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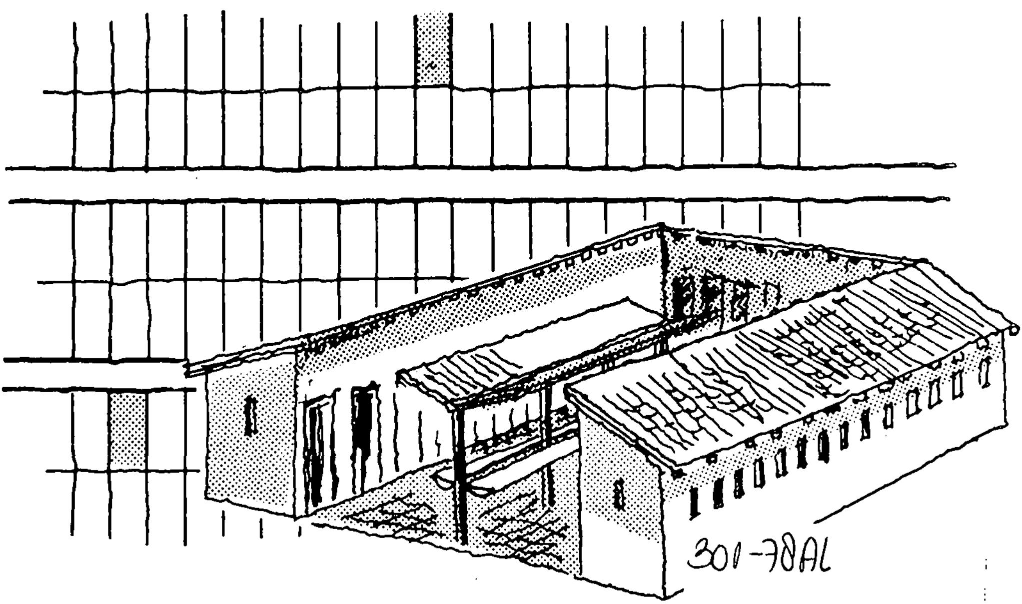

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NOTE

This is a second non-revised printing of the original text published first in March 1977.

A certain updating and definitely an extension of the sanitation concepts and the evaluation methodologies applied have been prepared for the London conference on Engineering, Science and Medicine in the Prevention of Tropical Water Related Desease, arranged by Royal Society of Tropical Medicine and Hygiene, International Association on Water Pollution Research, and Institution of Civil Engineers, 1-7 Great George Street, Westminster, London SW1, UK, 11 - 14 December, 1978.

Proceedings from this conference will be published by Pergamon Press, and the paper "Appraisal of four alternative excreta removal systems for urban areas in developing countries" by Hansen, Therkelsen and Hansen should be read together with the present book.

ACKNOWLEDGEMENT

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Poul Harremoës, Technical University of Denmark strongly advocated that a study on alternative sanitation systems be made. Encouraging - and sometimes discouraging - advice was received from many participants in the WHO/DANIDA training courses on costal pollution control held in Denmark since 1970. Tadayuki Moroshita, Ministry of the Environment, Japan, Oladende O. Oladapo, Lagos State, Nigeria and Kasemsan Suwarnarat, Department of Health, Thailand, all contributed to the work by arranging technical demonstrations on waste and excreta disposal in their respective countries. G. Bactimann, WHO, Geneva, Alexander Velderman, NEDECO-DHV, Holland and John Hebo Nielsen, COWICONSULT, Denmark were kind enough to critically examine the draft report. To all these persons the authors wish to express their sincere appreciation of constructive and most valuable guidance.

Last but not least Lis Asger Hansen organized and typed the inhomogeneous manuscript and Niels Gerhard Nielsen applied his artistic touch to finalize the document. The authors are greatful to these two most amiable and inspiring personalities.

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1. SUMMARY

The main objective for the present report is to investigate alternatives to the traditional Western World sewerage (flush toilet & piped network) for the removal of human wastes from high density, low-cost housing in developing countries.

In order to form a basis for the study it was decided to apply the analysis to a 3oo ha area (chapter 5) in the outskirts of Metropolitan Lagos, Nigeria. The geographical conditions include a flat slope of the area and a very high ground water table together with a warm humid climate. However, since such conditions are common in urban areas in developing countries the results herein have a wider applicability. The study discusses 6 alternative technical systems for the removal of sanitary wastewater, and potential combinations of these alternatives were not investigated since detailed contoured maps were not available. The results therefore constitute a comparative evaluation of the different systems for a certain area rather than the "best" overall solution.

The work has been carried out solely on the initiative of the authors, and the proposed systems do not constitute actual design proposals. The choice of study area was made only to achieve a realistic basis for the model calculations.

The following systems have been developed to the extent it was deemed necessary to establish reliable cost estimates:

- o Full (conventional) sewerage
- o Aqua Privy (with piped disposal of the liquid phase)
- o House Vault, Japanese Stool
- o House Vault, Chiang Mai
- o Ablution Block
- o Multrum

The six systems selected for evaluation are based only on existing technology to allow a comparative evaluation in quantitative terms. It must be born in mind, however, that the "Multrum" in the proposed version, cf. chapter 11, has not been proven technically feasible, under the said conditions, and consequently this system does not fully qualify for comparison with the other alternatives.

Cost in monetary terms is an important decision criterion when alternative sanitation systems are considered. The cost sensitivity of each system has been evaluated with respect to:

- o Population density: 125-2oo-4oo p/hectar
- o Interest rate : o - 5 - 15%/year

In the case of the conventional sewerage alternative the influence on the total cost of some main parameters have been identified and discussed separately:

- o Water consumption (4oo, 25o and 125 ℓ/p,d)
- o Slope of main sewers (15, 8, 4 and 2 pm)
- o General slope of area (15 .... 3 pm)
- o Removal & replacement of road pavement

Certain evaluation criteria other than cost, such as local employment and purchase of equipment from foreign countries have also been included in the discussion.

The results indicate that there is no generally applicable optimal solution. Rather the results describe the influence of the project cost as related to basic planning parameters such as population density, service level and available terms for capital loans.On this basis the optimal alternatives for certain conditions are specified.

It is finally concluded that high density urban development favours a piped waste water removal system, though not necessarily of the traditional Western type. The aqua privy and the ablution block seem promising alternatives, particularly where staged development is necessary, e.g. due to scarcity of water supply or capital or both.

2. INTRODUCTION, BACKGROUND

Since 197o the World Health Organization, WHO, has arranged annual training courses on coastal pollution control. The courses are sponsored by the Danish International Development Agency and take place in Denmark with participants from many countries, particularly from the developing areas of Africa, South America, and South East Asia. Repeatedly the participants in their factual statements have described cases with unsatisfactory waste removal conditions that are related to high density "shanty" town settlement - located adjacent to or within capitals or major coastal cities in their home countries.

In such shanty towns people live under extremely primitive conditions, where high population density, poor sanitation, and a warm humid climate form ideal conditions for outbreak of epidemics. For parasitic diseases an endemic situation does often exist.

The solutions proposed to alleviate this problem typically include the conventional sewerage, for example the western type flush toilet with a subterranean piped collection network. This system unfortunately has two prerequisites:

1) Water. Self cleaning of the pipes necessitates a high water consumption (app. 5o ℓ/p,d)

2) Money. Substantial investments are necessary to introduce sewerage, particularly in already inhabited areas.

Often neither one of these resources is in plentiful supply in developing countries, and there are several examples of extensive planning of water supply and sewerage programmes that have never been implemented, because of such lack of money and adequate water supply cf. the case of Lagos, GILBERT /1/.

Based on these considerations the present report will evaluate and compare technical alternatives for waste water removal from human dwellings in urban areas. With special regard to developing regions low-cost solutions will be emphasized, and high priority will be given to economical evaluations and cost sensitivities to technical modifications.

It could be added that the report is not inventive as to new technology for water supply and waste water removal. Rather an attempt is made to evaluate existing technology and examine if the planners have any significantly different alternatives to choose among.

3. URBAN DEVELOPMENT IN SANITATION SYSTEMS

History in itself is fascinating but additionally it may give perspective to new proposals; and often experience of the past is most useful when planning ahead.

Along this line the authors found it interesting to study the development from pit latrines to full sewerage in the city of Copenhagen, a development that took place over a few hundred years and to a certain degree involved almost any concept for excreta removal that is practised even now-a-days in different parts of the world depending on cultural traditions and stage of development.

The sanitary history of Copenhagen will show a rather accidental development and demonstrate a case where poor management rather than technical failure would cause unsatisfactory sanitary conditions to the citizen.

3.1 Copenhagen 1765 - 1975

The first installation for excreta disposal in Copenhagen was the hole in the ground. Some of the holes were emptied by the "nightman" and his crew upon request from the houseowner. Others were simply covered after being filled up, and a new hole was dug in the ground in the backyard. The type of superstructure, the privy, depended on the wealth of the houseowner and the imagination of the carpenter.

Later it became common to have the privy inside the house, e.g. under the stairway. This convenience of direct privy access had some drawbacks, e.g. risk of odeur inside the house depending on storage time and ventilation of the privy. Particular inconvenience was experienced when the nightman emptied the storage tank, cf. figure 3.1.1. Sandwiches, warm beer, and a snaps was offered in order to avoid spilling

Figure 3.1.1 Emptying underground tanks involved potential risks of spillages and a very real risk of odeur, particularly if the tank was situated inside the house and under the stairway as shown on the painting from HILDEN /2/

and to obtain a swift operation. The nightman had a well paid but lowly esteemed job.

In 1756 it was decreed in Copenhagen that ground tanks be made with brickwalls, and as of 1795 it was no longer permitted to build the privy and ground tank inside the house. In these years the city experienced a growth of population which the existing system was not built for, cf. figure 3.1.2.

Advanced methods of emptying the ground tank were introduced in Germany already before 188o. Figure 3.1.3 shows one of the early versions of the now-a-days very common vacuum trucks, which offer possibilities for excreta removal without any health hazard or great risk of contamination of

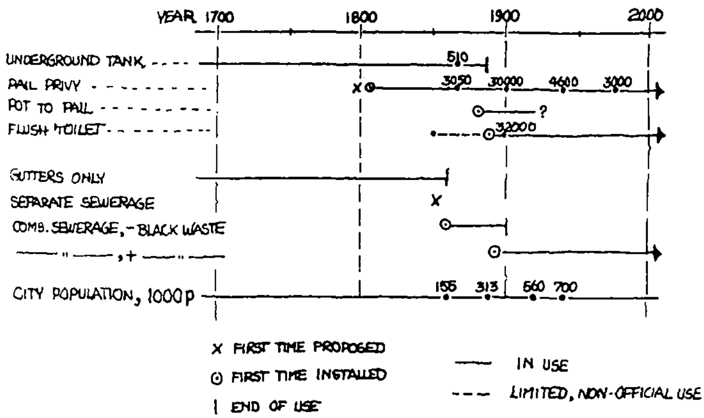


Figure 3.1.2 Sanitary development in Copenhagen, facts & dates

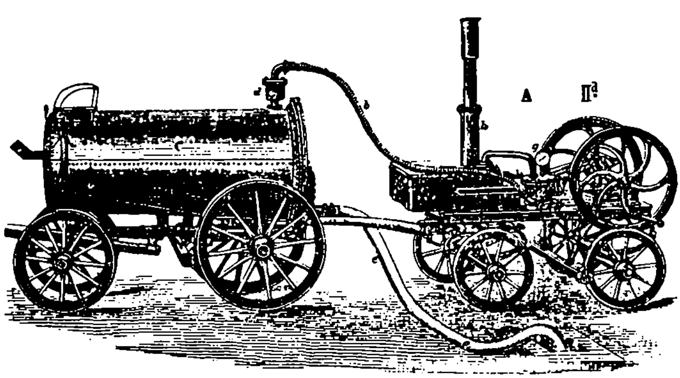


Figure 3.1.3 Vacuum trucks were available before 188o in Germany, operated either manually or automatically, cf. HEIDEN /3/

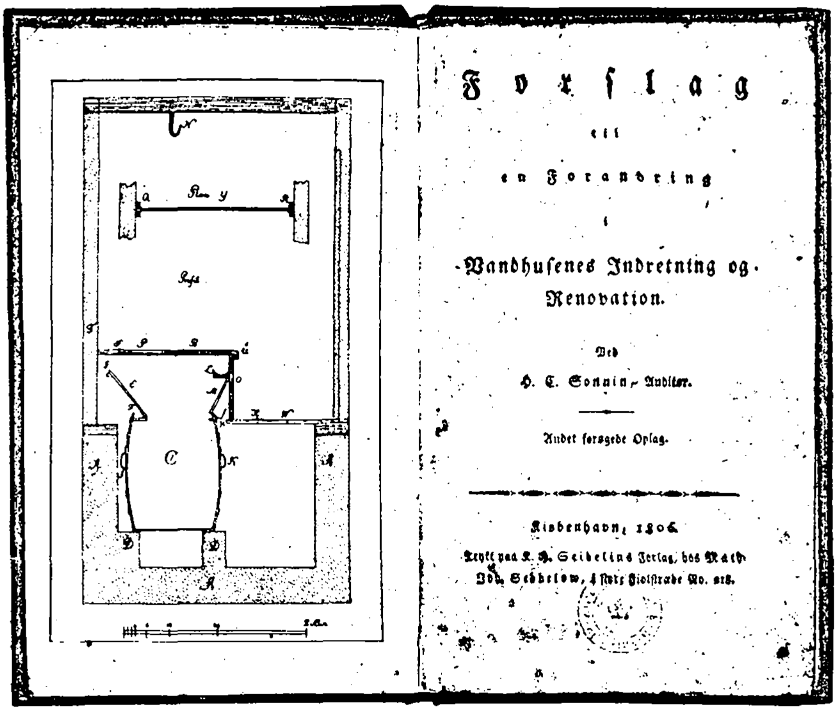


Figure 3.1.4 The pail privy was proposed in 1797 by H.C. Sonnin, Copenhagen, as an improvement to the ground tank. In the scale 1 alen = .63 meter

the environment during the operation. It is interesting that this concept of excreta removal is widely practised even in 1975 in Tokyo (more than 5o% of the population), whereas it never became a generally applied method of excreta removal in cities in Northern Europe. Actually, in Copenhagen the ground tank was practically outruled before the vacuum truck was technically available, cf. figure 3.1.2.

In 1797 SONNIN /4/ proposed a pail latrine, where storage of faeces would be in a portable pail instead of a permanent ground tank. Cf. figure 3.1.4. A refinement of the pail

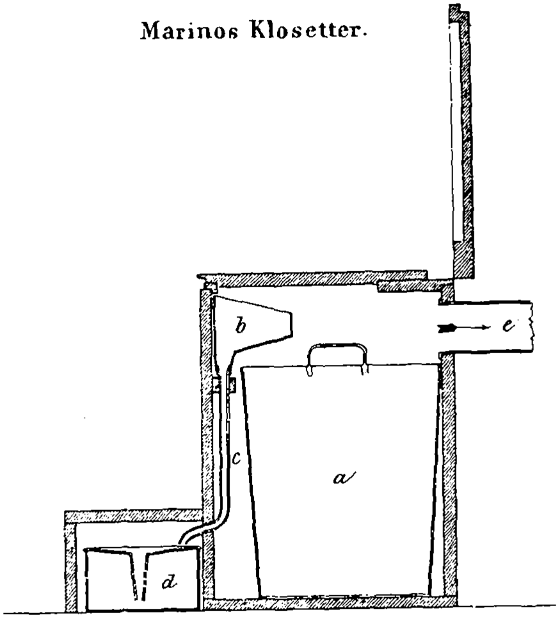


Figure 3.1.5 The Marino toilet was shown at the Health Conference 1858 in Copenhagen, HILDEN /3/

system as apposed to the ground tank was the separation of urine and faeces. The urine is proposed to be led to the yard. The pail would be emptied upon request from the house-owner.

Figure 3.1.5 shows the "Marino" toilet which was discussed at an international health conference in Copenhagen 1858. The "Marino" toilet was also available in an individual chamber version, cf. figure 3.1.6. Compared to Sonnin's proposal of 1797 the principal changes are very modest. The idea of separating urine and faeces was maintained, and it was proposed that this separation lengthens the intervals

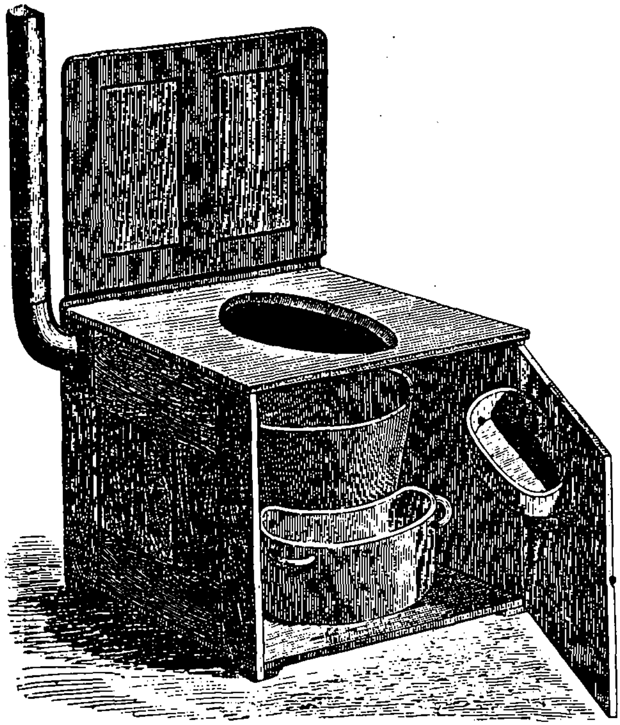


Figure 3.1.6 Chamber pot version of Marino's toilet, cf. figure 3.1.5

between emptying the pail. Further the decomposition of faeces would be slowed down and the odeur problem diminished in the absence of urine in the pail. Danish experiments in 185o showed that with separation of urine the fermentation of faeces is insignificant for several weeks, while fermentation is active already 12-18 hours after excretion, if urine be included, HYG.MED. /4/. A resolution from the above mentioned conference recommends that "excreta already separated by nature should be kept separate".

The proposed pail latrine was not readily introduced in practice, neither was the ground tank easily ruled out. In 1865 a survey showed 3o5o pail latrines and 51o ground tanks, cf. figure 3.1.2. From the same time another record shows - on the average - one stool per 4o-6o persons above age 1o years, HILDEN /2/. The last ground tank disappeared in 1885.

In the 188o's a new variation of the pail system emerged, a pot to pail procedure where the chamber pot was emptied daily into a storage pail, which would then be removed when filled. The advantage was the improved privacy and convenience of having the privy inside the dwelling without long storage. The disadvantage was that some pail privies now were emptied only rarely, and that some storage pails were overflowing due to the popularity of the chamber pot system, HILDEN /2/.

A sewerage system was first proposed in 1853 as a result of 6 years preparation. The proposal was rather revolutionary, consisting of two sewer lines, one for household waste water (grey waste) and storm water, and one for flush toilet sewage (black waste). Ultimately the black waste would be pumped into the deep waters of The Sound (between Sweden and Denmark), the storm water and the grey waste would be discharged into the harbour and the canals of Copenhagen.

1853 also became the year of the first cholera outbreak in Copenhagen, where 5ooo people died in three months. And in 1853 the proposal was turned down definitively! Sometimes development requires more than one disaster.

The pail system continued to be in use, cf. figure 3.1.2, and not until the turn of the century should the flush toilet and the combined sewerage become the dominating system for disposal of storm water, grey waste, and black waste. The change occurred during the 189o's, because the pail system failed in four respects, HILDEN /2/:

1. Insufficient cleaning of pails, so that risks of communicable diseases were enhanced, in particular because the empty pail would normally not be returned to the previous user.
2. Insufficient pail capacity with the result that chamber pot emptying caused overflow of the storage pail.
3. Improper decanting of storage pails within the city boundaries.
4. Ineffective ultimate disposal system. The pails were taken to depots within the city boundaries. Often the "depot" was simply the ground surface, where the night soil would accumulate until it was collected by farmers. The system remained poorly managed and the result was obnoxious accumulation of night soil within the city boundary.

The ultimate night soil disposal dilemma called for new solutions. In the 189o's the thinking centered around composting. But, since combined sewerage and flush toilets were now being introduced, the economic background for the composting system disappeared. Also, the slogan "tout a l'égout" was launched in the early 189o's and used by engineers as well as medical people. The number of pails peaked in the beginning of 19oo and then declined to 46oo prior to the Second World War.

3.2 Reversed Development

In Copenhagen the flush toilet was introduced only to become all-dominating in relatively short time. Possibly this is a fairly typical solution, at least for a majority of European and American cities. And quite obviously the flush toilet/sewage system has contributed significantly to the well-being of man in many urban communities, particularly in terms of:

a) less risk of transfer of pathogens when using the toilet.

b) less direct nuisance (odeur, sight) for the user of the toilet, for the person who handles the waste, and for other persons because the waste is piped underground out of the community instead of being transported above the surface within the city area.

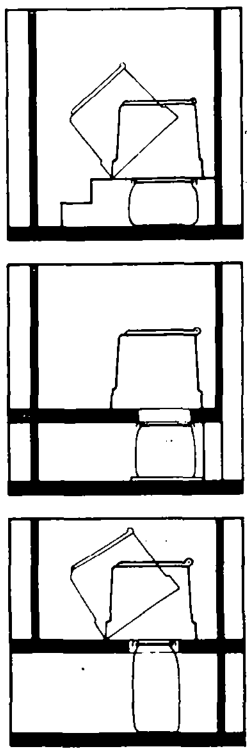
Some problems, though, are linked to the flush toilet/ sewage system. The water consumption is considerable and high quality drinking water is used only to convey faeces from the house to the treatment plant. Costs may be considered high, but in this respect the evaluation is very relative, in the industrialized and developed country the cost is relatively low; in the developing country the cost may be relatively high. The question of costs is dealt with in details in later chapters of this report.

Though very dominating the flush toilet is not all-exclusive in developed countries, for example the pail latrine is still in use in Copenhagen, cf. figure 3.1.2. Furthermore the number of pails is increasing, at least in Scandinavia, cf. figure 3.2.1; for a comprehensive list of alternatives see ORTEGA et al /5/. There seems to be a few typical reasons for seeking alternatives to the traditional flush toilet system even in highly industrialized countries:

1. An attempt to minimize water consumption, which may be in limited supply.
2. A considerable distance to an existing sewerage system, often the case for recreation houses in remote areas.
3. A need for provisional sanitation systems, e.g. in construction areas.

Items 1 and 2 have a certain bearing on conditions in developing countries. Firstly, the limited water supply may be very typical. Secondly, those who install an alternative system, e.g. in a recreation house in a remote area, are often people in a higher income class. This indicates very clearly that an alternative to the flush toilet system should not automatically be judged a "secondary solution".

PACTO



a)
INSTALLATION AS A COMPLETE UNIT ON TOP OF THE FLOOR

b)
INSTALLATION PARTLY UNDER THE FLOOR WITH ACCESS FROM OUTSIDE TO THE COLLECTION BAG

c)
INSTALLATION IN HOUSE WITH A BASEMENT

Figure 3.2.1 PACTO toilet (Sweden 1976), commercially available at 6oo $ as a complete "do it yourself" assembly kit. When comparing to figures 3.1.4 - 6 the question arises which one is more developed?

Such attitude has often been expressed in the context that already developed countries "should not export their secondary solutions to developing countries".

These considerations may justify a more careful examination of alternatives to the flush toilet system. They do not necessarily indicate that alternatives are preferable or feasible. It is the intention of this report to evaluate the feasibility of a few alternative sanitation systems for urban settlements in developing countries. And hopefully the findings will constitute factual information to those responsible for new proposals and decisions.

4. ALTERNATIVE WASTE DISPOSAL SCHEMES

4.1 Systems considered, systems excluded

Five different systems of excreta removal from human dwellings have been adopted for evaluation. Some of these systems are already widely used, e.g. the full severage and the house vault system; others have been proposed before but are only little used, e.g. the aqua privy system; and one system, the house bucket/ablution block system which is proposed in the present report, is a hybrid of different systems that are in use, but are not individually satisfactory solutions to urban development projects. The five systems involve 6 different technical approaches, which are briefly introduced below.

1° Full sewerage, FS

The full sewerage is identical to the waste water disposal system that is used conventionally at least in Europe and The United States. The system is included here because it is often referred to as the "only satisfactory long range solution", OLADAPO /6/. Further, the full sewerage will serve as a relevant reference when evaluating other schemes because of its wide application and generally high standard in terms of protection of human health.

Generally FS carries all liquid wastes including suspended and settleable solids. Consequently a relatively high flow is required to keep the system in operation without too frequent cloggings of sewers. A water consumption of app. 5o ℓ/p,d has been proposed, NEDECO /7/.

Details are found in chapter 7

2° Aqua privy, AP

The aqua privy system with piped liquid waste disposal has been proposed by VINCENT et al /8/ for low cost high density housing. Information on the system operation may be found in the same article, but generally the system does not have a widespread use and the information on experience is scarce.

AP will carry all liquid wastes, but settleable solids are precipitated in a subsurface holding tank on the plot before entering a pipe network that conveys the settled sewage to a treatment plant. Because of the tank settling pipes can be laid with very modest slopes withour causing clogging problems. Furthermore the minimum required pipediameter is smaller and there is no lower flow limit.

House vault and tanker truck

The house vault is widely used in Japan, e.g. more than 5o% of the Tokyo population was served by this system in the early 7o'ies, /9/. Even though the system is now systematically being replaced by the FS, the house vault will continue to be used for many years in Tokyo suburbs and other Japanese cities.

The subterranean house vault receives human excreta (faeces & urine) from a vertical duct coming from the stool in the in-house privy. No other wastes are introduced. The vault is emptied regularly (every 2 weeks) by a vacuum truck which transports the night soil to a treatment plant. Two versions of the house vault system will be considered here:

3° Japanese stool, JS

The system is briefly described above, and further details are given in chapter 9.

4° Chiang Mai Squatting Plate, CM

The CM system differs from the JS by a water seal between the privy and the vault to prevent possible odeur nuisance in the privy (a mini siphon is built into the concrete squatting slab). A small amount of water (.8 ℓ) is applied after each use of the privy. Therefore the vault volume for the CM system must be increased compared to the JS system. Further details are given in chapter 9.

5° House bucket and ablution block, AB

The house bucket system is still widely used, e.g. in Lagos/Nigeria, Accra/Ghana, Hong Kong and other places. Emptying necessitates public collection of nightsoil, e.g. every second day, but difficulties are numerous in the operation of this system, partly because it is considered low-status work to collect other people's latrine. Added to this social factor may be a religious consideration that one person ought not handle another persons excreta, ETHIOPIA /1o/. Dissatisfaction with the bucket/night-soil collection is reported e.g. in LAGOS /11/; but often the lack of satisfaction is also a reason for not reporting, and facts seem to be difficult to obtain on operation of the night soil system.

Public toilet blocks have been installed in several communities in Africa, e.g. in suburban areas of major cities like Tema in Ghana. The operation is often very dissatisfactory because the blocks are not sufficiently attended and users may by negligence or on purpose leave the block unattractive to subsequent users. Lack of use seems to be the rule rather than the exception with public toilet blocks built for community service in many African areas.

Based on such experiences it may seem optimistic to propose a system that combines the house bucket and the public toilet facility. Nevertheless, it is done on the premises that handling of the bucket be a business of the family and that the ablution block be sufficiently attended by public employees to maintain a satisfactory hygienic and aesthetical standard. The evaluation, chapters lo & 12, in terms of health, economy etc. will indicate whether this concept of waste disposal is promising compared to other alternatives.

6° Multrum, MR

The multrum is basically a compost tank where human excreta and possibly organic kitchen waste will decompose aerobically and provide a compost that could be disposed of on the plot. In this respect the multrum differs from all the other systems, where an off-the plot disposal is assumed.

Each plot has one privy placed on top of the compost container that is emptied from outside a few times a year. Further details are found in chapter 11.

Only the above six proposed schemes will be considered in the following. There may be other alternatives or variations that would qualify for further consideration, but the authors trust that six systems chosen provide a fair cross section of alternatives that are already wholely or partly in use, and which are often referred to in discussions on alternative sanitation systems in developing areas. The present report aims at providing further quantification and qualification of this discussion.

Many other systems are not included in the present evaluation, because they are deemed less acceptable in an urban high density development area with difficult ground conditions; examples are septic tanks and pit latrines. Such installations would typically need thorough consideration for more rural settlements or in urban areas with permeable soils, cf. the brief discussion in chapter 6 & 12.

4.2 Complete sanitation schemes

The evaluation of a sanitation system requires consideration of several stages of the water route such as:

1. Water withdrawal
2. Water treatment
3. Water supply
4. Water use } { Plot installations
5. Waste water removal
6. Waste water transport/collection
7. Waste water treatment
8. Waste water disposal
9. Sludge disposal

The analysis in chapters 7 through 11 of this report deals mainly with items 5 - 7, and mainly in technical and economical terms. A more complete evaluation is presented in chapter 12, and criteria for this evaluation are outlined in chapter 6.

5. STUDY AREA SELECTED

It was stated in chapter 2 that this study will discuss problems of disposal of sanitary waste in high density urban areas with difficult ground conditions. Since the Ebute Metta area in Lagos, the capital of Nigeria, fits this general description it has been selected as study area.

Lagos is a coastal city with extensive human settlements having only poor sanitary facilities. Certainly Lagos include also city areas with satisfactory and modern sanitation, but such facilities are not available to the majority of the population.

It seems typical for large cities in developing countries that these have a number of non-implemented master plans for water supply and sewerage and Lagos is no exception (7 comprehensive proposals in the period 1926 - 1965). Money was available for preparation of plans but little was spent on sanitary improvements!

The most recent master plan is that of GILBERT /1/ from 1966. With this report at hand, and based on personal experience of both authors, the city of Lagos was chosen as a model study area for a comparative evaluation of alternative sanitation system. The district of Ebute Metta within Lagos, cf. figure 5.1 & 5.2, would realistically meet the criteria with respect to climate, drainage, soil permeability, high ground water table etc. cf. subsequent chapter 6. By this choice the piped waste water systems will come out relatively expensive, because shoring of deep trenches will be necessary and construction more expensive. Consequently alternative sanitation systems, if they exist, would come out economically favourable in this particular comparison.

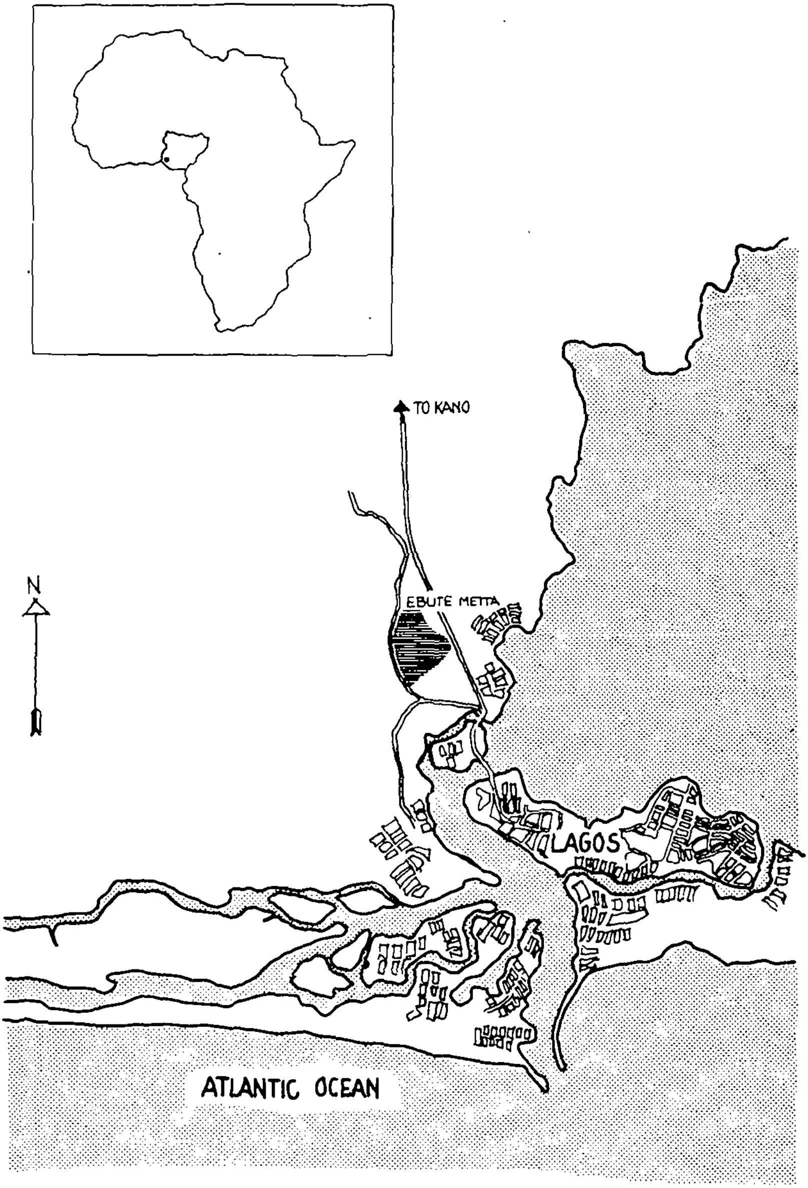
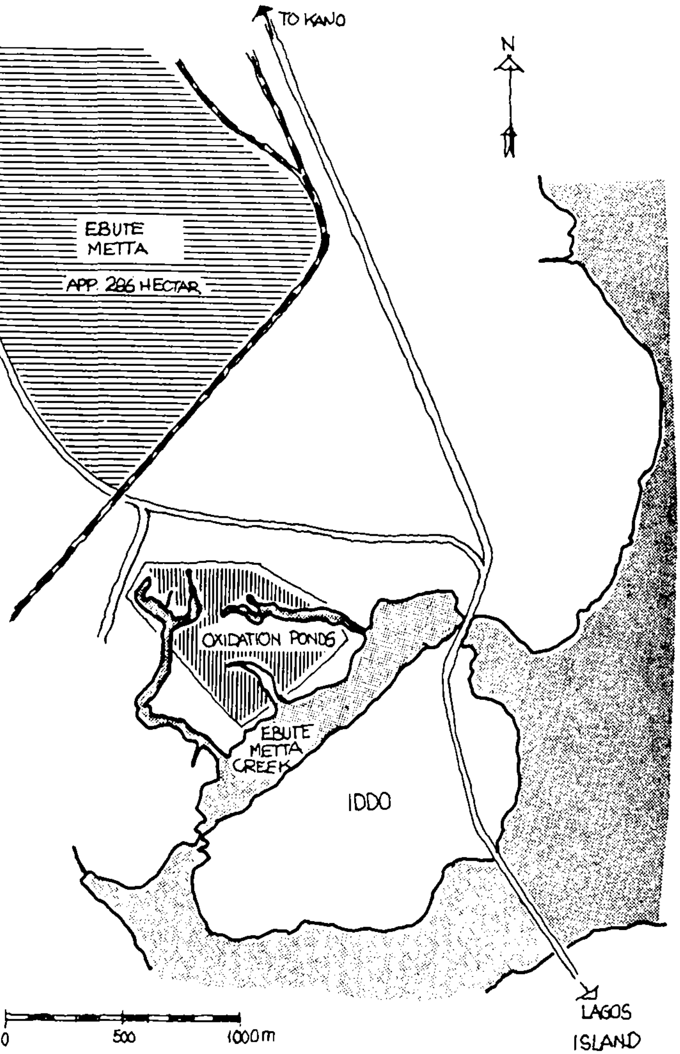


Figure 5.1 Study region chosen: Lagos, Nigeria



<u>Figure 5.2</u> Area selected for comparative evaluation of systems: Ebute Metta within metropolitan Lagos

Problems with control of communicable diseases are intensified in the warm and humid climate of Lagos; e.g. malaria is a permanent threat in the wet season. This therefore constitutes another reason to use the Ebute Metta as a model study area for alternative sanitation systems. The Ebute Metta area has a size of app. 286 hectars and would offer housing for only a limited number of people; in the present study between 36ooo and 114ooo depending on the population density, cf. chapter 6.

6. GENERAL EVALUATION CRITERIA & ASSUMPTIONS

A comparison of different technical alternatives necessitates a series of general assumptions and criteria. To the extent possible such criteria are presented here. Certain assumptions, though, that are specifically and exclusively used in a certain chapter may be presented only where applied first time. Detailed tables used for engineering design, e.g. regarding sewerage, are presented in the appendices, chapter 16.

6.1 Area and housing

Figure 6.1.1 shows the area within Lagos city boundaries which is chosen as a physical framework for evaluation of each technical alternative. The particular choice of this area is the responsibility of the authors; neither the city of Lagos nor any person in Lagos has expressed the wish to have this study carried out. The area was chosen, however, because of the geotechnical and climatical conditions, i.e. a clayish impermeable soil under tropical temperatures, high precipitation rates and high humidity. Further, several areas of Lagos do have a waste water collection system (night soil collection) which is deemed nonsatisfactory, cf. /11/.

Figure 6.1.1 & 6.1.2 proposes the town plan arrangement and the infrastructure within the road network. Also indicated are the three different population densities that will be used for comparative purposes throughout the report. The housing area totals app. 286 ha, cf. figure 6.1.1, and with due consideration to green belts etc the population numbers shown on figure 6.1.2 have been calculated. The principle of "one family per plot" is maintained throughout the report. This implies that one family occupies one plot of 2oo m^2, and there are 4o plots per hectar.

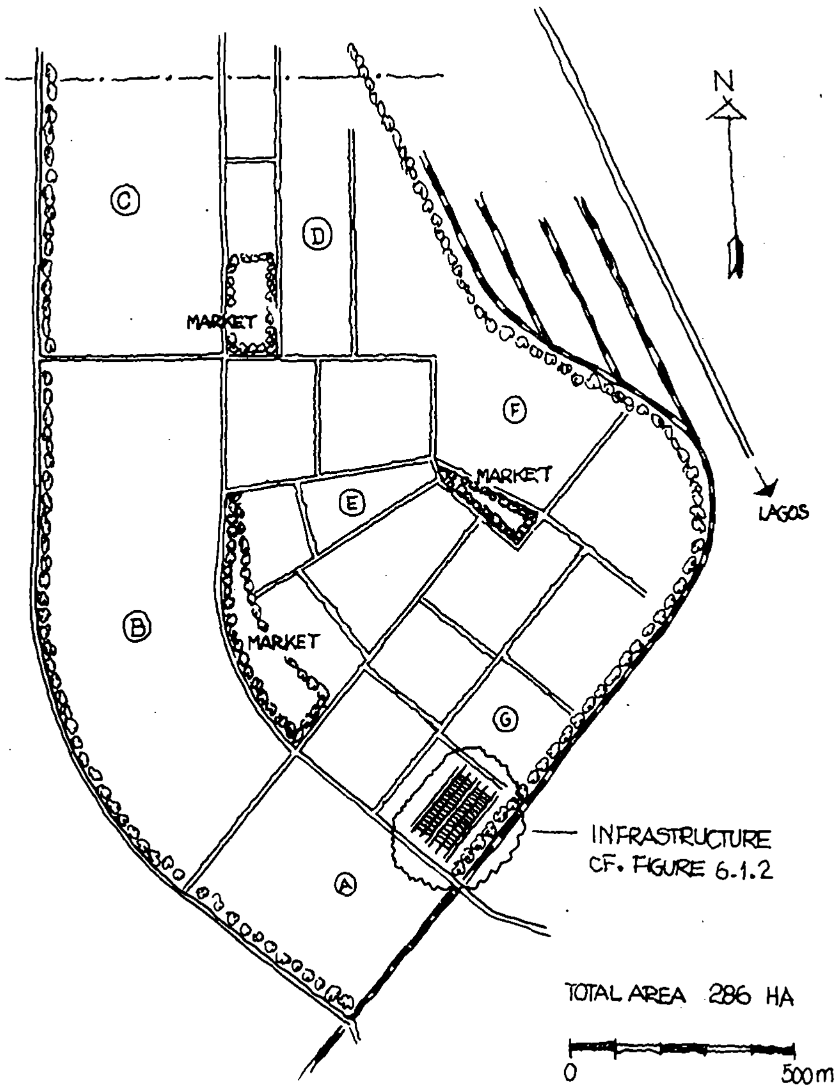


Figure 6.1.1 Lay-out of housing and transportation within the study area, cf. figures 5.1 & 5.2

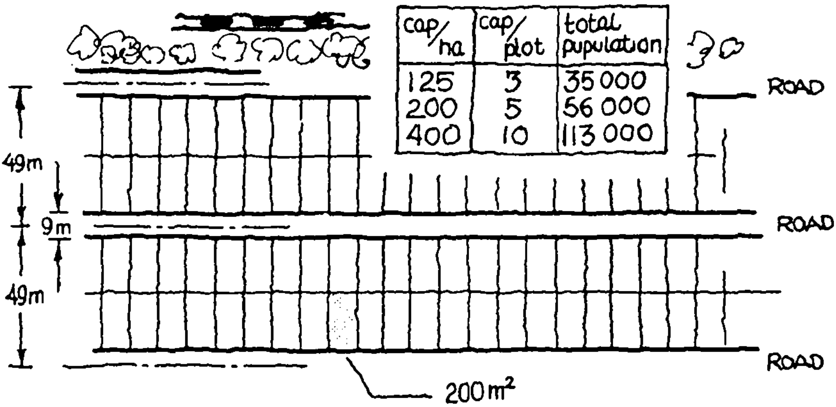


Figure 6.1.2 Detailed plot lay-out and alternative population densities

Another fundamental criterion is that maintenance of plot facilities be the responsibility of the occupants. This means for example that a house bucket (cf. the chamber pot system, chapter 3,) would be emptied by a family member and not by publicly employed labour. Thus, the night-soil bucket collection system is excluded from evaluation in the present report. Night soil collection systems are frequently operated with little success, e.g. in Lagos, and that there are often strong emotional reactions to the direct handling of the excreta from other households. Reasons for this attitude could be cultural and religious, ETHIOPIA /12/.

6.2 Water supply, WS

It is assumed that public water supply be provided, either by public standpipe or by plot connection:

1. Public standpipes spaced conveniently with regard to distance from individual plots, e.g. max. 15o m from any plot. The public standpipe water supply is satisfactory for the systems MR, AB, JS & CM.

2. Plot tap, at least one per plot. This would satisfy water requirements for waste water systems FS and AP. For JS and MR a plot tap should normally be avoided because waste water cannot be removed adequately.

The cost of the water supply is pertinent to the evaluation of the total costs of a sanitation service. Information as to water supply is unfortunately very poor, but some observations have been provided by WHITE et al. /13/ from studies in East Africa. In general piped water is delivered at a lower price per liter than is water otherwise supplied. The range of variation of prices is experienced to be very broad; WHITE et al. /13 p. 1oo/ find that even average prices vary from 6.5 $/cap,y (high density) to 12.6 $/cap,y (low density) for piped water in urban settlements. Though pertinent mostly to East African conditions and based on a limited number of observations, it is here assumed that WHITE's observations are applicable to indicate an order of magnitude for water supply in the present case.

Table 6.2.1 Assumed water comsumption & prices

Population	1) Plotpipe-supply			2) Standpipe-supply		
cap/ha	ℓ/p,d	$/p,y	$/plot,y	ℓ/p,d	$/p,y	$/plot,y
125	22o	13	39	14	2.5	8
2oo	17o	8	4o	1o	1.8	9
4oo	9o	7	7o	9	1.6	16

Notes: 1) WHITE /13, table 4.4/
2) WHITE /13, table 4.4 and figure 5.3/

6.3. Waste water removal, WWR

For the study area, figure 6.1.1, a general North-South slope of 3 pm is assumed so that the ground drains naturally into the lagoon. The ground water level is assumed o.5-1 meter below soil surface and the soil impermeable (clay). These conditions compare pretty well to actual conditions in the Lagos area, GILBERT/1/, and in many other major urban settlements in developing countries, e.g. Djakarta & Dacca.

Impermeable soil conditions are not favourable to the construction of a piped waste water removal system because of more difficult trenching. Consequently, when comparing different alternatives the price of a piped system is a priori relatively high and maybe too high for similar sewerage projects in areas with more slope, more permeable soil and lower groundwater table.

The structures for the conveyance of storm water are not included in the subsequent calculations. This assumption will not invalidate the comparison of alternative schemes since it applies equally to all proposals. Open or covered ditches could be suitable for the collection of storm water.

It is proposed that the waste water be treated in facultative oxidation ponds (figure 7.4.2). These ponds are conveniently located in the lowlands between the study area and the lagoon, figure 5.2. Details as to pond design and costs will be presented in the following chapters, particularly no. 7.

6.4. Health

The state of human health is controlled by a number of interrelated factors, and waste removal is only one of these. Others are climate, water supply, the general availability of medical care, vaccination programmes, vector control programmes, transportation/communication etc. Thus, the

Table 6.4.1 Identification of some disease transmission routes and biological agents involved

TRANSMISSION		EXAMPLES	
Route	Contraction	Bio-agent	Disease
	Ingestion	Salmonella	Typhoid
Water	Skinpenetration	Schistosomas mans., jap.,h.	Schistosomiasis
Anus-hand-hand-man	Ingestion	Shigella	Dysentery
Insect-vectors	Breed in water	Anopheline Culex p. fat. Aedes Aegypti	Malaria Filariasis Yellow & Dengue fever
Food contamination	Ingestion	Salmonella Vibrio Entamoebia	Salmonelloses Cholera Amoebiasis
Man-soil-man	Dirt intake & food contamination	Ascaris Trichuris Necator	Round worm Whip worm Hook worm
Air	Eye & Nose	Aesthetical nuisance, e.g. odeur and unsightly conditions	

assessment of the role of improved waste removal may be very difficult.

The subsequent assessment will be based on the assumption that the transmission of certain communicable diseases is identifiable by mode of contraction and/or type of biological agent (e.g. bacteria & viruses). Consequently, an improved health situation is assumed when a proposed waste removal system significantly impedes the use of a transmission route known to be common in the migration of a certain disease or disease vector. Table 6.4.1 shows a number of specific criteria for such evaluation.

The transmission routes and the model agents listed in table 6.4.1 indicate only a limited number of risks of impaired health; but the examples chosen are important per se, and they represent a number of important possibilities of disease contraction. The risk of infection by the water route is only one among many others, and possibly not the most important one in the present context.

It is important when evaluating improved sanitation to distinguish between effects of improved water supply and effects of improved waste water removal. In the subsequent evaluation water supply is assumed to be based on sources that are located outside the urbanized settlement. The question of separate effects of water supply and waste water removal will be dealt with specifically in chapter 12 (comparative evaluation).

6.5. Economy

6.5.1 Cost components

In order to obtain economically comparable alternatives the evaluation will include the following items:

1. Plot installation.	Facilities necessary to secure collection, e.g. sinks, possible tanks for settleables, laterals, and manholes for a piped system or holding tank or bucket for a non-piped system. Maintenance on the plot is the responsibility and expense of the occupants. Superstructures for privies are not included.
2. Collection	All public collection is included whether it be piped or hauled by truck. Basically collection comprises transport from plot to treatment plant.
3. Treatment	All treatment costs are included. Pumping after treatment and disposal into the lagoon (brackish) is however excluded.

Details relating to prices and costs are given below in chapters 7 - 11, and in the appendices, chapter 16.

6.5.2 Labour, wages

Calculations of labour costs will be based on table 6.5.2.1, which is obtained from ALUKO /14/. The order of magnitude of wages compares well to those suggested by WHO /15/.

Table 6.5.2.1 Wages, Nigeria 1974

Employee	$/month	$/year
Engineer	300	4000
Driver	80	1000
Labourer	50	600

6.5.3 Trucks & supplies

Prices for trucks for hauling of excreta are indicated in table 6.5.3.1.

Table 6.5.3.1 Vacuum truck purchase costs

Tank capacity m^3	Approximate cost 1975 $
2	10000
3	13000
5	18000

Note: Prices based on information from TOKYU /16/

To ensure uninterrupted collection service cost of spare parts and extra trucks must be included in any purchase to secure safe collection service also when some trucks are out of operation. Such details including repair are encountered for in subsequent chapters 7 - 11.

Operation costs (e.g. gasoline, tires, lubrication etc) have been established on the basis of long term records for vacuum trucks of the city of Copenhagen. Based on these records it is estimated that - excluding labour and repair - the per hour operation cost is o.7 $ (app. 4 Danish Kroner/ hour in 1975, stops and evacuation operation included).

6.5.4 Civil works

Detailed cost calculation tables are found in chapter 16, appendix 1.

6.5.5 Present value, user charges, and comparison of alternatives.

The economical evaluation of the different technical alternatives will be based partly on a present value, PV, calculation. The principle and applicability of PV is discussed by WILLIAMS & NASSAR /17/, and it involves a calculation of the present value of all future expenditures and revenues according to the following formula:

$$PV = \sum_q (R_q(1+i)^{-n_q} - E_q(1+i)^{-n_q}) \qquad (6.1)$$

where R_q = future revenue

E_q = future expenditure

n_q = number of years (or a different time period) between present time and future E_q or R_q

q = enumerator, securing that any R and E be included

i = pro anno interest rate. How to choose i is discussed below in section 6.5.6.

The use of (6.1) implies that an investor would react according to the following fundamental rules, cf. WILLIAMS /17/:

1) It is better to receive to-day than to-morrow, and it is better to pay later than soon.

2) It is better to pay less and to receive more.

Based on these principles and eq. (6.1) the investor will always choose among several alternatives the one with the greatest PV. If only expenditures are considered and PV comes out negative the best choice will be the alternative with the smallest negative PV. (In subsequent calculations the sign may be omitted if misinterpretations are not likely).

Calculation of PV may give guidance as to economically optimal alternatives. When combined with other decision variables, e.g. health & general service level (convenience), the overall optimum alternative may be chosen.

However, to the plot occupant or the user of a technical system the PV as calculated by the investor may be irrelevant, because there is no direct relation between a calculated PV figure and the occupant's benefit from the system installed. There may - on the other hand - be a close relation between the project PV and the user charges collected. To the plot owner who has to pay as well as to the investor who will want revenues to cover expenditures it may therefore make sense to indicate directly the user charges necessary to operate the system, e.g. in terms of \$/cap,y or \$/plot,y.

In order to reach such annual user charge the PV can be used according to the following equation:

$$A = PV \frac{i}{1 - (1+i)^{-N}} \qquad 6-2$$

where A = annual user charge
PV = present value, cf. eq. (6.1) above
i = interest rate (calculatory) on an annual basis, cf. 6.5.6 below
N = expected life time of the system.

Equation (6.2) simply requires that there be exact balance between future expenses (investments and O&M as already built into the PV) and revenues (= user charges) collected throughout the project period of N years that applies to both revenues and expenditures. And this requirement then defines the necessary user charge.

The present value, PV, as well as the user charge, A, will be presented below as results of the economical evaluation in chapter 12.

6.5.6 Interest rates

For use in PV-calculations or estimating user fees a monetary interest rate is necessary. There is no simple way of finding a relevant interest rate, which may depend on the money-market situation or on governmental policies or administrative decisions. Also, the question of inflationary prices becomes very important, when considering project periods where the period between initial investment and final back-payment may amount to e.g. 3o years.

In order to allow for flexible interpretation of economical calculations it is therefore decided to use 3 different interest rates o, 5, and 15%.

An interest rate of 0% indicates the total cash flows throughout the project period, all in prices for a year of reference; 1975 is used where possible in this report.
5% may be a realistic interest rate on the international capital market assuming that inflation does not have to be accounted for in the calculation of PV (real interest).

15% may be an interest rate that include "real" interest and compensation for inflation.

All three interest rates will be used in subsequent graphs and tables of results.

6.6 Criteria summarized

Climate	Tropical, humid
Soil	Impermeable, clayish, high groundwater table (less than 1 meter below surface)
Area	Approximately 286 hectar
Slope of area	3 pm average natural slope available (cost sensitivity to other slopes will also be investigated)
Population density	125, 2oo or 4oo persons/hectar 4o plots per hectar, each plot 2oo m^2 3,5 or lo persons/family. Each family handles and maintains the plot installations
Water supply	Public standpipes sufficient for MR, AB, JS & CM Plot tap indispensable for FS and normally required also for AP. For MR, AB, JS (& CM) the plot tap should not be provided
Waste water removal	Piped systems for FS, AP & AB. Non-piped for MR, JS & CM
Waste water treatment	Oxidation ponds and discharge to the nearby lagoon (brackish/saline water)
Economy	Technical alternatives will be based on a present value, PV, calculation assuming a project period of 3o years;cf. 6.5.5. Interest rates of o, 5 and 15% will be used to demonstrate how capital intensive alternatives may differ from labour intensive ditto. Assumptions as to unit wages and prices are presented in 6.5.2 & 3 and in chapter 16, appendix I.

7. FULL SEWERAGE

7.1. Introduction

The presumptive layout of the full sewerage scheme (FS) is indicated on figures 7.1.1 and 7.1.2.

In addition to the evaluation criteria that were mentioned in chapter 6 the following factors will be considered in the cost sensitivity analysis (section 7.6. - 7.9.)

1° Level of water consumption
2° Gradients of drainage pipes
3° Slope of drainage area
4° Replacement of paving

7.2. Plot installations

Only the cost of a flush toilet and a sink drain in each home has been included in the overall cost estimate. The unit costs applied are as follows:

Flush toilet, Eastern type	: $ 6o
Sink	: $ 2o
Installation & internal piping	: $ 6o
External piping 7 m ∅ loo mm, $ 8.o/m	: $ 6o
Total cost per home	$ 2oo

7.3. Sewage collection

7.3.1 Slope of area

The location of the main sewerlines are shown on figure 7.1.1. In the absence of a contoured map it has been assumed that there is an average 3 pm slope available for the trunk sewers. The cost implication of this assumption is evaluated below in section 7.8.

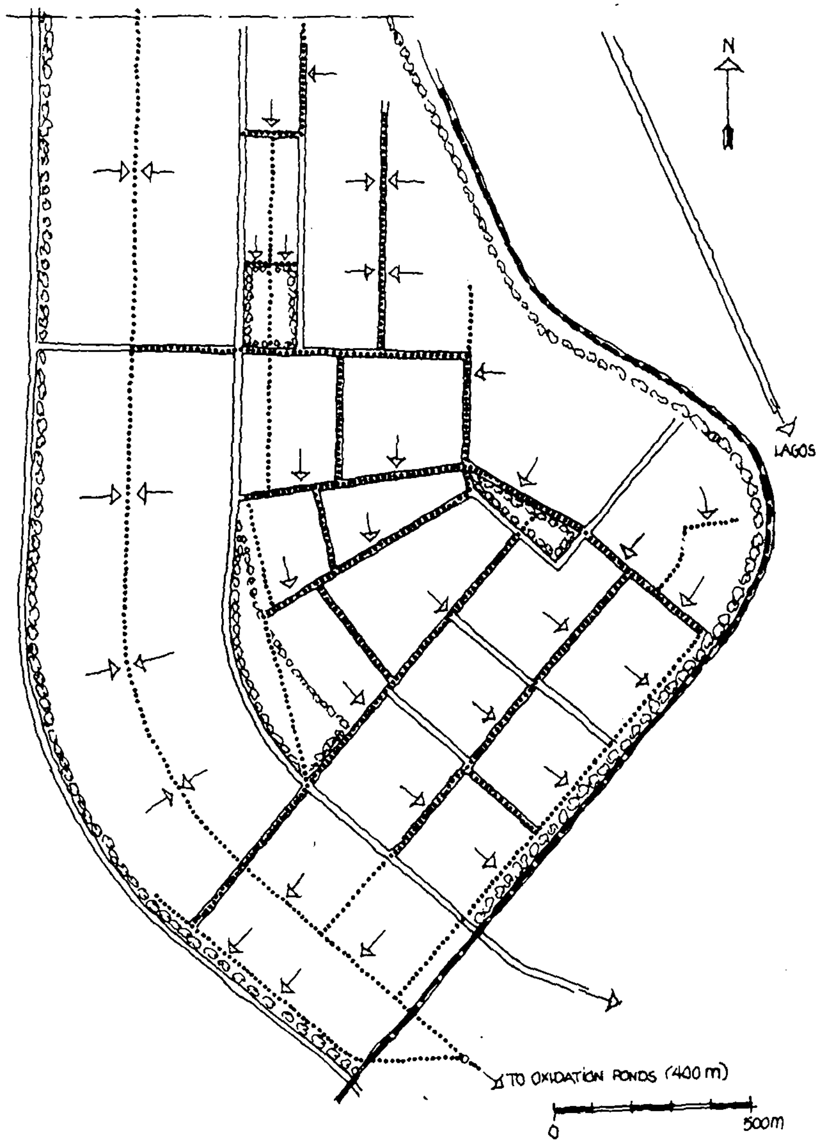


Figure 7.1.1 Major trunk sewers, FS system

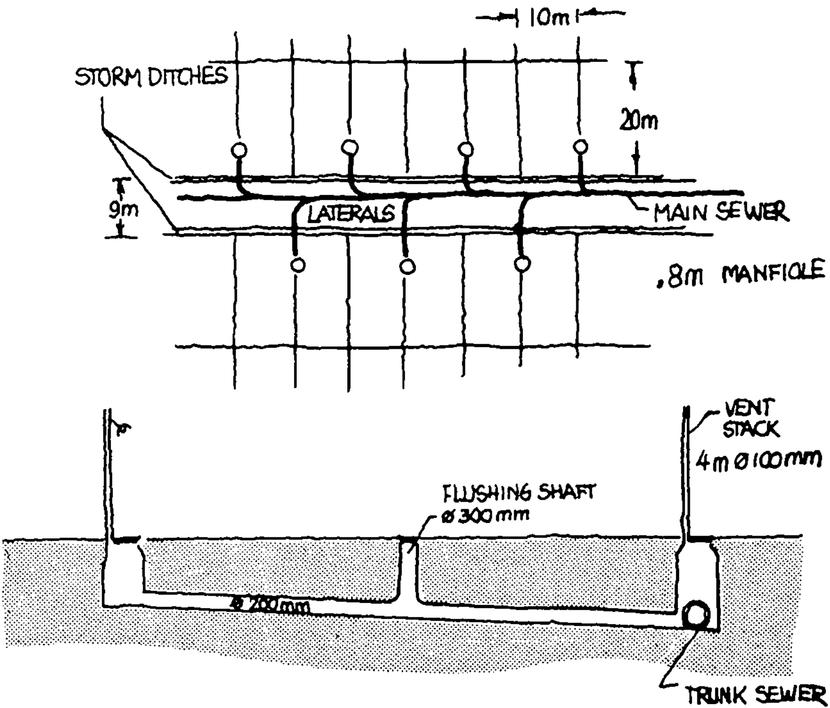


Figure 7.1.2 Plot drainage, laterals, mains, and trunk sewers, FS system

Table 7.2.1 Total cost of FS plot installations

Density cap/ha	Cap/plot	Cost/cap $	1) Total area cost, $ 1ooo
125	3	65	23oo
2oo	5	5o	23oo
4oo	1o	2o	23oo

Note: 1) Based on a total area of approx. 284 ha and an extra expenditure of $ 3o ooo for public toilets. The assumed lifetime for toilet facilities is 3o years. Maintenance is carried out by the home owners and is thus not included.

7.3.2 Network

The structure of plot drainage is shown on figure 7.1.2 Lateral lines extending 5 m onto the plots are connected to the main sewer by a 45^{o} bend and a 45^{o} junction piece. A manhole is located in the trunk sewerline at every junction of a mainline and at any change of slope or direction. In addition manholes are located at the top of all main and trunk sewers.

The storm water collection system is proposed to consist of concrete lined road side ditches at a gradient of approximately 2 pm. The design of the storm water system is not included in this study, but it is necessary to ensure that the storm water ditches do not conflict with the sanitary system. In the upper reaches the storm water ditches will be quite shallow and the sanitary sewers are easily located underneath. Since the slope of main sewers in the base design is considerably greater (8 & 15 pm) than the hydraulic gradient for the storm water ditches the sanitary sewers can always be located below the bottom of the storm water ditches also in the lower reaches of the system. In special cases, particularly when a flat slope of the main sewers (2 - 4 pm) is being considered, special designs have to be implemented probably involving wide rectangular channels (covered) or narrow deep channels (covered) in the middle of the road when trunk sewer connections enter from only one side.

7.3.3 Cleaning

Modern jet cleaning equipment is proposed for maintenance, therefore merely a flushing shaft is required midway on the main-sewer lines to facilitate cleaning.

7.3.4 Corrosion

Due to the generally high temperatures in the Lagos area and the low sewage flows in the initial development phases concrete pipes are likely to be attacked by Hydrogen Sulphide from the microbial activity in stale sewage, ref. /18/. Since ventilation through the manhole covers is provided, the corrosion hazard is not critical when the network is designed at a steep slope (15 pm). However, when smaller slopes are employed plastic pipes are alternatively proposed. The total price of sewer installation is essentially the same irrespective of whether the pipematerial is concrete or plastic.

7.3.5 Design criteria

Pipes. Sewers for this scheme are circular rubber ring jointed concrete pipes that are or can be produced locally. Where corrosion is deemed to be a problem, plastic pipes are alternatively proposed. Connections should be made by prefabricated 45° junction pieces.

Minimum pipe size is 15o mm in order to avoid blockings. This minimum diameter is rather large, but it has been experienced that objects such as tin cans, rags and vegetable debris (maize cobs) are often found in sewers.

Manholes. Details of manholes are shown in Appendix I. The different types are used as indicated below:

Type I: Shallow manhole ø .8 m, used for laterals only. (one per 2 plots)

Type II: Used on mains and trunk sewers, ø 25o - 6oo mm.

Type III: Used for trunk sewers larger than ø 6oo mm.

<u>Shafts.</u> Narrow flushing shaft ø 3oo mm with rounded junction piece to the sewer line. It is used for jet cleaning purposes only

<u>Laterals.</u>

Diameter	:	15o mm
Slope	:	2o pm
Min. depth	:	o.9 m

<u>Mains.</u>

Diameter minimum	:	2oo mm
Slope	:	15 pm (base design)

Such great slope will provide selfcleaning and keep pipe corrosion at a minimum. Due to proximity of the sources slugs of water will surge through the near part of the main, thus providing additional cleaning. The sensitivity of cost to slope is discussed later in this chapter.

<u>Trunks.</u>

Diameter, minimum	:	25o mm
Slope	:	3 pm

For upper ends where the flow is low the slope will have to be increased to ensure self-cleaning, but this will appear to be insignificant to the total estimated cost.

<u>Flow.</u> Dry weather flow, DWF = 85% of the water consumption, cf. /33/

Infiltration inflow. I = 3oooo US gal./mile, day
I = o.3 ℓ/s, ha, cf. /33/.

The infiltration depends essentially on the quality of the pipes and in particular on the craftmanship during construction, both of which are presently unknown. For rubber ring jointed pipes the above is a conservative estimate.

<u>Capacity.</u>

Lateral and main sewers	:	5 DWF + I
Trunk sewers	:	4 DWF + I
Large trunk sewers	:	(2-4) DWF + I

Hydraulics.

$$\frac{Q}{F} = V = 72\ R^{0.625}\ I^{0.5} \qquad \text{cf. /34/}$$

Q = flow, m^3/s

F = cross sectional area, m^2

R = hydraulic radius, m

I = slope, m/m

7.3.6 System design

A design based on a water consumption of 4oo ℓ/cap,d is initially developed. Lower levels of water consumption (25o and 125 ℓ/cap,d) are thereafter assumed, in order to identify and discuss the design in terms of sensitivity to this parameter.

It appears from the estimates in tables 7.3.6.2 and 7.3.6.3 that the areas where the slope of the trunk sewer will have to be increased to maintain self cleansing are insignificantly small, and this aspect will therefore not change the cost computation.

Table 7.3.6.1 Design of laterals and mains

Density cap/ha	Laterals ø 15o mm m/ha	Mains ø 2oo mm m/ha	DWF ℓ/s,ha	3) Required mains' capacity ℓ/s	Capacity of ø 2oo at 15 pm ℓ/s	4) Maintenance
125	19o	2oo	1) o.5	3.5	42	min
2oo	19o	2oo	o.8	5.4	42	min
4oo	19o	2oo	1.6	1o.4	42	min
pub. space	5o	5o	2) 3.o	14	42	min

Notes:
1) 4oo x o.85 = 34o ℓ/cap,d
2) 6o ℓ/cap, 8 h
3) 5 DWF + I per ha. Mains are max. 25o m so one main drains at most 1.25 ha.
4) Cleaning of occasional blockings. The mains are deemed to be self-cleaning with respect to sand.

Table 7.3.6.2 Capacity of trunk sewers

Diameter	Capacity at 3 pm slope	Area drainage capacity 1)		
		125 cap/ha	200 cap/ha	400 cap/ha
mm	ℓ/s	ha	ha	ha
250	33	14.3	9.4	4.9
300	54	23.5	15.4	8.1
350	81	35.2	23.1	12.1
400	115	50	32.9	17.2
450	155	67.4	44.3	23.1
500	210	91.3	60	31.3
600	330	143.5	94.3	49.3
700	500	217.4	142.9	74.6
800	720	313	205.7	107.5
900	980	426.1	280	146.3
1000	1150	500	328.6	171.6
1100	1650	720	471.4	246.3

Note: 1) 4 DWF + I, DWF = 340 ℓ/cap,d, I = 0.3 ℓ/s, ha

Table 7.3.6.3 Self-cleaning of trunk sewers

Density	2 DWF 1)	For self-cleaning $\tau = 0.25\ kg/m^2$ 2)		For self-cleaning $\tau = 0.15\ kg/m^2$	
		flow	3) area	flow	3) area
cap/ha	ℓ/s,ha	ℓ/s	ha	ℓ/s	ha
125	1.0	14	14	7	7
200	1.6	14	9	7	4.5
400	3.2	14	3	7	1.5

Notes: 1) Flow deemed available once a day for pipe cleaning.

2) Eroding force $\tau = \gamma \cdot R \cdot I$ cf/32/, kg/m^2

γ = density of water kg/m^3

R = hydraulic radius, m

I = slope, m/m

Min. τ values of 0.15 - 0.25 kg/m^2 are recommended under various conditions, cf /32/, to maintain self-cleaning with respect to sand.

3) The area downstream of which the trunk sewer is deemed to be self-cleaning at a slope of 3 pm

Table 7.3.6.4: Quantitative summary of sewers.

	Pipe diam. (mm)	Accumulated length, meters 125 cap/ha	200 cap/ha	400 cap/ha
Laterals	150	55030	55030	55030
Mains	200	57890	57890	57890
Trunks	250	5710	4190	2710
	300	2140	1890	1340
	350	560	1690	1100
	400	1230	510	1770
	450	1590	1120	910
	500	750	1250	350
	600	250	1400	1470
	700	1115	180	1750
	800	0	1115	830
	900	0	0	0
	1000	0	0	265
	1100	0	0	850
Trunks total		13345	13345	13345

Notes:

		Number of manholes		
Manhole	Type I	55o3	55o3	55o3
	Type II	517	512	463
	Type III	13	18	68
	Flushing shaft	25o	25o	25o

Table 7.3.6.5 Cost of Sewage collection System

Pop. density cap/ha	1) Laterals $ 1ooo	2) Main sewers $ 1ooo	3) Trunk sewers $ 1ooo	4) Manholes $ 1ooo	Sewers total $ 1ooo	5) Mainten-ance $ 1ooo
125	1486	2489	1167	112o	6262	3
2oo	1486	2489	123o	1122	6327	3
4oo	1486	2489	1416	1134	6525	3

Notes: 1) ø 15o mm, average depth 2.5 m at $ 27/m
2) ø 2oo mm, average depth 3.o m at $ 43/m
3) Average depth 5.o m price see appendix I table 1
4) Prices from appendix I, table 2
5) Occational cleaning of blockings by a contractor (supplies $ 1ooo + labour $ 2ooo/y)

At a slope of 3 pm a substantial flow is necessary to maintain self-cleaning.

The trunk sewer system is designed on basis of table 7.3.6.2 (see appandix II, chapter 16, for the summarized calculations).

The sewerage design results and preliminary cost estimates are summarized in tables 7.3.6.4 and 7.3.6.5.

7.4 Sewage treatment

Sewage treatment is provided by facultative oxidation ponds which are designed according to the South African design procedure /19, 2o, 21/, with primary and secondary ponds.

The ponds are proposed to be built in the low-lying area to the South of Ebute Metta as indicated on figure 5.2. The objective of treatment is to reduce the noxious character of the sewage, and an engineering criteria for the effluent quality has been assessed to 2o mg/ℓ BOD_5.

7.4.1 Design of oxidation ponds

The following assumptions are made in the design of oxidation ponds:

- Organic loading 55 g BOD_5/cap day /22/.
- Evaporation approximates precipitation.
- Effluent quality better than BOD_5=2o mg/ℓ
- The effluent is discharged into the brackish Ebute Metta Creek, which again discharges immediately into the Lagos Lagoon and subsequently into the sea.
- Basic equations /2o/

$$P = \frac{P_o}{K_T \cdot R + 1}$$

$K_T = 1.2 \cdot 1.085^{-(35-T)}$ T in ^oC

$P_1 = \frac{750}{2 \cdot depth+8}$ (empiric formula for strong sewage)

P = concentration, mg BOD_5/ℓ

R = retention time, d

Table 7.4.1.1 Sizing of oxidation ponds

Pop. density cap/ha	1) Hydraulic loading m^3/d	2) Influent BOD mg/ℓ	Total detention d	Total surface area ha	3) Surface loading BOD kg/ha
125	19600	100	6	8.3	237
200	26900	115	6.7	12.8	246
400	46400	135	7.7	25.3	249

Notes: 1) Based on DWF + I (DWF = 340 ℓ/cap, d I = 0.3 ℓ/s,ha)
2) Based on I = 0.3 ℓ/s,ha which is a conservative estimate
3) For comparison, surface loadings that are usually recommended are 150 - 350 kg BOD/ha /19/

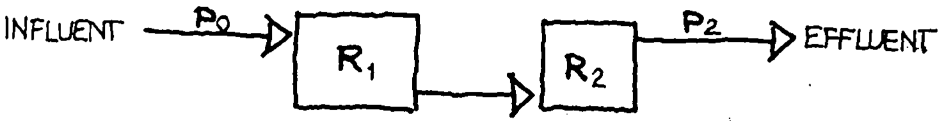


Figure 7.4.1.1 Principle layout for oxidation pond design

7.4.2 Pond construction and cost

Primary and secondary ponds are operated separately in parallèl, with a typical pond size of 1oo x 2oo m or 2 ha; cf. figure 7.4.2.1 below.

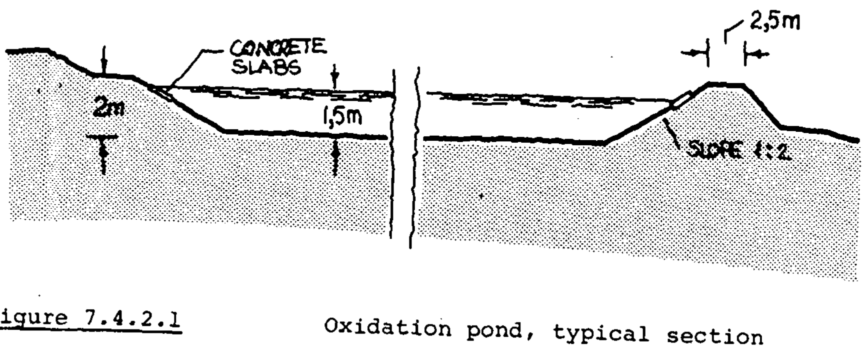


Figure 7.4.2.1 Oxidation pond, typical section

Table 7.4.2.1 Cost estimate for 1 oxidation pond of 2 ha

Excavation, 15ooo m^3:	\$ 225oo
Excavation, backfill and compact, 15ooo m^3, \$ 2.2/m^3	\$ 33ooo
Lining (clay or polyeth.), 2oooo m^2, \$ o.5/m^2	\$ 1oooo
Slabs at water level, 364 m^2, \$ 1o/m^2	\$ 37oo
In- & outlet works	\$ 1ooo
Fencing, 3oo m at \$ 15/m	\$ 45oo
Total	\$ 747oo
Construction cost of oxidation ponds is per ha	\$ 38ooo

Prices for miscellaneous civil works are found in chapter 16, appendix I, table AI.3.

Table 7.4.2.2: Cost of sewage treatment, FS system, 1ooo \$

Pop density	Pond area	INVESTMENT		O & M		
			1)	2) after		3)
cap/ha	ha	30 years	10 years	10 yrs.	supplies	labour
125	8.3	316	10	158	1	12
200	12.8	486	10	243	1	13
400	25.3	960	15	480	1	17

Notes:

1) Comminuters at plant inlet
2) Pond dredging at 50% of construction cost
3) 4 foremen, \$ 1200/yr.
   8 labourers, \$ 600/yr.
   Site upkeep 1 labourer per 2 ha pond.

7.5 Base design cost

Based on the cost estimates presented in the preceeding sections of this chapter a summary can now be made of costs for a base design. The summary appears from tables 7.5.1 - 2 and figure 7.5.1.

Table 7.5.1 Summary of cost of FS base design, 1000 $

Density		INVESTMENT				O & M	
					after	Annual	
cap/ha		30 yr.life	10 yr.	5 yr.	10 yrs.	supplies	labour
125	PI	2300	-	-	-	-	-
	C	6262	-	-	-	1	2
	T	316	10	-	158	1	12
200	PI	2300	-	-	-	-	-
	C	6327	-	-	-	1	2
	T	486	10	-	243	1	13
400	PI	2300	-	-	-	-	-
	C	6525	-	-	-	1	2
	T	960	15	-	480	1	17

Table 7.5.2 Preliminary PV of FS, base design

Density		0 %		15%	
cap/ha		Invest	O&M	Invest	O&M
125	PI	2300		2300	
	C	6262	90	6262	19
	T	346	706	329	134
	SUBTOT.	8908	796	8891	153
	TOT.	9704		9044	
200	PI	2300		2300	
	C	6327	90	6327	19
	T	516	906	499	167
	SUBTOT.	9143	996	9126	186
	TOT.	10139		9312	
400	PI	2300		2300	
	C	6525	90	6525	19
	T	1005	1500	980	267
	SUBTOT.	1590		9805	286
	TOT.	11420		10091	

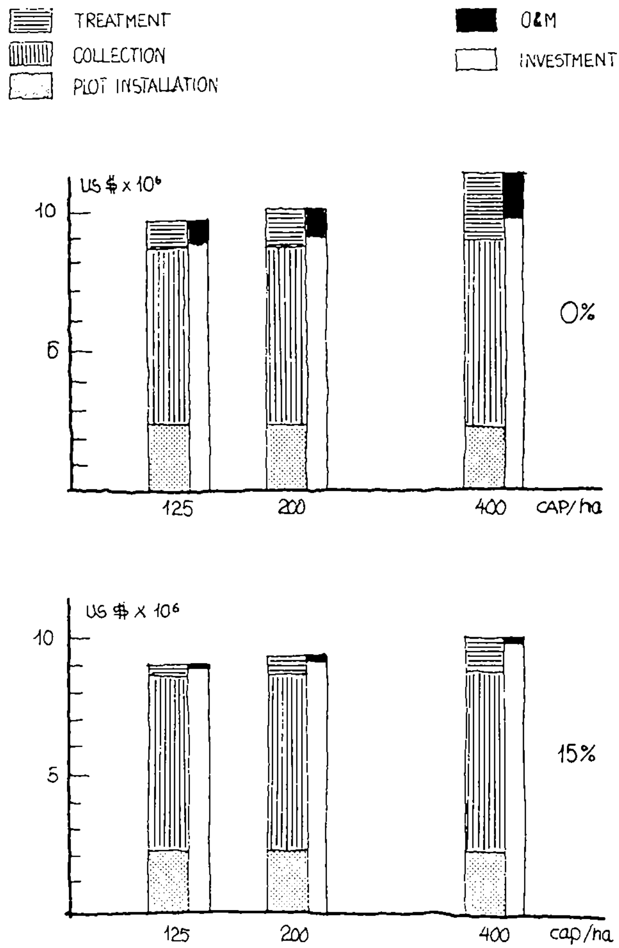


Figure 7.5.1 Major FS cost components (PV) preliminary analyses, cf. table 7.5.2

7.6 Cost sensitivity to level of water consumption.

The system has been redesigned for lower levels of water consumption 250 ℓ/p,d and 125 ℓ/p,d

At the lower flows the trunk sewer system will require slightly more maintenance. Trunk sewers amount to 13 km.

Table 7.6.1 Cost elements of FS system at decreased flows

Water cons. ℓ/p,d	Density p/ha	1) Laterals $ 1ooo	1) Main sewers $ 1ooo	Trunk sewers $ 1ooo	Man-holes $ 1ooo	Sewers total $ 1ooo
25o	125	1486	2481	112o	1119	6214
	2oo	1486	2489	1167	112o	6262
	4oo	1486	2489	123o	1122	6327
125	125	1486	2489	11oo	1118	6193
	2oo	1486	2489	112o	1119	6214
	4oo	1486	2489	1167	112o	6262

Notes: 1) Laterals and mains are restricted by min. sizes ø 15o mm and ø 2oo mm respectively

These can be routinely flushed twice a year with jet cleaning equipment within the minimum maintenance budget of $ 3ooo/year.

It appears from figure 7.6.1 that the total cost, PV, of a sewerage project is virtually insensitive to the level of water consumption for which it is designed. This is because mainly the larger trunk sewers and to a minor extent the sewage treatment works are influenced by the design flow, and these components are not the primary contributers to the total project cost (figure 7.5.1)

Table 7.6.2 Sizing of oxidation ponds at decreased flow

Waste water ℓ/p,d	Density p/ha	Hydraulic load m^3/d	Influent quality BOD mg/ℓ	1) Total pond detention d	Total pond surface ha	Cost of pond $ 1ooo
25o	125	121o5	165	9.o	7.7	292
	2oo	16615	19o	1o.o	11.8	45o
	4oo	2866o	22o	11.2	22.8	866
125	125	6o5o	325	14.8	6.4	244
	2oo	831o	38o	16.7	9.9	374
	4oo	1433o	44o	18.8	19.1	728

Note: 1) For design criteria and procedure see section 7.4

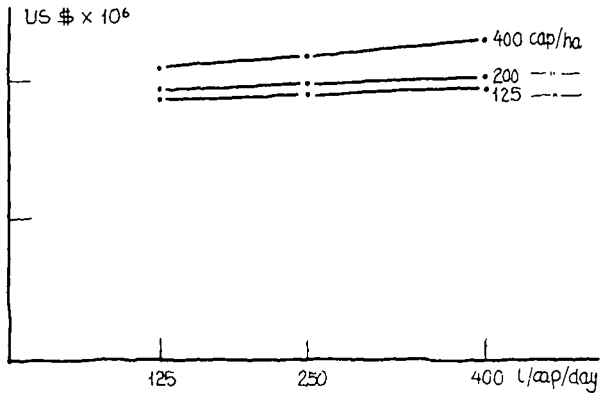


Figure 7.6.1 FS cost sensitivity to level of water consumption. Total cost (PV at i = 0%) is based on a 3o year life time, cf. tables 7.5.2 & 7.6.3

Table 7.6.3 Cost elements of FS at decreased flow

Waste water ℓ/p,d	Density p/ha	INVESTMENT, $ 1ooo 1) 3o yrs.	2) 1o yrs.	O & M, $ 1ooo 3) after 1o yrs.	annual supplies	labour
	125	88o6	8	146	2	14
25o	2oo	9o12	1o	225	2	15
	4oo	9493	1o	433	2	18
	125	8737	5	122	2	13
125	2oo	8888	8	187	2	14
	4oo	929o	1o	364	2	17

Notes:
1) House installations + sewers and oxidation ponds.
2) Comminutors at inlet to treatment works.
3) Dredging of oxidation ponds after 1o years operation at 5o% of construction cost.
4) Sewer maintenance (1+2) = $ 3ooo.
Maintenance on treatment works: 4 foremen at $ 12oo/yr., 8 labourers at $ 6oo/yr.
Site upkeep: 1 labourer per 2 ha pond
Supplies : $ 1ooo/yr.

7.7 Cost sensitivity to slope of sewer lines

It appears from figure 7.5.1 that sewer installations constitute the major portion of the project cost. This could potentially be decreased by designing sewerlines at smaller gradients. (Tables 7.7.1, 7.7.2 and 7.7.3)

Laterals:

Slopes of less than the recommended 2o pm are not desirable because frequent blockings, insufficient earth cover or complications with the storm ditches may result.

Main sewers:

A potential decrease in the required min. slope for mains of 15 pm would effect requirements for higher levels of maintenance. At very small slopes a routine cleaning should be done several times per year. South African experience with similar sewer districts suggests that jet cleaning once a year is adequate to maintain small sewers laid at 4 pm slope.

Trunk sewers:

Minimum slopes for large trunk sewers are in the range of o.5 - 1.o% Very small savings, if any, can be generated by decreasing the slopes to such low values. (see tables 7.3.6.2 - 5).

Figure 7.7.1 summarizes the tables and demonstrates that slope of main sewers is a key factor in estimating the total cost of sewerage.

Table 7.7.1 Design of FS main sewers at small slopes

1) Slope	2) Design flow ℓ/s	3) Capacity of main sewer ℓ/s	4) Velocity at 2 x DWF m/s	5) τ at 2 x DWF kg/m²	6) Level of maintenance
2 pm	5.4	15	o.25	o.o7	Regular routine
4 pm	5.4	22	o.3	o.13	basis
8 pm	5.4	3o	o.4	o.21	Occational clean-
15 pm	5.4	41	o.45	o.32	ing by contractor

Notes:
1) Based on 2oo cap/ha and DWF = 34o ℓ/s, ha
2) Based on 5 DWF + I and 1.25 ha per main. (I = o.3 ℓ/s, ha)
3) Diameter 2oo mm
4) Velocity in the lower end of main sewer
5) $\tau = \gamma RI$, see section 7. For the recommended range for τ to ensure self cleaning at o.15 - o.25 kg/m² and similarly for the velocity o.45 - o.6 m/s
6) The regular maintenance should be ensured for 4 pm slope and probably even for 8 pm slope if the level of water consumption is significantly lower than assumed 4oo ℓ/cap,d.

Table 7.7.2 Cost components of FS at varying gradients

Slope	Laterals depth m	Laterals cost $1ooo	Mains depth m	Mains cost $1ooo	Trunk depth m	Trunk cost $1ooo	Man-holes $1ooo	Sewers total $1ooo	Mainte-nance
2 o/oo	1.1	6o5	1.3	81o	1.5	388	738	2541	1 truck
4 o/oo	1.3	715	1.5	984	2.o	482	849	3o3o	1 truck
8 o/oo	1.5	825	2.o	139o	3.o	743	988	3946	occati-
15 o/oo	2.5	1486	3.o	2489	5.o	123o	1122	6327	onal

Notes: Prices from appendix I, chapter 16
Estimates are made for 2oo p/ha and DWF=34o ℓ/p,d

Table 7.7.3 Total cost components of FS at small slopes

Slope	INVESTMENT			O&M		
	1)	1)	2)	1) After	2) Annual	
pm	3o year	1o year	5 year	1o year	supplies	labour
2	5327	1o	3o	243	3	17
4	5816	1o	15	243	2	15
8	6732	1o	-	243	2	15
15	9113	1o	-	243	2	15

Notes: 1) Cost of sewage treatment from table 7.4.2.2
Cost of house installations from table 7.2.1

2) One jet cleaning truck can service an estimated 6oo m sewer per day. Based on 2oo effective working days, per year a total of 12o km sewer line can be cleaned i.e. all main and trunk sewers can be cleaned twice a year with one jet cleaning unit in operation. It is estimated that 2 cleanings per year is adequate for 4 pm slope, whereas 2 units are recommended for 2 pm to increase the cleaning frequency and to ensure operation in case of a long term breakdown. Price per unit is (lifetime 5 years):

Vehicle	$ 1oooo + 5o% for spare parts
Supplies	$ o.7 per operating hour: $ 14oo/y
1 driver	$ 1ooo/y
2 labourers	$ 6oo/y

7.8 Cost sensitivity to slope of land

In the case of flat land conditions extra expenses are incurred due to increased investments that result from deep excavations and due to increased pumping of sewage.

In the previous sections a natural land slope of 3 pm was assumed. An estimate of the economical impact of other land slopes can be made using data that are already developed.

7.8.1 Pumping

In situations with no natural slope of the area, the re-

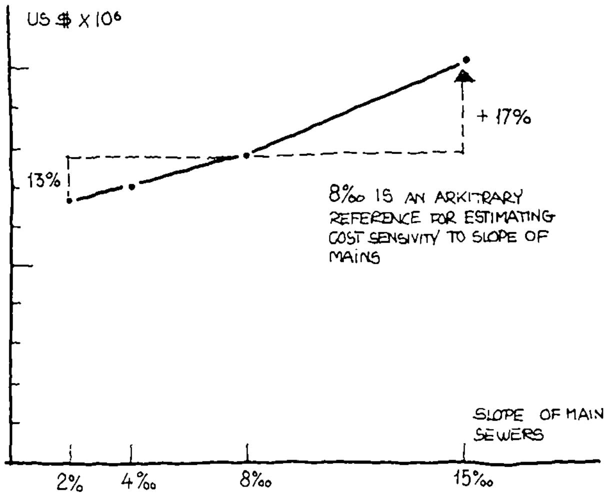


Figure 7.7.1 FS cost sensitivity to slope of main sewers. PV as based on 2oo p/ha DWF = 34o ℓ/p,d; i = 0%, and time horizon = 3o y; cf. cost data table 7.7.3

quired gradients of mains would greatly influence the amount of pumping. The trunk gradients would influence the total cost to a very limited extent (section 7.7). The necessary lift of sewage can be estimated from table 7.7.2 assuming that all sewage is collected by gravity down to the treatment plant and then pumped up in one operation (which is possibly not the most economical solution to the sewarage design, because deep excavation is relatively expensive, but it does give a fair estimate of total pumping cost involved). Using these data table 7.8.1.1 can be established.

Comparing the figures in table 7.8.1.1 with those in table 7.7.3 and figure 7.7.1 it appears that pumping costs will be negligible. The order of magnitude will be 2 - 3% of

Table 7.8.1.1 FS pumping of sewage for different gradients of mains, 1ooo $/y

Density cap/ha	Flow m^3/d	Mains' gradients & pumping lift, pm & m 4&2.5	8&4	15&7
125	196oo	3.o	4.8	8.3
2oo	269oo	4.1	6.5	11.5
4oo	464oo	7.1	11.3	19.8

Notes: Sewage 34o ℓ/p,d
Infiltration .3 ℓ/s,ha
Power .o4 $/kWh
Efficiency .6
The lift indicated is based on the average trunk depth, cf. table 7.7.2, + compensation for trunk slope and head loss through treatment plant.

total project cost for slope of mains in the range of 4 to 15 pm.

Tables 7.7.2 and 7.7.3 were established under the assumption that land slope averages 3 pm. It is now realized that changes of mains' gradients in the interval of 8 - 15 pm do not significantly change the fact that pumping costs are neglible in the context of total present value of sewerage. Consequently, in terms of pumping it would have no essential economical effect whether land slope changes from 3 pm to O pm. If land slope become greater than 3 pm the pumping expenses approach Zero and become even less important.

Pumping was not included in the previous cost sensitivity analysis, and this simplification seems justified.

7.8.2 Escavation

Escavation costs are of greatest importance to sewerage investment and total cost. This is particularly true for flat land and high ground water level. Further, it was realized in section 7.7 that cost is especially sensitive to gradients of main sewers.

Therefore, if the land slope exceeds that required for main sewers there would be possibilities for savings in the order of magnitude of 35% as demonstrated on figure 7.7.1. The reference used for figure 7.7.1 is a land slope of 3% but this does not significantly change the fact demonstrated, namely that great slope requirements become very costly (due to escavation) unless the land as such offers the necessary slope.

Deep excavation could be avoided by installation of pumping stations which is common practice in areas with flat slope. Savings in excavation are then counteracted by extra expenses for installation and maintenance of pumping stations though possibly with a net saving with present prices on energy supply. This consideration will not be carried further in the present report.

The conclusion is that land slope available should be compared to main sewer slope requirements. A price difference of 3o - 5o% might easily be realized when comparing sewerage at a gently sloping to a flat development area.

7.9. Cost effects of road pavement

If a sewer project should be installed in an existing city the price of sewer construction would include extensive road repair. In order to appreciate the magnitude of this potential extra cost a rough estimate has been developed below.

The price of removing and replacing pavement is estimated to $ 37/m sewer, /23/.

If the main sewers are placed in roads, rather than on the private plots the potential extra expenditure is:

looooo at \$ 37/m = \$ 3.7 x lo^6 which represents approximately 4o% of the project cost.

If the main sewers are placed off the roads where possible the total length of sewerline in roads is only 4oooo m which represents \$ 1.5 x lo^6 or 15% of the project cost.

7.lo. Preliminary FS evaluations

In order to compare the full sewerage alternative to other alternatives (chap. 8 - 11) the following representative parameters have been chosen:

Water consumption 25o ℓ/cap x day
Average gradient of main sewers 8 pm
Average slope of area 3 pm

Data are extracted from tables in the preceding chapters and summarized in tables 7.lo.1 & 2 below.

Table 7.lo.1 Cost components of FS system

Density cap/ha	1) Laterals \$ 1ooo	2) Main sewers \$ 1ooo	3) Trunk sewers \$ 1ooo	Manholes \$ 1ooo	Total sewer \$ 1ooo	4) Maintenance
125	825	139o	677	988	388o	Regular mainten-
2oo	825	139o	7o5	988	39o8	ance,
4oo	825	139o	743	988	3946	1 truck

Notes: 1) Average depth for laterals 1.5 m
2) Average depth for mains 2.o m
3) Average depth for trunks 3.o m
4) Routine maintenance is preferred rather than the services of a contractor, because the waste flow may not be as high as assumed in the design.

Table 7.10.2 Summary of the cost of FS

Density cap/ha		INVESTMENT, $ 1000 30 yr.	10 yr.	5 yr.	O&M, $ 1000 After 10 yr.	Annual supplies	labour
	PI	2300					
125	C	3880		15		1	2
	T	292	8		146	1	12
	PI	2300					
200	C	3908		15		1	2
	T	450	8		225	1	13
	PI	2300					
400	C	3946		15		1	2
	T	866	10		433	1	16

Table 7.10.3 PV for FS, 1000 $

Density cap/ha		0% Inv.	O&M	5% Inv.	O&M	15% Inv.	O&M
	PI	2300	-	2300	-	2300	-
	C	3970	90	3933	46	3909	20
125	T	316	682	308	201	302	131
	SUBTOT.	6586	772	6541	247	6511	151
	TOT.	7358		6788		6662	
	PI	2300	-	2300	-	2300	-
	C	3998	90	3961	46	3937	20
200	T	474	870	466	438	460	162
	SUBTOT.	6772	960	6727	484	6697	182
	TOT.	7732		7211		6879	
	PI	2300	-	2300	-	2300	-
	C	4036	90	3999	46	3975	20
400	T	896	1376	886	690	879	246
	SUBTOT.	7232	1466	7185	736	7154	266
	TOT.	8698		7921		7420	

Table 7.1o.4 FS annual O&M, labour employment

Density cap/ha	Labourers employed no	Cost supplies $ 1ooo	Cost wages $ 1ooo
125	19	2	14
2oo	21	2	15
4oo	26	2	18

Table 7.1o.5 PV per capita for FS, $ total

Density	O%		5%		15%	
cap/ha	Tot.	O&M	Tot.	O&M	Tot.	O&M
125	21o	22	194	7	19o	4
2oo	138	17	129	9	123	3
4oo	76	13	69	6	65	2

Table 7.1o.6 FS user charge, 3o years, $/y

Density	O%		5%		15%	
cap/ha	plot	cap.	plot	cap.	plot	cap.
125	22	7	39	13	9o	3o
2oo	23	5	42	8	93	19
4oo	26	3	46	5	1oo	1o

Note: Figures are derived from table 7.1o.3 totals using the criteria developed in chapter 6

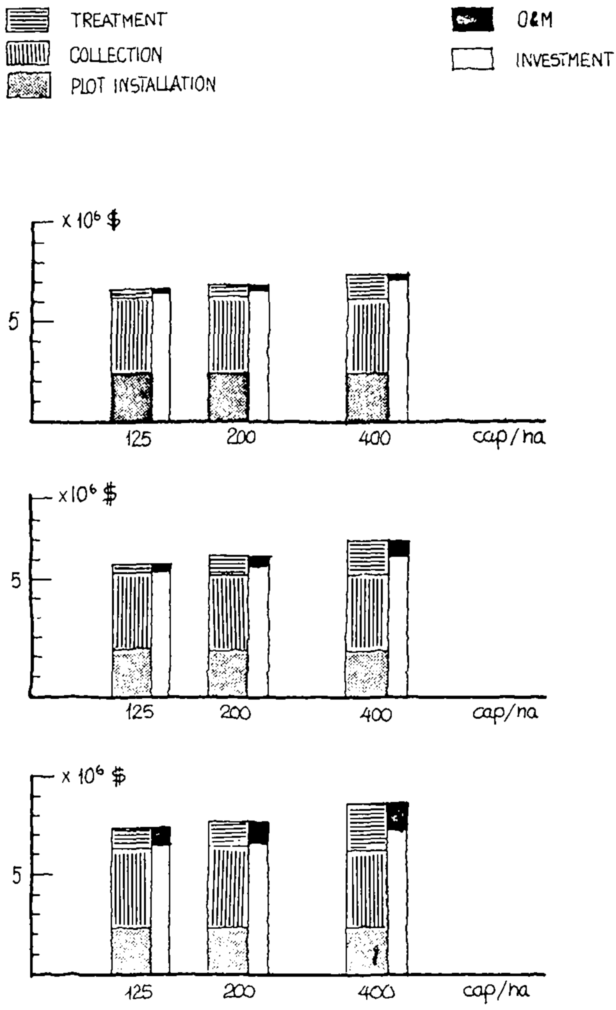


Figure 7.1o.1 Differentiated cost of FS, PV results for 3o y planning horizon. Cf. table 7.1o.3

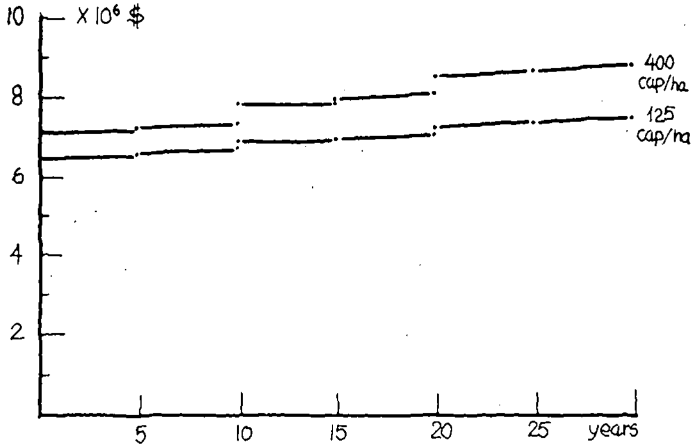


Figure 7.lo.2 FS, accumulated total cash flow at i = 0%

8. AQUA PRIVY WITH PIPED LIQUID DISPOSAL

8.1. Design considerations

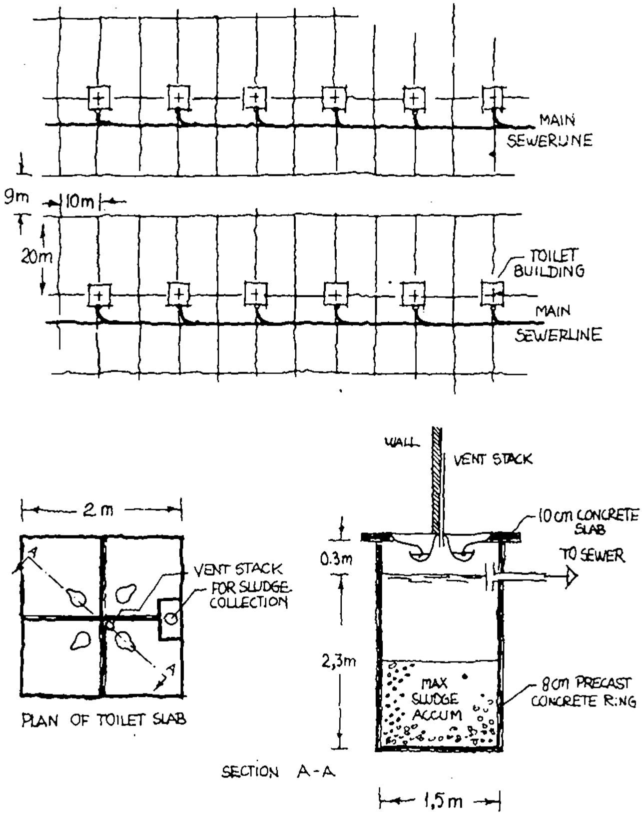
This scheme proposes that the toilet buildings for 4 plots are located adjacently as shown on figure 8.1.1 below. Thereby the need for laterals has been eliminated. A settling tank is installed underneath each toilet module to allow the sewer system that receive the overflow to be designed with a very small gradient. Latrine waste, sink waste water, shower water and kitchen waste water is led to the tank. The combination of shower and toilet slab in the same privy compartment would be a convenient option for the plot owner. This combination of toilet & shower is being used for example in Thailand. The tank accumulates settleable solids, whereas the supernatant overflows to the sewer. The mean solids residence time is 1 - 3 month, so considerable anaerobic microbial liquefaction of organic solids will take place. The sludge is collected by vacuum truck on a regular service basis e.g. biannual emptying and disposed into the primary ponds at the treatment works. Needless to say, the success of this scheme depends on the ability of the local authority to operate the maintenance scheme.

8.2. Plot installation, design & cost

The sludge accumulation in privies may be estimated as follows:

Settleable suspended solids = 9o g/cap,d /22/
Microbial activity effects 5o% solids reduction.
The annual sludge accumulation (6% solids) is then
o.5 x 9o x 1/1ooo x 365 x 1oo/6 = 275 ℓ/cap,y.
The hydraulic detention time is in excess of 24 hrs.

Cost estimations are then carried out in the following tables 8.2.1 & 8.2.2.



<u>Figure 8.1.1</u>   Aqua privy, details and plot situation

8.3. Sewage collection

The collection "main" sewer line is aligned alongside the row of toilet buildings and the privy overflow connects to the sewer with a 45° junction piece; figure 8.1.1.

The privies will retain sand and heavy organic material thus only a minimum maintenance will be required for sewers in spite of the low gradient (4 pm). In this alternative it is

Table 8.2.1 Design of aqua privy

Density cap/ha	1) cap/unit	2) Emptying no/mo	3) Estim. sludge accum. m^3	3) Required tank volume m^3	4) Cost per unit $	5) Cost of add. sink $
125	12	6	1.7	4.5	45o	5o
2oo	2o	4	1.8	4.5	45o	5o
4oo	4o	2	1.8	4.5	45o	5o

Notes: 1) lo aqua privy units per ha (one per 4 plots)
2) Tanker truck capacity 2.o m^3
3) Liquid volume 4.o m^3, see design figure 8.1.1
4) Cost includes tank, cover slab with 4 CM squatting plates, connection to the main sewer and a 4 m tall vent stack. A shower head is located in each stall but the price of this installation is exclusive.
5) A sink for washing clothes and utensils is installed, and the drain led to the privy tank.

Table 8.2.2 Maintenance of Privies

Density	1) Emptyings	2) Trucks required	3) Cost of trucks 5 yr life	Annual O&M	
				4) Supplies	5) Labour
cap/ha	No/y	No	$ 1ooo	$ 1ooo	$ 1ooo
125	56oo	4	6o	4	9
2oo	84oo	6	9o	7	13
4oo	16ooo	12	18o	14	26

Notes: 1) Based on a total of 28oo aqua privies
2) 1 privy emptying per truck load.
7 privies serviced per 8 hour day. (1 hour round trip per privy visit and 1 hour start up and close down).
25o working days per year less 2o% for breakdown and service. I.e. 14oo privy services per truck per year.
3) Price of one truck is $ 1oooo + 5o% for spare parts during the 5 year lifetime.
4) Supplies for trucks is $ o.7 per operating hour
5) Per truck 1 Driver 1ooo $/y
2 Labourers 12oo $/y

Table 8.3.1 Design of AP main sewers

Population density cap/ha	1) Design flow ℓ/sec	Required diameter mm	Capacity at a slope of 4 pm ℓ/sec	2) Sewer maintenance
125	1.2	1oo	3.5	minimum
2oo	1.8	1oo	3.5	-
4oo	3.3	15o	1o	-

Notes: 1) 5 DWF + I based on a wasteflow of DWF og 1o5 ℓ/cap x day and infiltration I = o.3 ℓ/sec, ha and a maximum drainage area of 1.25 ha.
2) Cleaning of occational blockings by a contractor.

Table 8.3.2 Cost of AP sewage collection

Population density cap/ha	Cost of 1) mains $ 1ooo	2) trunks $ 1ooo	manholes $ 1ooo	3) Total sewerage $ 1ooo	4) O&M annual supplies $ 1ooo	labour $ 1ooo
125	695	46o	35o	15o5	1	2
2oo	695	46o	35o	15o5	1	2
4oo	868	47o	35o	1688	1	2

Notes: 1) Total length of main sewers 5789o m. Slope 4 o/oo i.e. average depth 1.5 m at \$ 12/m for Ø 1oo mm and \$ 15/m for Ø 15o mm
2) Average depth 2.o m
3) 3o y lifetime
4) Cleaning of occational blockings

imperative that plastic pipes are used for sewers to avoid corrosion damages. Should future development require that toilets are transferred into the houses, then the system can be adapted to this situation. Flush toilets are easily connected to the privy. (The tank can also easily be converted to a regular manhole should future increased inflow favourize a regular sewer system rather than an aqua privy system. In this latter case considerable sewer maintenance would be required in the light of the flat slope at which the collecting sewer is designed, and generally this modification may not be possible without changing the piped network drastically.

8.4. Sewage treatment facilities

Sewage treatment is carried out in a system of facultative oxidation ponds. The sludge that has been collected from the tanks is to be disposed into the primary ponds. No net BOD reduction is taken into account from the short retention in the tanks because the organic acids - that are formed from microbial liquefaction of organic solids in the tank - will not be degraded to any large extent until the liquid phase reaches the treatment ponds.

Design and cost considerations appear from table 8.4.1

8.5. Preliminary AP evaluation

Basic data for further comparison between AP and other systems are presented in tables 8.5.1 - 5 and figures 8.5.1 & 2.

Table 8.4.1 Design and cost of oxidation ponds for AP

Population density	1) Hydr. load	2) Infl. qual. BOD	Total deten- tion	Pond area	3) Cost of ponds	4) Cost of station. equipm.	5) lo yr. O&M	6) Annual O&M supplies	labour
cap/ha	m^3/day	mg/ℓ	d	ha	$ 1ooo	$ 1ooo	$ 1ooo	$ 1ooo	$ 1ooo
125	7471	265	12.8	6.8	258	lo	129	1	11
2oo	9726	325	14.9	lo.3	392	lo	196	1	12
4oo	15741	4oo	17.6	19.6	745	15	372	1	16

Notes: 1) Waste flow = lo5 ℓ/cap x day + infiltration I = o.15 ℓ/sec, ha
2) The collected sludge is disposed into the primary ponds, so the organic load is 55 g BOD/cap,d (see chapter 7)
3) Based on $ 38ooo per ha pond (chapter 7)
4) Equipment for sludge disposal and truck cleaning, lo year lifetime is assumed.
5) Dredging of ponds after lo years of operation (5o% of construction cost)
6) 4 foremen at $ 12oo/y
8 labourers at $ 6oo/y
Site upkeep 1 labourer at $ 6oo/2 ha pond

Table 8.5.1 Summary of AP cost components

Density cap/ha	Cost item	INVESTMENT, $ 1000 30 y.	10 y.	5 y.	O&M, $ 1000 after 10 y.	annual supplies	annual labour
	PI	1400					
125	1) C	1505		60		5	11
	T	258	10		129	1	11
	PI	1400					
200	C	1505		90		8	15
	T	392	10		196	1	12
	PI	1400					
400	C	1688		180		15	28
	T	745	15		372	1	16

Note: 1) Sewers + sewer and privy maintenance.

Table 8.5.2 PV of AP system, 1000 $

Density cap/ha	Cost item	i = 0% Invest $ 1000	i = 0% O&M $ 1000	i = 5% Invest $ 1000	i = 5% O&M $ 1000	i = 15% Invest $ 1000	i = 15% O&M $ 1000
	PI	1400	-	1400	-	1400	-
125	C	1865	480	1718	246	1623	105
	T	288	618	278	312	271	119
	SUBTOTAL	3553	1098	3396	558	3294	224
	TOTAL	4651		3954		3518	
	PI	1400	-	1400	-	1400	-
200	C	2045	690	1825	354	1681	-
	T	422	782	412	394	405	146
	SUBTOTAL	3867	1472	3637	748	3486	297
	TOTAL	5339		4385		3783	
	PI	1400	-	1400	-	1400	-
400	C	2768	1290	2327	661	2041	283
	T	790	1255	775	630	765	227
	SUBTOTAL	4958	2545	4502	1291	4206	510
	TOTAL	7503		5793		4716	

Table 8.5.3 Annual O&M, labour employment for AP

Density cap/ha	Labourers employed No.	Cost Supplies $ 1ooo	Cost Wages $ 1ooo
125	16	6	22
2oo	18	9	27
4oo	23	16	44

Table 8.5.4 PV/cap for AP

Pop. density cap/ha	i=0% Tot	i=0% O&M	i=5% Tot	i=5% O&M	i=15% Tot	i=15% O&M
125	133	31	113	16	lol	6
2oo	95	26	78	13	68	5
4oo	66	22	51	11	41	4

Table 8.5.5 User charge for AP

Pop. density cap/ha	i=0% plot	i=0% cap	i=5% plot	i=5% cap	i=15% plot	i=15% cap
125	14	5	23	7	47	16
2oo	16	3	25	5	51	lo
4oo	22	2	33	3	64	6

Note: Figures are derived from table 8.5.2 using criteria from chapter 6 concerning constant annual expenditures for users

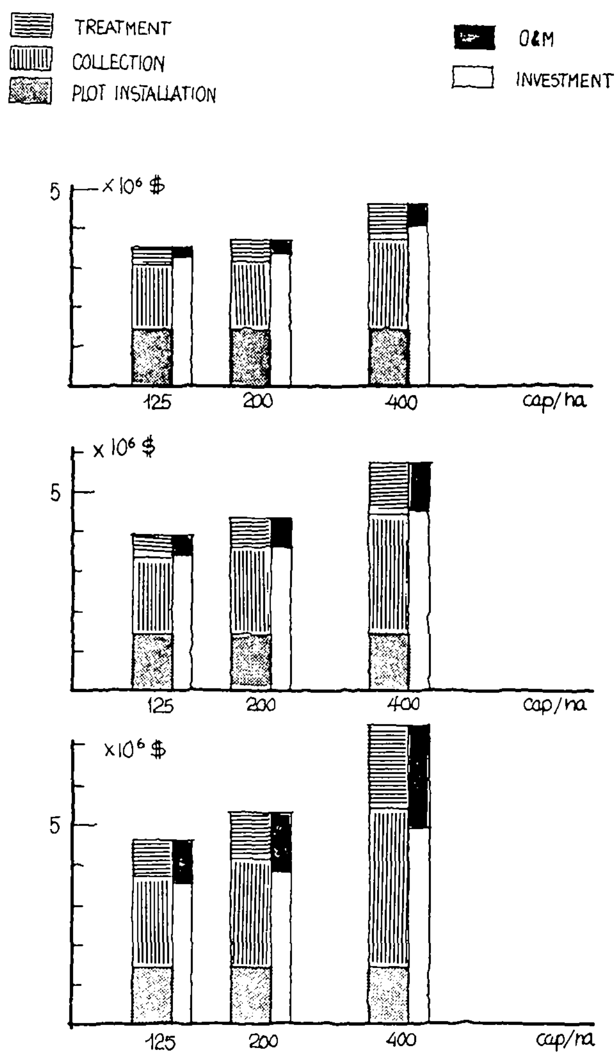


Figure 8.5.1 Differentiated total PV for the AP system 3o y planning horizon, cf. table 8.5.2

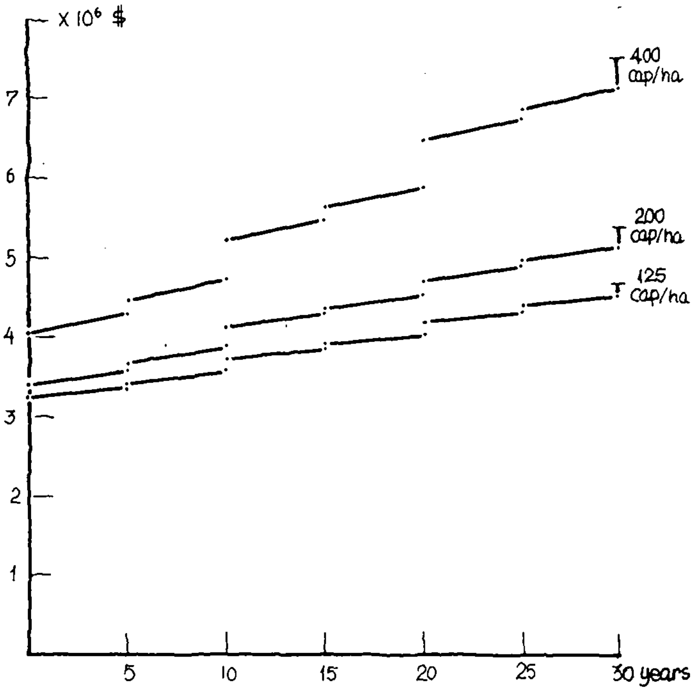


Figure 8.5.2 Accumulated total cash flow for AP at i = 0%

9. HOUSE VAULT & VACUUM TRUCK

9.1. Design & components

Basically the system consists of a house tank which is used for accumulation of only excreta from the users and which is emptied regularly by a vacuum truck, e.g. every 2 weeks. There are two modifications, cf. figure 9.1.1.

a) Chiang Mai Squatting Plate with siphon, i.e. a water seal between privy and vault. The flush is manual and demands little water, e.g. o.8 ℓ/visit.

b) Japanese Stool with only a vertical chute between privy and vault. No flushing is applied.

Two families (plots) are supposed to share the same vault, but each family has its own privy above the vault.

These systems do not include disposal of wash water (grey waste), which is presumed disposed indiscriminately or into the storm water system.

9.2. Vaults

Information is given on vault capacities & costs in table 9.2.1. Based on the information in table 9.2.1 the price for vaults and basic privy installation is then summarized in table 9.2.2.

9.3. Vacuum trucks, Crews and Service areas

Truck prices are found in table 6.5.3.1. According to Japanese practice the 2 m^3 tank (1.8 m^3 normally used in Tokyo and suburbs) is the preferable one; it is very flexible in traffic and it does not require heavy duty pavement. The 2 m^3 tank will be used in subsequent calculations.

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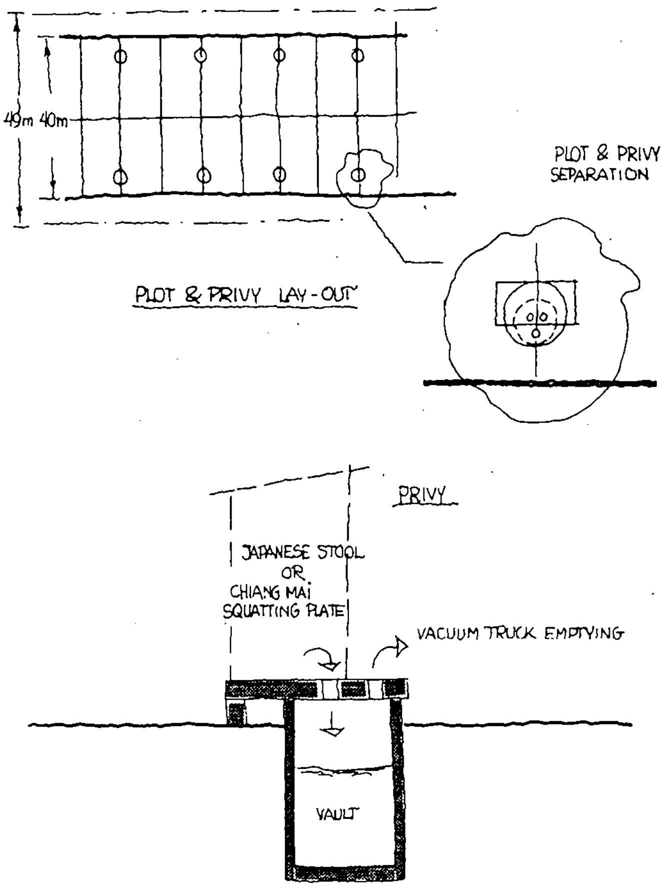


Figure 9.1.1 Plot & privy lay-out for JS & CM systems

The calculation of collection costs is indicated in table 9.3.2 and basic data are found in chapter 6.

9.4. Treatment ponds

Treatment of night soil from vaults will be done in oxidation

Table 9.2.1 JS & CM vault capacities and costs

System	Density cap/ha	3) Cap/vault no	4) Accul. vol. 2w m^3	3w m^3	5) Depth m	Diam. m	6) Cost $
1)	125	6	.12	.18	.7	.6	100
JS	200	10	.20	.30	1.1	.6	130
	400	20	.40	.60	1.2	.8	100
2)	125	6	.6	.9	1.2	1.0	150
CM	200	10	1.0	1.5	1.3	1.2	200
	400	20	2.0	3.0	1.7	1.5	300

Notes: 1) 1.4 liter/cap,d cf KAWASAKI /24/ and PRA /25/
2) 1.4 liter/cap,d + .8 ℓ flushing water/visit; 7 visits/day = 1.4 + 5.6 = 7 ℓ/cap,d
3) 2 families per vault
4) Collection every 2 or 3 weeks. Vault design for 3 weeks storage. Truck design for 2 weeks accumulation.
5) Depth of vault = necessary depth for storage during 3 weeks (50% margin with respect to emptying). Slab 0.1 meter above ground to avoid surface water infiltration.
6) Prices are estimated from chapter 7: Full sewerage. The cost includes stool or squatting plate but not superstructure for the privy.

Table 9.2.2 JS & CM, plot development costs, vaults & slabs.

System	Density cap/ha	Vaults/ha no	Cost/ha $	Total 1000 $
	125	20	2000	573
JS	200	20	2600	745
	400	20	3600	1031
	125	20	3000	859
CM	200	20	4000	1146
	400	20	6000	1718

Note: Total area = 286.4 ha

Table 9.3.1 Number of trucks necessary for JS & CM

System	Cap/ ha	1) Vaults per full truck	2) 3) Round trips	2) Vaults per 2 w per truck	4) Area served by one truck	5) Trucks & crews
	no	no	no/d	no	ha/2w	no
	125	17	1+	26o	13	26
JS	2oo	1o	2+	23o	11.5	3o
	4oo	5	4	2oo	1o	34
	125	3	5	15o	7.5	46
CM	2oo	2	6	12o	6	57
	4oo	1	8	8o	4	86

Notes:
1) cf. table 9.2.1, 2 week's accumulation
2) 2 weeks = 1o days of 8 hours = 8o hours
1 service necessary each 2 weeks = min. reg.
Vault service = 15 min including travel between two sites- Truck emptying = 45 min., including round trip. Start up and close down = 45 min. including travel to first and from last vault. The garage is located adjacent to the empt. station.
3) A truck is not necessarily full when returning last time during the day. This is accounted for in the number of vaults emptied and hectars served. In an actual situation the sizing and routing of trucks would be done through a simple operation research procedure. This would yield an optimal solution, whereby the area served would be greater than or equal to the one indicated above. A more detailed and refined approach is not justified at this preliminary stage of analysis.
4) 2o vaults/ha
5) Indicated in this coloumn is the theoritically necessary number + 2o% in order to give time for maintenance and repair (1 day out of each week). Also crews mechanics, shop labour etc are increased accordingly. Total area 286.4 ha.

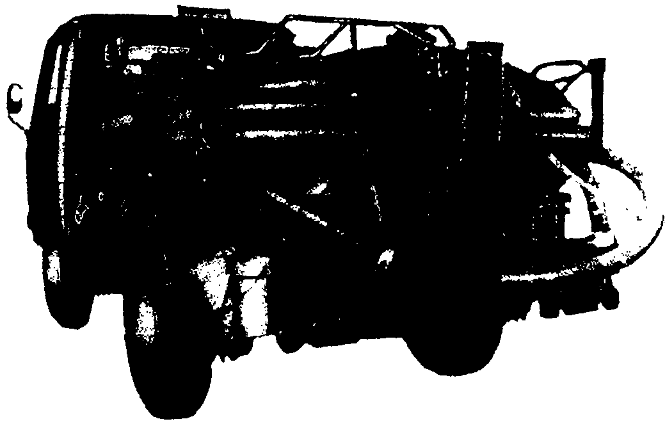


Figure 9.3.1 Vacuum truck. Used in Tokyo and suburbs for night soil collection. 1.8 m^3 tank capacity is very common. Courtesy: Tokyu Car Man. Co

ponds as proposed by SHAW /26/, who found that for a pond loading of 3ooo cap/ha and under South African climatic conditions the pond will function satisfactorily. Make-up water may be necessary depending on precipitation/evaporation ratios in which case no effluents will appear. Pumping will be required for proper stirring and mixing in the pond during discharge of new soil.

The ponds proposed in chapter 7 have a surface loading of app. 45oo persons/ha, i.e. somewhat higher than proposed by SHAW for night soil treatment under South African conditions. There is no information available to compare these loading rates under tropical conditions, but it is assumed for the present that previous calculations (chapter 7) apply also to night soil ponds. Maintenance and operation routines are assumed to be more labour demanding; an estimated 1oo% increase of O&M is used in subsequent calculations.

Table 9.3.2 Collection costs, trucks & labour for JS & CM systems

System	Density	1) Trucks & crews	2) Garages repair shops	3) Truck purch. & parts	Operation & Mainten. 4) Gas oil tires	5) Crew	Total annual O&M
	cap/ha	no	1ooo$	1ooo$	1ooo$	1ooo$	1ooo$
	125	26	13	39o	38	42	8o
JS	2oo	3o	15	45o	44	48	92
	4oo	34	17	51o	5o	54	1o4
	125	46	23	69o	67	74	141
CM	2oo	57	29	855	83	91	174
	4oo	86	43	129o	125	138	263

Notes:
1) A crew consists of 1 driver and 1 labourer. Wages are found in table 6.5.2.1 Truck numbers from table 9.3.1
2) Repair shops (shelters) and parking facilities are estimated at a total price of 1o $/m^2 and a need of 5o m^2/truck. Life time is estimated at 1o years.
3) Trucks have an estimated life-time of 5 years. 5o% has been added to the purchase price in order to make up for spare parts.
4) .7 $/hour of operation. 8o hours operation in 2 weeks gives app. 21oo hours/year,truck.
5) Ref. table 9.3.3. Also, note 5) in table 9.3.1 as to safety with truck and crew numbers.

9.5. Preliminary JS & CM evaluations

An economic evaluation of the night soil systems JS and CM is presented in tables 9.5.1 & 2.

Table 9.4.1 Night soil treatment costs for JS & CM systems

System	Density	1) Total pond surface	Invest, 1ooo $		O & M, 1ooo $	
			ponds 3o y life	pumps 1o y life	2) annual	3) each 1o y
	cap/ha	ha				
JS	125	8	31o	2o	26+1	16o
&	2oo	13	49o	3o	28+2	24o
CM	4oo	25	95o	4o	36+4	48o

Notes:
1) Cf. table 7.5.4; 45oo persons/ha pond
2) O & M is estimated to be twice the annual cost for sewage treatment, cf. table 7.5.4. Cost for extra pumping is added for cleaning of tanks, loading apron etc. Continous agitation of ponds is not considered necessary.

Table 9.5.3 Employment and annual O&M for JS & CM

System	Density	Labour employed	Trucks used	Cost	
				Suppl.	Wages
	cap/ha	no	no	1ooo $	1ooo $
	125	84	26	39	68
JS	2oo	97	3o	46	76
	4oo	117	34	54	9o
	125	124	46	68	1oo
CM	2oo	151	57	85	119
	4oo	221	86	129	174

Note: 1o-year pond remodeling is not included in O&M

Tables 9.5.1 & 2 Summary of Costs, PV for JS & CM.

Density	System	Cost item 1) 2)	Actual costs, 1000 $, 1975 $ & prices: Vaults ponds	Stat. equipm.	Mobile equipm.	O&M		Total PV in 1000 $: 30 year planning horizon given interest rate of		
cap/ha			30 y life	10 y life	5 y life	annual	each 10 y	0%	5%	15%
125	JS	PI	573	0	0	0	0	573	573	573
		C	0	13	390	80	0	4779	2641	1307
		T	310	20	0	27	160	1500	923	563
		Total	883	33	390	107	160	6852	4137	2443
	CM	PI	859	0	0	0	0	859	859	859
		C	0	23	690	141	0	8439	4663	2308
		T	310	20	0	27	160	1500	923	563
		Total	1169	43	690	168	160	10798	6445	3730
200	JS	PI	745	0	0	0	0	745	745	745
		C	0	15	450	92	0	5505	3042	1506
		T	490	30	0	30	240	1960	1449	800
		Total	1235	45	450	122	240	8210	5236	3051
	CM	PI	1146	0	0	0	0	1146	1146	1146
		C	0	29	855	174	0	10437	5767	2857
		T	490	30	0	30	240	1960	1449	800
		Total	1636	59	855	204	240	13543	8362	4803
400	JS	PI	1031	0	0	0	0	1031	1031	1031
		C	0	17	510	104	0	6231	3443	1705
		T	950	40	0	40	480	3230	2120	1434
		Total	1981	57	510	144	480	10492	6594	4170
	CM	PI	1718	0	0	0	0	1718	1718	1718
		C	0	43	1290	263	0	15759	8708	4312
		T	950	40	0	40	480	3230	2120	1434
		Total	2668	83	1290	303	480	20707	2546	7464

Notes: 1) Plot development includes vault & slab, not superstructure
2) Treatment does not include outfall constructions.

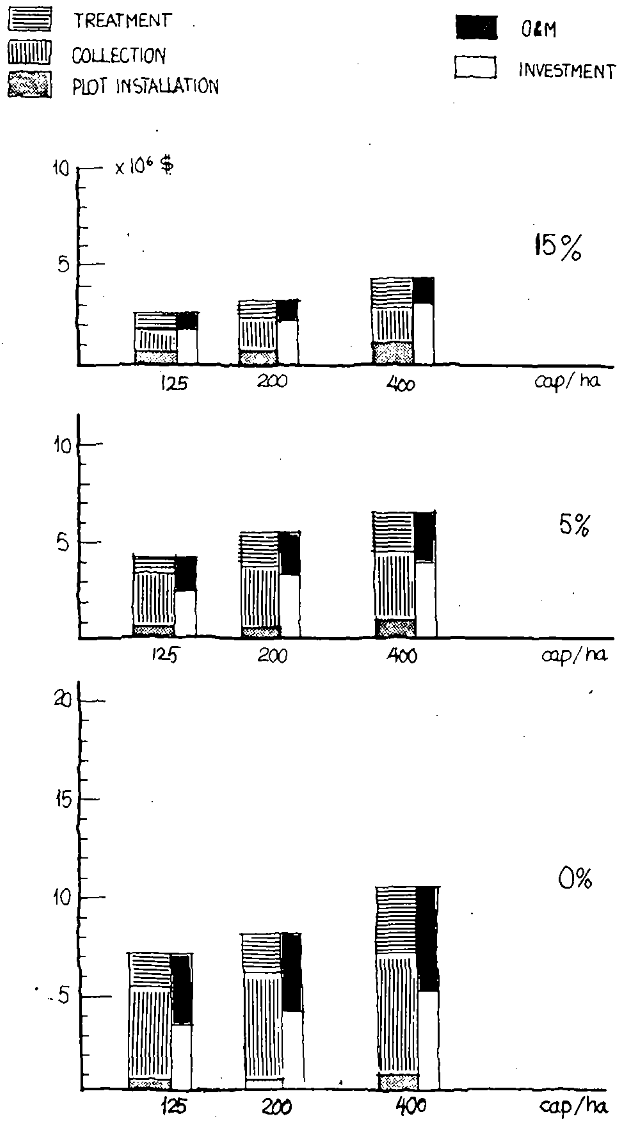
Table 9.5.4 PV for CM and JS systems

System	Density	0%		5%		15%	
	cap/ha	Tot.	O&M	Tot.	O&M	Tot.	O&M
JS	125	196	101	118	52	70	22
	200	147	74	94	38	54	16
	400	92	46	58	24	37	10
CM	125	309	153	184	78	107	33
	200	242	118	149	60	86	25
	400	182	88	110	45	65	19

Table 9.5.5 User and plot charges for JS & CM systems, $/y

System	Density	0%		5%		15%	
	cap/ha	plot	cap	plot	cap	plot	cap
JS	125	20	7	24	8	33	11
	200	24	5	30	6	41	8
	400	31	3	38	4	56	6
CM	125	32	11	37	12	50	17
	200	40	8	48	10	65	13
	400	61	6	72	7	101	10

Note: Values derived from figures 9.5.1 and 9.5.2 using criteria from chapter 6 concerning constant annual user expenditures.



<u>Figure 9.5.1</u> PV for JS system, 3o y planning horizon, cf. table 9.5.1

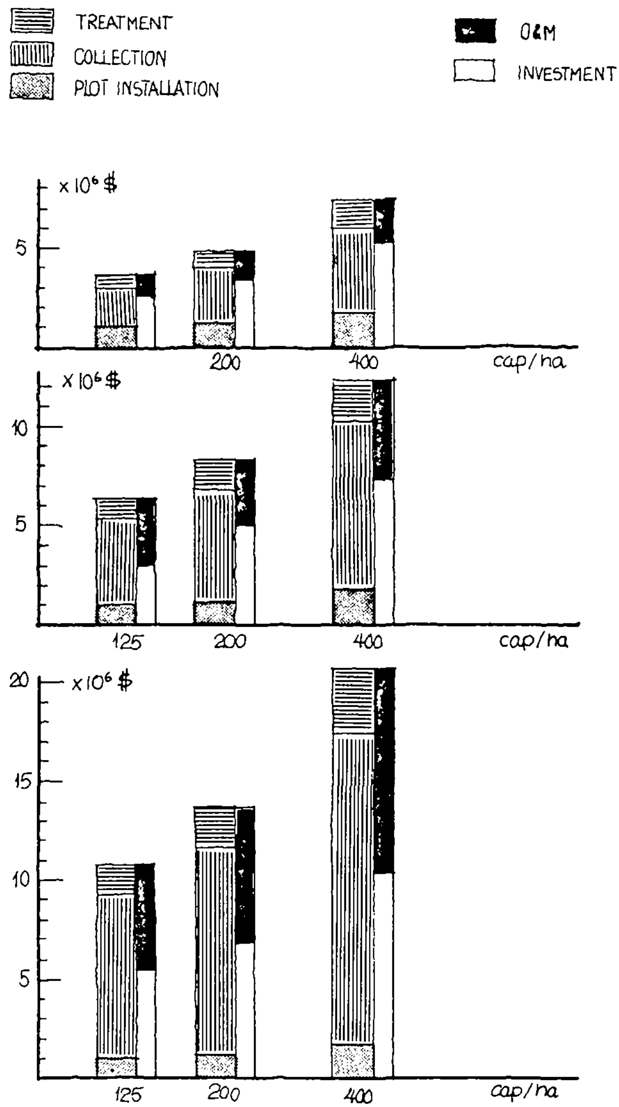


Figure 9.5.2 PV for CM system, 3o y planning horizon, cf. table 9.5.2

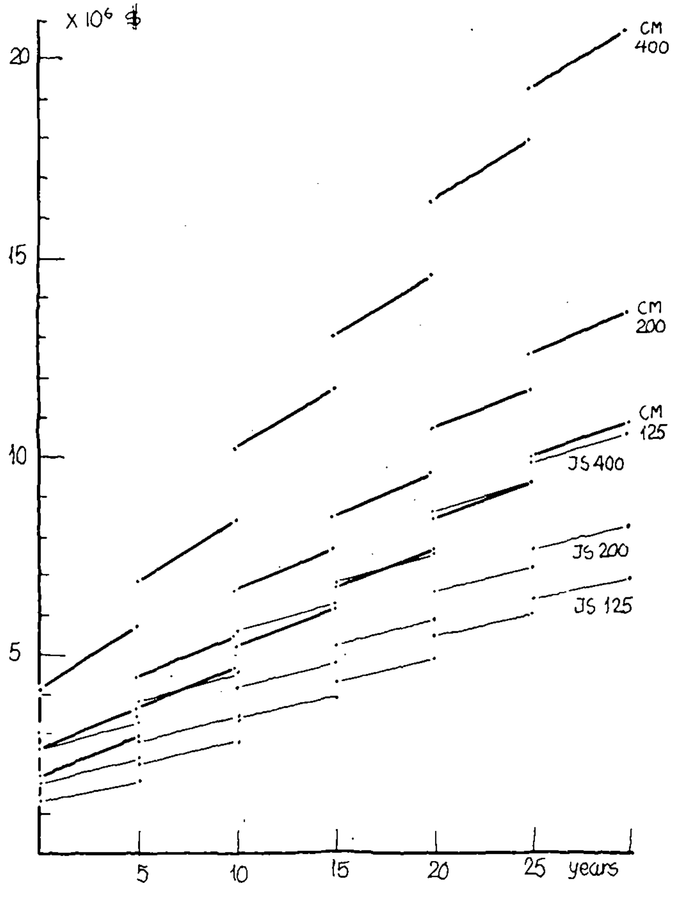


Figure 9.5.3 Accumulated total cash flow at i = O% for JS and CM systems

10. HOUSE BUCKET & ABLUTION BLOCK

lo.1. Design considerations

The proposed scheme introduces the following:

1. Each house has bucket latrine. A member of the household carries the bucket to an emptying and cleaning facility at most 15o m away (in the ablution block).
2. Ablution blocks serving 5oo people are built regularly spaced in the community - maximum distance from a home to the nearest ablution facility is 15o m, cf. figure lo.1.1. The blocks include stand pipes for drinking water, toilets, showers, facilities for washing clothes and utensils and an arrangement for emptying and cleaning the private latrine buckets. The whole ablution block is cleaned and maintained by full time employed attendants.
3. Individual houses are not provided with piped water or piped drainage.

In the context of this report drainage facilities alone are considered; however the superstructure for the ablution block has been included in the cost estimates, because the structure is a public facility in this design, contrary to what is the case when the toilet and washing facilities are located on the individual plots.

lo.2. Ablution block

Tables lo.2.1 and lo.2.2 summarize the data for the ablution block system.

lo.3. Sewage collection system

The collection system includes sewers from all ablution blocks inclusive of manhole adjacent to the buildings. Maintenance of the pipe system is included. Sewers are designed

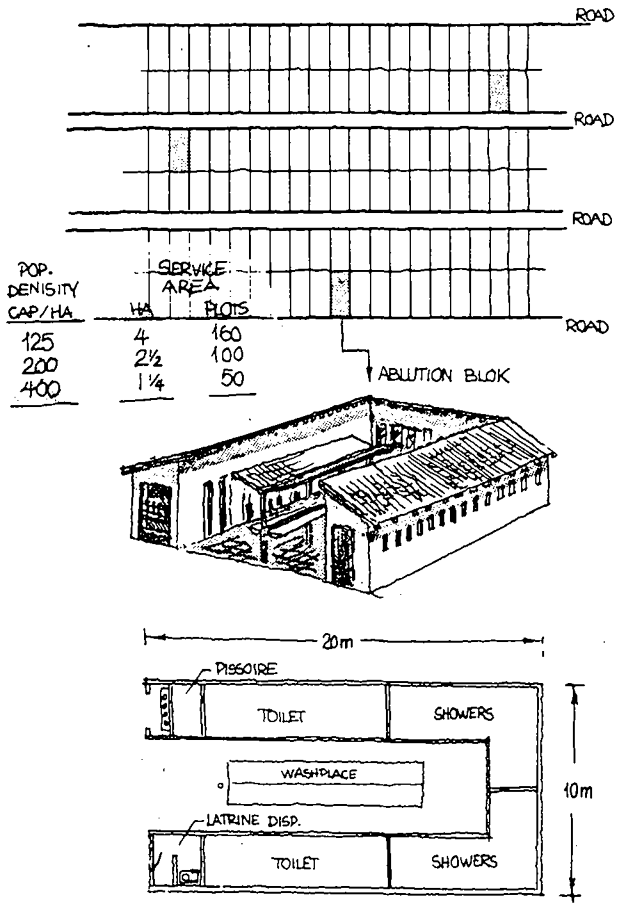


Figure lo.1.1 House Bucket & Ablution Block, system AB, lay-out and details

for flat slopes (4 pm) and a high level of maintenance (regular cleaning). The intermittant and high water usage at the facility also provides a good condition for selfcleaning of the collection system.

Collection costs are calculated and presented in table lo.3.2.

Table lo.2.1 AB specifications

Facility	Capacity cap/unit no	Area per unit m^2	Required units no	Area per AB m^2
Toilet	5o	1.5	lo	15
Pissoire	25o	5.	2	lo
Wash places	2o	2.	25	5o
Showers	25	2.	2o	4o
Latrine disp. & bucket clean.	25o	5.	2	lo
Extra space				25
Total floor area required per ablution block:				15o

Table lo.2.2 Cost of sanitary installations per ablution block.

Facility	INVESTMENT 3o yr $	lo yr $	5 yr $	O&M Supplies $/y	Labour $/y
1) Latrine bucket			16oo		
			1ooo		
			5oo		
2) Slabs, drains & toilets	9ooo				
3) Super structure	6ooo				
Maintenance and cleaning				4) 5oo	5) 18oo

Notes: 1) One bucket at $ lo per plot, price for the respective population densities; cf. figure lo.1.1

2) Includes concrete slab with drains, toilets and all internal drain pipes at $ 6o/m^2

3) Includes concrete walls, partitions, doors and a light sheet metal roof at $ 4o/m^2

4) Detergents & brooms

5) Attention 16 h/d, week and holidays
3 labourers at $ 6oo per year

Table lo.3.1 Design of sewers from ablution blocks

Density cap/ha	Service area per block ha	1) Spacing of sewers Ø 15o mm	2) Max. flow ℓ/s	Pipe cap. at 4 o/oo ℓ/s	3) Cleaning frequency per yr.
125	4	4th road	1.8	lo	6-8 times
2oo	2.5	2nd road	1.8	lo	4-6 times
4oo	1.25	2nd road	3.6	lo	4-6 times

Notes: 1) Minimum size to avoid blockings
2) 6DWF + I, based on DWF = 5o ℓ/p,d and I = o.1 ℓ/s, ha. If water is free of charge the flow is likely to increase to 75-1oo ℓ/p,d. The drainage system is adequate to accomodate this flow.
3) One sewercleaning vehicle in continous operation 2oo operating days per year. One unit is estimated to service 6oo-9oo m sewer per day.

lo.4. Sewage treatment system

Facultative oxidation ponds are used for sewage treatment. Comminuters are installed at inlet works. The treated effluent is disposed into the brackish Ebute Metta Creek.

Tables lo.4.1 & 2 give data on pond design and sewage treatment.

lo.5. Preliminary AB evaluations

Basic data for further comparison between AB and other systems are presented in tables lo.5.1 - 5.

Table 1o.3.2 Cost of sewage collection for AB

Density	Blocks	Sewers from blocks		Trunk sewers 2)	Manholes 3)	Sewers total	Trucks 4)	O&M	
			1)					5) Supplies	6) Labour
cap/ha	no	m	$ 1ooo	$ 1ooo	$ 1ooo	$ 1ooo	$ 1ooo	$ 1ooo	$ 1ooo
125	7o	1o9oo	164	44o	3o	634	15	1	2
2oo	115	174oo	261	45o	42	753	15	1	2
4oo	23o	174oo	261	46o	49	77o	15	1	2

Notes: 1) Ø 15o mm sewer at average depth 1.5 m at $ 15/m
2) Average depth 2.o m, min. size for trunk sewer is Ø 25o mm
3) 1 type I manhole at $ 1oo per block
1 type II manhole at $ 3oo per connection to trunksewer + 5 additional
4) Price of 1 sewer cleaning unit is $ 1oooo + 5o% for spare parts during 5 yr lifetime
5) Supplies are estimated to $ o.7 per operating hour
6) 1 driver at $ 1ooo/y and 2 labourers at $ 6oo/y, cf. chapter 6.

Table lo.4.1 Sizing of treatment ponds for AB system

Density cap/ha	1) Hydraulic load m^3/day	2) Influent quality mg/ℓ BOD_5	3) Total detention $time_d$	Total surface area ha
125	4264	461	19.7	6
2oo	5338	59o	24.3	lo
4oo	82o2	77o	3o.7	18

Notes: 1) Based on an estimated DWF = 5o ℓ/cap,d + infiltration I = o.1 ℓ/sec, ha
2) Organic load 55 g BOD/cap,d (see chapter 7)
3) Assumed min. month temperature 22 °C; design procedure is indicated in chapter 7

Table lo.4.2 Cost of sewage treatment for AB system

	INVESTMENT		O&M		
Density cap/ha	Pond construc. 3o y life 1) $ 1ooo	Comminuters lo y life $ 1ooo	lo y 2) $ 1ooo	annual suppl. $ 1ooo	3) labour $ 1ooo
125	228	5	114	1	lo
2oo	38o	5	19o	1	11
4oo	684	8	342	1	13

Notes: 1) Based on $ 38ooo/ha pond surface
2) Dredging of ponds after lo years at 5o% of construction cost.
3) 4 foremen at $ 12oo/y
6 attendants at $ 6oo/y
Site upkeep 1 labourer/2 ha pond

Table lo.5.1 Cost components for AB system

Density cap/ha	Cost item	INVESTM., $ looo life 3o y	lo y	5 y	O&M, $ looo After lo y	Supplies	Labour
	PI	lo5o	-	112	-	35	126
125	C	634	-	15	-	1	2
	T	228	5	-	114	1	lo
	TOTAL	1912	5	127	114	175	
	PI	1725	-	115	-	57	2o7
2oo	C	753	-	15	-	1	2
	T	38o	5	-	19o	1	11
	TOTAL	2858	5	13o	19o	279	
	PI	345o	-	115	-	115	414
4oo	C	77o	-	15	-	1	2
	T	684	8	-	342	1	13
	TOTAL	49o4	8	13o	342	546	

Table lo.5.2 PV for AB system, 1975 $,
3o y planning horizon

Density cap/ha	Cost item	i=0% Invest $ looo	i=0% O&M $ looo	i=5% Invest $ looo	i=5% O&M $ looo	i=15% Invest $ looo	i=15% O&M $ looo
	PI	1722	483o	1448	2475	127o	lo58
125	C	724	9o	687	46	663	2o
	T	243	558	238	282	235	lo8
	SUBTOT.	2689	5478	2373	28o3	2168	1186
	TOTAL	8167		5176		3354	
	PI	2415	792o	2133	4o58	195o	1734
2oo	C	843	9o	8o6	46	782	2o
	T	395	74o	39o	373	387	138
	SUBTOT.	3653	875o	3329	4477	3119	1892
	TOTAL	124o3		78o6		5oll	
	PI	414o	1587o	3858	8131	3675	3476
4oo	C	86o	9o	823	46	799	2o
	T	7o8	llo4	7oo	554	694	198
	SUBTOT.	57o8	17o64	5381	8731	5168	3694
	TOTAL	22772		14112		8862	

Table lo.5.3 Annual O&M for AB system

Pop. density cap/ha	Labour empl. no	Trucks used no	Cost Suppl. 1ooo $	Cost Labour 1ooo $
125	226	1	37	138
2oo	362	1	59	22o
4oo	711	1	117	429

Note: Pond remodeling every lo years is not included in O&M

Table lo.5.4 PV/cap for AB system, US $/cap

Pop. density cap/ha	i=0% Tot.	i=0% O&M	i=5% Tot.	i=5% O&M	i=15% Tot.	i=15% O&M
125	233	157	148	8o	96	34
2oo	221	156	139	8o	89	34
4oo	2oo	151	124	77	78	33

Table lo.5.5 User and plot charges for AB system, $/cap,y or $/plot,y

Pop. density cap/ha	i=0% plot	i=0% cap	i=5% plot	i=5% cap	i=15% plot	i=15% cap
125	24	8	3o	lo	45	15
2oo	37	7	45	9	68	14
4oo	67	7	81	8	12o	12

Note: Values from table lo.5.2 using criteria from chapter 6 regarding constant annual user expenses.

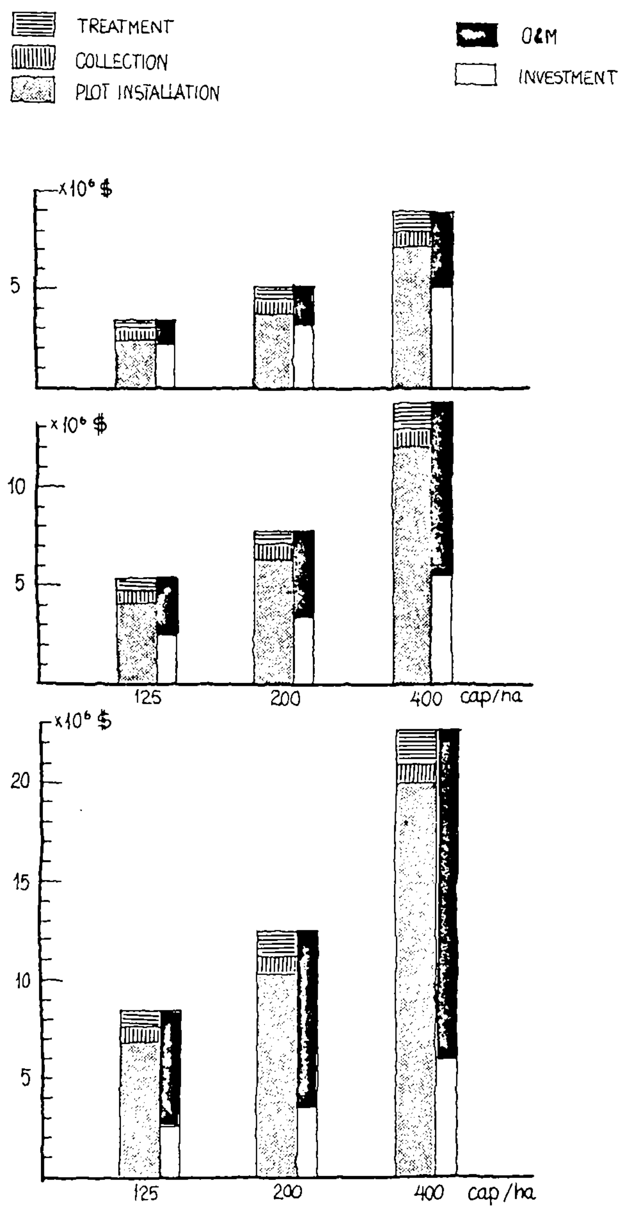
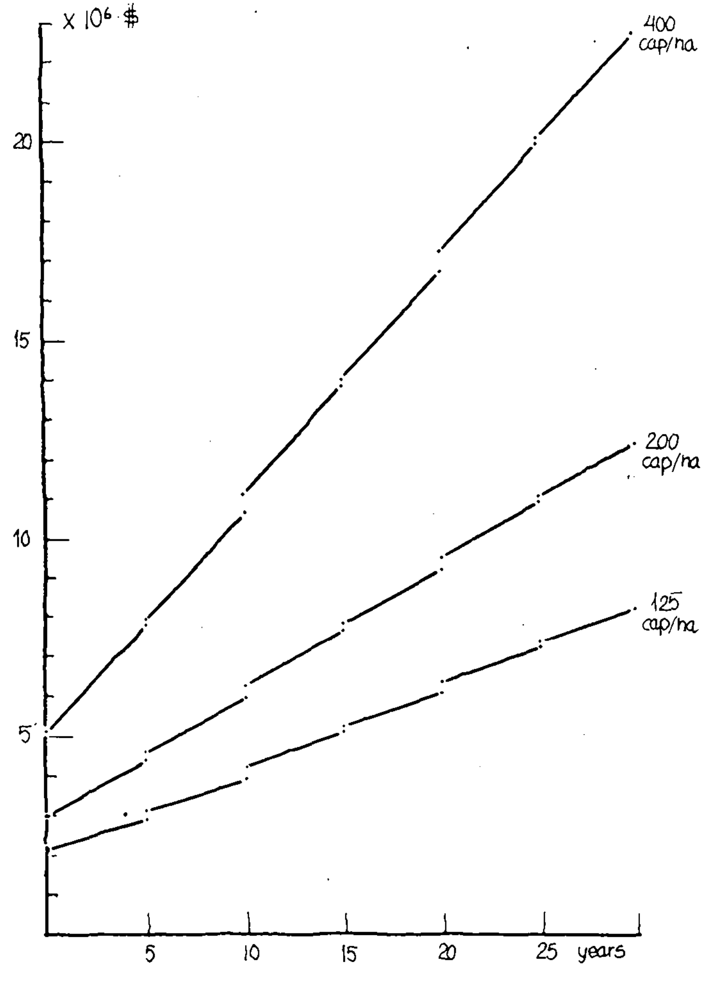


Figure 1o.5.1 Differentiated total PV of AB system, 3o y planning horizon, cf. table 1o.5.2



<u>Figure lo.5.2</u> AB system, accumulated total cash flow at i = O%

11. MULTRUM

The Multrum was proposed in 1969 as an excreta disposal unit for low-cost high-density housing in developing urban areas, see SCANPLAN /27 & 28/.

11.1. Stage of development

Figure 11.1.1 shows the Multrum set-up and the construction of a composting compartment directly underneath the toilet slab in the privy. The scale is indicated and shows that the compartment volume is approximately .8 m^3. The principles of the Multrum have been applied for many years in the Swedish "Clivus Multrum", but the Clivus system has a composting compartment volume of several cubicmeters. In Sweden the Clivus system is used primarily in remote dwellings or recreational housing areas, and often the use is intermittant. The Multrum with a small composting compartment and under tropical climatical conditions in high density housing areas is sofar unproven. The system is presently being investigated in field scale in Tanzania and Mali with international financial support, cf. IDRS /29/.

Comments as to feasibility of the Multrum concept can be only fragmentary and speculative at this stage of development and testing. Some questions and problems that are expected to be solved through the on-going African experiments are listed below:

1. Will the composting process work under the specified conditions?
2. Which dimensions and volumes should be applied for an optimal composting process to take place?
3. Will natural ventilation be adequate to supply oxygen for composting without drying out the excreta and thereby retard the microbial processes.

4. Will odeur and fly nuisance be controllable?

5. Can organic kitchen waste be introduced into the Multrum system as proposed in the Swedish Clivus system?

6. What is an environmentally safe routine for emptying the compost compartment and spreading it on the ground?

In addition to these general problems are specific questions regarding modifications related to local circumstances such as:

Temperatures
Humidity
Stagnant air
Poor maintenance
Surface-and ground water infiltration
Survival of pathogens and larvae of worms

Considering all these yet unanswered questions it is noted that the authors have severe reservations regarding the feasibility of the Multrum in a high-density urban development. But the system is included in the study because it may economically represent a lowcost extreme, that is useful for the overall evaluation of alternative sanitary systems. Also, the public attention devoted to this system in recent years would justify the discussion.

11.2. Plot development

The plot development is described without details because of the many unknowns mentioned above. Water supply is assumed to be through public stand pipes and there are no particular provisions included for waste water disposal on the plot.

Each plot must have been designated by one complete Multrum unit because proper use of the facility is critical to the success of the system.

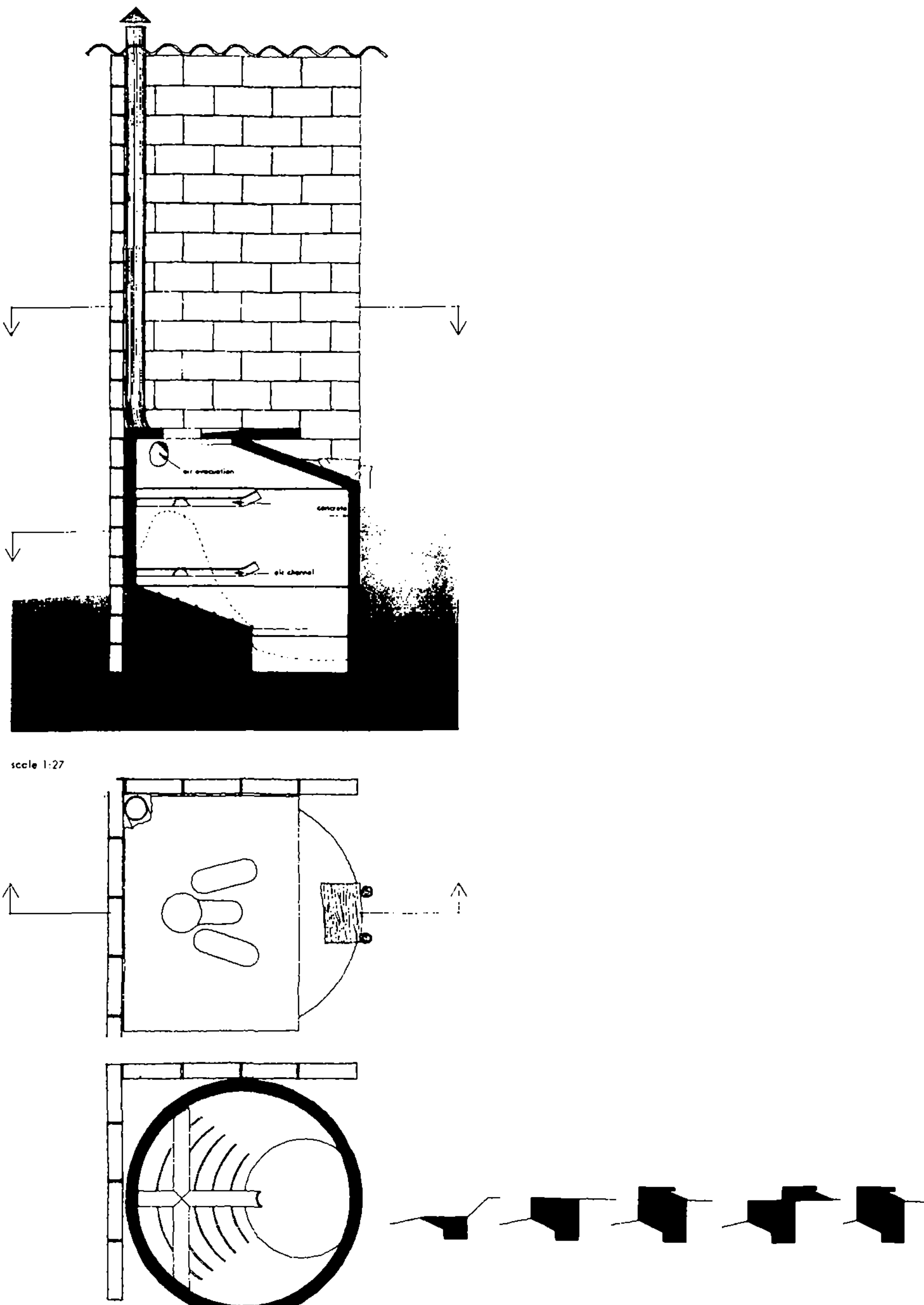


Figure 11.1.1 MR system concepts. Photocopy from WIINBLAD /27/

A rough estimate for the construction of the compost tank is developed. Based mainly on the indications of table 9.2.1 the following costs are assumed:

Pop. density	Compost tank
125 cap/ha	2oo $
2oo cap/ha	25o $
4oo cap/ha	3oo $

Pre-manufactured modular units would be provided for each plot.

There are no accounted expenses for waste water removal and treatment due to the on-plot disposal of compost. With a good quality concrete construction a 3o years life time of the multrum can be anticipated. Maintenance is limited to annual or biannual compost removal and spreading on the plot.

11.3. Preliminary MR evaluation

Basic data for further evaluation, cf. chapter 12, are developed in the following tables 11.3.1-3. Graphical presentation of these data is found only in chapter 12.

Table 11.3.1 PV for MR system, 3o year lifetime

Density cap/ha	i=0% $ looo	i=5% $ looo	i=15% $ looo
125	226o	226o	226o
2oo	2846	2846	2846
4oo	339o	339o	339o

Table 11.3.2 PV/cap for MR system

Density cap/ha	i=0% \$/cap	i=5% \$/cap	i=15% \$/cap
125	65	65	65
2oo	51	51	51
4oo	3o	3o	3o

Table 11.3.3 Annual user & plot charges for MR system.

Density	i=0%		i=5%		i=15%	
cap/ha	\$/plot	\$/cap	\$/plot	\$/cap	\$/plot	\$/cap
125	7	2.2	13	4.2	3o	1o
2oo	9	1.7	17	3.3	39	8
4oo	1o	1.o	2o	2.o	46	5

12. COMPARATIVE EVALUATION

Based on assumptions and calculatory results obtained for the five different sanitation systems, chapters 7 through 11, a comparative evaluation can be made.

It should be born in mind that the basic assumptions with respect to area, storm water, ground water level, slope of land, and population densities are identical for all five systems evaluated, cf. chapter 6. A few introductory remarks may be necessary:

1. For actual problems where the situation is similar to the one delineated through the above basic assumptions it may be relevant to use the results of this report for a relative consideration; e.g. compare economy and use terms like "% more expensive", etc.

2. For specific problems where the above basic assumptions do not apply properly the results should be used with extreme care. For example, if the ground water level is far below the surface the price of sewerage may be lower than indicated in this report.

3. Use of absolute values, e.g. $/p,y for full sewerage per se should be avoided. Chapter 6 presents the data used for wages, trucks etc., but these data are typical only for an "average developing country" that may resemble those of an African state. Each country or region has its own characteristics that must be assessed properly before absolute numbers can be derived.

12.1 Service levels, WS & WWR

A summarized comparison of the different systems with respect to water supply (WS) and wastewater removal (WWR) is presented in table 12.1.1.

The FS requirement for a minimum water consumption (approximately 5o ℓ/c,d) to secure proper function may severely limit the feasibility at an early stage of development. Only FS has such requirement for water consumption, because the settleables in this case are conveyed by the water to the treatment plant. On-plot water supply, preferably a multitap installation, is necessary.

None of the other WWR systems have this high water demand, which makes these very attractive where water is in short supply. Less water consumption may cause loss of convenience to the plot inhabitants depending on which sanitary system is chosen.

Assuming AP as a substitute for FS the loss of convenience to the plot family is fairly modest if recognizeable at all. It should be added that the AP design has provision for increased future water consumption. The biannual removal of sludge deposits from the privy tank will hardly cause inconvenience to the plot inhabitants.

Table 12.1.1 WS & WWR service levels

System	WS		WWR
	Type	Sanitary cons. ℓ/p,d	Removal mode, waste type
			1) 2)
FS	plot, multitap	5o (min.)	piped, grey & black
AP	plot	flex 4)	piped, grey & black
CM	plot or public	8	truck, only black
JS	public stand	o 5)	truck, only black
AB	public stand	flex	piped, grey & black
MR	public stand	o	on site, black & org.3)

Notes: 1) grey means waste water from cooking and washing etc.
2) black means human excreta
3) org. means organic waste, e.g. from cooking.
4) flex means flexible water consumption e.g. anything from 1oℓ/p,d and higher.
5) o means that no water is added to the excreta during the visit to the privy

JS does not require addition of water for handling of excreta. CM requires the addition of some 8 liters/p,d for flushing the siphon, but this modest amount does not require a plot tap, and the water needs not be high quality drinking water. A significant drop in service level should be acknowledged when proposing CM or JS systems in stead of FS or AP, because the former do not include provision for plot disposal of grey waste.

AB involves piped water supply via public standpipes and piped waste water removal, but in practice there will be no limits to the water consumption/capita. Grey waste will be generated and disposed of at the ablution block, which is a superior service compared to CM & JS. However, the overall service level of AB is inferior to that of FS or AP, because there is no provision for immediate plot disposal. It should emphasize that AB can be developed into a plot pipe connected system like FS or AP, if future development makes such change desireable.

The MR would require no water usage. Wet waste organics from food preparation may favourably be added to the composting tank, and among the six alternatives only MR will handle this particular waste flow. Similarly to JS & CM this system offers significantly less convenience to the plot inhabitants than the FS & AP alternatives. It is even debatable whether the service level is comparable to that of JS & CM, and only practical experience from actual use over a long period of time can clarify this question.

12.2 Health

The health evaluation criteria were introduced in chapter 6. Based on these concepts table 12.2.1 has been established. Some transmission routes are identified and examples of infectious diseases and their agents are listed.

Table 12.2.1 Health improvement following sanitary excreta removal at least public stand pipe water supply.

TRANSMISSION Route	Contraction	EXAMPLES Bio-agent	Disease	IMPROVED HEALTH INDICATION FS*	AP*	CM	JS	AB*	MR
	Ingestion	Salmonella	Typhoid	o	o	o	o	o	o
Water	Skinpenetration	Schistosomas	Schistosomiasis	+	+	+	(+)	(+)	(+)
		mans., jap.,h		+	+	+	(+)	(+)	(+)
Anus-hand-hand-man	Ingestion	Shigella	Dysentery	+	+	+	+	+	+
Insect-vectors	Breed in water	Anopheline	Malaria	+	+	+	+	+	+
		Culex p. fat.	Filariasis	+	+	+	+	+	+
		Aedes Aegypti	Yellow & Dengue fever	+	+	+	+	+	+
Food contamination	Ingestion	Salmonella	Salmonelloses	+	+	+	+	+	+
		Vibrio	Cholera	+	+	+	+	+	+
		Entamoebia	Amoebiasis	+	+	+	+	+	+
Man-soil-man	Dirt intake & food contamination	Ascaris	Round worm	+	+	+	+	+	?
		Trichuris	Whip worm	+	+	+	+	+	?
		Necator	Hook worm	+	+	+	+	+	?
Air	Eye & Nose	Aesthetical nuisance, e.g. odeur and unsightly conditions		+	+	+	+	+	?

Notes: o = no improvement) As compared to on the plot or
+ = significant improvement) other indiscriminate
- = impairment) disposal of non treated excreta
* = tap water available at sanitary site

The systems that are discussed herein would all significantly reduce the possibility of transmission of pathogens to man. Modification to this statement is necessary only for the Multrum, where evidence is still lacking with respect to technical feasibility and inactivation of pathogens.

Table 12.2.1 suggests that there is little if any difference between the systems in terms of improved health, compared to a situation with indiscriminate disposal. The indispensable prerequisite for this statement is that the systems are operated and maintained properly. Even though this requirement may be difficult to fulfil , it applies equally to all systems that an education programme is necessary, and that the task of education would tend to be similar for all systems.

Due to lack of data on operation the MR may turn out less satisfactory in terms of health protection compared to the other five systems.

Table 12.2.1 merely suggests qualitatively that health improvements are associated with the introduction of a sanitary waste disposal system. A more quantitative evaluation of the effect of introducing sanitary installations is found in a case study from the Philippines, WHO /3o/. Four different communities all within larger urban settlements were studied. Cholera was known to be a problem through existing records, and it was used as the model disease for the comparative study which is outlined briefly in table 12.2.2

Table 12.2.3 shows a significant drop in occurrences of cholera as a result of improved sanitary conditions. There is no marked difference between effects of water supply or excreta removal or both.

Obviously there is a lower limit to the number of cholera cases even with improved sanitary installations. One possible explanation lies in the fact that the communities observed

Table 12.2.2 Background data for a Philippine study. Cholera reduction as related to sanitary improvements.

Location name	Sanitary installations in test period	Families no	Persons no
West Visayan	Non treated drinking water from a non controlled well with sucking pump. No latrines	134	743
Dawis	Municipally piped drinking water. No latrines but indiscriminate disposal of excreta; ocean beaches nearby with tidal action	135	8o3
Magsungay	Drinking water from public wells with hand pump. Excreta removal using CM squatting plate over a bottomless barrel placed in sandy soil. When the barrel is full it is moved. 4 families per privy	128	787
Sibucao	Piped drinking water. Excreta removal at public buildings with flush toilet and septic-tank. Ultimately the effluents go to a canal and into the sea. 3o persons/toilet.	135	756

Table 12.2.3 Results of a Philippine study on Cholera.

Location	Sanitary Improvement DW	EXR	Population no	Cases of Cholera no	%
West Visayan	Control		743	115	15
Dawis	+	-	8o3	33	4
Magsemgay	-	+	787	35	4
Sibucao	+	+	756	33	4

Notes: DW - improved drinking water supply publicly piped and controlled
EXR - improved excreta removal system

are in daily contact and interaction with neighbouring communities and persons, which means that pathogen transmission is not controlled only by local precautions.

Another observation in the Philippine study is that cholera propagation and persistance in the community is limited where sanitary improvements are made compared to the control community.

The conclusion to these statements and findings is that an excreta removal system may significantly reduce the occurrence of communicable diseases, and at least cholera seems an illustrative example in this respect. Introducing an excreta removal system will probably not eliminate communicable diseases, because societal interaction creates many transmission routes that are not controlled through proper excreta removal.

Referring to tables 12.2.1 - 3 it is assumed that any of the systems FS, AP, JS, CM, AB, and MR could significantly decrease incidents of disease outbreak and propagation if installed in a community. The basis for comparison is a similar community without a well planned excreta disposal system. There will always be a background disease rate depending on the general situation in the country or region, where the observed community is situated. Further, with the data available there is no firm ground for a ranking of the systems in terms of disease transmission risks. Only for the Multrum, MR, is it necessary to make the reservation that feasibility still remains to be proven through practical investigation.

12.3. Economy

The economical evaluation is based on data presented at the end of each of the separate chapters 7 - 11. It should be recalled that data for full sewerage, FS, have been established after careful examination of cost sensitivity to slope of pipes, water, consumption, etc in order to obtain a relevant basis for comparison with other alternatives.

12.3.1. Present value, PV, calculations

All calculations refer to a 3o year planning period over which all investments are depreciated (periods of 5, lo and 3o years are used according to expected life time of equipment and civil works).

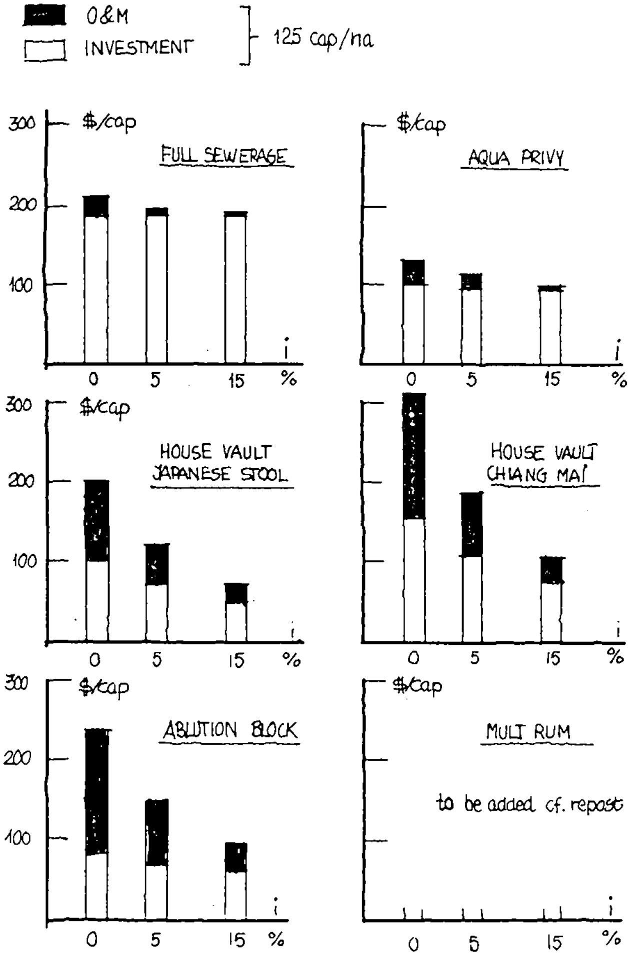


Figure 12.3.1.1 PV/cap, all systems compared, 125 cap/ha. 3o y planning horizon at different interest rates

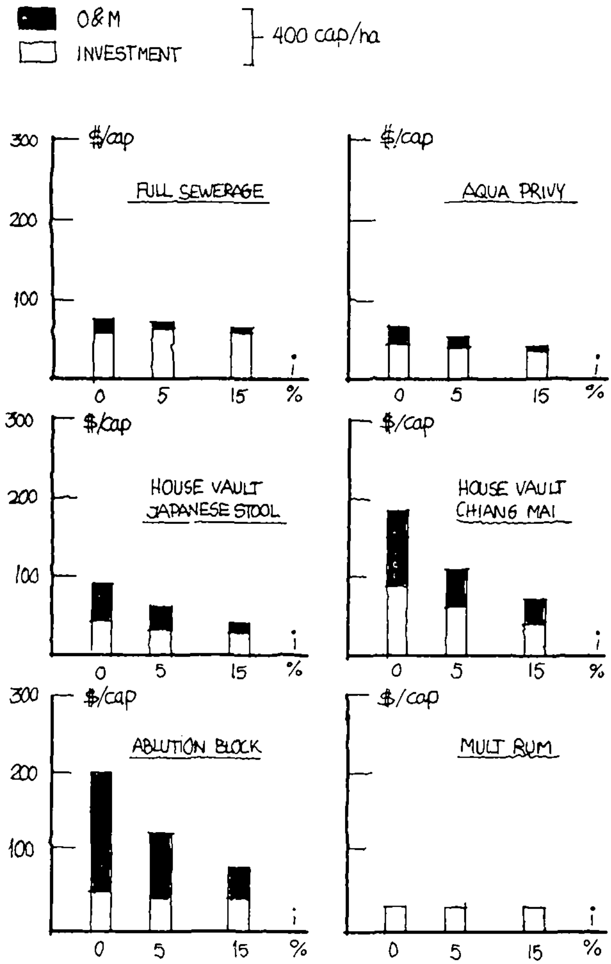


Figure 12.3.1.2 PV/cap, all systems compared, 4oo cap/ha. 3o y planning horizon at different interest rates

PV calculations indicate the amount of money that would initially be required to cover all future expenses (investments and O&M) within the project period. For an interest rate equal to zero, i = o, PV is the total cash outflow over the project period. For i > o PV is smaller than for i = o, which reflects the fact that postponed payments require less present cash.

Figures 12.3.1.1 - 2 indicate the calculated PV's in $/cap for 125 and 4oo cap/ha and how PV is composed of investments and O&M expenses. MR requires the total investment right at the beginning of the project period, and consequently there is no effect of varying interest rate. FS and AP are similar with only insignificant O&M contributions to PV.

For AB, JS and CM the PV's are composed of initial investments, postponed investments, and regular O&M, where the latter contributes greatly to the total PV. Consequently PV becomes particularly sensitive to changes in interest rate.

Distinction between investment and O&M is very important since O&M is an on-going and often labour intensive activity. Investments may also be labour demanding, but the employment period will be of relatively short duration. O&M therefore tends to constitute a cash flow that is recycled locally, whereas investments demand a cash flow out of the community.

PV is often used by investors to evaluate "best choice" of investments. The calculation of PV involves all future revenues and expenses within the project period. In figures 12.3.1.1 - 2 only expenditures are considered, and consequently the best choice would be the one with the least PV indicated in the figures.

It should be stressed that PV is only an economical decision criterion. Many other aspects such as employment or scarcity of water may pose important criteria that need separate evaluation.

12.3.2 User charges

It is assumed that within the project period (3o years) an annual user charge must be paid. The charge should be constant (1975 $), and payments are calculated respectively per plot and per capita. The charge is calculated so that present value of all future charge will equal that of all future expenditures with due regard to time of expenditure and interest rate, cf. chapter 6. Results of such calculations are presented in chapters 7 - 11, cf. tables 7.1o.5, 8.5.5, 9.5.5, 1o.5.5, and 11.3.3. Figures 12.3.2.1 - 2 give a graphical representation of these tables.

Obviously the user vill face wide ranges of annual sanitation expenditures depending on the technical alternative. This is demonstrated in table 12.3.2.1

Table 12.3.2.1 User charge ranges for alternative sanitation systems.

Basis cap/ha	Interest rate %	Cheapest[1] $/y	Most expensive $/y
	o	AP 5	CM 11
125	5	AP 7	FS 13
	15	JS 11	FS 3o
	o	AP 3	CM 8
2oo	5	AP 5	CM 1o
	15	JS 8	FS 19
	o	AP 2	AB 7
4oo	5	AP 3	AB 8
	15	AP & JS 6	AB 12

Notes: 1) MR would give the cheapest solution but is excluded from the comparison since the feasibility is yet un-proven.

The plot charges are derived easily when multiplying by 3, 5, and 1o for 125, 2oo and 4oo cap/ha respectively

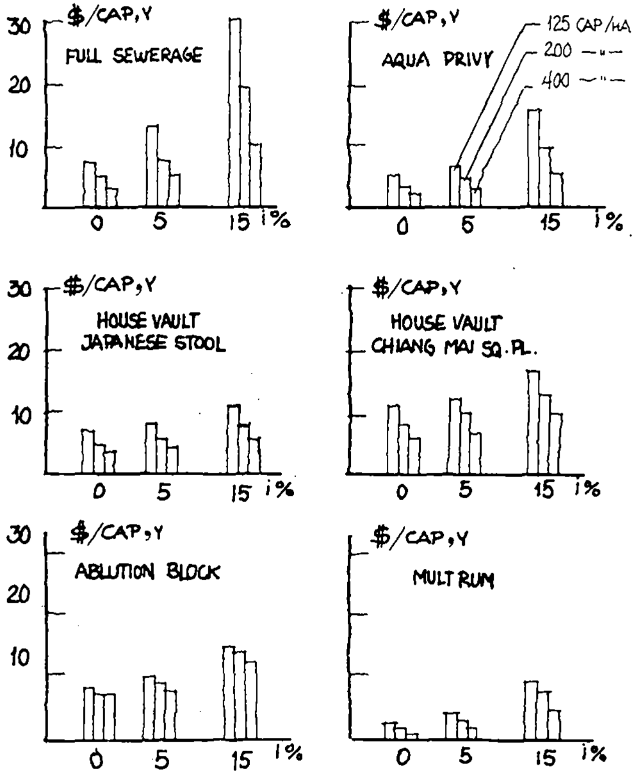


Figure 12.3.2.1 User charges, $/cap,y, all systems compared. 3o y planning horizon at different population densities and interest rates

Table 12.3.2.1 shows the cheapest and the most expensive alternatives, and it appears that expenditure per capita per year depends very much on population density as well as interest rate. A total range from 2 to 3o $/cap,y is found.

Generally the most expensive alternative is 2 - 3 times as expensive as the cheapest, cf. 8/3, lo/5, and 19/8 for

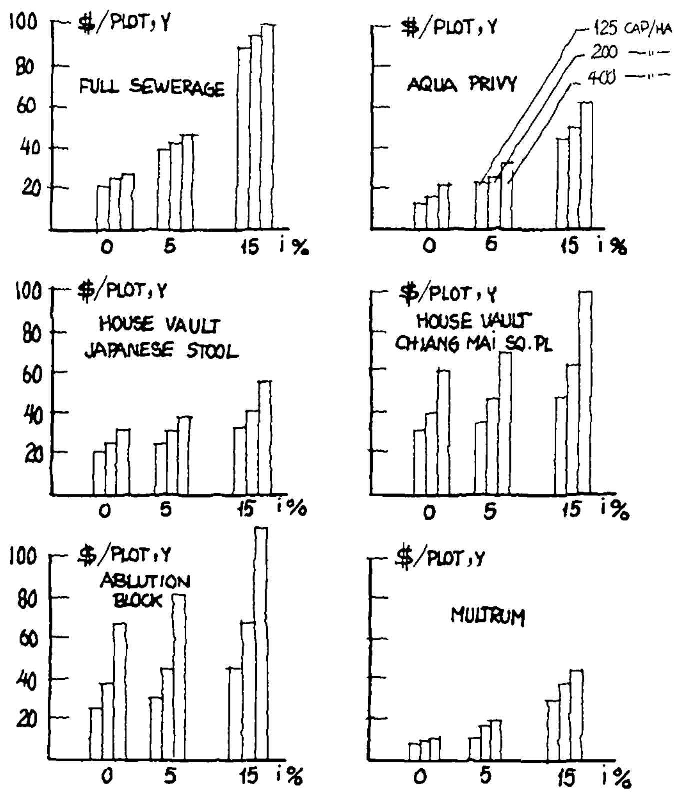


Figure 12.3.2.2 User charges, $/plot,y, all systems compared. 3o y planning horizon at different population densities and interest rates

2oo cap/ha, table 12.3.2.1. This is valid for any interest rate and any population density. Similarly, for any chosen population density the expenditure may vary with a factor 2 - 3 depending on the applied interest rate.

For any population density and reasonably low interest rates the AP system is cheapest (next only to MR). For high population densities (4oo cap/ha) the AP is cheapest even at high interest rates. Table 12.3.2.1 shows AP price levels of 2-3 $/cap,y for 4oo cap/ha, and 5-7 $/y for 125 cap/ha for interest rates o-5%. For densities of 4oo cap/ha the price rises to 6 $/cap,y for i = 15%. At high interest rates the JS becomes competitive to AP for population densities 2oo and 125 cap/ha. The reason is obviously that AP requires major investments at the beginning of the project period, which involves relatively high interest payment over the project period. JS has investments (trucks in particular) distributed regularly over the entire project period, which requires a relatively low total interest payment.

The most expensive alternative is either FS, CM, or AB. For small population densities and low interest rates CM becomes most expensive; cf. table 12.3.2.1 for 125 and 2oo cap/ha. With increasing interest rate FS becomes most expensive, and again the reason is that essentially all capital must be invested at the beginning of the project period. At high population densities the AB becomes the most expensive alternative.

Between most expensive and cheapest alternative are systems that would need other than economical justification for possible selection, cf. below and the discussion above as to O&M and investment.

12.3.4 Cost of water supply & waste water removal

The costs of water supply were indicated in chapter 6, table 6.2.1. Though data are scarce and relate mainly to East Africa it is deemed relevant to compare costs of water supply and waste water removal.

It should be kept in mind that the water supply prices are from existing distribution systems and a certain overhead for administration may be included, which is not the case for the waste water removal data.

Table 12.3.4.1 Relative WWR costs at different interest rates

	i=5%		i=15%	
Density	AP + WS	WWR cost ratio	JS + WS	WWR cost ratio
	1)		2)	
cap/ha	$/cap,y	%	$/cap,y	%
125	7 + 13	35	11 + 2.5	81
2oo	5 + 8	38	8 + 1.8	82
4oo	3 + 7	3o	6 + 1.6	79

Notes: 1) Cf. tables 12.3.2.1 & 6.2.1 from where it appears that AP is the most favourable WWR alternative at i=5%

2) Cf. tables 12.3.2.1 & 6.2.1 from where it appears that JS is the most favourable WWR alternative at i=15%

To indicate the magnitude of cost of WWR compared to WS + WWR table 12.3.4.1 lists the pertinent data for the economically most favourable WWR alternatives for two different interest rates.

Apparently the WWR cost ratio is influenced considerably by the interest rate. However, the reason is that the water supply is by plot tap in the case of AP, and by public stand pipe in the case of JS. Recalling the lack of accuracy of the water supply cost, cf. section 6.2, the above cost ratios are listed merely to inform about order of magnitude.

12.4. Employment, supplies & education

12.4.1. Employment

As mentioned already under economical considerations the same user expenditure may involve very different employment situations. Typically the FS & AP systems require limited O&M expenses and few labourers employed; whereas JS, CM & AB would require a considerable number of employees to secure proper operation, cf. table 12.4.1. For example, the labour force is lo-3o times bigger for AB than for FS, and the ratio is very much dependant on population densities. The MR system is by nature the least labour demanding of all systems under evaluation.

Table 12.4.1 Annual O&M in terms of labour, trucks and supplies.

SYSTEM	I T E M	PERSONS PER HECTAR 125	2oo	4oo
Full Sewerage	Labour no	19	21	26
	Trucks no 1)	1	1	1
	Labour looo $	14	15	18
	Supplies looo $	2	2	2
Aqua Privy	Labour no	16	18	23
	Trucks no 2)	5	7	13
	Labour looo $	22	27	44
	Supplies looo $	6	9	16
JS & Vault & Truck	Labour no	84	97	117
	Trucks no	26	3o	34
	Labour looo $	68	76	9o
	Supplies looo $	39	46	54
CM & Vault & Truck	Labour no	124	151	221
	Trucks no	46	57	86
	Labour looo $	1oo	119	174
	Supplies looo $	68	85	129
Bucket & Ablution	Labour no	226	362	711
	Trucks no	1	1	1
	Labour looo $	138	22o	429
	Supplies looo $	37	59	117
Multrum	Labour no			
	Trucks no		LOVE	
	Labour looo $			
	Supplies looo $			

Notes: 1) Jet cleaner
2) Jet cleaner + vacuum trucks

A labour demanding WWR system may be attractive at an early stage of development where migration towards the cities typically is associated with an unemployment situation. But a general conclusion should be avoided and decisions made only after careful examination of problems pertinent to the region under investigation.

It should be kept in mind that table 12.4.1 applies to O&M costs, and these are only a fraction of total costs (and user expenditures). Information as to O&M versus total cost can be found e.g. in figures 12.3.1.1 -2.

12.4.2 Supplies

The extremes of supply costs (fuel, electricity, tires, lubrication,...) are found in table 12.3.1 for FS (minimum ∿ 2000 $/y). In any case the supply is merely a fraction of total O&M, and particularly so for piped WWR systems such as FS & AP.

12.4.3. Education

One important component has been omitted from O&M, namely the cost of education of the users of new or remodeled sanitation facilities.

It must be emphasized that education, e.g. by sanitary inspectors, will be necessary over an extended period (years). Only through such education programmes will the health improvements become manifest after the installation of the system.

It is not justified to expect any significant difference in effects and costs between educational programmes pertinent to the different alternative sanitation schemes. Of course the elements of knowledge and practice will be different, but education will be necessary as to basic sanitary principles and disease control irrespective of which system may be introduced.

Therefore it is assumed that education and costs involved herein be included in any sanitation improvement programme. These costs have, however, been excluded in all calculations in the present report, because they would be essentially the same for all of the systems.

13. CONCLUSIONS

Waste water removal systems for urban housing in developing countries have been analyzed through a hypothetical case based on background data from Lagos, Nigeria. Some general and some specific conclusions can be drawn and these are presented below under individual headings. Six alternative systems have been considered:

- Full (conventional) sewerage (FS)
- Aqua privy with piped liquid disposal (AP)
- Japanese stool with vault, excluding sink waste (JS)
- Chiang Mai squatting plate, excluding sink waste (CM)
- House bucket and communal ablution blocks (AB)
- Multrum (MR)

Best alternative

There is no universally best alternative, but the precedent analysis has demonstrated how several circumstances are important in the search of an optimal solution, e.g. density of population, slope of ground, water supply, interest rates, service level etc.

Population density is of particular importance. At high densities, e.g. above 4oo cap/ha, the piped systems are more favourable from an economic point of view. At low densities, e.g. 125 cap/ha and less, the non-piped systems appear prefereable - particularly when interest payment is taken into consideration.

The case study has been regarded a virgin land development, but the conclusion above holds true for remodelling of existing urban settlements

It should be noted that full sewerage (FS) and "piped system" are not synonymous; if only a low water consumption is available a piped system is still feasible (AP and AB), whereas FS is not.

Health

For each of the systems FS, AP, JS, CM, and AB it may be concluded that they could significantly and equally well improve health relative to a situation without waste removal facilities. The prerequisite is good technical operation, which applies to all of the systems.

MR is not included in the statement because of lack of data.

General service

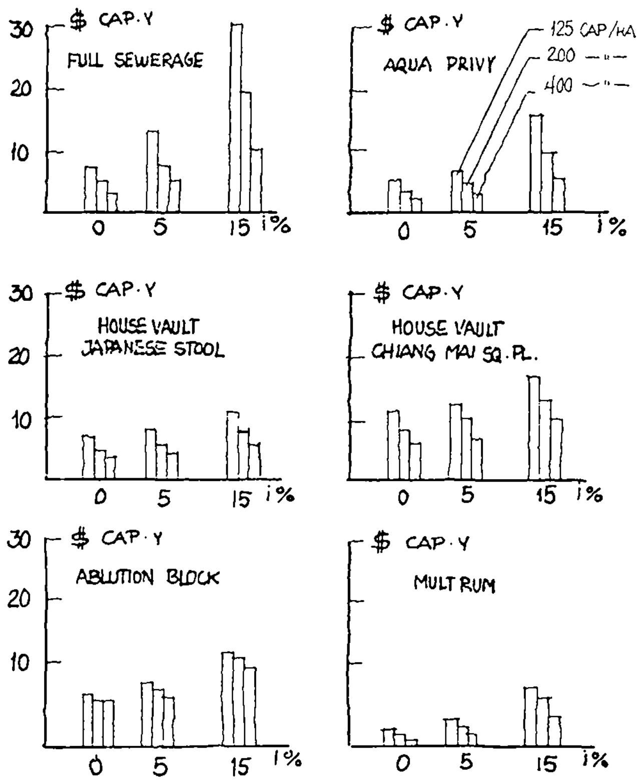
Parameters have not been established to quantitatively indicate the level of user service generated by each of the systems under consideration. The difficulty may be demonstrated for example, by asking which is more convenient or satisfactory, 1) to use a waterless JS installation on ones own plot, or 2) to use a flush toilet and have a wash stand available in a communal ablution block loo meter away?

A very rough categorization may be as follows, class I superior to II, etc:

Service class I: FS, AP
Service class II: JS, CM, AB
Service class III: MR (?)

User charges

In order to compare economically the different systems, the user charge is found to be an appropriate parameter, cf. section 12.3 above from which the subsequent figure is reproduced. The Multrum is merely of theoretical interest at the present, because the practical feasibility remains to be proven.



The user charges, as indicated in the figure, must be used with care because of the specific assumptions of this report, cf. chapter 6. A relative rather than an absolute evaluation is recommended. It is however interesting how the Aqua Privy appears favourable at low and medium interest rates, indicating that much attention should be paid to this system, particularly where little natural land slope and/or limited water supply discourage the use of other piped systems.

Full sewerage, FS

The FS system is evaluated in chapter 7. It should be re-emphasized that good technical performance of FS requires a minimum water consumption of app. 5o ℓ/c,d.

The base design calculations led to the following conclusions regarding cost sensitivities to different FS designs:

- lowering the water consumption would not significantly reduce the project cost, regardless of population density, cf. figure 7.6.1.
- decreasing average slope of mains from 15 pm to 2 pm could decrease total cost of sewerage (PV = present value, 3o year project period) by as much as 35%; cf. figure 7.7.1.
- natural land slope exceeding 3 pm may significantly reduce the total cost of sewerage; the potential PV reductions are equivalent to savings on main sewers provided that gradiants for these are acceptable.

Thus, FS cost is sensitive mainly to change of slope (ground, main sewer or both). The sensitivity of cost to the level of water consumption is insignificant.

In order to facilitate subsequent comparison the FS cost data were established as follows:

Density cap/ha	FS cost in $/capita,year i = o%	i = 5%	i = 15%	
125	7 (.7)	13 (.5)	3o (.6)	Figures in
2oo	5 (.6)	8 (.6)	19 (.5)	paranthesis
4oo	3 (.5)	5 (.4)	1o (.3)	indicate O&M

The $ values are transferred from table 7.1o.5, and it is demonstrated that FS user charges are extremely sensitive to interest rates. The cost of O&M is negligible particularly at high interest rates, and the FS system tends to be capital intensive and labour extensive.

Aqua Privy, AP

The AP system is described and evaluated in chapter 8. The unique feature of AP is its flexibility to water consumption, e.g. a range from 2o - 4oo ℓ/p,d is acceptable without technical malfunction.

Cost data of the AP may be summarized as follows:

Density	AP cost	$/capita, year		
cap/ha	i = 0%	i = 5%	i = 15%	
125	5 (1.2)	7 (1.o)	16 (1.o)	Figures in
2oo	3 (.8)	5 (.8)	lo (.7)	paranthesis
4oo	2 (.7)	3 (.6)	6 (.6)	indicate O&M

The $ values are transferred from table 8.5.5, and also in this case are the user charges highly influenced by interest rates. It appears that user charges will be used mainly to repay invested capital and interests, and only to a limited extent used for local employment, i.e. O&M.

House Vault, Japanese Stool, JS, and Chiang Mai, CM

There is virtually no minimum water consumption for these systems, but for CM a siphon flush requiring app. o.8 ℓ/visit must be encounted; cf. details in chapter 9.

Cost data for these two systems can be summarized as follows based on tables 9.5.5 and 9.5.4:

	Density	JS & CM cost	$/capita, year		
	cap/ha	i = 0%	i = 5%	i = 15%	
JS	125	7 (3.6)	8 (3.5)	11 (3.5)	
	2oo	5 (2.5)	6 (2.4)	8 (2.4)	
	4oo	3 (1.5)	4 (1.7)	6 (1.6)	Figures in
					paranthesis
CM	125	11 (5.4)	12 (5.1)	17 (5.2)	indicate O&M
	2oo	8 (3.9)	lo (4.o)	13 (3.8)	
	4oo	6 (2.9)	7 (2.9)	lo (2.9)	

With these two systems the user charge is less sensitive to interest rates, and a significant part, often 1/2, is kept in the society for O&M. It must be added though that the user charge as such tends to be higher than for AP, particularly at low interest rates. For high interest rates JS and CM require less user charge than FS.

Ablution Block, AB

This includes a piped sewer system, but there are no individual plot connections, neither for water supply nor for waste water removal. Details are given in chapter lo.

Cost data can be summarized as follows:

Density cap/ha	AB cost i = 0%	$/capita, year i = 5%	i = 15%	
125	8 (5)	lo (5)	15 (5)	Figures in
2oo	7 (5)	9 (5)	14 (5)	paranthesis
4oo	7 (5)	8 (5)	12 (5)	indicate O&M

For AB the user charge is less sensitive to interest rates than in the case of FS and AP. A significant proportion (3o - 7o%) of the user charge is used for local employment. The user charges as such are generally comparable to those of JS and CM; they are comparable to those of AP at high interest rates and low population densities, but much higher at low interest rates and high population densities.

While CM and JS do not naturally develop into a piped system, the AB system could easily and logically be converted to a FS system.

Even though the initial AB capital investment is significant and comparable to that of FS for high population densities (cf. figures 12.3.1 1&2) it may be advantageous to use AB as a predecessor of FS at an early stage of development, particularly because water consumption may initially be too low for FS and because the AB system includes washing facilities (removal also of grey waste water).

New developments, new technology

Piped systems such as AP and AB offer promising alternatives for waste water removal from planned high-density low-income urban developments, mainly because these can be operated at low cost and at the low water consumption rates that may be typical at early stages of development. New technology is not needed to establish AP or AB systems in pilot or full scale, and therefore pilot or full scale experiments should be initiated as soon as possible, e.g. in some developing regions in African countries.

When planning for very low density urban settlements the non-piped WWR systems would seem attractive. The MR is an interesting proposal which is now being tested in small-scale in Tanzania. For the individual disposal type system there may be ample room for other developments and testing of new WWR technology, probably with emphasis on water saving systems.

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15. ABBREVIATIONS

Generally applied abbreviations are the following:

AB	Ablution block, cf. chapter lo
AP	Aqua privy, cf. chapter 8
black waste	Human excreta
BOD	5-days biological oxygen demand
C	Collection of sewage
cap	capita, identical to p = person
CM	Chiang Mai house vault system, cf. chapter 9
d	day & night
DANIDA	Danish International Development Agency
DWF	Daily Water Flow , normally .85 x daily water consumption, e.g. 4oo→34o ℓ/p,d
FS	Full sewerage, cf. chapter 7
grey waste	Waste water from washing and cooking
h	hour
i	interest rate (calculatory), year basis, cf. chapter 6
JS	Japanese Stool house vault system, cf. chapter 9
kWh	kilo Watt hour - energy unit = 3.6×10^6 Joule
ℓ	liter
mg	milligramme, lo^{-6} kilogramme
mo	month
MR	Multrum, cf. chapter 11
no	number
O&M	Operation & Maintenance
p	person, cf. cap
PI	Plot Installation
pm	per mille = per 1ooo = o/oo
PV	Present Value of future expense or revenue, cf. chapter 6
T	Treatment of waste water
w	week

Cont.

WHO	World Health Organization
WS	Water Supply
WWR	Waste Water Removal
y	year = yr

16. APPENDICES

Two appendices, I & II, are presented below to further support the calculations on sewerage in chapter 7 and some of the generally applied unit costs.

Appendix I

Sewer prices have been based on American practice, cf. American Society of Civil Engineers: "Design and Construction of Sanitary and Storm Sewers", ASCE Manual of Engineering Practice no 37. Prices have been updated and tables for direct cost estimates prepared by FROISE /31/ whose work was used to establish table A I.1

Table A I.1 Prices of Sewers, \$/m, US \$ 1975, after FROISE /31/

Pipe diameter	Depth of sewer (m)						
mm	1.1	1.3	1.5	2.o	2.5	3.o	5.o
1oo	8	9	12	19	22	29	71
15o	11	13	15	22	27	34	77
2oo	13	14	17	24	3o	43	77
25o			2o	26	3o	45	79
3oo			2o	26	3o	45	81
35o			22	29	32	48	81
4oo			24	31	34	51	87
45o			28	34	37	54	9o
5oo			33	4o	43	59	99
6oo			4o	5o	53	69	1o9
7oo			55	62	68	85	131
8oo			7o	81	85	1o4	147
9oo				9o	97	113	166
1ooo				1o9	115	132	181
11oo				125	133	152	2o5

Note: Prices include dewatering, but exclude removal and replacement of pavement.

Table A I.2 Prices of manholes, $/unit, US $ 1975, after FROISE /31/

Type	Use	Depth (m) -1.o	1.o-1.8	2.o-4.o	4.o-
Type I	laterals only	1oo	-	-	-
Type II	Ø sewer 2oo-5oo mm	-	3oo	55o	1ooo
Type III	Ø sewer > 6oo mm	-	-	1ooo	125o
Flushing shaft	middle of mains for cleansing	-	1oo	15o	2oo

Prices for manholes have been estimated, FROISE/31/, according to the classification shown in table A I.2

Other unit costs have been estimated from own experiences in African countries or are obtained from persons or companies with experience in developing countries.

Table A I.3. Unit costs of miscellaneous civil works, US $ 1975

Item	Cost, $
Trenching (depth < 1.o m)	$1.2/m^3$
Excavation in bulk	$1.5/m^3$
Excavation in bulk incl. backfill, grading & compacting	$2.2/m^3$
Pond lining	$o.5/m^2$
Concrete slabs (8 cm)	$1o/m^2$
Fencing	15/m
Concrete placed incl. form & reinforcement	$1oo/m^3$
Concrete lining incl. mesh	$75/m^3$

Appendix II

The trunk sewer design calculation has been summarized in the following table A II.1. Figure A II.1 explains the symbols used and shows the area and the trunk sewer lay-out. Results are utilized in chapter 7.

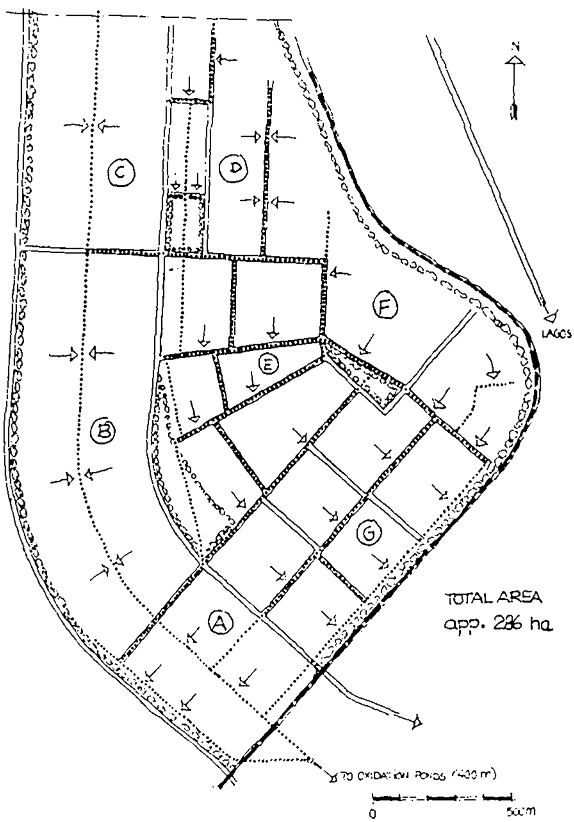

Figure A.II.1. Layout and Difinitions for sizing of trunk sewers.

Table A II.1. Sizing of trunk sewers

1) ESTATE	Residental drainage area, ha	Lateral	Main	A mm diam.	A m trunk	B mm diam.	B m trunk	C mm diam.	C m trunk 2)
OUTSIDE	286.4	0	0	700	400	800	400	1100	400
	272.8			700		800	170	1100	170
A.		4655							
	272.8		4900	700	50	800	50	1100	50
24.5 ha	218.2			700	230	800	230	1100	230
	189.4			700	265	800	265	1000	265
	13.6			250	900	300	450	400	320
						250	450	350	140
								300	150
								250	290
B.		12010							
	100.6		12640						
				600	250	700	180	800	580
63.2 ha				500	500	600	700	700	520
				450	350	500	270	600	240
				400	240	450	190		
C.		7545							
	39.7		7940						
39.7 ha				400	90	450	140	600	180
				350	240	400	200	500	180
				300	200	350	160	450	120
				250	300	300	120	400	120
						250	210	300	130
								250	100
E.		8540							
			8970						
	66.2			500	250	600	700	800	250
	38.6			450	780	500	330	700	780
43.1 ha				400	350	450	350	600	350
	15.5			300	250	300	350	400	280
				250	600	250	500	350	100

Table A II.1 Cont.

								300	140
								250	330
	9.8			250	550	250	550	350	100
								300	100
								250	350
	17.8			300	100	350	100	400	160
				250	410	300	100	350	100
						250	310	300	100
								250	150
D.		7485							
			7870						
	38.6			300	200	350	200	400	200
38.6 ha				250	990	300	320	350	320
						250	670	300	70
								250	620
				300	490	400	70	450	70
				250	370	350	390	400	350
						300	150	350	120
						250	250	300	100
								250	200
G.	77.3	8835							
			9300						
46.5 ha	54.6			450	460	500	650	700	450
				400	550	450	440	600	700
				350	320	400	240	500	170
				300	330	350	310	450	260
F.				250	1000	300	170	400	140
						250	850	350	120
30.8 ha		5960						300	430
			6270					250	390
				300	570	350	530	450	460
				250	590	300	230	400	200
						250	400	350	100
								300	120
								250	280

Notes: 1) Letters refer to markings on figure A II.1.
2) Based on 2 x DWF + I