A REVIEW OF PLANNING AND DESIGN OF SANITARY SERVICES IN NIGERIAN BUILDINGS

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ABSTRACT

A review of the procedures involved in the planning and design of the sanitary services installations which are commonly encountered in building projects, namely water supply; wastewater drainage, treatment and disposal; and rainwater drainage, is presented.

Recommendations are made towards improving the planning and design of these services in the peculiar circumstances of Nigeria; with a view to minimising the common problems associated with poor planning and design. Such problems include incessant interruption of water supplies, over-flowing of septic tanks and soakaway pits, and flooding of building premises during rainfall.

Some of the recommendations, among several others, are that (i) all water supplies from public mains should utilise a boosting arrangement (ii) all private borehole water supplies intended for potable use should undergo chemical and bacterial analysis (iii) the design of wastewater collection, treatment and disposal systems should take account of the low level of sanitation literacy prevalent in Nigeria.

1 INTRODUCTION

A major contributor to our poor sanitary conditions is the deplorable state of our sanitary services installations. The problems associated with these installations had been identified (1, 2). When the mains water supply, for instance, is interrupted for long hours, and sometimes for days, people usually spend useful time in search of water that would otherwise be used for other businesses. A lot of manhours are thereby wasted.

In the process of the search for water, road accidents and other mishaps occur, causing injuries and death to children and adults alike. Water scarcity also compels people to make do with polluted water from streams, ponds, wells, etc, with attendant health hazards. People go without washing their bodies and clothings for long

intervals. Toilets, sanitary appliances and utensils are left uncleaned for long intervals at times because of water scarcity.

The wastewater drainage, treatment and disposal systems equally pose their own problems. Several unhygienic methods are applied in wastewater disposal, such as the pit latrine, bucket latrine and direct defecation into useful water streams and land. Many of the systems are also poorly designed, constructed, operated and maintained.

Furthermore, our sanitary conditions are worsened by the existing poor rainwater drainage systems. Flooding of building premises is a very common occurrence. also, when ponding of a roof or ground surface occurs as a result of poor drainage the surface eventually loses its strength and texture.
These poor sanitary conditions are caused by poor design, construction and operation of sanitary installations in buildings.

The focus in this paper is the proper design of the very common sanitary installations, namely: water supply; waste drainage, treatment and disposal; and rainwater drainage.
Useful suggestions which had been applied beneficially in actual practice, in the peculiar situation of developing countries, are made.

2 WATER SUPPLY

2.1 Common Water Sources

Water supply to our buildings is usually from either of two sources: the town mains supply or a water borehole source provided within the premises.

A water borehole consists of a set of tubes sunk into the water bearing stratum of the ground, the lower end of the set of tubes being perforated in order to allow water to permeate into the tubes and be raised by a pump. Figure1 shows a sketch of a section through a typical water borehole.

Depending on the composition of the contents of the particular underground stratum from which it is obtained, water may appear clear, odourless and tasteless and be acceptable for some non-potable uses, such as fire fighting, without under-going chemical and bacterial analysis, as well as purification. However, all borehole supplies intended for potable use should undergo such tests and purification (3) that ensure compliance with recognised standards of freedom from harmful bacteria and toxic substances; whose presence in a water source may, or may not, impart colour, odour or taste to it.

The International Standards for Drinking Water produced by the World Health Organisation defines the limits for substances frequently found in water (4). Table 1 shows some of these limits. Other regulatory bodies such as the Environmental Protection Agency of the United States also specify their own limits.

Water from the town mains is supplied through pipes laid underground and running by streets, the mains being fed from the Water Authority's water station which comprises boreholes, filtration units, chemical treatment units, water pumps and storage tanks. Connections to individual premises are normally done under the permission and supervision of the Authority.

Due to the erratic nature of the mains supply system in the developing countries, many housing developers, now favour the private borehole supply, unlike the situation in developed countries where the city mains are favoured due to their reliable nature. It is therefore suggested that all our mains supplies in Nigeria be boosted by means of a low level break pressure tank and surface pump, and distribution effected from a high level storage by gravity. A simplified sketch of this arrangement is shown in Figure 2.

In the case of private borehole supply there is usually the need for water storage, and hence boosting, for the following reason.

(a) to allow for periods of interruption of supply due to pump failure, system maintenance, etc.
(b) to allow rest periods for the borehole pump
(c) to allow for recovery of the well after drawdown
(d) a low level storage would necessarily need to be boosted before distribution in order to achieve satisfactory system pressure.

2.2 WATER DISTRIBUTION

There are thus two common distribution arrangements in borehole supply schemes. One is the directly boosted system in which the borehole submersible pump discharges directly into a high level tank (thereby dispensing with the break tank) from which distribution to the buldings and premises is effected by virtue of gravity. The other is the indirectly boosted system shown in Figures 2.

The use of a low level storage and break pressure tank in borehole water supply schemes is known, in general, to reduce the submersible pump discharge head and thereby increase the pumping-rate of the borehole pump (5). The cost of the borehole pump could thereby be smaller. However, in making a choice between the two distribution arrangements, considerations of cost on both sides have to be made.

For instance, the cost of installing the break tank, surface booster pump, piping and accessories has to be checked against the cost of locating all the required storage at a high level together with the associated cost of constructing a sitructurally stable tower to carry this torage.

2.3 Design Considerations for Water Distribution in Buildings Projects

Some important aspects of the distribution system which could jeopardize functionality if not properly designed had been identified (1), and they include the following: the estimation of water demand, storage tank capacities, and height of storage tank; as well as the rating of the booster pump and the layout planning and sizing of the distribution pipes. These aspects are discussed below.

2.3.1 Water Demand

Water storage requirements are usually stipulated to cover a specified period of interruption of supply, usually 24 hours. For supply to individual buildings the British Standard Code of Practice (CP) 310 (6), for instance, stipulates the demands per head listed in Table 2 based on the use of the building, and Table 3 based on type of sanitary appliance. The latter table may be preferred where the project is intended for a multi-purpose use. Also, in the United States the National Plumbing Code (7) stipulateds demands for different building uses and appliance types.

An alternative approach to the use of such code requirements is to use past metered water consumption data obtained on a capital basis for buildings of a similar pattern of use to the one under consideration. For supply to individual buildings and premises by connection from public mains, it is suggested that larger water storage capacities than the estimated daily demands be installed since public supplies in Nigeria are sometimes interrupted for longer periods than 24 hours. An additional 100 to 200% of the estimated daily demands may be adequate.

For supplies to groups of houses, settlements and towns average annual usage values are specified. A usual range for domestic purposes in the United States is 50 to 60 U.S. gal (189 to 227L) per person per day (8). 200L per day is commonly recommended for purposes of planning of town water supply projects (9). Additional factors of 200% to 250% are usually utilised in the Nigerian situation (5).

A usual way of estimating future water demand for planning purposes for any given locality is to draw a graph of past consumption data for at least twenty years and then extrapolate the mean straight line into the future (10).

To the demands estimated for domestic use, water requ irements for public, commercial, industrial and agricultural uses have to be added where such uses are envisaged and a common distribution system is planned for. The requirements for such other uses are, again, stipulated by different authorities. For instance, in the

United States, the National Board of Fire Underwriters (11) suggest the following formula for fire fighting requirements-

$$Q = 1020\sqrt{P}\,(1 - 0.01\sqrt{P})$$

where Q = fire demand in United States gallons per minute

and P = population served in thousands.

2.3.2 Storage Tank Capacities

The total water demand (domestic plus other uses) of buildings, may be combined in a common set of storage tanks or separated in different tanks, depending on the intended water distribution configuration and taking economy and functionality into consideration.

For the boosted system incorporating a low level storage, the total demand is an addition of the capacities of the break tank and the high level storage, taking away the remnant volume at the lowest possible water level in each tank. Where only a high level storage is utilised, its capacity constitutes the total demand. For the former arrangement, the ratio for dividing the total demand between the two tanks should consider such factors as the cost of constructing a suitable supporting tower for the high level tank and the available spaces for the tanks.

2.3.3 Height Of Storage Tank

The available water pressure in a distribution network is determined by the height of the high level storage, taking discount of the pressure losses along the pipework. These losses arise as a result of the frictional resistance of the pipework; the head loss due to pipe fittings, valves and accessory equipment; and the head to be overcome during any vertical rise of water in the pipework.

In determining the height of storage tank the guiding principle is to ensure that the appliance or terminal which is most likely to be starved of water has a reasonable positive pressure head. Once this is achieved all other points in the system would also have adequate pressure.

Several codes, such as the American National plumbing Code (7), stipulate minimum pressures at fixture supply pipe outlets and these should be used in Nigerian designs.

Table 4 shows recommended flow rates and pressures at some fixture outlets (7). The corresponding heights of storage tanks should be such that, with appropriate system configurations and designs of the fixture outlets as contributory factors, such pressures can be attained.

While aiming at achieving such minimum pressures it should be noted that an excessively high storage may require the installation of pressure - reducing devices at an additional cost within the distribution system being served, in order to achieve

satisfactory performance. It would also bring about higher costs of constructing the supporting tower and of installing a more expensive surface booster, or submersible, pump of higher rating.

2.3.4 Booster Pump Rating

The parameters needed for rating a booster pump are the total head to be overcome by the pump and its discharge rate; the total head being an addition of the height above the pump of the high level storage (the static discharge head), the frictional head loss, the head loss due to pipe fittings and valves, and the height of the pump above the low level storage (the suction lift). The pump rating procedure is illustrated in available literature (8, 12, 13). It involves generating a graph of total head versus discharge rate for the boosting system (i. e. the system head curve) and then superimposing this graph onto the characteristic curve of a particular pump as obtained from the manufacturer. The point of intersection represents the design total head and discharge rate. A pump which has a characteristic curve having a wide plateau at the peak efficiency is usually preferred.

Also, in order to avoid cavitation during pumping it must be ensured that a pump which has a Net Positive Suction Head (NPSH) less than calculated for the boosting system under consideration in selected. The calculation of NPSH is illustrated in available literature (8, 12, 13).

As an illustration, Figure 3 shows a system head curve superimposed onto the characteristic curves of a range of pumps of a particular make. The efficiency and NPSH curves of this range of pumps are also shown in the figure.

The peak efficiency occurs at a discharge rate of 4.0m^3/h while the system head curve intersects the characteristic curves of pump2 and pump3 at 3.8m^3/h and 4.7m^3/h respectively with corresponding heads of 19.5m and 25.0m respectively.

If pump2 is chosen, the corresponding NPSH is 0.7m which value needs to be compared with the system NPSH, as explained, in order to eliminate cavitation. The choice of any particular pump, in addition, takes into account the discharge rate (and hence the time taken to fill the tank) and the head developed by the pump.

In an earlier paper, the author had generated sets of system head versus discharge curves for water boosting to different elevations of high level storage using commonly utilised system parameters such as pipe size, Hazen - Williams Coefficient, total length of horizontal pipework, etc (14).
Such curves can be used repeatedly for systems whose parameters can be

approximated to those utilised in generating the curves. Such ready-made curves have been found to be invaluable tools in speeding up the pump selection process.

2.3.5 Layout Planning and Sizing of Distribution Pipe

The layout of the distribution pipes should be planned to achieve a simple and economical arrangement. For example, in a situation where a single horizontal pipe run can supply appliances located in adjacent rooms, it would be uneconomical to have separate horizontal runs to supply the different rooms. And where a single riser can conveniently supply several floors in a multi-storey building, it would be uneconomical to use individual risers to supply the different floors.

Pipes should also be properly sized and appropriate pipe fittings provided in the distribution network so as to achieve the intended flow rates and pressures.

Distribution pipes are sized by utilising the available pressure from the distribution source (which in the recommended boosted arrangement is the high level storage), after discounting losses along the pipework, and the required flows which are stipulated in design codes (6,7). Pipe sizing procedures are illustrated in the literature (15, 16, 17).

In order to facilitate pipe sizing which is usually lengthy and time-consuming when done manually for a complex distribution system several computer software packages are being marketed (18, 19). Indigenous efforts, which should be patronised, are also being made in developing such software programmes. One of such efforts is that reported by Nwoye (20).

3. WASTE DRAINAGE, TREATMENT AND DISPOSAL

3.1 Waste Drainage Design

In designing drainage system, the intention is to quickly and quietly convey the waste to a treatment unit, and subsequently, to a disposal systme. When the slope of the drainage pipework is favourable, flow is achieved by virtue of gravity. On the other hand, if the pipework has a higher elevation downstream than some other point in the system, then the use of a sewage lift pump may be inevitable.

In order to maintain system simplicity and ease of maintenance the use of pumps in the dainage system should be avoided at both the architectural and sanitary services design stages, especially in developing countries where several maintenance problems abound.

Furthermore, the system should be properly ventilated so as to maintain an equilibrium of air pressure within the system. It should also not constitute a health hazard by generating a foul odour or by way of leakages.
The system should therefore be water-tight and trap seals for preventing air flow from the sewer back into the building should be installed at appropriate points on the drain lines of all sanitary appliances.

The internal (within the building) drainage system design entails specifying the types, sizes and locations of all waste pipes, vent pipes, trap seals and other system components, as well as determining the appropriate stack pipe system; there being two basic types of stack pipe system in common use within the building: the one-pipe and two-pipe systems. In the one-pipe system, all appliances are connected directly to a common discharge stack; whereas in the two-pipe system, discharges from soil appliances such as water closets and urinals are conveyed by means of a soil stack while a separate stack conveys the discharges from wash basins, baths and other culinary and ablutionary appliances. There are, however, modifications to these two basic stack pipe system.

A common approach to drainage pipe sizing within the building is to assign discharge units to each appliance and by aggregating, the total discharge units for a particular pipe section can ,be obtained. The procedure is illustrated in available literature (15, 16). By means of the discharge units and pipe gradient, the waste flow rate and drainage pipe size can be obtained from design tables and graphs..
The design for the drainage below ground involves planning the pipe layout, sizing the pipe system using the same principles as used in sizing the piping within the building, and specifying the depths of pipework and manholes in conformity with specified slopes. Also, pipe fittings needed for easing the sewage flow would be specified.

Sewage drainage system design is usually guided by codes of practice, such as CP 301 (21), CP 304 (22), and CP 305 (23), which should be utilised. Computer softwares are available for facilitating drainage design (24) and should be utilised.

3.2. Sewage Treatment and Disposal

Methods of domestic sewage treatment and disposal include the very common septic tank and soakway pit system and the very common septic tank and soakway pit system and the central sewage treatment plant. There are also the more crude methods such as the pit latrine and bucket latrine, that are prevalent in the remote village settlements (25) but which should not be recommended for our modern building projects. We shall consider the design of the two commonly utilised systems in urban settlements, namely, the septic tank and soakaway pit system and the central treatment plant.

3.2.1 The Septic Tank and Soakaway Pit system

This is the most common sewage treatment and disposal system in the urban settlements of developing countries.

In this system, the domestic waste is conveyed through the drainage network into the septic tank which is an underground covered tank (Figure 4). Heavy solids settle at the bottom of the tank and form a sludge, a fraction of which is digested by anaerobic bacterial action (26, 27, 28).

Lighter solids and grease which result from this action, float on the surface and form a scum. The liquid which remains after settling out of the sludge constitutes the effluent which is piped away for disposal, a build-up of sludge occurs in the tank. This is usually removed by a tanker vehicle at recommended time intervals.

Several authorities recommend equations for calculating capacities of septic tanks based on the number of full-time occupants of the building (upon which the sewage load depands) (29, 30). For instance, the British Standard Code of Pratice (30) recommends, for six-monthly desludging, the equation

$$C = 136.35N + 1818$$

where C = Septic tank capacity in litres

and N = number of persons served.

For part-time occupancy, for instance for premises occupied only during daytime, proportionate allowances are made on the number of full-time occupants. Thus for an office building occupied for 10 hours daily by 100 persons, the proportionate number of persons to be applied in the above equation is $\frac{42 \text{ (i.e10 x 100)}}{24}$.

There are also codes and design guides that recommend rules for determining the dimensions of the septic tank (i.e its lenght, width, depth, etc) (15, 31).

A comparison had been made of the several recommended equations for sizing septic tanks in consideration of their suitability for developing countries like Nigeria, in view of the more prevalent abuse of sewage treatment works than in the developed nations (32, 33). For instance, all sorts of indigestible matter are drained into the tanks and desludging of the tanks is not usually done regularly and punctually. A set of design equations had thereby been recommended for use in developing countries (32, 33), namely

$$C = \frac{1}{1000}(250N + 2000), \text{ in } m^3$$

Tank lenght, $L = 3\sqrt{6c}$, in m

Tank width, $W = L/3$, " "

Depth of liquid in the tank, $D = L/2$, " "

Effluent flows from the septic tank to the soakaway pit and soaks into the surrounding

subsoil under and around the pit.
The pit is usually lined with hollow concrete blocks to maintain its sides.
The blocks are laid in normal pattern and on their sides in alternate layers as shown in figure 5; an arrangement which provides adequte perforations that facilitate the soaking of the effluent into the soil. The pit's immediate surrounding is usually filled with hardcore, like broken stones, in order that the surrounding soil does not collapse through the perforations and enter the pit.

The pit capacity depends on the seepage qualities of the soil, and so site seepage tests as elaborated in available literature (31, 34, 35) should be conducted to determine percolation rates before sizing pits. From the percolation rate, a pit capacity which can take the daily effluent volume can be calculated, this colume being dependent on the daily water demand (2). Typical demand values are as listed in Table 2 (6).

The dimensioning of the soakaway pit, in relation to its capacity, takes into consideration such factors as compactness of construction, size of available land space and minimum depth of the pit.

In a bid to facilitate the design process computer programs had been written by the author to generate tables of capacities and dimensions (36, 37). Thus, by simply entering such tables using a specified number of building occupants or users of the system the required capacities and dimensions can be obtained and utilised in construction.

3.2.2 The Central Sewage Treatment Plant

This is a more hygienic and efficient treatment method than the septic tank. Whereas a maximum percentage digestion (in terms of reduction in Biochemical Oxygen Demand) in the region of 33% has been reported in the septic tank (38), values as high as 90% have been recorded for some central sewage treatment methods (39).

Central treatment plants can treat sewage from several buildings, and then dis charge treated effluent into a nearby watercourse or land drainage network, the effluent being of sufficiently high quality as to be free from risk of causing pollution. The common pollutants in domestic in domestic wastewaters include biodegrad able organics, suspended material, pathogenic micro-organisms, ammonia, and phosphates. The processes usually employed to remove the biodegradable organisms and suspended material which together form more than 95% by weight (27) of all pollutants found in typical domestic waste wates are illustrated in Figure 6. During the bioconversion process, the biodegradable organics are assimilated

by micro-organisms and partly converted to biomass and partly utilised for energy in metabolism; the production of biomass being more favoured in the presence of molecular oxygen than in anaerobic conditions.

As the biodegradable organics subsequently deplete in quantity, the micro-organisms turn to utilising their own biomass to derive the maintenance energy for living. This results in the decay of biomass and is referred to as endogenous respiration.

The biomass when suspended in the wastewater is referred to as activated sludge due to its highly adsorptive (active) nature towards suspended solids, resulting in the particulates formed during the bioconversion step together with the inert particles in the influent being incorporated into a mass of activated sludge floc by biflocculation.

Thus the most important step in wastewater treatment systems with respect to overall effluent properties is that of solids. separation. Solids separated from wastewaters are mostly organic in nature, the organic portion consisting of biomass and inert volatile material contained in the influent. As these organics are decomposable, and are therefore a potential nuisance to the environment, they need to be stabilised (or decomposed) in a controlled manner beyond a potential nuisance point. Solids stabilisation can be achieved by either aerobic or anaerobic processes.

In waste-water processing where phosphate or nitrogen removal (or the attainment of greater removal of organic material than is possible through bioconversion and solids separation) is an objective, effluent polishing (40, 41) is usually applied as a final step.

In addition to the processes outlined in Figure 6, pretreatment of influent by filtration using screens, and the chlorination of liquid effluent prior to discharge to a receiveing watercourse are normally incorporated in waste-water treatment systems.

The final domestic effluent is expected to meet specified limits in the concentration of the several pollutants. In this regard the establishment of the Federal Environmental Protection Agency (FEPA) by the Government is most welcome. FEPA (42) has stipulated limits for pollutants in effluents to be discharged into receiving waters as well as those for land application. Some of these limits are shown in Table 5.

Several designs of central wastewater treatment systems are in use; the more common of these being the activated sludge treatment, trickling filter treatment, facultative lagoon, and the aerated lagoon systems.

The characteristics and design principle of each system are illustrated in available literature (27, 38, 39). The factors which influence the choice of any particular system include the quality of the resulting effluent in relation to stipulated standards; reliability requirements such as system resistance to shock loads of organic and toxic materials, sensitivity to intermittent operations, operator skills and maintenance requirements; as well as cost considerations such as land requirements, capital cost, and operation and maintenance costs.

The system design of central wastewater treatment plants require the determination of the volume of flow on a capital basis, and the sizing of the component units of the plant in line with this flow, among other technical specifications.
The commonly applied rules in determining wastewater loads for central wastewater treatment plants are the formulae (27)

$$Q = 3.8 \times 10^{-3}\, p\left(50 + \frac{P}{200}\right) \quad \text{for } P \leq 10{,}000$$

where Q = average daily wastewater flow, m^3/day
P = population served
and $Q = 0.38P$ for $> 10{,}000$

Where the effluent disposal for the central plant is by way of a land drainage network the ability of the soil to absorb effluent must first be determined by a seepage test (31, 34, 35) to enable proper sizing of the disposal trench or beds to be done.

Basically, the tests involve determining the time take for water to percolate down a given depth in a hole of standard dimensions on the site under consideration.
The percolation rate may thus be expressed as V_p in S/mm.

For the disposal of a daily effluent Q m^3, the required land area, A m2 can be calculated from the relationship (43).

$$Q = \frac{24 \times 60 \times 60\ A}{1000\ V_p} = \frac{86.4\ A}{V_p}$$

$$\text{or} \quad A = \frac{Q\ V_p}{86.4} \quad m^2$$

3.2.3 Choosing Between a Septic tank and a central Treatment Plant

The situations which favour the use of a central plant over the septic tank include the following:

a Where an ordered common drainage network leading to a treatment plant can

be planned for several buildings within a settlement, village or town.

b Where the saw requires a high quality of treated effluent to be disposed. In the Nigerian situation, the guidelines set up by the FEPA (42) should be adhered to.

c Where the sewage load to be treated is too large as to make the construction of a single septic tank unviable in comparison with the installation of a single central plant.

d Where a suitable watercourse or land mass is available for the discharge of a large amount of treated effluent.

e Where a continuous electricity supply can be installed on site to power the plant.

On the other hand, the septic tank finds favour in isolated buildings and groups of buildings as it is easy to construct and simple to operate and maintain.

4 RAINWATER DRAINAGE

The design of rainwater drainage for building projects aims at achieving the convenient removal of rain under storm conditions from roofs of buildings, and from ground surfaces. The design condition for a given project is, therefore, that of maximum rainfall intensity accompanied by wind.

A common approach to estimating rainfall intensity is by means of the formula (15).

$$R = 150\ T^{-0.625}$$

where R = rainfall intensity in mm/h

and T = duration of storm in minutes

Other approaches have also been developed and are being utilised (44,45,46). In particular, it has been observed by Bilham (47) that rainfall intensities of 75mm/h occur once in four years for a duration of five minutes. This value is commonly used as a safe design rainfall rate to avoid flooding in buildings and premises in the United Kingdom.

Taking the case of Nigeria where the heaviest rains occur in the warm humid climatic zone (which covers Port Harcourt, Calabar, Warri, Benin and Lagos) with a recorded total yearly rainfall as high as 2788mm (48, 49) we observe that the daily maximum rainfall that ever occurs throughout the year rarely exceeds 130mm at any location (50) while the measured six-hourly maximum value rarely exceeds 60mm (51).

Obviously, therefore, the maximum hourly intensity of rainfall is less than 60mm at any location in Nigeria.
The rate of 75mm/h utilised in the United Kingdom thus provides an ample safety margin when applied for the Nigerian situation.

We shall consider the drainage of the two commonly encountered types of roof, namely the pitched roof and the flat roof, as well as the drainage of ground surfaces.

4.1 Pitched Roof

This type of roof, shown in figure 7, is usually covered with roofing sheets made of zinc, aluminium, asbestos cement or concrete. The troughs of the corrugations convey the rain to the roof's edge. In situations where rainfall from the edge of the roof is not onto an adjacent roof, wall or other impeding structure, it is usually convenient to discharge rain directly onto ground. Where, however, such an impediment exists rain from the roof's edge needs to be directed to fall at convenient points by means of eaves gutters and downpipes. Hpped ends of roofs are also drained in a similar manner (Figure 8)

Eaves gutters, usually made of the same types of material as used in making roofing sheets, may be segmental, halfround or O.G. (ogee) in section as shown in figure 9.

For simulteneously draining the pitched roofs of two adjacent buildings valley gutters (Figure 10) may be used.

The run-off from a pitched roof due to wind-blown rain is usually greater than it would be were the rain falling vertically without wind. This situation is illustrated in figure 11. A usual way of accounting for the effect of wind is to assume rainfall to be driven at an angle of two units vertically to one unit horizontally (52).
Thus, if the projected horizontal area of roof per metre run of gutter is X m^2 and the vertical elevational area of roof per metre run of gutter is Y m^2, then wind-blown rain is assumed to fall at an angle which establishes an effective area of roof of (x + y/2)m^2 per metre run of gutter.

It is worthy to note, in making the foregoing assumption, that the predominant direction and maximum speed of wind in a given locality are not of relevance. It is the slope of the roof that is of interest.

To facilitate the sizing of gutters and downpipes, graphs and tables giving effective areas to be drained for various gutter types and rainfall intensities are provided (7, 52) based on hydraulics principles.

4.2 Flat Roofs

Flat roofs, which are usually constructed in concrete or other materials are usually gently sloped towards gutters or roof outlets. A typical fall may be in the neighbourhood of 1 in 100. Outlets may be set in the roof itself or in a gutter into which the roof drains. Figure 12 illustrates a roof outlet.

Roof gutters and downpipes are sized in the same manner as for pitched roofs, excepting that the projected horizontal area is the effective area to be drained in this case.

For vertical walls extending above a flat roof and draining onto the roof (Figure 13), one half of the wall area is usually taken as the effective wall area to be drained (7).

Tables and graphs provide gutter and downpipe sizes for various gutter types and slopes , and rainfall intensities.

Table 6 (16) and Appendix 1 (15) illustrate these.

4.3 Ground surface Drainage

In designing surface water drainage, it is necessary to first determine the amount of rain run-off from the surface.

The equation

$$Q = \frac{APR}{60 \times 1000}$$

where Q = run-off in m^3 /min

A = area drained in m^2

P = run-off coefficient which is the percentage of imperviousness of the surface.

R = rainfall intensity in mm/h

is used to calcualte the rain run-off from a given area.

Run-off coefficients commonly used for some surface types are the following (15).

Roofs	=	0.95
Paved Roads and surfaces	=	0.75 - 0.90
Paths	=	0.50 - 0.75
Parks and Gardens	=	0.10

From these figures, we observe that, next to roofs, paved roads and surfaces produce the largest run-off for a given surface area and rainfally intensity. The amounts of run-off from gardens and other unpaved surfaces are usually very small in comparison.

In order to ease the flow from paved surfaces, various gradients are recommended for design (52). For paved access roads 1 in 20 to 1 in 30 is recommended for the longitudinal gradient while the camber or crossfall is 1 in 40. The minimum gradient

Central sewage treatment plants should be specified and designed following the guidelines set out;

g site seepage tests should be conducted to determine percolation rates to be used for design for effluent disposal wherever this method of disposal is chosen;

h relevant codes for engineering practice, as well as design guides elaborated in literature should be utilised in the design of sanitary services for our building projects;

i available design aids, such as water distribution pipe sizing computer software, computer - generated tables for septic tank and soakaway pit capacities and dimensions, etc, should also be utilised in facilitating the design of sanitary services; and

j building and planning approval authorities should ensure that all sanitary services designs are carried out by qualified engineers. The engineers should also monitor the execution of the sanitary installations to ensure conformity with approved designs. Furthermore, the authorities should formulate maintenance monitoring plans to ensure that building owners and occupants carry out prescribed periodic maintenance on sanitary installations.

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recommended for other paved surface is 1 in 60.

Paved surfaces are sometimes drained to ditches which are interconnected by underground piping or covered channels as shown in Figure 14. The design of drainage channels and pipes is based on hydraulics principles as elaborated in literature (953, 54, 55). Also tables and graphs of flow rate against drainage pipe size for various drainage gradients are available to facilitate surface water drainage pipe sizing.

As an illustration we utilise the chart in Appendix 2 (15) to determine the size of pipe which, when laid to a fall of 1 in 100; can drain a paved area of 1,000 m^2 at a maximum possible rainfall intensity of 75 mm/h.

The run-off coefficient is assumed to be 0.9

The run-off, $$Q = \frac{1{,}000 \times 0.9 \times 75}{60 \times 1{,}000} = 1.125\ m^3/min = 18.75\ L/S$$

By entering the chart horizontally at this flow rate and vertically at a gradient of 1 in 100 we obtain a 150mm diameter pipe, nearly full.

Rainwater drainage design is usually guided by codes of practice such as CP 308 (56). (56).

5 CONCLUSION AND RECOMMENDATIONS

The paper has highlighted the problems associated with poor sanitary services installations. It has also reviewed the planning and design procedures necessary for redressing the situation. The recommendations made towards improving the planning and design of the sanitary services for our buildings may be summarised as follows:

a all private borehole water supplies intended for potable use should undergo chemical and bacterial analysis to ensure compliance to recognised standards;

b all supplies from public mains should be boosted;

c care should be taken in the determination of storage tank capacities and height of high level tank tower during the design of water distribution systems. Equal attention should be given to the rating of the booster pump, and the layout planning and sizing of the distribution pipe system;

d the design of wastewater collection, treatment and disposal systems should take account of the low level of sanitation literacy prevalent in Nigeria;

e in wastewater drainage system design, simple gravity-flow system should be adopted;

f the criteria set out in the paper should guide the choice between the septic tank and the central sewage treatment plant for our building projects.

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TABLE 2: WATER DEMAND BASED ON USE OF BUILDING

Type of building	Storage in Litres	
Dwelling houses and flats	per resident	90
Hostels	per resident	90
Hotels	per resident	135
Offices without canteens	per head	35
Office with canteens	per head	45
Restaurants	per head per meal	8
Day schools	per head	27
Boarding schools	per resident	90
Nurses homes and medical quarters	per resident	110

* Source: British standard Code of Practice 310: Water Supply, 1965, London.

TABLE 3: WATER DEMAND BASED ON SANITARY APPLIANCE

Sanitary Appliance	Storage in Litres
Water closet	180
Sink	135-225
Wash basin	90-250
Shower	135-225
Urinal	135-250

*Source: British Standard Code of Practice 310: Water Supply, 1965, London.

TABLE 4 RECOMMENDED FIXTURE OUTLET FLOW RATES AND PRESSURES

Fixture Supply Outlet Serving	Flow rate		Fixture Pressure	
	(gpm)	(l/s)	(psi)	(m of Water)
Bathtub	4	(0.30)	8	5.62
Bidet	2	(0.15)	4	2.81
Shower	3	(0.23)	8	5.62
Urinal	15	(1.14)	15	10.54
Wash basin	3	(0.23)	8	5.62
Water closet with close coupled tank	3	(0.23)	8	5.62

*Source: National Plumbing Code Handbook by V. T. Manas, McGraw-Hill Book Co., New York, 1957.

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TABLE 1: INTERNATIONAL STANDARDS FOR DRINKING WATER

Substance or Characteristic	Highest Desirable Level	Maximum Permissible Level
Suspended matter (units of turbidity)	5	25
Total dissolved solids (mg/l	500	1500
Total hardness (mg/l $CaCo^3$)	100	500
P^H range	7.0 - 8.5	6.5 - 9.2
Cl^- (mg/l)	200	600
Ca (")	75	200
Cu (")	0.05	1.5
Fe (")	0.1	1.0
So_4^{2-} (")	200	400
Zn (")	5	15

* Source: World Health Organisation (cited in Water Supply by Twort, A. C., Hoather, R. C. and Law, E. M., Edward Arnold Ltd, London, 1982).

TABLE 5: EFFLUENT LIMITATION GUIDELINES IN NIGERIA (mg/l)

Parameter	Limit for Discharge into Surface Water	Limit for Land Application
Colour (Lovibond Units)	7	-
PH	6 - 9	6 - 9
BOD_5 at 20	30	50
Total suspended Solids	30	-
Total dissolved Solids	2000	2000
Chloride (as Cl^-)	600	600
Sulphate (as So_4^{2-})	500	1000
Sulphide (as S^{2-})	0.2	-
Cynaide (as CN^-)	0.1	-
Detergents (Linear alkylate sulphonate as Methylene blue active substance)	15	15
Oil and grease	10	20
Nitrate (as No_3^-)	20	-
Phosphate(as Po_4^{3-})	5	10
Calcium (as Ca^{2+})	200	-
Magnesium (as Mg^{2+})	200	-
Coliform (daily average)	400 MPN/100ml	500

*Source: Supplement to Federal Republic of Nigeria Official Gazette Extraordinary NO. 42 Vol. 78, 1991.

Dash (-) indicates that the particular parameter is of no relevance.

TABLE 6 : SIZE OF GUTTERS OF VARIOUS SLOPES FOR VARIOUS ROOF AREAS

Diameter of Semicircular Gutter +		Maximum Projected Roof Area for Gutters of Various Slopes							
		1/16 in. per ft (0.0052)		1.8 in. per ft (0.01042)		1/4 in. per ft (0.020083)		1/2 in. per ft (0.04167)	
in.	mm	Sq ft	m^2	Sq ft	m^2	Sq ft	m^2	Sq ft	m^2
3	75	170	16	240	22	340	32	480	45
4	100	360	33	510	48	720	67	1,020	95
5	125	625	58	880	82	1,250	116	1,770	164
6	150	960	89	1,360	126	1,920	178	2,770	257
7	175	1,380	128	1,950	181	2,760	256	3,900	362
8	200	1,990	185	2,800	260	3,980	370	5,600	520
10	250	3,600	334	5,100	474	7,200	669	10,000	929

+ Gutters other than semicircular may be used provided they have an equivalent cross-sectional area.

* Source : Standard Plumbing Engineering Design by L. C. Nielsen, McGraw - Hill Book Co., New York, 1982.

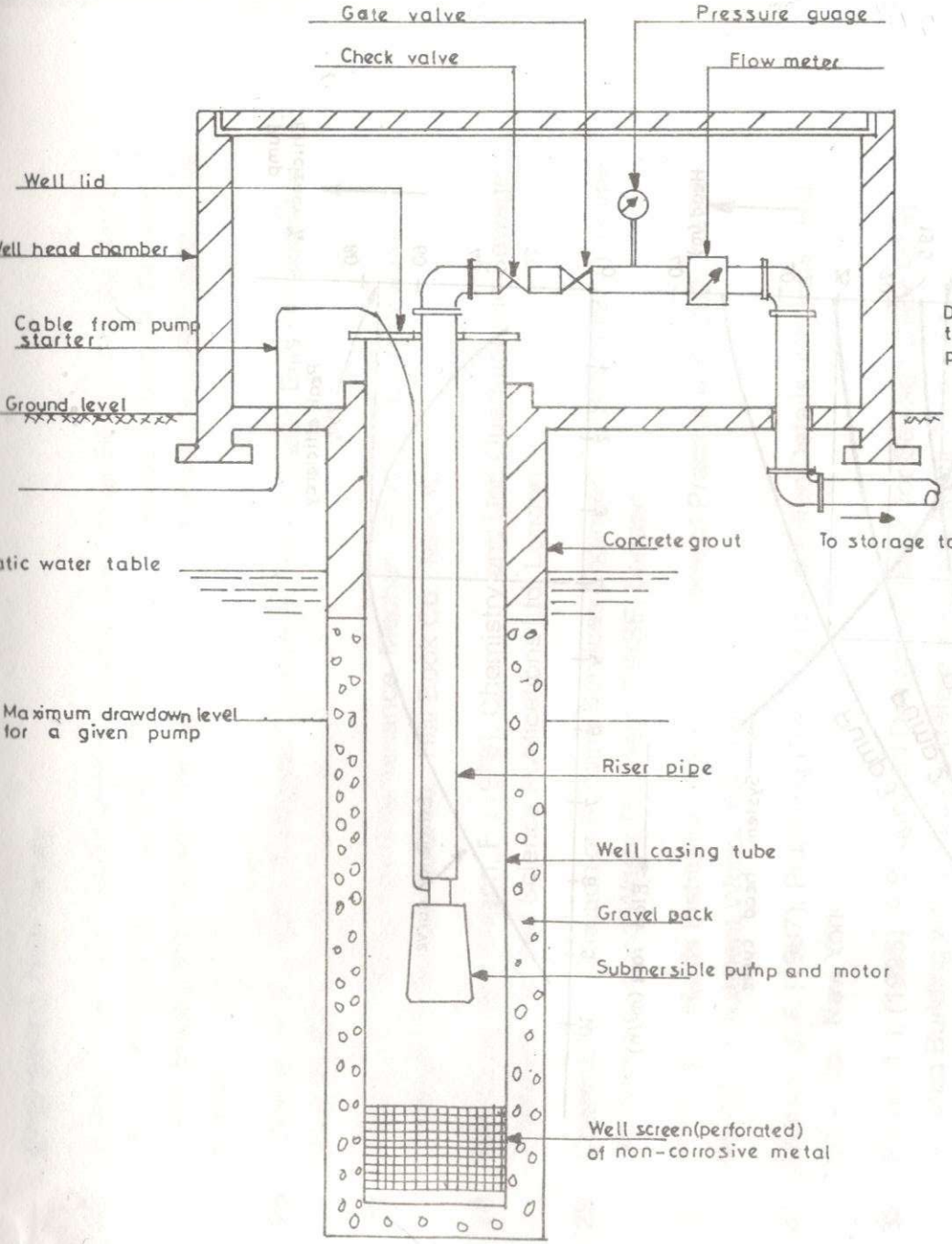


Figure 1: Section through water borehole (not to scale)

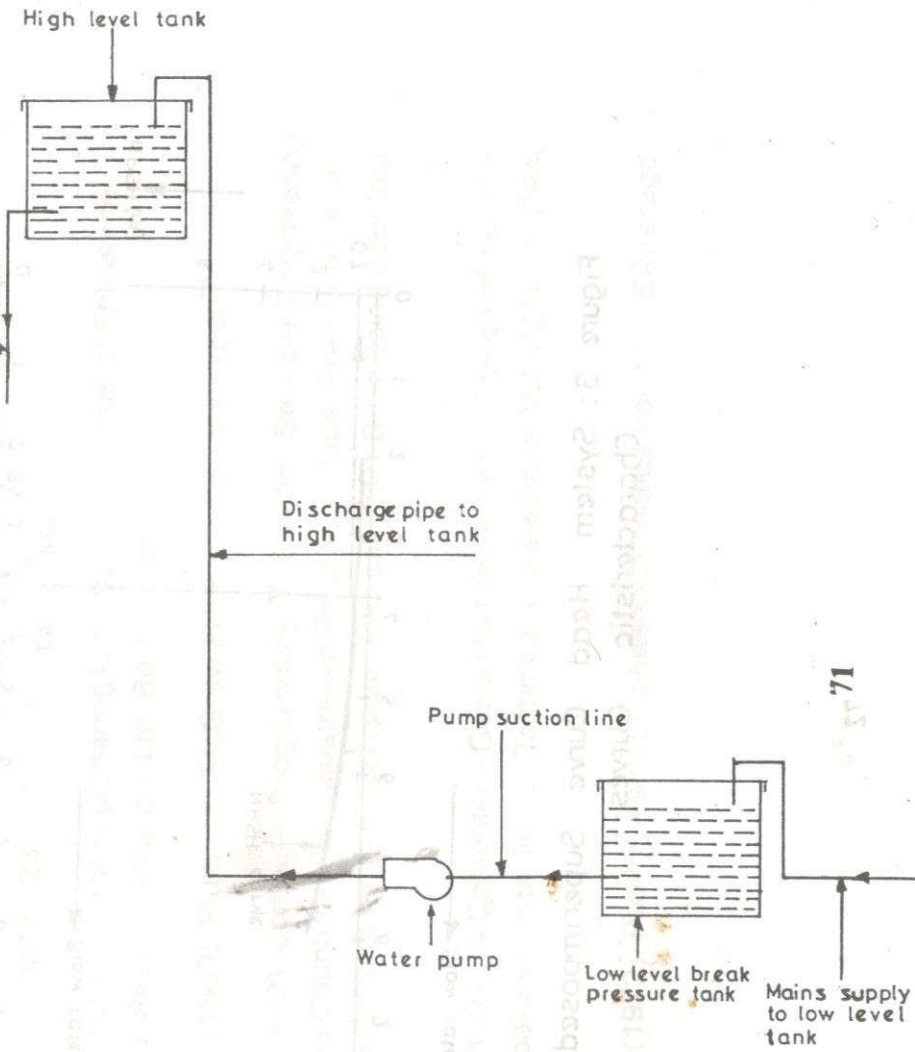


Figure 2: Sketch of boosted water distribution system (accesories and pipe fittings not shown)

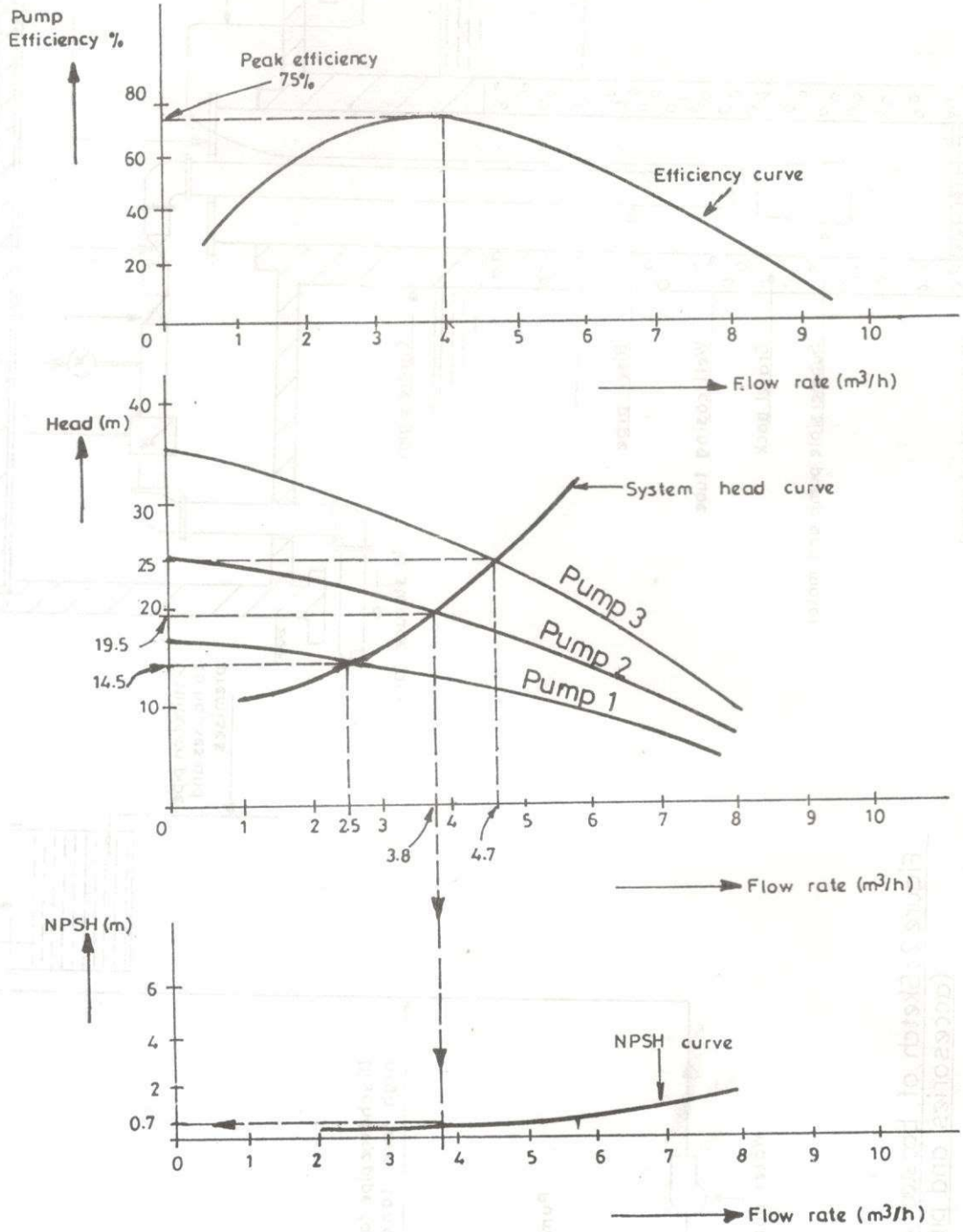
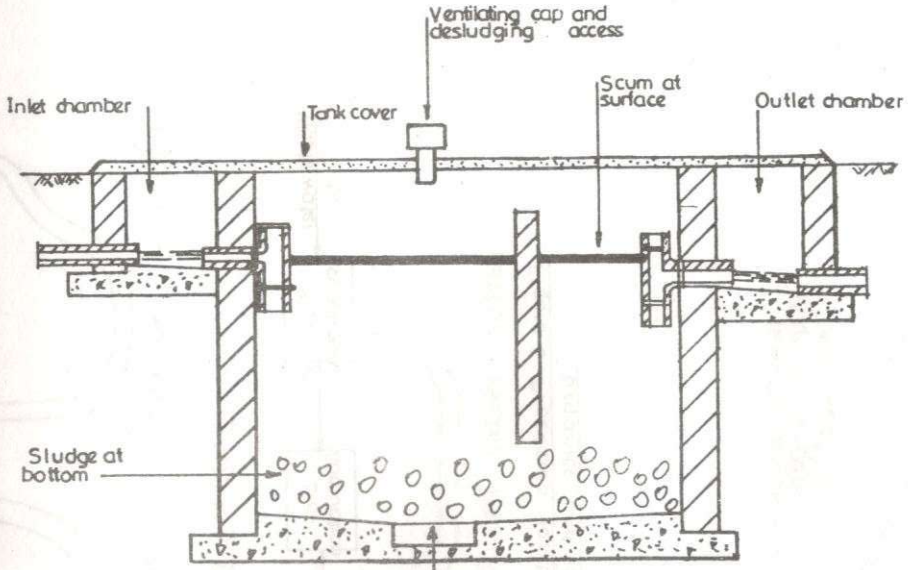
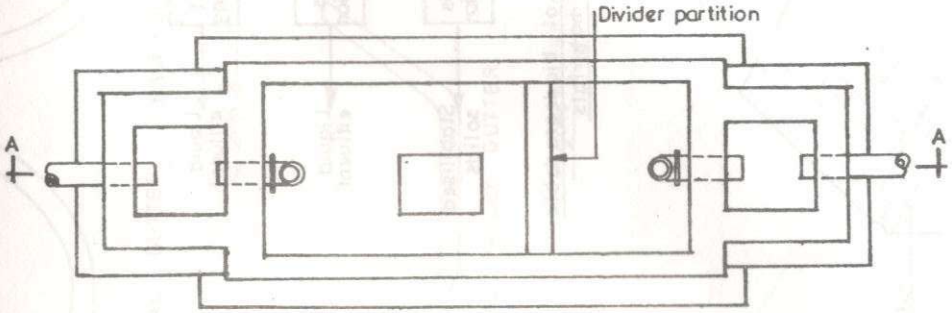


Figure 3: System Head Curve Superimposed onto Pump Characteristic Curves

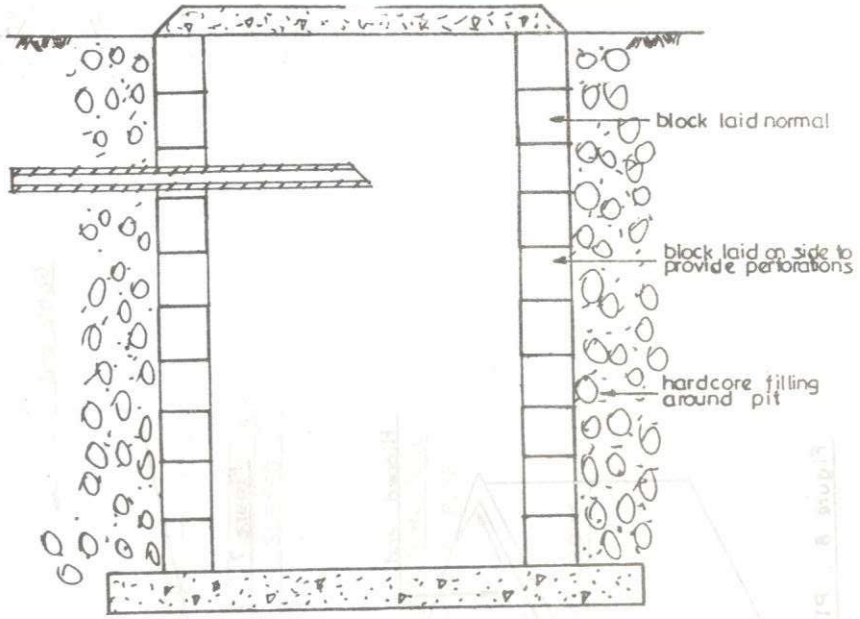


Section A-A

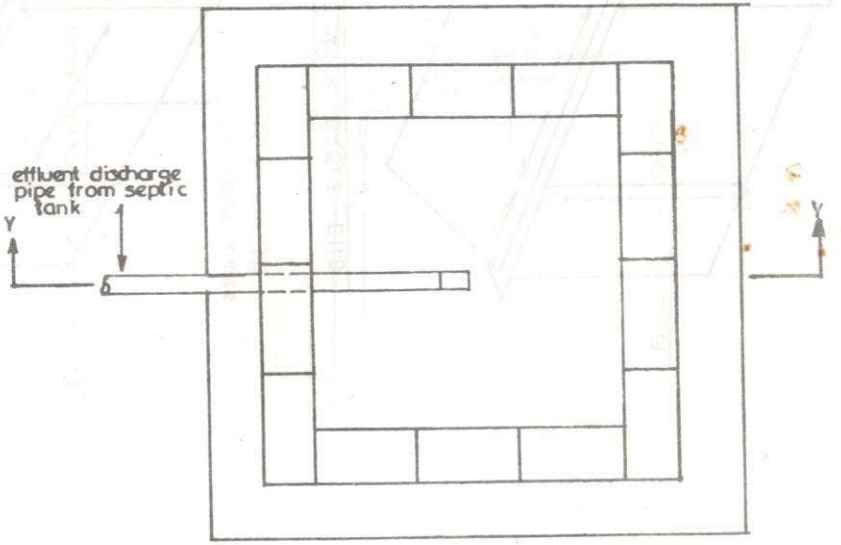


Plan View With Tank Cover Removed

Figure 4: A typical septic tank



Section Y-Y



Plan View With Pit Cover Removed

Figure 5: A typical soakaway pit

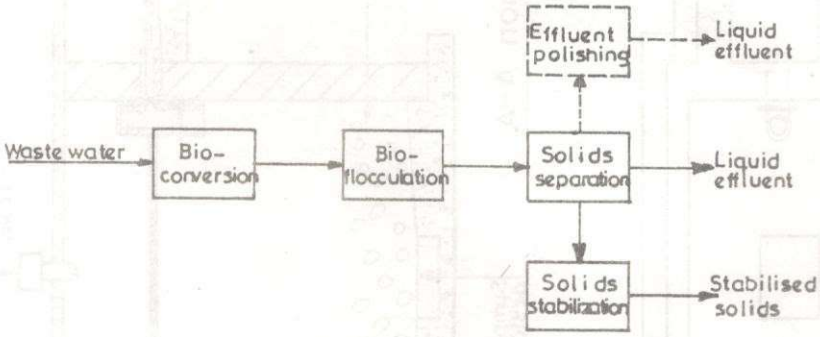


Figure 6 : Processes for the removal of biodegradable organics and suspended materials

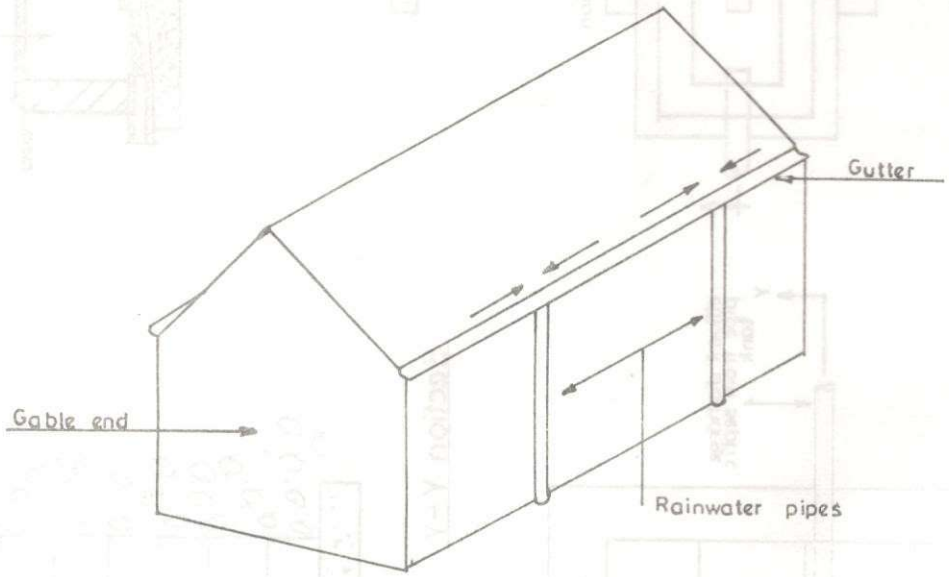


Figure 7 : PITCHED ROOF WITH GABLE END

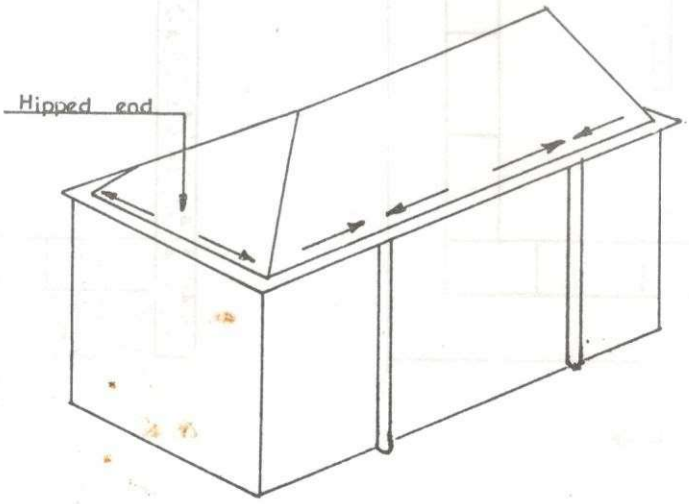


Figure 8 : PITCHED ROOF WITH HIPPED END

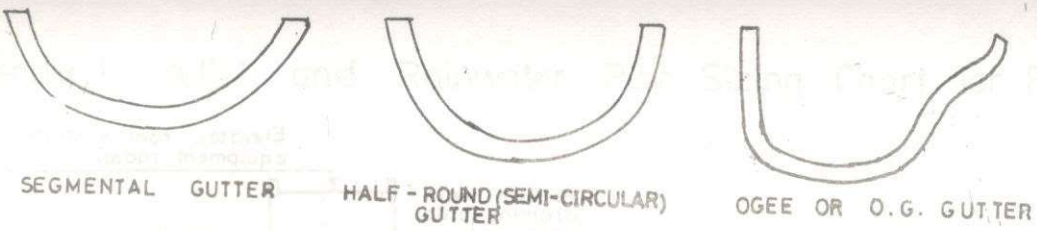


Figure 9 : EAVES GUTTER SECTIONS

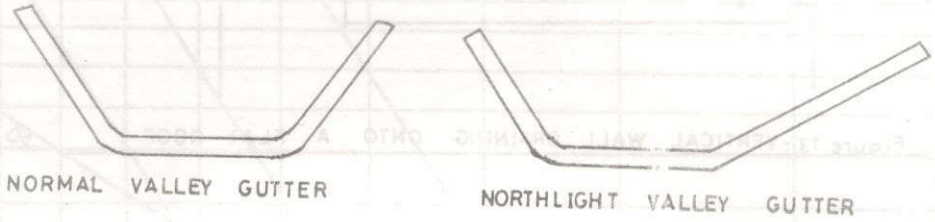


Figure 10: VALLEY GUTTERS

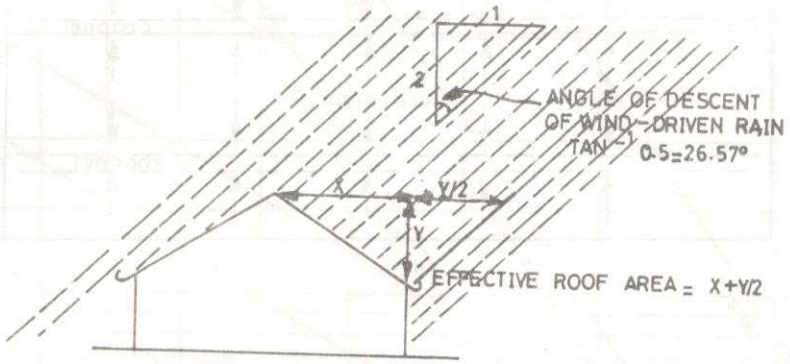


Figure 11: WIND-DRIVEN RAIN ON A PITCHED ROOF

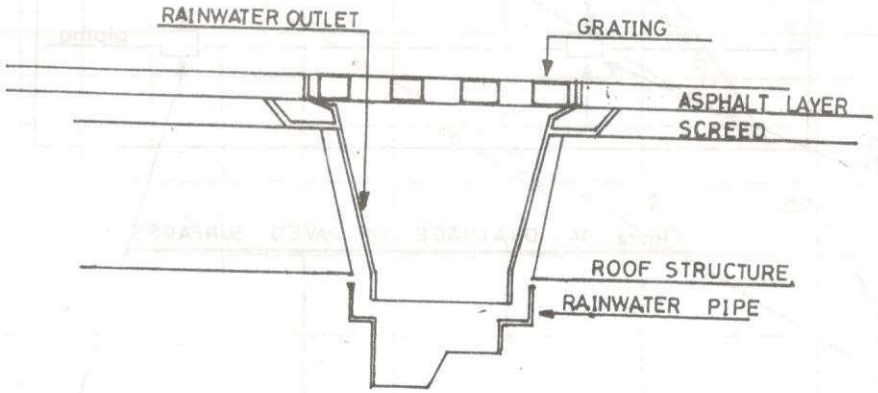


Figure 12: RAINWATER OUTLET SET IN FLAT ROOF

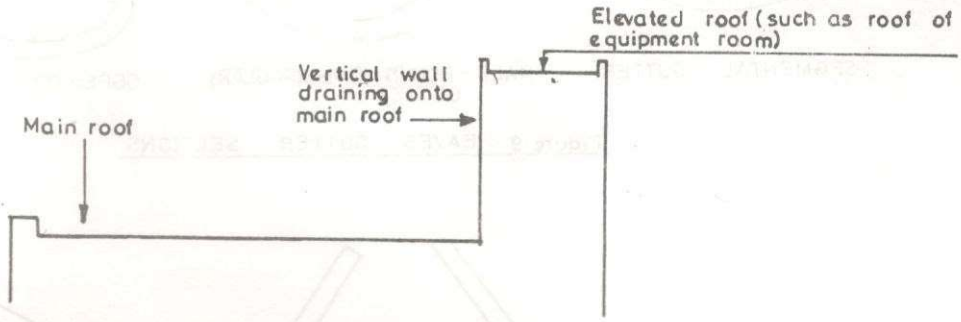
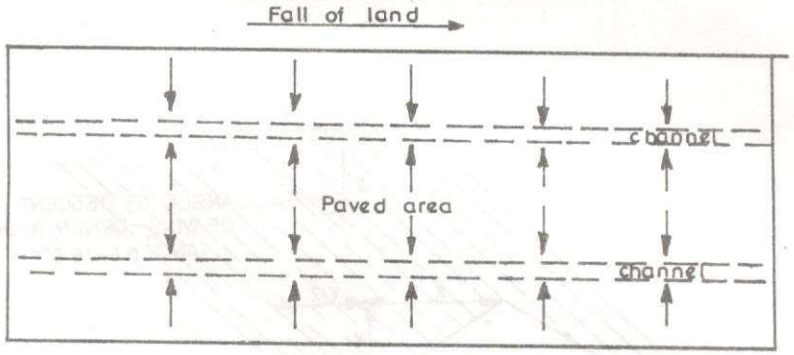


Figure 13 : VERTICAL WALL DRAINING ONTO A FLAT ROOF



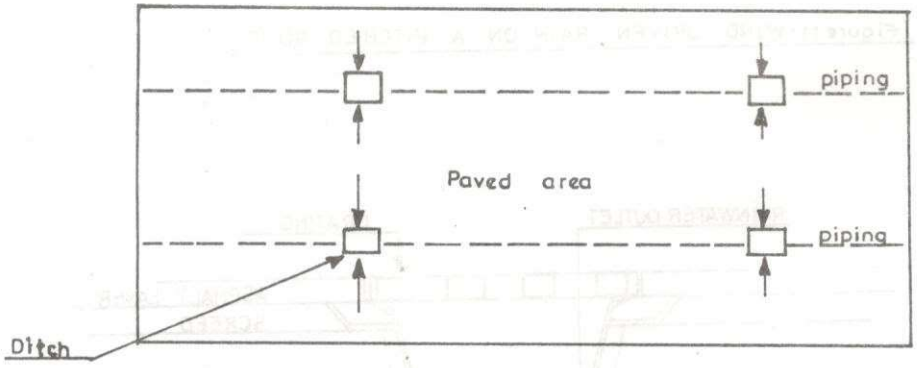
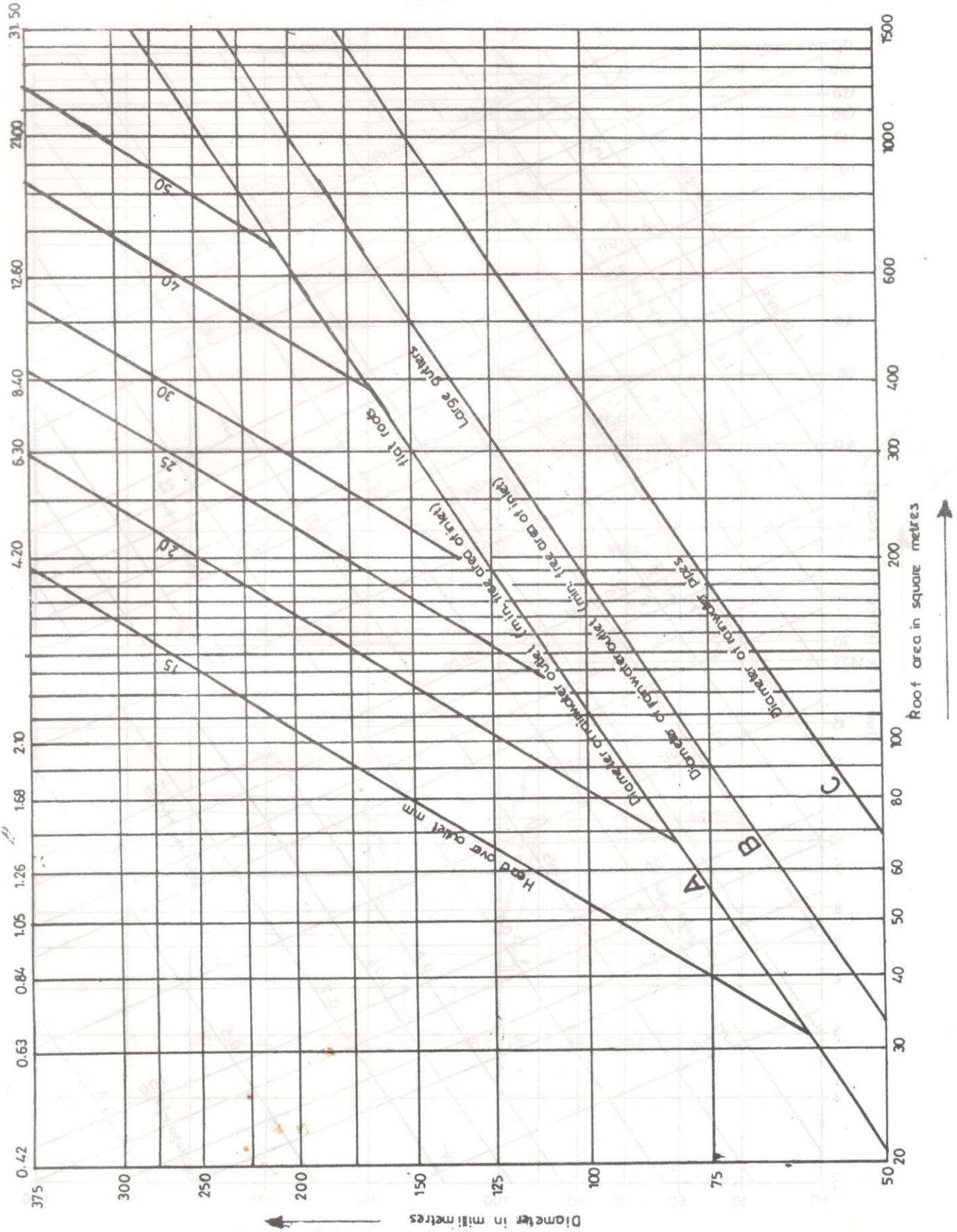


Figure 14 : DRAINAGE OF PAVED SURFACES

Appendix 1: Outlet and Rainwater Pipe Sizing Chart for Roof Drainage



Appendix 2 : Drain Sizing Chart for Ground Surfaces

