problem.

As far as the number of dimensions in space is concerned a distinction can be made into: ID (one dimensional model), 2D (two dimensional models), 3D (three dimensional models)

Furthermore models are distinguished into steady models and time dependent model

1 D models: *(variation in t and x) This type of models is often used for e.g. water flow, sediment transport and morphology, flows in tidal estuaries, control of irrigation systems, Simulation of dam break waves, flood propagation through channels and reservoirs.*

2D Models: (variations in t, x, and y) This type of models is often used for e.g. sectional models of structures, models to study salt intrusion and water quality, tidal flows, detailed flows in rivers (e.g. circulations), short-wave modeling.

3D Models: (variations in t, x, y and z) This type of models is often used for e.g. study of discharge from power station of hot water plume into ocean, physically-based, distributed hydrological model.

The next step in Schematisation is the choice of dependant variables. These variables are the unknown quantities which vary in time and space and whose values we wish to calculate by modeling e.g. flow (Q), depth (h), stage (H), Concentration (I), etc.

3. Equations

The variations in the dependent variables can be described by equations, which relate all the dependent and independent variables. The equations are generally derived from balances in control elements, e.g. mass, momentum, energy, heat, population balances. This leads, in general, to one or more ordinary or partial differential equations.

4. Solution Methods

There are many solution methods for differential equations. In hydraulic engineering numerical methods are most commonly used. These include: - finite difference, - finite element, boundary element methods,

5. Computer program

Once a solution method has been selected, a computer program has to be written. In this program the computer is instructed to execute all the steps or the solution method in the correct sequence and to realize input and Output of a wide variety of data. Tile data must be collected from prototype or experiment to help determine the values or model parameters and to determine the boundary conditions to be specified during the model simulations.

6. Calibration

A series of simulations with the model for different values of parameters leads to choice of parameter values, which give the best comparison between, measured and computed results.

7. Verification

After the model has been calibrated satisfactorily it should then be run once more with a completely new set of data for a final verification of the suitability of the model. This run should accurately reproduce the measured data without having to adjust any of the parameter values.

8. Simulation

Finally the model is ready for application to the objectives of the study, which was defined at the initiation of the model development.

16.15 Distinguishing Feature of Computer Aided Design and Drafting (CAD & CADD) Technician

The work involves responsibility for creating, verifying, modifying and producing civil engineering design plans, sketches, layouts, schematic drawings, record drawings, graphic representations and plans for water and sewer capital projects, utilizing computer-aided design and drafting (CADD) software technology such as AutoCAD. Projects may include all technical aspects of the civil engineering field, but will primarily consist of water distribution, sewer collection, storm water collection, pumping, management, and treatment.

The incumbent also assists with the digitization of existing records, digital collection of field records, and provides support and training to other engineering staff in the use of AutoCAD, ArcPad, Trimble GPS and similar technologies. Additionally, the incumbent performs technical engineering and survey work in the field, including construction record survey and drawing, construction layout, and measurements to verify drawings, plans, sketches, etc. Additionally, the incumbent will assist in daily records management for current records, plus assist and coordinate in the preservation and archival of historic records of water utilities, sewer utilities, stormwater utilities, Water and Sewer buildings and facilities. The incumbent also acts as a liaison with contractors, consultants, public utilities, and property owners regarding projects. This position differs from that of an Assistant Civil Engineer by virtue of the fact that a CADD Technician performs technical work rather than professional engineering tasks. The work is performed under direct supervision of a higher level Engineer in accordance with standard engineering principles, practices and techniques. Does related work as required.

Typical Work Activities

Drafts, reviews, verifies and modifies digital and hardcopy civil engineering design plans, sketches, layouts, schematic drawings, record drawings, graphic representations and plans for water and sewer capital projects, using CADD software; Conducts field measurement checks to verify drawings and plans, using levels, theodolites, tapes
and other engineering equipment; Updates AutoCAD master files with project as-built drawing submittals from project managers upon project completion;
Queries a computer data base and retrieves data for use in engineering projects and
water/wastewater installations; Trains staff in the use of computer-aided design and drafting software; Digitizes hand and field records using software such as ARCPAD;
Updates, maintains and archives water and sewer plan files;
Prepares graphics for inclusion in project reports, proposals and presentations;
Contacts a variety of public and private agencies or employees to obtain and relay information relative to construction and/or survey projects;
Creates and maintains a variety of records and reports relative to the work.

FULL PERFORMANCE KNOWLEDGES, SKILLS, ABILITIES, PERSONAL CHARACTERISTICS:

Good knowledge of computer-aided design and drafting software, including AutoCAD; good knowledge of modern engineering practices and techniques; good knowledge of surveying practices, techniques, tasks, equipment and terminology; working knowledge of the common practices, tools, equipment, terminology and safety precautions associated with construction projects and/or installations to water and wastewater

systems; working knowledge of federal, state and local rules and regulations relative to construction projects and installations to water and wastewater systems; skill in using computer-aided design and drafting software such as AutoCAD;
skill in operating optical and satellite-based surveying instruments and equipment; ability to organize and maintain accurate records and files; ability to understand and interpret complex oral instructions and/or written directions; ability to establish and maintain effective working relationships with others; ability to communicate effectively both orally and in writing; ability to successfully work with and serve a diverse local community; physical condition commensurate with the demands of the position.

16.16 Examples of CAD

16.16.1 Sewer Design based on Spreadsheet

Out of several formulae commonly used to design sewers, the manning formula is probably the most widely used. Although recent research has demonstrated some limitations of the Manning formula; for example, it takes insufficient allowance for the independence to flow caused by slimes which can build up in sewers, nevertheless it is recommended that it be used for sewer designing in Kenya.

The Manning formula is; $(1/n)\ R^{2/3}S^{1/2}$

Where; n= Manning Constant
R=Hydraulic Radius in m
S=Hydraulic Gradient in m/m

The calculations are done based on an Excel spreadsheet where the introduced data are the ground levels with the corresponding chainages and the discharges. The output of the spreadsheet calculation is the velocity of flow, frictional losses and the dynamic and static pressures along the sewer.

The spreadsheet is made out of different sheets which are linked. The sheet, which we call "sewer" contains the pipe characteristics and is linked with the sheets for calculation of the losses, pressures and flow velocities. The number of sheets for calculation depends on the number of lines as each line has its own sheet. For the clarity of working, the sheets are named according to the sewer line.

16.16.2 PC Based Simplified Sewer Design

The purpose of the program is to aid the design of simplified sewerage systems. It seeks to do this by:

(1.) automating – and thus speeding up – the necessary design calculations;
(2.) providing a tool for analysing different design permutations / configurations; and
(3.) being suitable for training / learning purposes

The program should only be used by engineers who are "computer-literate." It is not really suitable for use by others.

16.16.3 System requirements

The program will run on any of the following Microsoft Windows-based operating systems:

- Windows 95
- Windows 98
- Windows 2000
- Windows NT4

It will **not** run on computers running the Windows 3.1 operating system.

The monitor screen resolution must be 800 x 600 pixels or greater (this is a very common resolution – only one step up from the minimum possible).

The hardware requirements are not demanding – any PC capable of running one of the above versions of Windows should be able to run this program.

16.16.4 Obtaining the program

There is an Internet website from where the program can be downloaded; its address (URL) (which is case sensitive) is:

http://www.efm.leeds.ac.uk/CIVE/Sewerage

From this site, you can obtain the most up-to-date version of, and latest information on, the program.

16.16.5 The definition of a sewer network used in the program

The program requires the sewer network to be described as a series of linked sewer pipes. The sewers may only be linked in a *tree* type manner – that is, the network expands from the most downstream point branching at junctions to several upstream ends. And there must be no loops in the network. (The program has built-in automatic checks which show warnings if any network is entered that cannot be calculated.)To cater for designing individual sections of a sewer network (which may be drawn together at the end of the design process), the network may be split into sub networks, termed *sub-nets* in the program. Sub-nets may join other sub-nets at "drop" junctions – i.e. those at which the sewers are not necessarily at the same level. The typical case would be for a small branch sewer sub-network to be designed. When complete, this needs to be linked into a main street/collector sewer that may be at a much lower level.

In summary:

· A **network** consists of one or more **sub-nets** which may join at drop junctions. A **sub-net** is a tree structure of sewers whose ends join at continuous levels, i.e. without drop junctions. In general sub-nets correspond to condominial systems and these then form the input into public collector sewers.

16.16.6 The minimum information necessary to use the program

As a minimum to get your basic layout entered, you will need the length of the sewer and number of people connected to it (this may be described in terms of the number of houses connected together with a mean number of people per house, or in terms of a single total number of people who live in houses connected to the sewer). To be of practical use you will also need the ground levels at the end of each sewer length.

16.16.7 Getting started

If you really want to jump straight into a design, you can go directly, where four step-by-step examples are presented. The next sections describe in detail the various screens contained within the program. On first starting the program you are presented with the screen shown in Figure 16.1.

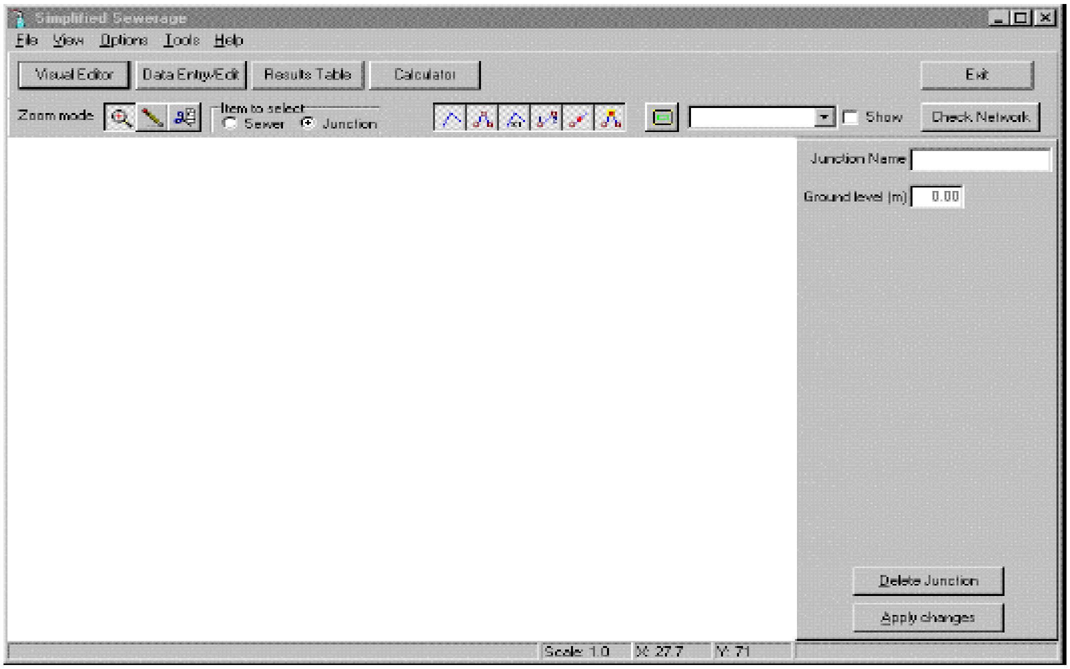


Figure 16.1 The Initial Screen

Below the main menu bar are four buttons (Figure 16.2) which allow you to switch
Between the four main screens of the program:

Figure 16.2 The main Tool Button bar

The first button takes you to the ***Visual Editor*** screen. This is the screen that is shown when you first start the program – see Figure 12.1. Here you can draw the sewer network on-screen and also edit all the network description parameters. It provides the normal means of entering all the necessary design data.

An alternative to this method of entering data is by using the ***Data Entry/Edit*** screen – see below. You may use either or both of these screens to edit your network data. The second button switches to the ***Data Entry/Edit*** screen. This is the alternative, table-based method for editing the sewer network description.

The third button takes you to the ***Results Table*** screen – a table of the detailed design results for the sewer network. Here you may also change some of the design calculations and recalculate to show these changes.

The fourth button displays the ***Calculator*** screen. On this screen you can see the details of calculations performed for each sewer in the network and adjust the parameters to examine possible design changes.

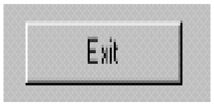


Figure 16.3 Exit button

There is a fifth button on this toolbar – the ***Exit*** button (Figure 16.3), to allow you to leave the program. You will be prompted for confirmation to prevent accidental closure of the program or closure before first saving your edited network.

16.16.8 Visual Editor Screen

This screen (Figure 16.4) allows you to draw the sewer network on-screen and also edit all the network description parameters. This screen should be used as the main set-up screen for any network that you wish to develop. From this screen all network changes can be made. An alternative method of changing/specifying the network is provided using the ***Data Entry/Edit*** screen where all changes are made by entries in tables and boxes. Changes made on one screen are automatically changed on the other.

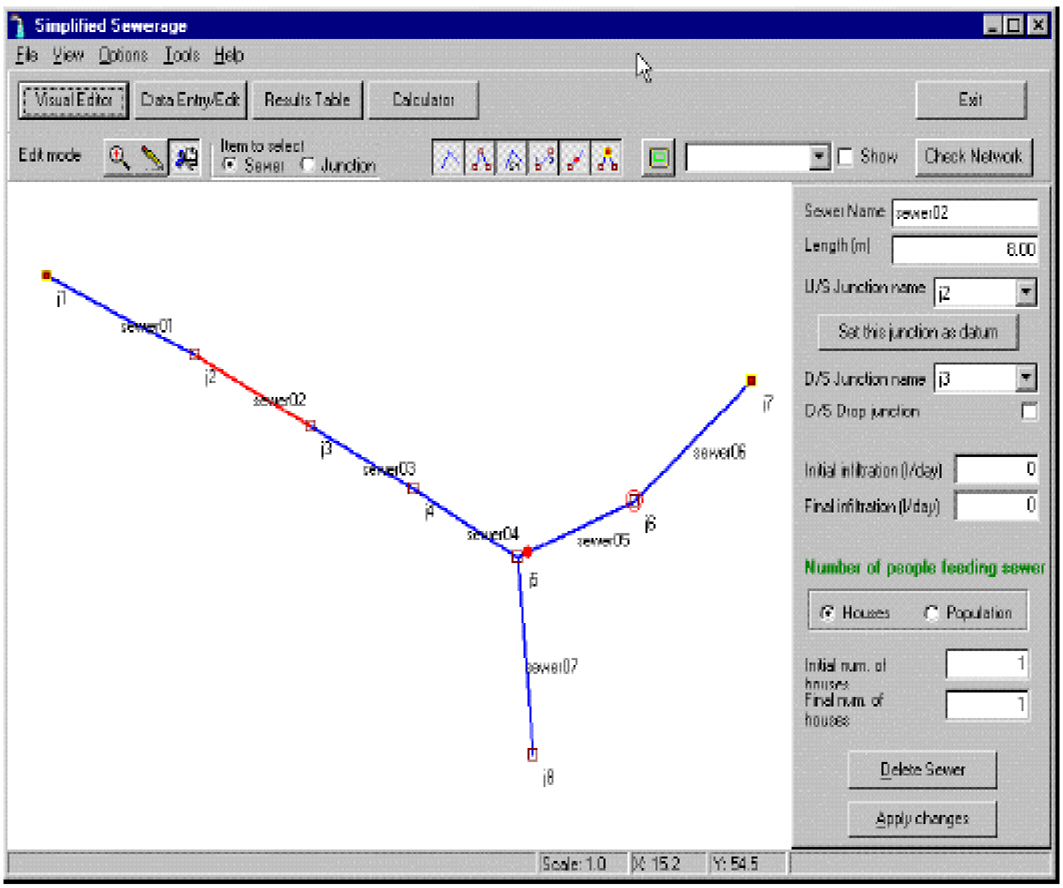
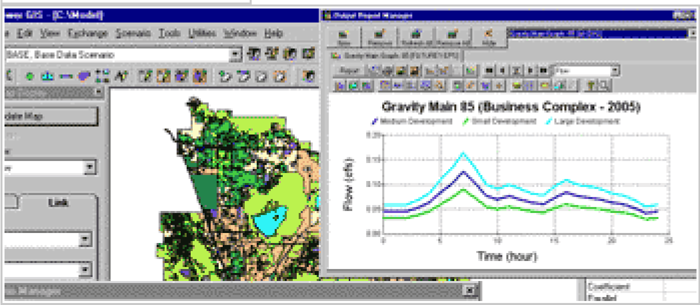


Figure 16.4 Visual Editor Screen

Note that this screen does **not** draw the network to scale. Lengths drawn on the screen do **not** necessarily represent the actual length on the ground, or the actual length entered in the data; thus changing the data values in the program will not change the view on the screen. The reason for this is that simplified sewerage is designed without requiring a detailed survey of positions of sewer ends (only levels are required); hence the co-ordinates of sewer ends are not usually known. It would, of course, make sense to draw any network on the screen to look *similar* to that being designed on the ground.

Sanitary, Storm and Combined Sewer Design and Modeling

H2OMAP Sewer
H2OMAP Sewer Suite
H2OMAP Sewer Pro
H2OMAP Sewer Suite Pro



Batch Run and Compare Multiple Scenarios Instantly
(Click to enlarge the picture)

H_2OMAP Sewer Usage:

- Master Planning
- CMOM Program Planning
- Facility Sizing
- Wet-Well Design
- Facility Management
- Outlet Configuration
- Operations Study
- Pump Cycling
- Real-time Simulation
- Infrastructure Rehabilitation/Replacement
- Peak Flow Effects
- Flow Capacity Evaluation
- Overflow Management
- Infiltration/Inflow Modeling
- System Deficiency Identification
- System Improvements
- BOD Modeling
- Odor Control
- Source Tracing
- Pollutant Contribution Analysis
- Discharge Permit Compliance
- Corrosion Control
- Treatment Process Efficiency Evaluation
- Sewage Age Improvement
- Sediment Modeling and Impact Analysis
- Sensitivity Analysis
- Alternative Analyses
- Dynamic Loading Comparison
- Flow Monitoring Design
- Model Calibration
- Operator Training
- Capital Improvement Program Development
- Capital Budgeting
- Impact Fees Determination
- System Expansion
- New System Design
- System Layout Mapping
- Dynamic Profiling
- Project Management

$H_2$0MAP Sewer is a powerful, stand-alone GIS-based computer program for use in the planning, design, analysis, and expansion of sanitary, storm and combined sewer collection systems. The program can be effectively used to model both dry-weather and wet-weather flows and determine the most cost-effective and reliable method of wastewater collection. Built using advanced Object-oriented Geospatial Component model, $H_2$0MAP Sewer seamlessly views, manipulates, and exchanges GIS data sets with incredible ease and simplicity.

Utilizing a rigorous and versatile network modeling solution, $H_2$0MAP Sewer supports geo-coding and multiple mapping layers which can be imported from one of many data sources including Computer-Aided Design (CAD) drawings (e.g., dwg, dgn, dxf), CAD world files, standard GIS formats (Shape files, Generate files, MID/MIF files, and Arc Info coverage), Vector Product Format (VPF) files, Spatial Database Engine (ArcSDE) Layers, attribute tables, grid data, image files, ODBC, and Comma Separated Delimited Text (CSV) files. The program also directly supports the new geo-database standard of ArcGIS through an ArcSDE connection.

$H_2$0MAP Sewer surpasses its competitors with fast and comprehensive hydraulic and dynamic water quality computational capabilities such as steady-state analysis using various peaking factors and automated system design, along with sophisticated dynamic solution of the St. Venant's equations considering flow attenuation, time of concentration, source tracing as well as pollutant, BOD, sediment transport/deposition, hydrogen sulfide and corrosion modeling. It utilizes a full-featured, state-of-the-art hydraulic, water quality and sediment transport computational engine. Through the use of extensive scenario management functionality, the program is also capable of analyzing existing or proposed sewage collection systems.

$H_2$0MAP Sewer Pro adds very sophisticated rainfall-runoff modeling for representative peak flow computation and dynamic routing of combined sanitary sewerage and storm water throughout the collection system, equipping wastewater utilities with a powerful tool for predicting unwanted overflow events and backups and developing sound remedial alternatives for eliminating CSO, SSO and storm water problems, meeting environmental regulations, and improving system management, operation and community relations.

The program also offers a complete, database-driven open-architecture environment, which makes it easy to manage and distribute geospatial data as well as exchange important modeling information with other applications and enterprise GIS systems. Because the program creates and stores all graphic data in a native shape file format, $H_2$0MAP Sewer instantly becomes an integral part of your data warehouse, compatible with any other GIS application. You can now visualize your sewer collection system at a glance and develop informed, cost-effective and reliable solutions concerning it.

There are three main parts to this screen: the toolbar (Figure 16.5), which allows setting of the editing and display options for the network.

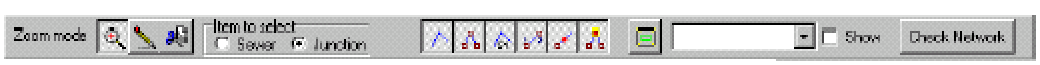


Figure 16.5 Visual Edit toolbar.

The details panel to the right provide the means for giving, and allowing changes to, details of either the selected sewer or junction the initially blank white panel (Figure 16.1) is where the network will be displayed and the status bar at the bottom displays such information as scale factor, and the mouse co-ordinates.

8.1 SANSYS FOR SEWERAGE- Sanitary Sewerage System Design, Analysis and Management Software

SANSYS will help you analyze, design and manage your sanitary sewerage system. It is a comprehensive graphic information system (GIS) for your sewer infrastructure.

As a municipal planning tool, SANSYS will simulate the effects of zoning changes. As a design tool for land development, you may quickly determine suitable pipe sizes and velocities. As a maintenance management tool, your pipe and manhole inspections may be recorded and color-coded to help you to prioritize the work of your infrastructure upgrading projects. By using zoning classifications, population densities, industrial densities, contributing areas and infiltration allowances, you may size pipes for a new development or find infiltration problems of an existing system.

Key Plans & Profiles

On key plans and profiles, powerful keyed manholes have features which allow you to view or label data, or to locate manholes and pipes on a spreadsheet or an AutoCAD drawing. They let you navigate in a simple way with little effort. Spreadsheets

SANSYS provides you with an easy way of selecting, entering, changing, and reviewing data. Spreadsheet style data management offers descriptive prompts and help screens as you move from cell to cell.

AutoCAD

SANSYS stands alone, but if you have AutoCAD the two will work together. Specify your own base plan for capturing manhole and pipe locations. Our toolbar makes it easy to locate or edit sewerage system components within SANSYS and AutoCAD. SANSYS produces detailed drawings of your model to your specifications using the same color-coding as the key plans and profiles.

SANSYS Features

- integrated spreadsheet style data editing and management
- key plans and profiles with keyed manholes
- AutoCAD R12, R13, R14 input and output
- import or export ASCII files
- append models
- formatted report
- manhole and pipe sorting and search facilities
- pipe length calculations from coordinates
- invert calculations from ground elevations
- pipe grade calculations from lengths and inverts
- comprehensive cross-reference checks
- model and design criteria descriptions

Sewerage Model

This gravity network model will calculate flows using many choices and combinations of peaking factors.

- peaking factor formulae:
- Babbitt with variations
- Harmon (Ten State) with variations
- constant and exponential
- specified allowable ranges
- variations for commercial, non-residential flows
- pipes, manholes entered in any order
- manhole alphanumeric identifiers, coordinates, invert elevations, ground elevations, head losses
- pipe connections, lengths, grades, and diameters
- Manning's n roughness coefficients
- pipe location descriptions
- fixed length option for curved or irregular alignments
- models may be linked together
- zoned contributing areas
- overflow diversions
- forcemains
- metric, Imperial or U.S. units of measurement

Loads

- residential, per capita flows
- population, industrial and commercial densities
- zoning definitions with choice of density units
- contributing populations
- infiltration by area or pipe size and length
- pumped and measured flows

Results

- average and peak flows
- flows from infiltration
- existing and required pipe capacities
- required pipe diameters
- full pipe and peak flow velocities
- part flow as remaining capacity, peak flow/capacity or depth/diameter
- hydraulic grades for surcharging pipes
- overflows diverted
- summary totals for areas and populations
- list of under capacity pipes
- list of pipes with inadequate scour velocity
- totals of pipe lengths by diameter
- pipe Inventory Management

As a maintenance management tool, your manhole and pipe inspection data may be recorded and color-coded to help you to prioritize the work of your infrastructure upgrading projects. If you have another GIS, comprehensive export utilities help you keep your system updated.

- installation and inspection dates
- video reference
- cleaning history
- inspection comments
- condition marks
- material type
- coordinate and invert elevation sources

Key Plans & Profiles

- labels display your choice of information when your cursor is placed over a keyed manhole
- point-and-click locating of keyed manholes between spreadsheets, key plans, key profiles and AutoCAD
- double-click a keyed manhole to edit it and its pipe
- image zoom, copy to clipboard, save, print
- insert labels or legend on image
- size pipe lines to correspond to diameters

Pipe color-coding (see below)

AutoCAD

- link to AutoCAD for pipe, manhole and area digitizing and input on base plans with dynamic data exchange
- user specified manhole sizes, text sizes, layer colors, text font and text aspect ratio

- plan showing flow directions, undersized pipes, forcemains and overflows with labels for pipes, manholes, existing and proposed diameters, capacities, velocities, peak flows, part flows
- profiles showing pipes, hydraulic grade lines and manholes with labels for manholes, lengths, existing and proposed diameters, grades and inverts
- title, user descriptions and legend
- pipe color-coding (see below)
- Color-Coded Plans
- undersized pipes
- diameters
- capacities
- part flows:
- peak flow/capacity
- remaining capacity
- depth/diameter
- materials
- position sources
- conditions
- Size
- 3000 pipes and manholes per model
- 10 character manhole/pipe identifiers
- 10 infiltration rate codes
- 20 overflows and diversions
- 30 zoning categories

9.0 ENVIRONMENTAL IMPACT ASSESSMENT

9.1 Introduction

- EIA is the study of the effects of a proposed action on the environment. The “environment” is taken to include all aspects of the natural and human environment.

Ethics

- EIA identifies both negative and positive impacts on development activities and how they affect the people, their property and the environment. EIA in addition identifies measures to mitigate the negative Impacts, while maximizing on the positive ones. EIA is essentially a preventive process, which seeks to minimize adverse impact on the environment.

- The EIA Studies covers all physical, biological, social, economic and other impacts arising from an action on the environment. The studies vary from action and include studies on weather, flora and fauna, soil erosion, human health, urban irrigation and employment.

- EIA seeks solutions to all feasible alternatives that represents an optimum mix or balance of environmental and social-economic costs and benefits.

- The technical work involved in EIA is the estimation of the changes in environmental quality which may be expected as a result of the proposed action i.e. EIA attempts to weigh environmental effects on a common basis with social-economic costs and benefits in the overall project evaluation.

- The ultimate objective of EIA is to aid judgmental decision making by giving the decision maker a clear picture of the alternatives which are considered, the environmental changes which are predicted, the trade-offs of advantages and disadvantages for each alternative.

9.2 Objectives

The objective of Environmental Impact Assessment (EIA) Study is to ensure that the project conforms to regulations of the “National Environmental Management Authority (NEMA)” of Kenya and in general of the prevailing Kenya’s environmental rules and regulations.

9.3 Legal framework

- Environmental Management Coordination Act (EMCA) 1999 give authoritative legislature framework within which the environmental protection and management shall take place.
- The Environmental Impact Assessment and Audit Regulations, 2003 outlines how an EIA Study has to be carried, how it is written and what needs to be presented to NEMA
- The Environmental Management and Coordination (Water Quality Regulation), 2006 gives the required Quality which any water source has to meet and the quality of sewage discharges to be met
- The Environmental Impact Assessment (EIA) is guided by the Environmental and coordination Act, 1999 and the above regulation and standard through National Environmental Management Authority (NEMA). The Act became effective on 14th January 2000 after receiving Presidential Assert on 6th January 2000.

- Only Sewerage is included in EMCA schedule 2 as required to undertake full EIA Study and is therefore, expected to comply with the EIA requirement under the same Act ,itss regulations and Standard.

- The preparation of EIA Study and subsequent approval procedures are set out in the Environmental (Impact Assessment and Audit), Regulation 2003.

9.4 EIA Regulations

In accordance with NEMA regulations, a proponent shall submit to the Authority an environmental impact assessment study report incorporating but not limited to the following information.

(i) The proposed location of the project
(ii) The objective of the project
(iii) The technology, procedures and process to be used, in the implementation of the project
(iv) The materials to be used in the construction and implementation of the project.
(v) The products and by-products and waste generated by the project
(vi) The environmental effects of the project including the socio-cultural impacts, effects and direct, indirect, cumulative, irreversible, short-term and long-term effects anticipated.
(vii) A concise description of national environmental legislative and regulatory framework, baseline information and any other information related to the project.
(viii) A description of the potentially affected environment.
(ix) Alternative technologies and processes available and reasons for preferring the chosen technology and processes.
(x) An analysis alternative including projects site, design and technologies and reasons for preferring the proposed site, design and technologies.
(xi) An environmental management plan proposing the measures for eliminating, minimizing or mitigating adverse impacts on the environment, including the cost, time frame and responsibility to implement the measures.
(xii) The provision of an action plan for the prevention and management of foreseeable accidents and hazardous activities in the cause of carrying out activities or major industrial and other development projects.,
(xiii) The measures to prevent health hazards and to ensure security in the working environment for employers and for the management of emergencies.
(xiv) An Identification of gaps in the knowledge and uncertainties which were encountered in compiling the information.
(xv) An economic and social analysis of the project
(xvi) An indication of whether the environment of any other state is likely to be affected and the available alternative and mitigating measures.
(xvii) Any other matters as the authority may require.

9.5 Reasons for EIA study

Reasons for carrying out an EIA will include:

- Assurance of adequate procedures for managing environmental risks and compliance with procedures.
- Improved statutory compliance
- Identification of environmental risks and problem areas, early warning and prevention of potential adverse environmental effects.
- Improved planning through the identification of future and potential capital, operating and maintenance costs, associated with environmental activities.
- Improved preparation for emergency and crisis situation management.
- Improved corporate image and positive public relations
- Enhancement of environmental awareness and responsibility throughout the corporate hierarchy.
- Improved relations with regulatory authorities
- Facilitation of obtaining insurance coverage for environmental impairment liability.

9.6 EIA Requirements

9.6.1 EIA Expert

Criteria for listing

Below is the criterion for listing EIA Experts.

	Category expert	Assignment	Requirements
1	Lead Experts	Entitled to Conduct EAI studies independently under their name as professional	1. **Minimum 2nd** degree in an field **and:** Trained in EIA and 2 years of experience in conducting EIA or 2. 5 years research work in any discipline **and** 2 years experience in conduction EIA OR Must have undergone EIA course for a minimum period of 21 3. n days and must have worked for at least 15 years
3	Associate Experts	Entitled to undertake EIA studies only under the guidance of a Lead Expert.	3. **Phd**. In environmental related studies **and** 2 years research experience or 4. **Phd** in any discipline, an environmental course **and** 2 years research experience. or
			5. **Engineers** with an environmental course **and** 2 years of working experience.
			6. **2nd degree** in any discipline **and** 2 years of working experience or **First degree** in Environmental Sciences, environmental Law, Natural Resources Management, Social Sciences **and** 3 years of field experience or **Diploma** in Environmental Management Studies **and** 4 years of field experience.

However the following should be noted: -

- The criteria for listing of EIA Experts can at a later stage be revised to fit the National capacity requirement.
- The rational of selecting the Lead and Associate Expert is mainly based on the experience in conducting EIA studies. One finishes a minimum of 2 years as Associate Expert. Preference is given on number of EIA studies undertaken rather than the years.

9.6.2 Procedures for registering with NEMA

An applicant should prepare the following:-

- An application in the subscribed form to the Director General NEMA
- A detailed CV mentioning the academic performance signed and dated highlighting relevant experience in the field of EIA.
- Three letters of recommendation (references) from different recognized experts of Institutions acting in the field of environmental management and/or EIA.
- A detailed list of studies, publications and consultancies undertaken indicating whether acting in a team, his role and contribution.
- Any other relevant materials which may help appreciate the qualification of the applicant.

9.6.3 Fees

The charge is KShs6,000/= to 20,000/= per year as per attached schedule:-

Fees

		Application fee (KShs.)		Annual fee (KShs.)	
		citizen	Non citizen	citizen	Non citizen
1	Application by individual expert:				
	(a) Lead Expert (b) Associate Expert	3,000 2,000	9,000 6,000	5,000 3,000	15,000 9,000
2	Application by a firm of Experts:				
	(a) The firm's fee	5,000	15,000	20,000	60,000
3.	Inspection of records/register ………. 200 per record/register				
4.	Environmental Impact Assessment Licence ….. 0.1% of the total cost of the project				
5.	Surrender, transfer or variation of Environmental Impact Assessment licence …..5,000				

Source: - Kenya Gazette Legal Notice No.101 of June 2003.

9.6.4 Licences for EIA

9.6.5 EIA licence fees

To obtain EIA licence, KShs. 5,000 to 500,000 is paid as detailed in schedule below.

9.6.5.1 Inspection Records Fee

Between KShs.200 to 500 is required to inspect records. However if photocopy is to be obtained, and additional KShs.4 per page is paid attached is the schedule.

		Inspection fees (KSh.)	Extraction fee (KSh.)
1.	Environmental Impact Assessment Report	200	4 per page
2.	Environment Audit Report	200	4 per page
3.	Environmental Impact Assessment Study Report	500	4 per page
4.	Register of Experts	500	Not for extraction
5.	Access to Data Banks	500	Not for extraction
6.	Register of projects	500	Not for extraction
7.	Other/public documents	500	4 per page.

9.7 TERMS OF REFERENCE (TOR)

9.7.1 TOR of EIA Study

Below is as Sample TOR:-

Task 1: - Description of the proposed Project

Provide a full description of the project: location; general layout; unit process description and diagram; size in terms of population and population equivalents, present and projected; number and types of connected industries; anticipated influent and effluent characteristics; preconstruction and construction activities; schedule, staffing and support facilities and services; operation and maintenance activities; required off-site investments; and life span.

Task 2:- Description of the Environment

Assemble, evaluate and present baseline data on the environment characteristics of the study area. Include information on any changes anticipated before the project commences.

(a) Physical Environment: geology (General description for overall study area and details for land application sites); monthly average temperatures, rainfall and runoff characteristics; description of the receiving waters (identity of streams, lakes, or marine waters; annual average discharge or current data by month, chemical quality; existing discharges or withdrawals)

(b) Biological environment: terrestrial communities in areas affected by construction, facility siting, land application or disposal; aquatic, estuarine or marine communities in affected waters; rare or endangered species; sensitive habitats, including parks or preserves, significant natural sites, species of commercial importance in land application sites and receiving waters.

(c) Sociocultural environment: present and projected population; present land use; planned development activities; community structure; present and projected employment by industrial category; distribution of income, goods and services; recreation; public health; cultural properties; indigenous peoples; customs, aspirations and attitudes.

Task 3:-Legislative and Regulatory Considerations

Describe the pertinent regulations and standards governing environment quality, pollutant discharges to surface waters and land, industrial discharges to public sewers, water reclamation and reuse, agricultural and landscape use of sludge, health and safety, protection of sensitive areas, protection of endangered species, siting, land use control, etc; at international, national, regional and local levels.

Task 4:- Determination of the Potential Impacts of the proposed Project

In this Analysis, distinguish between significant positive and negative impacts, direct and indirect impacts, and immediate and long term impacts. Identify Impacts which are unavoidable or irreversible.
Wherever possible, describe impacts quantitatively, in terms of environmental costs and benefits. Assign economic values when feasible. Characterize the extent and quality of available data, explaining significant information deficiencies and any uncertainties associated with predictions of impact. If possible, give the TOR for Studies to obtain the missing information.

Special attention would be given to:

- The extent to which receiving water quality standards and/ or beneficial use objectives will be achieved with the proposed type and level of treatment.
- The length of stream or expanse of lake or marine waters which will be positively or negatively affected by the discharge, and the magnitude of the changes in water quality parameters.

- Projected quantitative changes in beneficial uses, such as fisheries (species composition, productivity), recreation and tourism (visitor-days, overnights, expenditures), and waters available for portable supply, irrigation, and industrial use.
- Sanitation and Public Health benefits anticipated.

Task 5:- Analysis of Alternatives to the Proposed Project

Describe alternatives that were examined in the course of developing the proposed project and identify other alternatives which would achieve the same objectives. The concept of alternatives extends to siting and design, technology selection, construction techniques and phasing, and operating and maintenance procedures. Compare alternatives in terms of potential environmental impacts, land and energy requirements, capital and operating costs, reliability, suitability under local conditions, and institutional, training, and monitoring requirements. When describing the impacts, indicate which are irreversible or unavoidable and which can be mitigated. To the extent possible, quantify the costs and benefits of each alternative of not constructing the project, in order to demonstrate environmental conditions without it.

Task 6:- Development of Management Plan to Mitigate Negative Impacts

Recommend Feasible and Cost Effective measures to prevent or reduce significant negative impacts to acceptable levels. Estimate the impacts and costs of those measures, and of the institutional and training requirements to implement them. Consider compensation to affected parties for impacts which cannot be mitigated. Prepare a management plan including proposed work programs, budget estimates, schedules, staffing and training requirements, and other necessary support services to implement the mitigating measures.

9.7.2 Environmental Audit

The Audit shall be carried out through questionnaires, site visits and Test Analysis and in the manner shown hereunder:-

(a) Initial Auditing

The Auditor in conduction an Initial Audit shall:-

- make a precise description of the project
- present the objective, the scope and the criteria of Audit
- collect and review all relevant environmental law and regulator frameworks on health and safety, sustainable use of natural resources and or acceptable national and international standards
- Verity the level of compliance by owner with conditions of regulations 31(iii)
- Evaluate knowledge and awareness of and responsibility for application of relevant legislation on the laws:
- Review existing project documentation related to all infrastructural facilities and designs.
- Examine monitoring programmes parameters, procedures in place for control and corrective actions.
- Evaluate the relationship with the Authority or other relevant bodies
- Inspect all buildings, premises and yards in which manufacturing, testing, transportation within and without the project area s well as storage and disposal of goods is carried out and give a record of all significant environmental risks associated with such activities.

Example and seek views on health and safety issues form both the project employees, the local and other potentially affected communities and

Prepare a proritised list of health and environmental concerns of past and on going activities and provide recommendations on estimated costs, corrective and rehabilitation measures and test implementation

9.8 Methodogology

9.8.1 Definitions Related TO EIA

9.8.1.1 EIA

EIA refers to a critical examination of the effects of a project on the environment before its implementation.

9.8.1.2 Screening

A process used to determine whether a project requires an Environmental Assessment and what type and level of Assessment would be necessary.

9.8.1.3 Scoping

A procedure for attempting to ensure that an Environmental Assessment focuses on the key Environmental issues associated with a project on omitting irrelevant material.

9.8.1.4 Environmental Management Plan (EMP)

An Action Plan or system which addresses the how, when, who, where and what of integrating environmental mitigation and monitoring measures throughout an existing or proposed operation or activity. It encompasses all the elements that are sometimes addressed separately in mitigation, monitoring and Action Plans.

9.8.1.5 Environmental Audit

A management too comprising a systematic document, periodic and objective evaluation of how well a project, organization or equipment is performing with the aim of helping to safeguard the environment. The Audit should facilitate management control of environmental practices and assess compliance with policy objectives and regulatory requirements.

9.8.1.6 Strategic Environmental Assessment (SEA)

A similar technique to EIA but normally applied to policies, plans, programmes and groups of projects. SEA provides the potential opportunity to avoid the preparation and implementation of inappropriate plans, programmes and projects and assists in the identification and evaluation of projects.

Alternatives and Identification of cumulative effects, SEA comprises two main types:-
Sectorial SEA (Applied when many new projects fall within one sector) and
Regional SEA (Applied when broad economic development is planned with one region)

9.8.1.7 Project

A project is defined as a specific set of Human activities in a particular location and time frame and intended to achieve any objective(s)

9.8.1.8 Environment

The term “Environment is used in its broadest possible sense to embrace not only physical and biological systems, but also socio economic systems and their inter-relationships.

9.8.2 Levels of Environmental Impact Assessment

9.8.2.1 Small projects

Small scale projects whose potential Adverse Environmental Impacts can easily be identified and for which mitigation measures can readily be prescribed and can be included in the Design and/or Implementation of the project. This type of project will be normally approved on the basis of mitigation measures without the need of a detailed EIA study requiring field investigations.

9.8.2.2 Projects suspected

Projects for which there is some level of uncertainty on the nature and level of Impacts. Decisions will be made during the scooping phase whether the project requires a full study or a partial study. The review at this stage also examines various alternatives, so that the decision maker can select options which do not have significant environmental impacts.

9.8.2.3 Projects with significant Impacts

Projects which clearly have significant adverse impacts. The study will determine the nature and the extent of these impacts and examine the alternatives and the mitigation measures. Conducting such EIA requires greater public participation.

9.8.2.4 Strategic Environmental Assessment (SEA)

SEA can be undertaken at a range of levels, from policies to programmes and plans.

9.8.2.5 Auditing and monitoring

An environmental Audit differs for EIA which aims to predict Environmental Impacts in that it is multidisciplinary process of objectively reviewing the environmental performance of an operating project or enterprise including its processes, material storage, operating procedures and Environmental Management to identify potential environmental impacts and liabilities.

9.8.3 Approach Principles

9.8.3.1 General

Focus on the main issues

- Avoid covering too many topics in too much detail
- Screen to limit to only the most likely and most serious of the environmental impacts.
- Work on mitigation measures that are workable and acceptable

  - Involve the appropriate persons and groups
- Resourceful actors for EIA Study
- Decision makers

  - Link information to decision about the project
  - Organize EIA early enough to provide information to improve basic designs, and progress through the several stages of the project planning and implementation.

  - Present clear options for mitigation measures and sound environmental management.

The approach to EIA study should include:

Preliminaries activities
Impact identification
Baseline study
Impact evaluation
Mitigation measures
Assessment
Documentation
Decision making
Post auditing.

9.8.3.2 Preliminary activities

Identification of the activities and the actors in the project that will include:-
Decision makers
Project team

Description of actions

Review of existing legislation

9.8.3.3 Impact Identification

- Check list of impacts
- Selection of important impacts based on
  - Magnitude
  - Extent
  - Significance and
  -

9.8.4 Special sensitivity

9.8.4.1 Baseline Study

Data prior to proposed action – bench mark

9.8.4.2 Impact Evaluation

Quantification or qualification of impacts

9.8.4.3 Mitigation measures

Identification of actions to eliminate or minimize adverse environmental impacts.

9.8.4.4 Assessment of alternatives

Combination of environmental losses and gains with the socio-economic costs and benefits for each alternative.

9.8.4.5 Documentation

- Reference documents that will confirm detailed record of the work done in the EIA.
- Working documents that convey information for immediate action.

9.8.4.6 Decision making

Decision making to be based on:-

- Socio-economic and environmental considerations
- Political considerations
- Choice not between “bad” and “good” but between “good” and “better”. However, the decision maker must:

9.8.4.7 Accept one of the project alternatives

Request further study or
Reject ant proposed action altogether.

9.8.4.8 Post Audits

Conducted to determine the closeness or how accurate are the predations of the EIA.

- Provide information in a form useful to decision makers
    - Terms and formats of conclusions of EIA Must be understandable by decision makers.

9.9 REPORT WRITING

9.9.1 EIA Scoping

(a) Executive summary
(b) The proposed project
(c) Approach and methodology
(d) Potential alternatives
(e) Study limits
(f) Environmental policy, legislative and planning framework
(g) Existing environmental conditions
(h) Scoping the environmental impacts
(i) Scooping the assessment
(j) Scooping the environmental management aspects
(k) Recommendations
(l) Technical appendices
(m) Administrative appendices – this will be consultants itinerary, etc

9.9.2 Full EIA Study

(1) Executive summary

(2) Presentation of the project
- project selection
- project location
- project description and associated activities
- alternatives.

(3) Approval and methodology
- General approach
- Assumptions, uncertainties and constraints.

(4) Environmental policy, legislative and Planning Framework

- Environmental policy and legislative framework
- Consultation and public participation
- Institutional capacity
- Relevant ongoing projects

(5) Existing environmental conditions

(6) Assessment of environmental impacts and mitigation measures

- Prediction of potential environmental impacts and benefits
- Enhancements

(7) Environmental Management Implementation

- Environmental monitoring
- Environmental management capacity
- Environmental management

(8) Conclusion and recommendations

(9) Technical appendices

(10) Administrative appendices

9.10 EIA FORMS

Attached are forms in relation with EIA.

FORM 1 (r.6)

THE ENVIRONMENT MANAGEMENT AND COORDINATION ACT
SUBMISSION OF PROJECT REPORT

PART A: DETAILS OF PROPONENT

A1 Name of proponent (Person or Firm ……………………………..

A2 PIN No. ………………………………………………………..

A3 Address ………………………………………………………..

A4 Name of contact person ………………………………………….

A5 Telephone No. ……………….. A6 Fax No. …………………..

A7 E-mail ……………………………………………………………

PART B: DETAILS OF THE PROJECT REPORT

B1 Title of the proposed project…………………………………….

…………………………………………………………………

B2 Objectives and scope of the project ……………………………..

…………………………………………………………………

…………………………………………………………………

B3 Description of the activities ……………………………………..

…………………………………………………………………

B4 Location of the proposed project ………………………………..

PART C: DECLARATION BY THE PROPONENT

I hereby certify that the particulars given above are correct and true to the best of my knowledge………………………………….

Name …………………………….. Position ……………………………

Signature ……………………………

PART D: DETAILS OF ENVIRONMENTAL IMPACT ASSESSMENT EXPERT

Name (Individual/firm) …………………………………………………………

Certificate of registration No. ……………………………………………………

Address ………………………………………………………………………..

Tel: ………………………. Fax: ……………………… E-mail: ……………………

PART E: FOR OFFICIAL USE

Approved/not approved ……………………………………………………………….

Comments: ………………………………………………………………………….

……………………………………………………………………………………….

……………………………………………………………………………………….

……………………………………………………………………………………….

Officer: ………………. Sign: ………………….. Date:………………………

NB: 1. If the Project Report does not contain sufficient information required under the Environmental (Impact Assessment and Audit) Regulations the applicant may be requested to give further information concerning the project or be notified of any defects in the application and maybe required to provide the additional information.

2. Any person who fraudulently makes a false statement in a project report or alters the project report commits an offence.

Important Notes: Please submit the following:

(a) Three copies of this form

(b) 10 copies of the project report

(c) The prescribed fees to:

Director-General,
The National Environment Management Authority,
Kapiti Road, South C,
P.O. Box 47146,
NAIROBI.

Tel. 254-02-609013/27/79 or 608900 Fax. 254-02-608997

FORM 2 (r.19)

Application Reference No.

FOR OFFICIAL USE

THE ENVIRONMENT MANAGEMENT CO-ORDINATION ACT

SUBMISSION OF ENVIRONMENTAL IMPACT ASSESSMENT STUDY REPORT

PART A: DETAILS OF PROPONENT

A1. Name of proponent (Person or Firm) ………………………………………..

A2 PIN No. ……………………………………………………………………

A3 Address. ……………………………………………………………………

A4 Name of contact person ……………………………………………………

A5 Telephone No. ………………….. A6 Fax No. ………………………………

A7 E-mail ………………………………………………………………………

PART B: DETAILS OF THE ENVIRONMENTAL IMPACT ASSESSMENT STUDY REPORT.

B1 Title of the propose project ……………………………………………………

…………………………………………………………………………………….

B2 Objectives and scope of the project ……………………………………………

…………………………………………………………………………………….

B3 Description of the activities ……………………………………………………

…………………………………………………………………………………….

B4 Location of the proposed project ………………………………………………

…………………………………………………………………………………….

PART C: DECLARATION BY THE PROPONENT

I hereby certify that the particulars given above are correct and true to the best of my knowledge.

Name ………………………………. Designation ………………………………

PART D: DETAILS OF ENVIRONMENTAL IMPACT ASSESSMENT EXPERT

Name (Individual/firm) …………………………………………………………..

Certificate of registration No. ………………………………………………………

Address ……………………………………………………………………………….

Tel: ………………. Fax: ……………………… E-mail ………………………

PART E: OFFICIAL USE

Approved/not approved ………………………………………………………………

Comments: ……………………………………………………………………………

…………………………………………………………………………………………

…………………………………………………………………………………………

…………………………………………………………………………………………

Officer……………………… Sign …………………… Date ……………

Important Notes: Please submit the following:

(a) Three copies of this form

(b) 10 copies of the project study report

(c) the prescribed fees to:

Director-General,

The National Environment Management Authority,

Kapiti Road, South C,

P.O. Box 47146,

NAIROBI.

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

FORM 3 (r.24)

Application Reference No: ……………………………..

Registration No.: ……………………………………….

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

ENVIRONMENTAL IMPACT ASSESSMENT LICENCE

This is to certify that the Project Report/Environmental Impact Assessment Study Report ……………………………………… received from …………………………………. (name of individual/firm) ………………………………..(address) submitted to the National Environment Management Authority in accordance with the Environmental Impact Assessment & Audit Regulations regarding ………………………………….

……………………………………… (title of project whose objective is to carry on

………………………………………………………………………………………..

………………………………………………………………………………………..

………………………………………………………………………………………..

……………………………………………………………………………………….

(briefly describe purpose) located at ………………………………….. (locality and District) has been reviewed and a licence is hereby issued for implementation of the project, subject to attached conditions.

Date this …………………… day ………………………… of 20……………………….

Signature ……………………………………………………………………………….

(seal)

Director-General

The National Environmental Management Authority.

Conditions of Licence:

1. This licence is valid for a period of ………………….. (time within which the project should commence) from the date hereof.

2. The Director General shall be notified of any transfer/variation/surrender of this licence.

FORM 4

Application Reference No. ……………………..

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

APPLICATION FOR REGISTRATION AS AN ENVIRONMENTAL IMPACT ASSESMENT/AUDIT EXPERT

PART A: DETAILS OF APPLICANT

A1 Name of proponent (Individual or Firm) ………………………………….

A2. Nationality …………………………………………………………..

A3 PIN No. ……………………………………………………………..

A4 Firm (Local/Foreign) ……………………………………………………..

A5 Address ……………………………………………………………..

……………………………………………………………………………..

A6 Telephone No. ……………………….. A7. Fax No. ……………………

A8 E-mail ……………………………………………………………….

A9 Applicants academic/professional qualifications: ……………………….

……………………………………………………………………………

A10 List of professional and their academic/professional qualifications and their nationalities (where applicable)

…………………………………………………………………………….

….………………………………………………………………………….

……………………………………………………………………………

A11 Experience in Environmental Impact Assessment related activities: ……

……………………………………………………………………………

……………………………………………………………………………

……………………………………………………………………………

A12 Application for registration of Lead Expert or Associate.

PART B: DECLARATION BY APPLICANT

B1 I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief.

…………………..…….. ………………………….. …………………………

Signature of applicant *Full name in Block letters* *Position*

On behalf of ………………….. ……………………………
Firm name and seal Date

PART C: FOR OFFICIAL USE

Approved/Not approved ……………………………………………………….

…………………………………………………………………………………….

Comments ……………………………………………………………………..

……………………………………………………………………………………

……………………………………………………………………………………

Official ………………… Sign …………….. Date …………………..

Important Notes: Please submit the following

(a) Application Form in duplicate

(b) Curriculum vitae of all applicants: and

(c) the prescribed fee, to:

Director-General,

The National Environment Management Authority (NEMA),

Kipiti Road, South C,

P.O. Box 47146,

NAIROBI, KENYA

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

E-mail …………………………….

FORM 5 (r.14(4))

Application Reference No: …………………..

Registration No: …………………………….

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF REGISTRATION AS AN ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT EXPERT

This is to certify M/s ………………………………………………………… of ……………………………………………………………………………………………… ……………………….. (Address) has been registered as an Environmental Impact Assessment Expert in accordance with the provisions of the Environment Management and Coordination Act and is authorized to practice in the capacity of a Lead Expert/Associate Expert/Firm of Experts (Type) …………………………………………

………………………………………………………………………………………..

Date this ………………… day ………………….. of 20……………..

Signature ……………………………………………..

(seal)

Director – General.

The National Environmental Management Authority

FORM 6 (r.15(1))

Application Reference No. …………………….

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

APPLICATION FOR LICENCE TO PRACTICE AS AN ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT EXPERT

PART A: DETAILS OF APPLICANT

A1 Name of proponent (Individual of Firm)…………………………………….

A2 Nationality ……………………………………………………………….

A3 PIN No. ……………………………………………………………….

A4 Firm (Local/Foreign) ……………………………………………………….

A5 Business registration No. (where applicable) ……………………………..

Date ………………………………..

A6 Address ………………………………………………………………

A7 Telephone No. ………………………… A8 Fax No. …………………

A9 E-mail……………………………………………………………………….

A10 Applicants academic/professional qualifications: …………………………

…………………………………………………………………………..

A11 List of professionals and their academic/professional qualifications and their nationalities (where applicable) ………………………………………………

……………………………………………………………………………..

……………………………………………………………………………..

A12 Experience in Environmental Impact Assessment related activities…………

…………………………………………………………………………….

…………………………………………………………………………….

……………………………………………………………………………..

…………………………………………………………………………….

PART B: DECLARATION BY APPLICANT

B1 I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief.

………………………. ………………………. ……………………..

Signature of applicant ***Full Name in Block letters*** ***Position***

On behalf of ……………………………… ……………………………..

Firm Name and seal ***Date***

PART C FOR OFFICIAL USE

Approved/Not approved ……………………………………………………………….

Comments ………………………………………………………………………………

………………………………………………………………………………………..

………………………………………………………………………………………..

……………………………………………………………………………………….

Official …………………. Sign ………………… Date …………………

Important Notes

Please submit the following:

(a) Application Form in duplicate

(b) Curriculum vitae of all applicants; and

(c) The prescribed fee, to

Director-General,

The National Environment Management Authority (NEMA),

Kapiti Road, South C,

P.O. Box 47146,

NAIROBI, KENYA.

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

E-mail …………………………….

FORM 7 **(r.15(2))**

Application Reference No.:…………………………

Licence No.: ………………………………………..

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINAITON ACT

ENVIRONMENTAL IMPACT ASSESSMENT/AUDIT PRACTICING LICENCE

M/s ……………………………………………………….(Individual or firm) of Address ……………………………………………………………………….

………………………………………………………………………………

………………………………………………………………………………

……………………………………………………………………… is licenced to practice in the capacity of a (Lead Expert/Associate Expert/Firm of Experts) ……………………..

……………………………………………………… in accordance with the provisions of the Environmental Management and Coordination Act.

Date this …………………….. day ……………………. of 20 …………………

Signature ………………………………………………………………………

(Seal)

Director-General,

The National Environmental Management Authority.

Conditions of Licence:

1. This licence expires on 31st December, 20…………….

FORM 8 **(r.21)**

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINAITON ACT

NOTICE TO THE PUBLIC TO SUBMIT COMMENTS ON AN ENVIRONMENTAL IMPACT ASSESSMENT STUDY REPORT.

Pursuant to Regulation 21 of the Environmental (Impact Assessment and Audit) Regulations, the National Environmental Impact Assessment Study Report for the implementation of the proposed project………………………………………………

…………………………………………………………………………………………

………………………………………………………………………………………….

……………………………………………………………(brief description of project)

at ……………………………………………………………………………………….

……………………………………………………………..(locality) of …………………………………… District. The said project anticipate the following impact ……………………………………………………………………………………

…………………………………………………………………………………………

(describe anticipated impacts and proposed mitigation measures).

The full report of the proposed project may be inspected during working hours at:

- **(a) The NEMA Headquarters.**
- **(b) …………………………..**
- **(c) …………………………..**

NEMA invites members of the public to submit oral or written comments within …………days of the date of publication of this notice to assist the Authority in the approval process of the project to:

- **(a) Director – General, NEMA,**
- **(b) …………………………….**
- **(a) …………………………….**

Dated this …………………….day…………………..of 20……….

Signature ………………………………………………

(seal)

Director-General,

The National Environmental Management Authority.

FORM 9 **(r.25)**

Application reference No. ……………………………

Licence No. ………………………………..

FOR OFFICIAL USE

THE ENVIRONMENT MANAGEMENT AND CO-ORDINATION ACT

APPLICATION FOR VARIATION OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

PART A: PREVIOUS APPLICATION

No previous application for variation of an environmental impact assessment licence.*

The Environmental Impact assessment licence was previously amended.*

PART B: DETAILS OF APPLICANT

B1 Name (Individual or firm): ………………………………………………………

B2 Business Registration No: ……………………………………………… ……

B3 Address: ………………………………………………………………………

B4 Name of contact person: …………………………………………………………

B5 Position of contact person: ………………………………………………………

B6 Address of contact person: ………………………………………………………

Telephone No.: ……………………… Fax: …………………. No: ……………

E-mail: ……………………………………………………………………….

PART C: DETAILS OF CURRENT ENVIRONMENTAL IMPACT ASSESSEMENT LICENCE.

C1 Name of the current Environmental Impact Assessment licence holder ………….

……………………………………………………………………………………..

……………………………………………………………………………………..

C2 Application No. o the current Environmental Impact Assessment licence ………..

………………………………………………………………………………………

C3 Date of issue of the current Environmental Impact Assessment licence …………..

………………………………………………………………………………………

PART D PROPOSED VARIATION STO THE CONDITIONS IN CURRENT ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

D1 Conditions in the current Environmental Impact Assessment licence…………….

………………………………………………………………………………………

……………………………………………………………………………………...

………………………………………………………………………………………

D2 Proposed variation(s)……………………………………………………………..

………………………………………………………………………………………

………………………………………………………………………………………

………………………………………………………………………………………

D3: Reason for variation(s) …………………………………………………………

………………………………………………………………………………………

……

D4 Describe the environmental changes arising from the proposed variation(s) ……..

………………………………………………………………………………………

……

D5 Describe how the environment and the community might be affected by the proposed variation(s)……………………………………………………………….

………………………………………………………………………………………

………………………………………………………………………………………

………………………………………………………………………………………

D6 Describe how and to what extent the environmental performance requirements set out in the EIA report previously approved or project profile previously approved or project profile previously submitted for this project may be affected ……………………………………………………………………………………..

………………………………………………………………………………………

………………………………………………………………………………………

D7 Describe any additional measures proposed to eliminate, reduce or control any adverse environmental impact arising from the proposed variation(s) and to meet the requirements in the Technical Memorandum on Environmental Impact Assessment Process……………………………………………………………

..…………………………………………………………………………………….

……………………………………………………………………………………..

……………………………………………………………………………………..

PART E: DECLARATION BY APPLICANT.

I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief. I understand the environmental impact assessment licence may be suspended, varied or cancelled if any information given above is false, misleading or incomplete.

…………………….. …………………….. ……………………………

Name ***Position*** ***Signature***

on behalf of ……………………………….. …………………………………….

Company name and seal ***Date***

PART F: OFFICIAL USE

Approved/Not approved ……………………………………………………..

……………………………………………………………………………..

Comments …………………………………………………………………….

………………………………………………………………………………

………………………………………………………………………………

………………………………………………………………………………

Officer …………….. Signature………………… Date ………………..

Important Notes

Please submit:-

(a) 3 copies of this completed Form: and

(b) The prescribed fee to:

Director-General,

The National Environment Management Authority,

Kapiti Road, South C,

P.O. Box 47146,

NAIROBI.

Tel. 254-02-609013/27/79 or 608999 Fax. 254-02-608997

E-mail …………………………….

***Delete where applicable**

FORM 10 **(r.25)**

Application Reference No…………………….

Certificate No…………………………………

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF VARIATION OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

This is to certify that the Environmental Impact Assessment Licence No. ………………

…………………….. Issued on …………………………….(date) to ………………….

…………………………………………….(name of individual/firm) of ……………

…………………………………………………………………………(address) regarding …………………………………………………………………………………….

…………………….(title of project) whose objective is to ……………………………..

………………………………………………………………………………………

…………………………………………………………(briefly describe purpose) located at …………………………………………………….(locality and District) has been varied to …………………………………………………………………………

…………………………………………………………………………………………

…………………………………………………………………………………………

…………………………(nature of variation) with effect from …………………….(date of variation) in accordance with the provisions of the Act.

Dated this ………………………… day …………………..of 20……………..

Signature ……………………………

(Seal)

Director-General

The National Environmental Management Authority.

FORM 11 **(r.26)**

Application reference No……………………

Licence No:…………………………………

FOR OFFICIAL USE

ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

NOTIFICATION OF TRANSFER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

PART A: DETAILS OF CURRENT LICENCE

A1 Name of the current Environmental Impact Assessment licence holder…………

………………………………………………………………………………………

A2 PIN No. ………………………………………………………………………….

A3 Address ……………………………… A4 Tel: …………………………………

A5 Fax No. ……………………………….A6. E-mail ……………………………..

A7 Application No. of current Environmental Impact Assessment licence ……….

……………………………………………………………………………………..

A8 Date of issue of current Environmental Impact Assessment licence ………….

…………………………………………………………………………………….

PART B DETAILS OF THE TRANSFEREE

B1 Name (Individual/firm)……………………………………………………………

B2 PIN No. ………………………………………………………………………….

B3 Address …………………..………………… A4 Tel: …………………………

B5 Fax No. …………………………. B6. E-mail …………………………………

B7 Name of contact person……………………………………………………………

B8 Capacity of transferee to run the project (financial, technological, manpower) …..

………………………………………………………………………………………………
………………………………………………………………………………………………
……………………………………………………………………………

PART C: REASONS(S) FOR TRANSFER OF LICENCE

………………………………………………………………………………………

……………………………………………………………………………………….

…………………………………………………………………………………………………….

PART D: DECLARATION BY RANFEROR AND TRANSFEREE

It is hereby notified that ……………………………………. of …………………… on this day of ……………………….. transferred EIA licence No………………….to ……..

…………………………………………..of …………………………………………… who will assume his responsibility for all liability under this project.

Transferor	**Transferee**
Name …………………………..	**Name …………………………..**
Address …………………………	**Address ………………………..**
Signed …………………………..	**Signed …………………………**
Date ………………………………..	**Date ……………………………**

PART E: FOR OFFICIAL USE

Approved/Not approved……………………………………………………………………

……………………………………………………………………………………………

Comments …………………………………………………………………………….

……………………………………………………………………………………………

………………………………………………………………………………………………………
…………………………………………………………………………………………

Officer………………… Signature ………………… Date…………………………

Important Notes:

1. **Where an Environmental Impact Assessment licence is transferred, the person to whom it is transferred and the person transferring it shall jointly notify the Director-General, of the transfer.**

2. **The person holding an environmental impact assessment licence assumes responsibility for the transfer of the licence only in respect of the project to which this licence was issued.**

3. **Any transfer of an environmental impact assessment licence, shall take effect on the date the Director General is notified.**

4. **This Form must be submitted in quadruplets with**
5. **Prescribed fees, to:**

Director General,

The National Environment Management Authority,

Kapiti Road, South C,

P.O. Box 47146,

NAIROBI.

Tel. 254-02-609013/27/79 **Fax. 254-02-608997**

E-mail ……………………………….

FORM 12 **(r.26)**

Application Reference No: ………………………….

Certificate No. ………………………………………

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF TRANSFER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE

This is to certify that the Environmental Impact Assessment Licence No………………..

Issued on …………………………………… (date) to ………………………………….

………………………………………. (name of previous holder) of …………………….

…………………..(address) regarding …………………………………………………… (title of project) ……………………………………………………………………..

whose objective is to …………………………………………………………………….

………………………………………………………………………………………

………………………………………………………………………………………

(briefly describe purpose) located at ………………………………………..(locality and District) has been transferred to ……………………………………………….(name of new holder) of …………………………………………………..(address) with effect from ……………………………………………………………..(date of transfer) in accordance with the provision of the Act.

Dated this …………………….day ……………………of 20……………

Signature …………………………………….

(seal)

Director – General,

The National Environmental Management Authority.

Important notes

1. **The transferee as well as the transferor of a licence under this regulation shall be liable for all liabilities and the observance of all obligations imposed by the transfer in respect of the licence transferred.**

2. **The transferor shall not be responsible for any future liabilities or any obligations so imposed with regard to the licence form the date the transfer is approved.**

FORM 13 (r.27)

Application reference No: …………………..

Licence No: …………………………………

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

NOTIFICATION OF SURRENDER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

PART A: PROPONENT DETAILS

A1 Name: (Individual or firm) …………………………………………….

A2 PIN No. ……………………………………………………………….

A3 Address ……………………………………………………………….

A4 Name of contact person ………………………………………………

A5 Position of contact person ……………………………………………

A6 Address …………………… Tel …………………. Fax No…………

E-mail ……………………………….

PART B: DETAILS OF THE CURRENT ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

B1 Environmental Impact Licence No…………………………………..

B2 Title of project under the current Environmental Impact licence

……………………………………………………………………

……………………………………………………………………

……………………………………………………………………

B3 Please state the following details of the Environmental Impact Assessment licence to be surrendered.

(a) Scope/scale of project(s)……………………………………………………

…………………………………………………………………………….

…………………………………………………………………………….

(b) Conditions on the EIA licence ……………………………………………….

…………………………………………………………………………….

…………………………………………………………………………….

……………………………………………………………………………………….

PART C: REASON(S) FOR SURRENDER

……………………………………………………………………………………………….
……………………………………………………………………………………………….
……………………………………………………………………………………………….
……………………………………………………………………………………………….

PART D: DECLARATION BY PROPONENT

I hereby certify that the particulars given above are correct and true to the best of my knowledge and belief.

…………………………. ………………………………… ……………………..
Name of applicant *Full name in block letters* *Position*

on behalf of ………………………………. ………………………….
Company name and seal *Date*

PART E: FOR OFFICIAL USE

Approved/Not approved ………………………………………………………………
…………………………………………………………………………………………
Comments ……………………………………………………………………………
…………………………………………………………………………………………
…………………………………………………………………………………………
…………………………………………………………………………………………
Officer ………………….. Signature ……………………. Date…………………

Important Notes:

Intent to surrender an environmental impact assessment licence should be communicated to the Authority atleast six months before the date of surrender.

FORM 14 (r.27)

Application Reference No.: …………………….

Certificate No.: …………………………………

FOR OFFICIAL USE

THE ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

CERTIFICATE OF SURRENDER OF ENVIRONMENTAL IMPACT ASSESSMENT LICENCE.

This is to certify that the Environmental Impact Assessment Licence No:………………

Issued on ………………………………..(date) to ………………………………………

(name of individual/firm) of …………………………..………..(address) regarding ………………………………………………………………………………………(title of project) whose objective is to ………………………………………………….(briefly describe purpose) located at ……………………………………………………(locality and District) has been duly surrendered with effect from ………………………………… (date) to the National Environment Management Authority in accordance with the provisions of the Act.

Dated this ……………………… day ………………………. of 20………..

Signature ………………………………………..

(seal)

Director-General

The National Environmental Management Authority.

Important Note:

A surrender shall be without prejudice to any liabilities or obligations which have accrued on the holder of the licence prior to the date of surrender.

FORM 15 (r.29)

Form No. ……………………………..

Reference No. …………………………

FOR OFFICIAL USE

ENVIRONMENTAL MANAGEMENT AND CO-ORDINATION ACT

NATIONAL ENVIRONMENT MANAGEMENT AUTHORITY

APPLICATION FOR ACCESS TO INFORMATION

PART A: DETAILS OF APPLLICANT

A1. Name: ……………………………………………………………….

Address: ……………………………………………………………..

……………………………………………………………………….

……………………………………………………………………….

……………………………………………………………………….

Telephone: …………………………. Fax …………………………

E-mail ………………………………………………………………

Profession ……………………………………………………….

Date ……………………………………………………………….

A2 NAME OF EMPLOYER (if applicable)……………………………………………

Address: ……………………………………………………………………………

…………………………………………………………………………………………

Telephone: ……………………………. Fax: ……………………………………

E-mail ……………………………………………………………………………….

Designation ……………………………………………………………………….

PART B: INFORMATION DETAILS

B1. TYPE OF INFORMATION REQUIRED (tick as appropriate)

☐ Project Report

☐ Environmental Impact Assessment Study Report
☐ Environmental Audit Report
☐ Strategic Environmental Assessment Report
☐ Environmental Monitoring Report
☐ Record of Decision (ROD) for Environmental Impact Assessment Approvals
☐ Licences for Project Reports
☐ Licences for Environmental Impact Assessment
☐ Environmental Impact Assessment Experts (Individual)
☐ Environmental Impact Assessment Experts (Firms)

B2 DOCUMENT

Title of the document ……………………………………………………………………

Author ………………………………………………………………………………..

Year …………………………………………………………………………………..

B3 HOW THE INFORMATION IS EXTRACTED?

☐ Reading
☐ Environmental Impact Assessment Experts (Individual)

B4 PURPOSE FOR REQUIRING THE INFORMATION

☐ Educational	☐ Research
☐ Affected party	☐ Interested party

Important note:

A prescribed fee of KShs.200 will be charged for access to information per record/register.

10.0 REPORT WRITING

10.1 Introduction

10.1.1 Purpose

Design reports have two main functions, namely

(i) to record and present all particulars of a project in a clear and concise way for future reference and for information to planners or other parties interested in the project
(ii) to present basic data assumptions and conclusions regarding the project to enable superiors and other interested parties involved in the project to supervise and approve the design

10.1.2 Report Standard

The form and contents of the reports will vary considerably depending on the complexibility and size of the water supply projects.

However, reports shall always be compiled to satisfy the two requirements as described above.

Large and complex projects should be reported on in two stages namely preliminary Design and Final Design of which details are given later in this chapter.

For small and uncomplicated schemes one Design Report is often adequate.

10.2 Contents

10.2.1 Prefeasibility, Feasibility and Preliminary and Final Design Reports

The following list of contents will generally be applicable for medium and large size projects. However the contents of the report should always be modified to suit the requirements of the particular project.

Front Cover Page

The following Information is contained in front:-

- Title of the Project: The Project Title should be clearly stated so that the type of project being proposed can easily be identified e.g Multipurpose Irrigation Project. The Title should indicate the following:

  (a) The project's general location including its regional and national setting, and
  (b) Phase of the Project, if applicable

- Project Client: It is important that Project client be clearly stated. The report should also include name of the Consultant (s) or other organizations that were called upon to carry out initial assessment
- Consultant
- Date

Table of Content

This will be the entire format of the report

(a) List of Tables
(b) List of figures
(c) Acknowledgement/Preface
(d) Acronyms

Chapters, which will consist of:-

Chapter 1 Executive Summary

Presents an Overview of the Project which should be adequate to allow a decision is made by superiors and policy makers.

Chapter 2 Introduction

This Chapter briefly explains the reasons for the Report and how it was prepared

2.1 Project Genesis

- ❖ Describe how the proposed Project idea was developed
- ❖ Indicate which agencies, have responsibility for the promotion of the Project
- ❖ List and explain briefly previous studies and reports on the project (particularly the project identification report) prepared by others
- ❖ Make Reference to related long term plans for the sector, regional development, Land use, water resources development, rural development, primary health care, etc

2.2 Organization and Management of the Study

- ❖ Explain how the present study was carried out
- ❖ Indicate which agencies are responsible for the various elements of work
- ❖ Present a timetable for the Study and indicate the level of effort

2.3 Scope and Status of this Report

- ❖ Explain how this Report fits in the Overall process of Project Preparation
- ❖ Identify data Limitation
- ❖ List Interim Reports or notes submitted during the Study and summarize any guidance provided by the responsible Project Authority
- ❖ Explain whether the Report is intended to be used to obtain approval in principle for the proposed project. If so, the report needs to be more comprehensive and less tentative in its conclusion than in cases to be initiated shortly after this study report is completed.

3.0 Description of Area

3.1 Location
3.2 Climate (Rainfall, temperature, evaporation, wind regime , sunshine hours etc)
3.3 Topography and Geology
3.4 Drainage Regime
3.5 Existing Community based NGO,s
3.6 Existing Self Help Groups and siustainability

3.7 Community Participation Mechanisms and Experience

4.0 Socio-economic Infrastructure

4.1 Administration
4.2 Education
4.3 Health Facilities
4.4 Transport
4.5 Commerce and Industry
4.6 Agriculture

5.0 **Existing Water Supply**

5.1 Location, Source and ownership
5.2 Consumers
5.3 Reliability and Constraints
5.4 Technical and Economic assessment of the supply

6.0 **Existing Sanitation**

6.1 Location, Source and ownership
6.2 Consumers
6.3 Reliability and Constraints
5.4 Technical and Economic assessment of the Sanitation Mode

7.0 **Consumer projections**

7.1 Design Period
7.2 Human population
7.3 Institutions
7.4 Commerce and Industry

8.0 **Water Demand**

8.1 Human demand
8.2 Institutional demand
8.3 Industrial demand
8.4 Total Sewage Generated

9.0 Socio-economic Study

9.1 Social Structure
9.2 Cultural Values
9.3 Sanitation and hygiene System and Promotions
9.4 Water Usage , Demand and Trend
9.5 Existing Economic Activities
9.6 Existing Income and Distribution
9.7 Type of housing and distribution
9.8 Willingness and ability to pay for water and Sanitation Services
9.9 Conclusions

10.0 Hydrology

10.1 Groundwater sources
10.2 Surfacewater source
10.3 Other possible sources

11.0 **Possible Applicable Sanitation Mode**

11.1 Onsite Sanitation
11.2 Off site
11.3 Recommended Sanitation Option

12.0 Initial Environmental Assessment

(Should be written in line with Chapter 17)

13.0 **Cost Estimation**
13.1 Capital
13.2 Operation and Maintenance
13.3 Revenue
13.4 Tariff Setting
13.5 Sustainability
13.6 Loan Repayment schedule at Interval of Five years

Appendices

- Calculations
- Drawing Index Sheets
- Name of People who undertook the Study and the role played
- List of Reference
- Etc

Note: - If there has been other Reports e.g Preliminary, the Final Design Report should be a summary of Preliminary Design Reports. It should be accompanied by working drawings.

APPENDIX 1 GLOSSARY

GLOSSARY

Sanitation

Interventions (usually construction of facilities such as latrines) that improve the management of excreta.

Excreta

Faeces and urine

Hygiene

The practice of keeping oneself and the surrounding environment clean

Aqua-privy

Latrine in which excrete fall directly through a submerged pipe into a watertight settling chamber below the floor, and from which effluent overflows to a soakaway or drain.

Desludging

- Removing settled solids from pits, vaults, tanks and septic tanks, digestion
- Decomposition of organic matter in wet conditions.

Nightsoil

- Human excrete, with or without anal cleaning material, which are deposited in a bucket or other receptacle for manual removal (often taking place at night).

Offset pit

- Pit that is partially or wholly displaced from its superstructure.

Overhung latrine

• Latrine sited such that excrete falls directly into the sea or other body of water.

Scum

• Layer of suspended solids less dense than water and floating on top of liquid waste from which they have separated by flotation

Sedimentation

- Process by which suspended solids denser than water settle as sludge

Vault

- Watertight tank for storage of excrete

Water closet (WC)

- Pan from which excrete is flushed by water into a drain.

Water seal

- Water held in a U-shaped pipe or hemispherical bowl connecting a pan to pipe, channel or pit to prevent the escape of gases and insects from the sewer or pit.

Aerobic — A term used to describe either an environment in which oxygen is present **,** or to describe an organism requiring, or not destroyed by, the presence of free 9elemental) oxygen

Anaerobic — A term used to describe either an environment in which oxygen is absent, or to describe an organism requiring, or not destroyed by, the absence of air or free (elemental) oxygen

Anaerobic Decomposition — Putrefaction or septic decay

Contaminate — To introduce pathogens

Domestic Sewage — Sewage produced as a result of normal domestic activities and principally derived from dwellings, business buildings' institutions and the like; it includes human excreta

Excreta — Liquid and solid human or animal body wastes -that is, faeces and urine

Faeces — Solid human excreta

Industrial Effluent — Sewage produced from industrial processes or agricultural activities

Infect — To introduce disease **to a new victim**

Mechanical Carriers — Creatures which, without being themselves diseased, carry pathogens on their bodies

Organic material — The waste products or remains of living organisms

Pathogens	Organisms which cause disease
Pollute	To lower the quality of a water relative to a particular use or in such a way as to affect adversely the aquatic environment
Property Drain	Drains usually within a property boundary which convey sewage from sanitary fitments to a sewer or sewage tank
Property Laterals	Those parts of property drains which lie outside the boundary
Public Sewer	A sewer provided and operated by Water Service Provider
Sanitary	Conducive to health; usually applied to methods of sewage disposal
Sanitary Facilities	arrangements for sanitary sewage disposal, including sinks, baths, water closets and similar fitments
Sewage	Used water containing organic and inorganic matter and, especially in tropical countries, usually also pathogens; sewage normally comprises both domestic sewage and industrial effluents and may or may not be diluted by groundwater or storm water
Sewerage	A sewerage system plus all plant for pumping and treating sewage before disposal
Sewerage System	A network of sewers and perhaps pumping mains leading to a sewage treatment works
Storm Water	Surface water produced by heavy rainfall
Sullage	Water dirtied as a result of washing or food preparation; for example, the liquid wastes from sinks, baths and showers
Surface Water	Water on or flowing over the surface of the ground
Surface Water Drains	Purpose made drains or ditches for collecting and transporting surface water to a watercourse or soakage area
Urine	Liquid human excreta
Water Closet	A small room with sanitary fitments furnished with a water supply to flush away wastes; that is, a water closet provides water borne sanitation
Water Resources	The total amount of water available for use**,** in any way, by a human society
Watersheds	The boundaries enclosing and separating drainage area

APPENDIX 2

REVIEW OF FINDINGS OF PREVIOUS

INVESTIGATIONS AND REPORTS ON

PER CAPITA WATER DEMAND IN KENYA

LIST OF TABLES

Introduction

A total of 11 previous studies have been reviewed to obtain their findings and also enable the determination of historical per capita trend in Kenya.

(a) Design and Selection Criteria for community Water Supplies

(Report No.4 by World Health Organization (WHO) dated October, 1972.)

Report No.4 presented operation charts for all schemes, which were being operated by Ministry of Water Development. These charts were completed each month for each supply and gave details of number of connections, fuel costs, chemical costs, wages and water consumption figures. The data gave the following figures: -

Table 3.a.1: - Per Capita Water Consumption

	1968 – 1969	1970 - 1971
Urban centres	86 litres/head/day	103litres/head/day
Rural centres	67 litres/head/day	76.5 litres/head/day
Market and Local centres	30 litres/head/day	38 litres/head/day

The results were plotted and showed that there was a large variation within each group as indicated in Table 3. a.2.

- It was concluded that the Average Consumption rate was therefore unsuitable to adopt as a standard figure.

Arising from the above, the following consumption rates were proposed as a guide for design and it was suggested that the values could be adjusted as necessary after considering the particular conditions and the present consumption trends: -

High class housing	300 litres/head/day
Average urban housing	150 litres/head/day
Low cost:	75 litres/head/day
Squatters and people served by stand pipe or kiosks.	20 litres/head/day

It was suggested that the requirements of Industries should be assessed separately according to the plans of the particular area for extensions to existing industries and new industries.

In cases where augmentation is required to serve large additional areas and Development Plans are uncertain, the following overall consumption rates was proposed to be used to allow for all types of users: -

Urban centres	135 litres/head/day
Rural centres	110 litres/head/day

For commercial and Administrative units, the following was proposed: -

Administration offices per person	25 litres/day
Hospitals, high class, per bed	400
Hospitals, medium class, per bed	200
Hospitals, low class, per bed	100
Dispensaries per out-patient	10
Boarding schools per person	50
Day schools per person	25
Hotels, high class per bed	600
Hotels, medium class, per bed	300
Hotels, low class, per bed, without toilet	50

Industrial requirements were noted to vary with the type of industry. These were to be studied separately compared with overseas figures.

Table 3.a.2: - Variation of per capita Water Demand

CONSUMPTION DATA						
A. Urban Centres WP Operated Scheme (Excluding Mombasa)						
	Population 1969	**Average Consumption 1968-69 (In 1000 lts)**	**Consumption/ head (In lts)**	**Average Consumption 1970-71 (In 1000 lts)**	**Consumption/ head (In lts)**	**%Increase in Total Consumption**
Central Province						
Port Hall	4,800	516	107	533	105	3
Nyeri	26,000	1,590	61	1,730	64	9.5
Rift						
Kabarnet	54,000	74	14	71	13	-3.5
Kericho	10,100	746	75	842	79	13
Naivasha	6,900	450	65	705	97	57
Ngong	1,590	96	61	123	74	28
North Eastern						
Garissa	Not known	220		495		125
Coast						
Malindi	6,000	962	161	1,110	174	15
Nyanza						
Kisii	6,000	61	103	77	123	28
Homa Bay	3,200	314	99	456	137	45
Eastern						
Babu	3,930	526	134	632	153	20
Kitui	3,020	232	77	313	98	34
Machakos	6,300	615	97	865	130	41
Meru	4,470	500	112	595	105	-1
Western						
Bungona	4,400	297	68	304	65	2
Busia	1,100	69	64	142	123	104
Kakamega	6,200	495	80	676	105	36
Average			**86**		**103**	**32**

B. Rural Centres WD Operated Schemes

	Population 1969	Average Consumption 1968-69 (In 1000 lts)	Consumption/ head (In lts)	Average Consumption 1970-71 (In 1000 lts)	Consumption/ head (In lts)	%Increase in Total Consumption
Central Province						
Gatundu	800	82	105	110	130	31
Kandara	510	20	40	25	48	25
Kangema	850	20	25	44	49	110
Karatina	10,000	184	182	250	236	36
Kiambu	2,700	347	127	450	159	30
Kigango	5,900	385	65	654	105	70
Kikuyu	1,500	183	123	210	133	15
Kerugoya	1,900	153	80.5	198	99	30
Kianyaga	500	42	83.5	65	104	30
Limuru	1,200	83	69	103	82	23
Mukurweini	2,000	30	14.5	35	17	19
Othaya	4,240	39	9	50	11	40
Ruiru	1,670	169	100	194	110	15
Coast						
Gede Watamu	2,100	154	73	257	115	17
Kilifi	1,200	135	113	221	175	63
Kwale	1,100	91	83	96	83	6
Mariakani	4,000	230	57	220	52	-4
Lamu	7,400	154	21	169	22	10
Voi	5,300	331	62	413	74	25
Wundanyi	4,300	80	18	116	26	44
Rift Valley						
Bomet	630	29	45	30	45	5
Eldama Ravine	2,700	61	23	62	22	2
Gilgil	4,200	165	39	185	42	12
Kajiado	1,800	284	159	368	190	30
Kapsabet	2,300	129	56	151	62	17
Kapenguria	1,700	82	48	77	43	-6
Londiani	3,000	90	30	96	30	7
Lumwa	2,600	80	30	81	30	2
Lodwar	Not known	110	_	80	_	-27
Maralal	3,900	93	24	103	25	11

	Population 1969	Average Consumption 1968-69 (In 1000 lts)	Consumption/ head (In lts)	Average Consumption 1970-71 (In 1000 lts)	Consumption/ head (In lts)	%Increase in Total Consumption
Molo	4,200	327	76	340	76	4
Nandi Hills	Not known	73	_	86	_	17
Narok	2,600	146	56	171	62	17
Sotik	800	84	105	108	127	29
Average			**67**		**81**	**26**
Nyanza						
Bondo	Not known	63	_	78	_	23
Maseno	1,280	304	240	327	243	7
Migori	2,000	43	21	46	22	10
Ukwala	1,000	33	33	31	29	-6
Eastern						
Chuka	2,500	44	18	71	27	63
Isiolo	8,200	145	18	193	23	34
Marsabit	5,600	77	14	186	32	124
Maua	1,000	18	18	23	21	26
Mwingi	2,500	32	13	57	21	18
Nkubu	3,000	54	18	75	24	39
Western						
Butere	1,000	75	75	62	59	-16
Kimili	700	113	161	118	155	4
Average			**67**		**76.5**	**24**

C. Market and Local Centres						
	Population 1969	**Average Consumption 1968-69 (In 1000 lts)**	**Consumption/ head (In lts)**	**Average Consumption 1970-71 (In 1000 lts)**	**Consumption/ head (In lts)**	**%Increase in Total Consumption**
Central						
Kigumo	420	21	51	27	62	27
Saba Saba	1,000	24	24	27	26	14
Rift						
Elburgon	5,300	96	18	123	23	28
Kapkatet	590	16	27	18	28	15
Kijabe	Not known	3	_	3	_	15
Kinangop	Not known	950	_	1,280	_	34
Coast						
Kinango	2,400	35	15	53	21	50
Mazeras	2,800	50	18	114	39	126
Nyanza						
Nyamira	250	87	35	87	33	0
Western						
Vihiga	500	26	52	39	73	49
Average			**30**		**38**	**36**

(b) Ministry of Water Development Design Manual dated May 1978.

This was the Standard Book used by the Government of Kenya for regulating and coordinating the water sector from year 1978 to 1983

Water Consumption

The consumption rates as recommended in WHO report 4 were proposed as a guide for design and could be adjusted as necessary after considering the particular conditions and the consumption trend. The rates represented the expected maximum demand and expected leakages in the distribution system.

Requirements of Industries were to be assessed separately according to the development plan of the particular area.

An addition of 5% was to be provided for water used in treatment plants.

(c) Ministry of Water Development Design Manual dated August, 1986.

This was the Standard Book used by the Government of Kenya for regulating and

(iii) Water Consumption rates

General

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

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

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

(d) **Ministry of Water and Irrigation Practice manual for water supply services dated October 2005.**

This is the standard book currently being used to Design Water Supplies in Kenya. The Study looked at facilities in WHO Report No. 4 and compared the values deduced in After Care Study of 1998.

It concluded that the values as proposed in Ministry's Design Manual 1986 be adopted since the values appeared to be valid as per After Care deduced per capita values despite of belief to the contrary.

The reason was cited as majority of our water supplies were constructed twenty to forty years ago and have hence surpassed their life span. There was lack of maintenance and new investment. This implied that rationing was the order of the day.

In view of the above the forces of supply and demand were not allowed to come into play.

Table 3.c.1: - Consumption Rates

Consumer	Unit	Rural Areas			Urban Areas		
		High potential	Medium potential	Low potential	High class housing	Medium class housing	Low class housing
People with individual connections	1/head/ day	60	50	40	250	150	75
People without individual connections	1/head/ day	20	15	10	_	_	20
Livestock	1/head/ day	50			_		
Boarding schools	1/head/ day	50					
Day schools with WC Without WC	1/head/ day	25 5					
Hospitals Regional District Other	1/bed/ day	400) 200) 100)			+20 1 per outpatient and day (Minimum 5000 1/day)		
Dispensary and Health Centre	1/day	5000					
Hotels High class Medium class Low class	1/bed/ day	600 300 50					
Administrative offices	1/head/ day	25					
Bars	1/day	500					
Shops	1/day	100					
Unspecified	1/ha/				20,000		

(e) **The After Care Study on the National Water master plan dated November, 1998 financed by JICA.**

The After Care Study Report used 141 Urban centres and 50 districts to estimate the National Average per capita water consumed as 100 l/p/d/. The summary is as below: -

Table 3.e.1 National Per Capita Water Consumed as per 1996

Service Area	Total Population (1998)	Service Coverage (%)	Water served population	Water supply (m^3/day)	Per capita consumption (l/p/d)
Urban Centre (141)	5,280x10^3	94	4,974x10^3	709x10^3	143
Rural Areas	22,240x10^3	44	9,724x10^3	750x10^3	78
Total/ average	**27,520x10^3**	**53**	**14,640x10^3**	**1,459x10^3**	**100**

Using about 30 urban centres from the said After Care Study and using Excel, the average Urban Water Demand per capita is estimated as 141l/p/d.
The details are as in table 3.e.2 below: -

Table 3.e.2: - Summary of Per Capita Urban Water Consumption

No.	Name of Urban Centre	Municipal area in Km^2	Urban Population	Urban population connected to Water Supply	Quantity of Water Supplied in m^3/day	Per Capita Water Consumption in lpd.
1.	Bungoma	55	70,000	36,000	2,620	72.77777778
2.	Kisumu	604ha	231,687	280,845	14,565	51.86134701
3.	Nakuru	12.9	231,687	304,561	41,120	135.0140038
4.	Nairobi	693	2,240,000	1,784,577	326,700	183.0685927
5.	Kiambu	58	7,500	8,058	490	60.80913378
6.	Limuru	N/A	3,000	1,958	660	337.0786517
7.	Thika	93	155,770	120,000	24,000	200
8.	Muranga	25	30,000	24,000	2,000	83.33333333
9.	Nyahururu	104	60,000	50,000	3,000	60
10	Karatina	N/A	7,299	14,533	1,300	89.45159293
11	Nyeri	200	142,000	40,000	7,000	175
12	Mombasa	282	580,000	370,764	180,200	486.0234543
13	Voi	16	15,772	20,300	2,700	133.0049261
14	Embu	80	45,000	35,000	45,000	128,5714286
15	Isiolo	N/A	26,968	36,000	4,700	130.5555556
16	Athi River	684	50,000	12,500	2,000	160
17	Machakos	520	154,006	80,000	1,740	21.75
18	Meru	N/A	124,412	16,330	4,700	28.78138396
19	Kisii	29	65,000	45,000	4,000	88.88888889
20	Homa Bay	197	75,000	43,000	750	17.44186047
21	Ngong	N/A	15,000	6,000	1,260	210
22	Kericho	66	80,000	58,723	5,246	89.33467296
23	Nyanyuki	147	55,000	43,100	11,250	261.0208817
24	Naivasha	9.89	60,000	46,000	762	16.56521739
25	Kitale	92	75,000	60,000	9,000	150
26	Eldoret	147	220,000	90,000	25,000	277.7777778
27	Kapsabet+ Baraton	14	20,000	7,000	1,100	157.1428571
28	Webuye	69	60,000	40,000	1,700	42.5
29	Busia	25	48,000	17,267	2,072	119.9976834
30	Kakamega	51	110,000	27,826	7,000	251.5632861
National Urban Average						**140.6438102**

Similarly using the same information, per capita sewage generated in urban areas is estimated at about 120 lpd.

The details are as in table 3.e.3 below: -

Table 3.e.3: - Summary of per capita urban sewage generated

No.	Name of Urban Centre	Municipal area in Km^2	Urban Population	Urban population connected to Sewer	Quantity of sewage flow in m^3/day	Per Capita Sewage flow in lpd.
1.	Bungoma	55	70,000	12,6000	772	61.26984
2.	Kisumu	604ha	231,687	130,000	15,000	115.3846
3.	Nakuru	12.9	231,687	123,500	8,223	66.583
4.	Nairobi	693	2,240,000	1,000,000	217,000	217
5.	Kiambu	58	7,500	2,250	150	66.66667
6.	Limuru	N/A	3,000	2,100	495	235.7143
7.	Thika	93	155,770	87,230	10,000	114.6395
8.	Muranga	25	30,000	10,500	870	82,85714
9.	Nyahururu	104	60,000	18,000	3,436	190.8889
10	Karatina	N/A	7,299	5,109	561	109.8062
11	Nyeri	200	142,000	37,100	2,500	67.38544
12	Mombasa	282	580,000	69,600	4,590	65.94828
13	Voi	16	15,772	700 Facility dry		0
14	Embu	80	45,000	9,000	682	75.77778
15	Isiolo	N/A	26,968	1,700	595	350
16	Athi River	684	50,000	12,500	1,700	136
17	Machakos	520	154,006	8,000	150	18.75
18	Meru	N/A	124,412	800	500	625
19	Kisii	29	65,000	13,000	972	74.76923
20	Homa Bay	197	75,000	15,000	1,045	69,66667
21	Ngong	N/A	15,000	750	Very little flow and ponds neglected	0
22	Kericho	66	80,000	41,600	2,000	48.07692
23	Nyanyuki	147	55,000	24,750	8,000	323.2323
24	Naivasha	9.89	60,000	30,000 dry		0
25	Kitale	92	75,000	37,500	4,760	126.9333
26	Eldoret	147	220,000	70,400	6,000	85.22727
27	Kapsabet+ Baraton	14	20,000	8,000	591	73.875
28	Webuye	69	60,000	12,000	433	36.08333
29	Busia	25	48,000	9,600	600	62.5
30	Kakamega	51	110,000	51,700	5,000	96.7118
National Urban Average						**119.8916**

Further, correlating sewage generated and water consumed from the same source, it has been found that about 85% of water used in Urban Areas end up as sewage and has to be treated before being disposed. The details are as in table 3.e.4 below: -

Table 3.e.4: - Summary of per capita urban water and sewage correlation

No.	Name of Urban Centre	Municipal area in Km^2	Urban Popula-tion	Per Capita Water consumption in lpd	Per Capita Sewage Flow in lpd	Correlation
1.	Bungoma	55	70,000	72.77777778	61.26984	0.841875682
2.	Kisumu	604ha	231,687	51.86134701	115.3846	2.224867306
3.	Nakuru	12.9	231,687	135.0140038	66.583	0.493156222
4.	Nairobi	693	2,240,000	183.0685927	217	1.185348053
5.	Kiambu	58	7,500	60.80913378	66.66667	1.096326531
6.	Limuru	N/A	3,000	337.0786517	235.7143	0.699285714
7.	Thika	93	155,770	200	114.6395	0.573197295
8.	Muranga	25	30,000	83.33333333	82.85714	0.994285714
9.	Nyahururu	104	60,000	60	190.8889	3.181481481
10	Karatina	N/A	7,299	89.45159293	109.8062	1.227549121
11	Nyeri	200	142,000	175	67.38544	0.385059684
12	Mombasa	282	580,000	486.0234543	65.94828	0.135689493
13	Voi	16	15,772	133.0049261	0	0
14	Embu	80	45,000	128.5714286	75.77778	0.589382716
15	Isiolo	N/A	26,968	130.5555556	350	2.680851064
16	Athi River	684	50,000	160	136	0.85
17	Machakos	520	154,006	21.75	18.75	0.862068966
18	Meru	N/A	124,412	28.78138396	625	21.71542553
19	Kisii	29	65,000	88.88888889	74.76923	0.841153846
20	Homa Bay	197	75,000	17.44186047	69.66667	3.994222222
21	Ngong	N/A	15,000	210	0	0
22	Kericho	66	80,000	89.33467296	48.07692	0.538166442
23	Nyanyuki	147	55,000	261.0208817	323.2323	1,238338945
24	Naivasha	9.89	60,000	16.56521739	0	0
25	Kitale	92	75,000	150	126.9333	0.846222222
26	Eldoret	147	220,000	277,7777778	85,22727	0.306818182
27	Kapsabet+ Baraton	14	20,000	157.1428571	73.875	0.470113636
28	Webuye	69	60,000	42.5	36.08333	0.849019608
29	Busia	25	48,000	119.9976834	62.5	0.520843388
30	Kakamega	51	110,000	251.5632861	96.7118	0.384443216
National Urban Average				**140.6438102**	**119.8916**	**0.852448345**

(f) Water Practice Manual by Bernard J. Dangerfield dated January 1983.

This seems to be the recognized Standard book World Wide up to date i.e. from 1983 to 2006. The Ministry's Standards have tended to ensure that the per capita adopted is within the required range as set out in this book.

The International Agencies e.g. Japan International Cooperation Agency (JICA) do recognize this ranges for Developing Countries Kenya being one of them.

(i) Requirement for survival

It is not possible to determine a precise figure for the minimum per capita water requirement. Requirements will vary depending upon the climate and prevailing social conditions.

The normal bodily water requirements for an adult in a temperate climate are about 2.2 l/day though some of this will be obtained from food. In a hot humid climate the requirement can exceed 9l/day. Apart from this, water is required for personal, ablutions, cooking and washing dishes, laundering, house cleaning, toilet flushing (if any), and other uses such as watering gardens and animals.

The Table 3.f.1 below gives the minimum water likely to be acceptable to consumers: -

Table 3.f.1: - Estimated minimum water requirements in (l/head/day)

Source	**Male, Maldives**		**Kathmaandu, Nepal**
	Private wells	**Piped**	**Standpipes**
Drinking cooking, dishwashing, house cleaning	7-15	15	10.5
Laundering	8-10	5	5
Ablutions	20-40	44.5	17.5
Toilet flushing			
Cistern flush	15	45	-
Hand flush	8	17.5	2.5
Other uses	-	8	4
Total	**43 - 73**	**90 - 117.5**	**39.5**

The use of water is likely to be at minimum where supplies are taken from standpipes, tankers or hand pumps.

Table 3.f.2-1.f.5 below shows the result of survey taken from various countries: -

Table 3.e.2: - Results of Survey of Standpipe and tanker supplies from various Countries

Location	Per Capita consumption l/head/day			Type of supply
	Min.	Mean	Max.	
Lima, pueblos, Jovenes,	-	30	-	Tanker
Hong Kong	-	32	68	Standpipe
Kathandu	7	31	47	Standpipe
Male, Maldives Republic	-	25	-	Standpipe
Cairo	-	14	-	Standpipe
Egypt, Provincial towns	12	32	46	Standpipe
Lesotho Provincial towns	20	-	36	Outside tap
St. Lucia	_	45	-	Standpipe

From table 3.f.2, it is suggested that average per capita consumption for water taken from **standpipes** is about **30l/head/day.**

A World Health Organization publication on the Design of standpipes recommends a figure of between **20 and 60l/head/day** for Design purposes.

Further, figures for hand pump supplies in Bangladesh show Per Capital consumption ranging from 12l/head/day where 100 or more people use one pump, to over 40l/head/day where a single family uses a pump alone.

Table 3.f.3: - Results of Survey of Domestic Per Capita Consumption using Individual Connection from various Countries

Place	Country	Domestic per Capita consumption, 1/head/day	Year	Growth in domestic Per Capita consumption, 1/head/day/year.
Male	Maldives	40-100	1981	
Kathmanda	Nepal	96	1973	
Cairo	Egypt	157	1966	
		181	1976	2.6
Port Said	Egypt	127	1966	
Provincial towns	Egypt	39-149	1978	
Provincial towns	Lesotho	107-158	1977	
Istanbul	Turkey	119	1976	
Lima	Peru	212	1980	
Santa Cruz	Bolivia	124	1981	
Sucte	Bolivia	98	1981	
Camiri	Bolivia	137	1982	
Paris	France	143	1948	
		159	1978	0.5
Cape Town	South Africa	140	1978	
Amsterdam	Netherlands	91	1948	
		123	1978	1.1

Reference: Binnie & Partners[3, 4, 8,12,13], John Taylor & Sons and Binnie & Partner[7], CONNAL[14], and Reed[15].

Table 3.f.4: - Summary of Measurements of Domestic Per Capita Consumption from various Countries according to Housing Class

Housing Class	Description	Range of per capita consumption [(a)], 1/head/day
High	**Detached houses, luxury apartments having 2 or more WCs, and 3 or more taps per household**	**260 – 150**
Middle	**Houses and apartments having at least 1 WC and 2 taps per household.**	**160-110**
Lower	**Tenements, government rehousing, shared houses, having at least 1 tap per household but sharing WC.**	**70* - 55** ***Frequently higher due to wastage.**

The above figures are based on tests carried out between 1970 and 1978 in the following places: Instanbul (Turkey), Sakaka (S. Arabia), Lesotho (Africa), Cairo (Egypt). Palembang (Indonesia), Hong Kong, Alexandria and Port Said (Egypt), also in Camiri, Bolivia in 1981.

Reference: CONNAI[14], Twort[16].

Notes:

(a) Exclusive of avoidable consumer wastage

(b) Figures for the same type of property gave consumptions of 90 1/head/day where there were fewer than 15 persons per meter to 233 1/head/day where over 35 persons were supplied through the meter.

(c) In government low cost housing blocks consumption averages 50 1/head/day where households have individual meters, but is about 110 1/head/day where washing facilities are shared.

Table 3.f.5: - Results of Per Capita Water Use Survey

Water use	Total Consumption 1/week
3 showers/person/day, 20 1/shower	2100
4 uses of WC/person/day, 10 1/use	2800
2 hand washing/person/day, 2 1/wash	140
3 meals/family/day, 10 1/meal	210
2 clothes washes/week, 150 1/wash	300
1 garden irrigation/week, 100 1/irrigation	100
1 car wash/week, 100 1/wash	100
1 floor wash/day, 2 1/wash	14
Total per family per week	5764
Average per Capita	**165 1/head/day**

Reference: CONNAL[14]

b) Per Capita Domestic Water Consumption

Table 3.f.3 above shows Average Domestic Per Capita Consumption where a 24-hour piped supply is provided.

Table 3.f.5 illustrates water use per a "typical family".
It should be noted that because of the wide variation in Domestic Per Capita Consumption, it is not generally advisable to adopt a figure from another area as the basis of forecast.

(h) WHO Report No.9

This is the Standard Book used for Design of Sewers in Kenya. Table 1.g.1 below gives the anticipated Daily contribution of Domestic Sewage: -

Table 1.h.1: - Anticipated Daily contribution of Domestic Sewage from various types of Establishments

Type of Establishment			Unit (per)	Factor (percent)	Anticipated quantity of Sewage Produced Daily.	
	Litres	Gallons			Litres	Gallons
High Class Housing	300	66	head	75	Say 220	48
Average Urban Housing	150	33	head	80	120	26
Low Cost Housing	75	16.5	head	85	Say 65	14
Communal Latrines/ ablution blocks (serving low cost dwellings which do not have individual facilities)	Say 70	15.5	head	85	Say 60	13
Public latrines	-	-	-	-	Say 1,500 litres (330 gallons) for each water closet or group of three urinals	
Day schools, shops and offices	Say 25	5.5	head	85	Say 21	5
Hospitals, high class	400	88	bed	80	320	70
Hospitals, Medium class	200	44	bed	80	160	35
Hospitals, low class	100	22	bed	80	80	17.5
Hotels, high class	600	132	bed	80	480	106
Hotels, others	300	66	bed	80	240	52.5
Dispensaries	50	11	Out patient	80	40	9
Non-resident factory workers	Say 25	5.5	head	80	20	4.5

Peak Factor

In Kenya, the ratio between the peak daily rate of flow and the average daily flow of domestic sewage is of the order of $2^1/_2$.

However, it would be uneconomical to use this factor to design all the sewers in a large sewerage network.

Sewage takes a finite time to flow through a sewerage system; as a result, peak flow rates tend to decrease as the extent of the network and therefore the population served increase.

Babbitt, in his book "Sewerage and Sewage Treatment", suggests that the relationship between peak sewage flows and population served is

$$M = \frac{5}{P^{1}/_{5}}$$

Where; M is the ratio of peak flow to average flow rate, and
P is the contributing population in thousands.

In practice, it is usually more convenient to link peak flows to pipe sizes and the following peak flow factors are suggested for use in Kenya.

Table 3.g.2 Suggested Sewage Peak Factors for Kenya

	Pipe Size	Peak Flow Factor
A	For sewers 300 millimetres (12 inches) in diameter or less	$2^1/_2$
B	For sewers exceeding 300 millimetres (12 inches) but less than 600 millimetres (24 inches) in diameter	2
C	For sewers 600 millimetres (24 inches) in diameter or more	$1^1/_2$

(i) Nairobi Mater Plan for Sewer, Sanitation and Drainage.

(By Otieno Odongo & Partners, Gath & Wanjohi in Association with SWECO dated September, 1998.)

a) Selection and Classification of industries

From the NCC water consumption records (Oct-Dec 1995) industries consuming at least 100 m^3/month were selected. Using the NCC water consumption records and the industrial inventory obtained from the Ministry of Commerce and Industry a selection of the industries considered to be major producers of wastewater and those discharging the most polluted wastewater was done and a list drawn up. The criteria for selection manufactured in Kenya, their location/address, product manufactured and classification according to the International Standard Industrial Classification (ISIC). The distribution of the industries in the four main industrial area were taken into consideration.

The industries selected were then grouped into the following broad classification.

Class 1 Wastewaters from the manufacture of food and drink. The main characteristic of these is that the compounds they contain are natural organic compounds.

Class 2 Other organic wastewaters. This class includes wastewaters from such industries as paper, leather and wool, in which the raw materials used are animal or vegetable matter.

Class 3 Wastewater containing metals and cyanides. These are largely produced in the engineering industry.

Class 4 Chemical wastewaters. These arise largely from chemical and pharmaceutical industries and factories using chemicals as part of the manufacturing process.

Generally, wastewaters in Class 1 and Class 2 do not normally present problems of treatment provided sufficient hydraulic and biological capacity is available at the treatment works. Class 3 wastewaters can be toxic to fish and can inhibit biological processes used at sewage treatment works. Class 4 wastewaters may at one extreme contain quantities of complex organic compounds, such as pesticides, some of which are highly toxic in also very small concentrations and at the other extreme the wastewaters may be quite innocuous. It is the toxic wastewaters, which can give, rise to the greatest problems and if discharge to sewers is uncontrolled they can lead directly to difficulties or even breakdown of the biological process at sewage treatment works and to fish deaths in receiving waters. Even sub-lethal quantities may make water unsuitable as a source of potable supply.

The International Standard Industrial Classification (ISIC) has been used as a basis for the identification of class sub-groupings and Table 1.i.1 shows the Industrial Classification Index that has been produced.

Table 3.h.1: - Industrial Classification Index

Class	ISIC No.	Process
Class 1	3111	Slaughtering, preparing and preserving meat
	3112	Manufacture of dairy products
	3113	Canning and preserving of fruit and vegetables
	3114	Canning, preserving and processing of fish
	4115	Manufacture of vegetables and animal oil and fats
	3116	Grain mill products
	3117	Manufacture of bakery products
	3119	Manufacture of cocoa, chocolate and sugar confectionery
	3121	Manufacture of food products
	3122	Manufacture of prepared animal feeds
	3131	Distilling, rectifying and blending of spirits
	3133	Malt liquors and malt
	3134	Soft drinks and carbonated waters industries.
Class	**ISIC No.**	**Process**
Class 2	3140	Tobacco manufacture
	3231	Tanneries and leather finishing
	3240	Manufacture of footwear
	3411	Manufacture of pulp, paper and paperboard
	3419	Manufacture of pulp, paperboard and articles
	3420	Printing, publishing and allied industries
Class 3	3700	Basic metal industries
	3811	Manufacture of cutlery, hand tools and general hardware
	3812	Manufacture of furniture and fixtures primarily of metal
	3813	Manufacture of structural metal products
	3819	Manufacture of fabricated metal products except machinery and equipment
		Manufacture of electrical machinery, apparatus, appliances and supplies
	3830	Manufacture and assembly of motor vehicles
		Manufacture of aircraft and repair.
	3843	
	3845	
Class 4	3211	Spinning, weaving and finishing textiles
	3212	Manufacture of made-up textile goods except wearing apparel
	3220	Manufacture of wearing apparel except footwear
	3219	Manufacture of textiles
	3511	Manufacture of basic industrial chemical excluding fertilizer.
	3514	Manufacture of fertilizer and pesticides.
	3521	Manufacture of paints, varnishes and lacquers
	3522	Manufacture of drugs and machines
	3523	Manufacture of soap perfumes, cosmetics and other preparations.
	3529	Manufacture of chemical products NEC
	3530	Petroleum products
	3550	Manufacture of rubber products
	3560	Manufacture of plastic products
	3620	Manufacture of glass and glass products
	3691	Manufacture of structural clay products
	3699	Manufacture of non-metallic products NEC

Industrial Testing and Sampling of the Wastewater

A sampling and testing programme for the selected industries was prepared and a questionnaire drawn up by the study team was used to gather information about the quantity, strength, content and method of wastewater discharge. Products, processes and amount of water consumed were also established. Any wastewater pretreatment plant was carefully examined and snaps effluent samples taken for analysis. Industrial wastewater survey information is summarized in Table 1.i.2.

Table 3.h.2: - Summary of Industrial Water Demand in Nairobi as per 1995

Class No.	ISIC No.	Name of Industry	Water Consumption/Month in m^3/month	Water Consumption in m^3/day
1	3113	Trufoods	239.00	7.97
1	3115	East Africa Industries	1,624.00	54.13
1	3115	Kapa oil Refineries	321.00	10.70
1	3117	Elliot Bakeries	2,840.00	94.67
1	3117	House of Manji B	472.00	15.73
1	3117	Mothers Favourite	1,022.00	34.07
1	3117	House of Manji A	472.00	15.73
1	3119	Cadbury Schweppes	918.00	30.60
1	3121	Nestle Foods	1,336.00	44.53
1	3121	Deepa Industries	235.00	7.83
1	3122	Proctor and Allan	282.00	9.40
1	3131	Kenya Wines Agencies	4,475.00	149.17
1	3131	Kenya Wines Agencies	7,118.00	237.27
1	3133	Kuguuru Foods	251.00	8.37
1	3133	Kenya Breweries	10,224.00	340.80
1	3134	Coca cola bottlers	7,118.00	237.27
2	3140	BAT	7,134.00	237.80
2	3231	Aziz Din Nabibux	115.00	3.83
2	3411	Silpack	168.00	5.60
2	3411	Tetra Pak	207.00	6.90
2	3420	Kenya Times Media	275.00	9.17
3	3700	Galsheet	1,675.00	55.83
3	3811	Metal Processing	288.00	9.60
3	3819	East A. Spectra	553.00	18.43
3	3830	Philips	2,391.00	79.70
3	3830	Battery Manufacture	211.00	7.03
3	3843	General Motors	2,500.00	83.33
3	3843	D.T. Dobie	471.00	15.70
3	3843	Simba Colt Motors	448.00	14.93
3	3843	CMC Hughes	497.00	16.57
4	3211	E.A. Fire Spinners	2,145.00	71.50
4	3211	Sunflag Dyeing	885.00	29.50
4	3511	Hoechst	3,842.00	128.07
4	3511	East African Oxygen	1,610.00	53.67
4	3514	Johnson Wax	391.00	13.03
4	3521	Crown Berger Paints	718.00	23.93

Class No.	ISIC No.	Name of Industry	Water Consumption/Month in m^3/month	Water Consumption in m^3/day
4	3522	Kenya Vet Vaccines	1,076.00	35.87
4	3522	Sterling Health	605.00	20.17
4	3522	Phoulene	9.00	0.30
4	3523	Colgate Palmolive	1,445.00	48.17
4	3523	Cussons (K) Ltd	4,449.00	148.30
4	3523	Elephant Soap	170.00	5.67
4	3523	Reckit and Coleman	321.00	10.70
4	3529	Kiwi Brand	326.00	10.87
4	3529	A Battery Spillage drain	211.00	7.03
4	3529	Associated Battery	211.00	7.03
4	3529	Henkel (K) Ltd	6,831.00	227.70
4	3550	Firestone (Sewer)	135.00	4.50
4	3550	Plastic and Rubber	1,207.00	40.23
4	3691	Mareba Enterprises	388.00	12.93

Source: - Service Demand Forecast of Nairobi and surrounding urban areas of Ngong, Kiserian, Ongata Rongai, Kikuyu, Mavoko & Ruiru by Athi Water Services Board dated December, 2006

APPENDIX 3 LIST OF REFERENCE

LIST OF REFERENCE USED

1. Benchmarking: Leveraging best Practice Strategies an American Productivity and Quality Center's (APQC) white paper for Senior Management based on the Internationally acclaimed Study Organizing and Benchmarking dated 1995
2. Benchmarking -Uncovering Best Practices and Learning from others (An overview of what Benchmarking is, code of conduct types, sources of information and setting up a process -From BRP(On line learning center from http://wwe.isixsigma.com/.
3. BPR Tutorial Series, BPR online learning center downloaded from bpr@prosci.com
4. National Environmental Sanitation and Hygiene Policy, Ministry of Health, Kenya dated July 2007.
5. The Sanitation challenge; Turning Commitment into Reality, W.H.O dated 2004
6. Appropriate Technology for Water Supply and Sanitation – A planners Guide by The World Bank dated December, 1980
7. Making Rural Water Supply sustainable by UNDP and World Bank, 1997
8. Guideline for Low Cost Sewerage Systems in Developing Countries by Infrastructure Development Institute-Japan dated August,2004
9. Participatory Hygiene and Sanitation Transformation by WHO and UNDP dated 2000
10. Urban Sanitation -A guide to Strategic Planning by Kevin Tayler, Jonathan Parkinson and Jeremy Colin dated 2003
11. Participatory Hygiene and Sanitation -Report on the prospective Review of Participatory Methods with a focus on PHAST, Kenya by UNDP/WHO
12. The Design of Shallow Sewer Systems by UNCHS Habitat, 1986
13. Simplified Sewerage Handbook
14. W.H.O Report No. 9
15. Service Demand Forecast of Nairobi and surrounding urban areas of Ngong, Kiserian, Ongata Rongai,Kikuyu, Mavoko & Ruiru by Uniconsult, Kenya dated December, 2006
16. Seminar on Sewage Treatment by University of Nairobi dated 16th November, 1973
17. A guide to the Development of On Site Sanitation, WHO, 1992
18. Japanese Effluent Standards
19. National Environment Agency.USA, 2002
20. Sewerage Manual of City of Portland, USA by Bureau of Environmental Services
21. Sewerage Manual for City of Los Angele, USA by Department of Public Works
22. Gravity Sanitary Sewer Design and Construction by American Society of Civil Engineers, 1979
23. Ministry of Water Development Rainfall Frequency Atlas of Kenya dated January, 1978
24. UPVC pipes KS 06-149 Metric series
25. Steel Pipes to ISO 65
26. Kenya Bureau of Standards (KEBS) Catalogue of Kenya Standards, 2005
27. Manual for Operation and Maintenance of Sewage Works by Public Works Department, Ministry of Interior, Government of Thailand dated, 1994
28. Working with Sewerage, HSE priced publications by Health and Safety Executive publication
29. Safe Work in confined spaces by HSE priced publications by Health and Safety Executive publication
30. Supply of sewer maintenance and laboratory equipment for Kisumu Sewerage Rehabilitation project by Runji JV Wanjohi Consultants dated February, 2001
31. Kapsabet Sewerage Project, Sewer Maintenance and Operation Manual by African Consulting Engineers, 2003
32. Ministry of Water and Irrigation Water Practice Manual, 2005
33. EIA Report of Further Development of Tseung Kwan O, China
34. Constructed Wetland Wikipedia, the Free Encyclopedia
35. Constructed Wetland Use in Non point Source Control by BMP, September, 1997
36. Waste Stabilization Ponds and Constructed Wetlands Design Manual by S. Kayombo, T.S.A. Mbwette, J.H.Y Katima, Prospective College of Engineering and Technology, University of Dar es salaam in conjuction with N. Ladergaard, S.E. Jorgensen, Danish University of Pharmaceutical Sciences section of Environmental Chemistry Copenhagen Denmark, 1998

37. Geotechnics of Waste Stabilization Ponds :An important piece of the wastewater treatment puzzle by Francisco silva-Tulla, ICF Consulting Senior Vice President Raul Flores Berrones, Mexican Institute of Water Tenchology, Research Engineering dated May, 2005
38. Design and Performance of Waste Stabilization Ponds by Hamzen Ramadan and Victor M. Ponce dated Friday, April, 2008
39. Sewage Lagoons -Waste Water Treatment Facility -Karst , Minnesota, USA -Journal Article dated 28th June, 1993
40. Soil as a Wastewater Treatment System; historical development by V.E Tzanakakis N.V. paranychianakis, National agricultural Research Foundation, Greece.
41. Waste Stabilization Ponds -A design Manual for Eastern Africa DD Mara, GP Alabaster, HW Pearson and SW mills in association with Lagoon Technology International, Leeds, England financed by Overseass Development Administration, 1992
42. Aftare Care Study, 1998
43. Design of Constructed Wetland for Tertiary Industrial Effluent Treatment Pilot Study at Pan African Paper Mills Ltd, Webuye -Final Design Report by Gath Consulting Engineers dated April, 2001
44. Consultancy Services for Design of a Constructed Wetland for Integrated Tertiary Municipal Effluent Treatment Pilot Project : Kisumu Town by BKS Engineering in Association with Capa Consult, June, 2001
45. Mombasa Wastewater Sea Outfall -Model Verification by WL/Delft Hydraulics, June, 1998.
46. Jurisdiction : City of Ithaca, Advert for recruitment of CAD and CADD Technician dated 2nd July, 2007.
47. Technical Document on WUASP Experiences by Ingrid De Loof in collaboration with Eng. C.K. Koske dated November, 2005
48. PC based Simplified Sewer Design by DFID dated January, 2001
49. The Environmental Management and Coordination (Water and Quality Regulations), 2006
50. The Environmental Management and Coordination Act, 1999
51. The Environmental (Impact Assessment and Audit) Regulations, 2003
52. The Environmental Management and Coordination (Water Quality) Regulation, 2006
53. Environmental Assessment Source Book, Wolrd Bank Technical Paper, 140 dated 1991.
54. Water Supply and Sanitation Project Preparation Handbook, World Bank Paper Number 12 dated 1983